Observed Long-Term Performance of Prefabricated Deck-Level Connections

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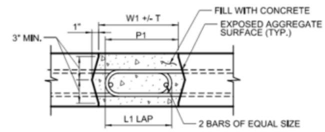
Purpose of this presentation

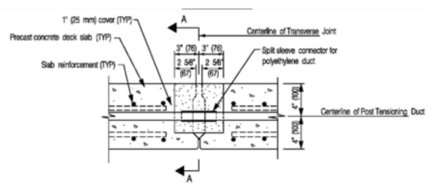
- Review the performance of deck-level connections
  - These connections represent the portions of an ABC design that have the most severe exposure.
- We will review the types of connection that are most commonly used.
- Review of three case studies from actual ABC projects with significant age and exposure
- Review how the 2018 AASHTO LRFD Guide Specifications for ABC address deck-level connections and how to achieve durability

Common types of deck level ABC connections

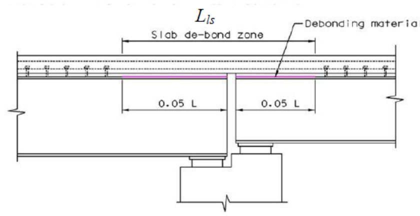
- Concrete closure joints with lapped bar reinforcement



- Post-tensioned joints with grouted shear keys



- Link slabs



Case Study – I-84 Ramp, Waterbury, CT

Deck replacement built in 1990
- Curved ramp structure (straight beams)
- 6 Span Bridge (700 ft) w/continuous spans
- ADT = 7100
- Full closure with detour - Constructed in 48 days
- Full-depth deck panels with longitudinal PT
- Deck protection: Woven glass fabric hot asphalt membrane with asphalt overlay

Case Study – I-84 Ramp, Waterbury, CT

Deck panel erection

Grouting shear keys

Case Study – I-84 Ramp, Waterbury, CT

Post-tensioning

PT Grouting

Case Study – I-84 Ramp, Waterbury, CT

CIP closures at joints

Grouted shear connector pockets
CIP parapets

Case Study – I-84 Ramp, Waterbury, CT

Construction complete

Case Study – I-84 Ramp, Waterbury, CT

- Performance after 34 years
- Notes from latest inspection report
  - Deck condition = 6
  - Spalls in closure pours at leaking deck joints
  - No issues with precast panels
  - No issues with leaking transverse PT joints
  - Leaking at deck expansion joints
  - Less than 1% overall deterioration

Case Study – I-84 Ramp, Waterbury, CT

2010 Condition
20 years in service

2023 Condition
33 years in service

Case Study – I-84 Ramp, Waterbury, CT

2023 Condition
33 years in service
CIP closure pours at leaking deck expansion joints

Case Study – I-84 Ramp, Waterbury, CT

- Overall performance
  - The performance of the precast panels system with longitudinal PT is exceptional
  - End closure pours have issues, but the deck joints are leaking
  - The precast panels should easily meet a 75-year service life
- Lesson Learned
  - Use better quality concrete for closure pours
  - Try to eliminate deck expansion joints
  - Maintain deck expansion joints

Case Study – Route 8 Viaduct, Seymour, CT

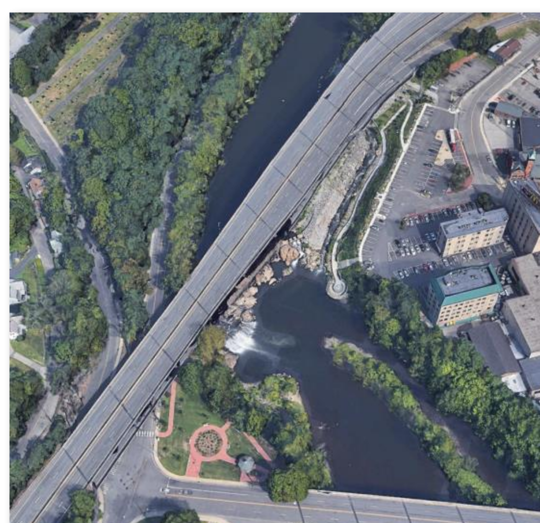
Site 1: Bridge 00588, Deck Replaced between 1992 and 1994

- Curved structure (straight beams)
- 47 Spans, ADT 42000
- Weekend closures with crossover – Built in 2 seasons
- Full-depth deck panels with longitudinal PT
- Details similar to the Waterbury Bridge
- Deck Protection: Sheet membrane with asphalt overlay

Case Study – Route 8 Viaduct, Seymour, CT

Site 2: Bridge 00587, Deck Replaced between 1992 and 1994

- Curved structure (straight beams)
- 20 Spans, ADT 42000
- Weekend closures with crossover
- Built in 2 seasons
- Full-depth deck panels with longitudinal PT
- Details similar to the Waterbury Bridge

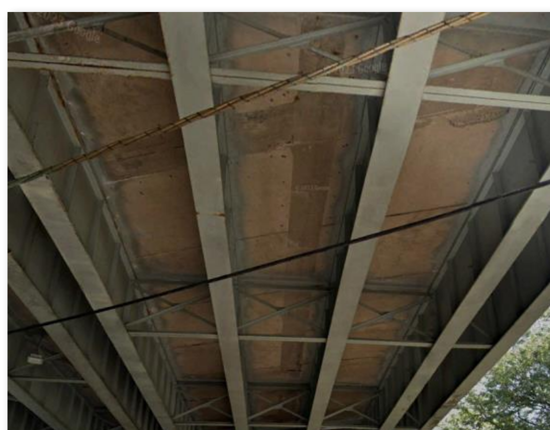


Case Study – Route 8 Viaduct, Seymour, CT

- Performance after 30 years
- Notes from latest inspection report
  - Deck condition: 5 to 6
  - Isolated issues with precast
    - Spalls
    - Minor joint leakage
    - Minor cracking
  - Significant issues with CIP closure pours
  - Leaking at deck expansion joints
    - Condition Rating = 4

Case Study – Route 8 Viaduct, Seymour, CT

Bridge 00588 deck condition
30 years in service



Bridge 00587 deck condition
30 years in service

Case Study – Route 8 Viaduct, Seymour, CT

Precast panels
30 years in service

CIP closure pour at leaking deck joint
30 years in service

Case Study – Route 8 Viaduct, Seymour, CT

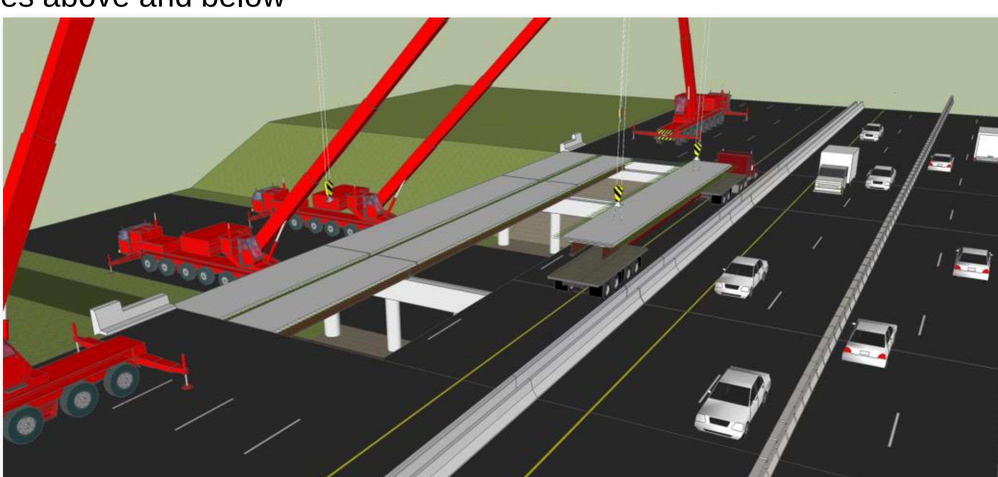
- Overall performance
  - The precast panels with longitudinal PT are performing well.
  - There are some issues, but not a significant percentage of the overall bridge.
  - The sheet membrane may not have been the best solution.
  - The end closure pours have issues, but the deck joints are leaking.
  - The rapid set concrete in the closure pours may not have been the highest quality.
  - The precast panels should meet a 75-year service life.
- Lessons Learned
  - Use better quality concrete for closure pours
  - Try to eliminate deck expansion joints
  - Maintain deck expansion joints

Case Study – 93Fast14, Medford, MA

14 Bridges at 7 Sites: Superstructure replacements built in 2011
- 41 spans total: 2 four span bridges, 1 single-span, 1 two-span, 10 three-span bridges
- ADT 175,000: 415± million vehicles and 40± million trucks since 2011
- Weekend closures with crossovers – Built in 10 weekends
- Modular deck beams with Link Slabs
- Deck Protection: Spray applied membrane with asphalt overlay
- Started as Design-Bid-Build, changed to Design-Build

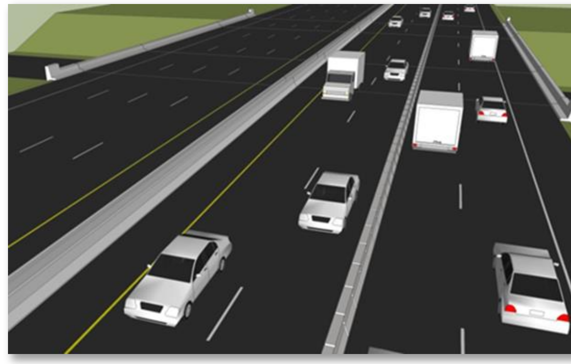
Anticipated Construction Methods

Cranes above and below



Traffic Management

- Close entire bound during weekends
  - 55 Hour closure
    - Give the entire other bound over to the contractor
  - Production: 2 bridges per weekend
- Crossovers were constructed at each end of the project limits
  - 6 miles x 4 lane work zone



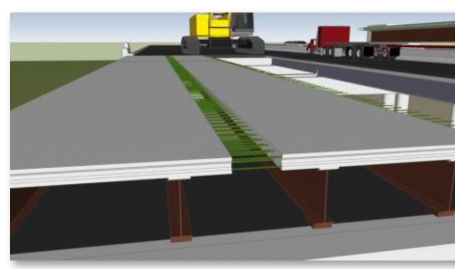
Deck Connections

<u>Options allowed</u>

1. Narrow Closure pour with straight bars and UHPC
2. Medium width-closure pour with hooked bars and grout
3. Wide pour with lapped bars and High Early Strength Concrete

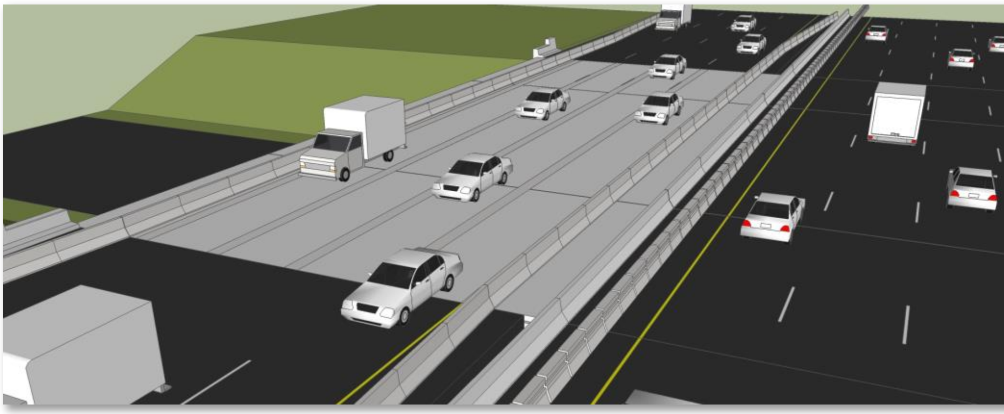
<u>Wide pour chosen</u>

- Reduced precast deck width:
  - Reduced shipping width and weight
  - Reduced crane pick weight
- Ease of installation
  - Easier to install with extended bars and longitudinal bars
  - More room for adjustment = Less Risk to the Contractor



Non-weekend work

- Installation of barriers: After the weekend work



Construction Sequence

Friday Night, 10:00 pm – Demolition Begins

Construction Sequence

Friday Night – Beam Removal and Cutting

Construction Sequence

Saturday Morning, 7:00 am – Demolition Complete

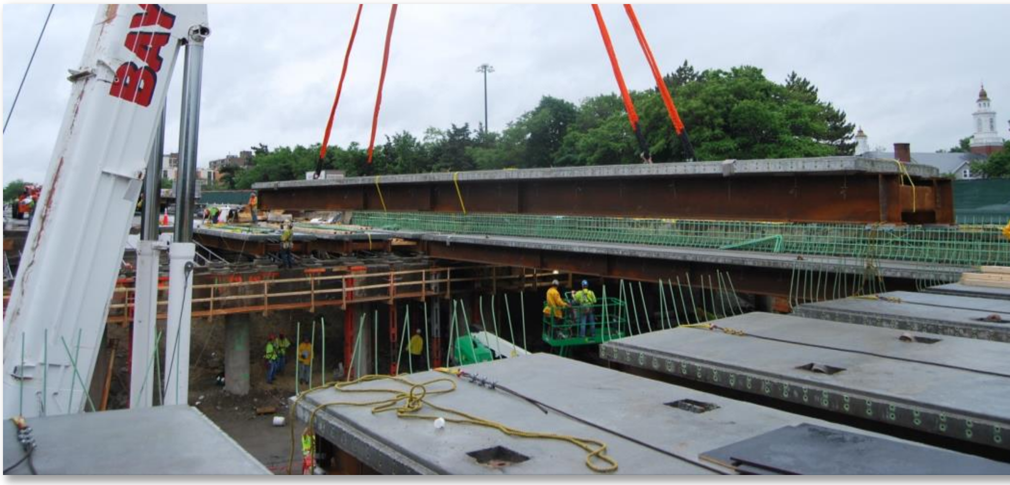
Construction Sequence

Saturday Morning, 11:30 am – Erection Begins

Construction Sequence

Saturday Afternoon – Erection Continues

Construction Sequence

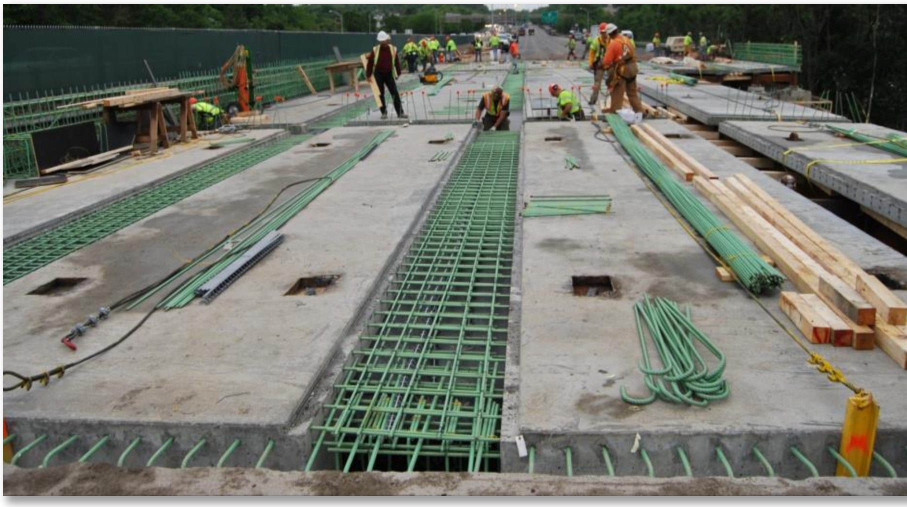


Saturday Afternoon – Erection Continues

Construction Sequence

Saturday Afternoon – Forming of Closure Joints

Construction Sequence



Saturday Afternoon – Placement of Closure Joint Reinforcing Steel

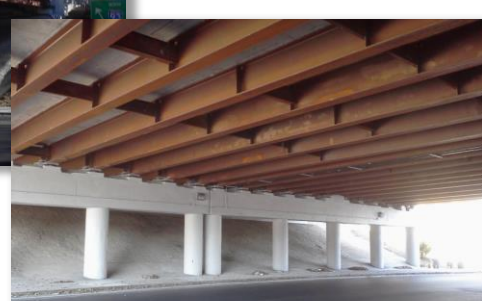
Construction Sequence

Sunday Morning – Placement of High Early Concrete

Construction Sequence

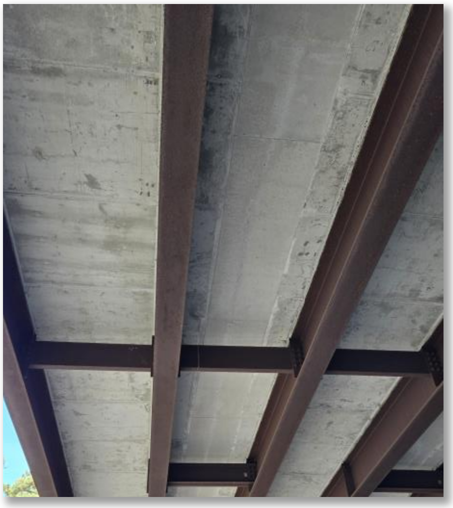
Sunday Morning – Curing of Closure Pours

Construction Sequence



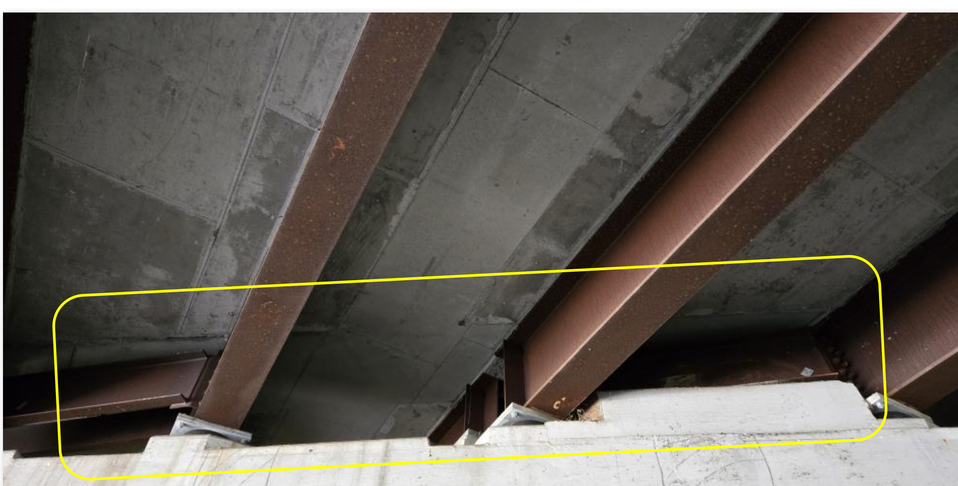
Completed Bridge in 2011

Case Study – 93Fast14, Medford, MA



Longitudinal Closure Joints in 2024 - 13 years in service

Case Study – 93Fast14, Medford, MA



Link slabs in 2024 - 13 years in service

Case Study – 93Fast14, Medford, MA

Link slabs in 2024 - 13 years in service

Case Study – 93Fast14, Medford, MA

Link slabs in 2024 - 13 years in service

Case Study – 93Fast14, Medford, MA

Overlay in 2024 - 13 years in service
Minor cracking at link slabs (not leaking)

Case Study – 93Fast14, Medford, MA

- Overall performance
  - The performance of the longitudinal closure joints is exceptional
  - Deck condition: 6 to 7
  - The performance of the link slabs is excellent
    - Even with 40+ Million truck cycles
  - The superstructure should easily meet a 75-year service life
- Lesson Learned
  - Use high quality concrete for closure pours
  - Link slabs are a durable alternative to expansion joints and continuity

Review of the 2018 AASHTO LRFD Guide Specifications for ABC

- Details covered today…

  …are covered in this specification.

  …have designs that are consistent with this specification.

AASHTO LRFD Guide Specifications for ABC

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.

Closure Joint with Lapped Hooked Bars

3.6.2.2—Hooked Reinforcing Bars in Closure Joints

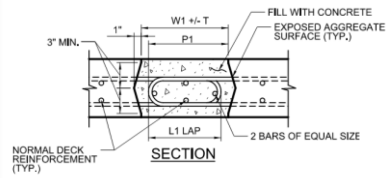
Hooked bars may be lapped within the closure joint. The minimum length of the lap between adjacent hooked reinforcing bars shall be equal to the hook development length, l_{hb}, specified in *AASHTO LRFD Bridge Design Specifications*. The detailing of the hooks shall include at least one transverse bar 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



Section 3: Link Slab Connections

3.6.9—LINK SLABS

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

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

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

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

C3.6.9

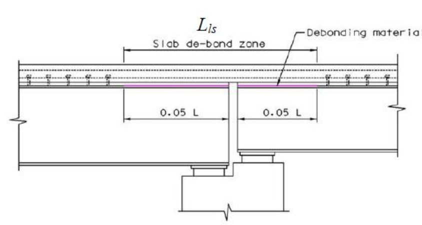
Links slabs are a means of providing a jointless bridge without continuity. The deck is continuous across the substructure element, but the supporting beams or girders are not. The basic design approach for link slabs is to design them to accommodate anticipated beam end rotations (Caner, A. and Zia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

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

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

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

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



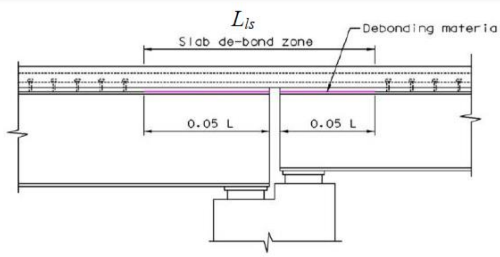
Section 3: Link Slab Connections

3.6.9.1—Primary Reinforcement

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

C3.6.9.1

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



Section 3: Link Slab Connections

3.6.9.2—Link Slab Reinforcement

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

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

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

C3.6.9.2

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

Link slabs do not provide measurable resistance to negative bending moments, and their stiffness is significantly less than the adjacent composite beams, therefore the joint can be assumed to be a theoretical pinned connection. This approach is similar to the assumption made with the design of an integral abutment bridge.

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

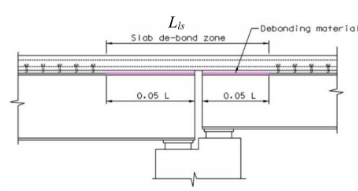


Figure C3.6.9.2-1—Links Slab Debond Zone

Section 3: Link Slab Connections

The design bending moment in the link slab per beam M_{ls} shall be calculated as follows:

$$M_{ls} = 2E_{ls}I_{ls}\Theta/L_{ls} \quad (3.6.9.2\text{-}1)$$

where:

E_{ls} = modulus of elasticity of the link slab (ksi)
I_{ls} = moment of inertia of the link slab using gross gross section properties (in.^{4})

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

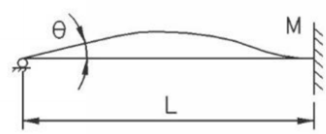
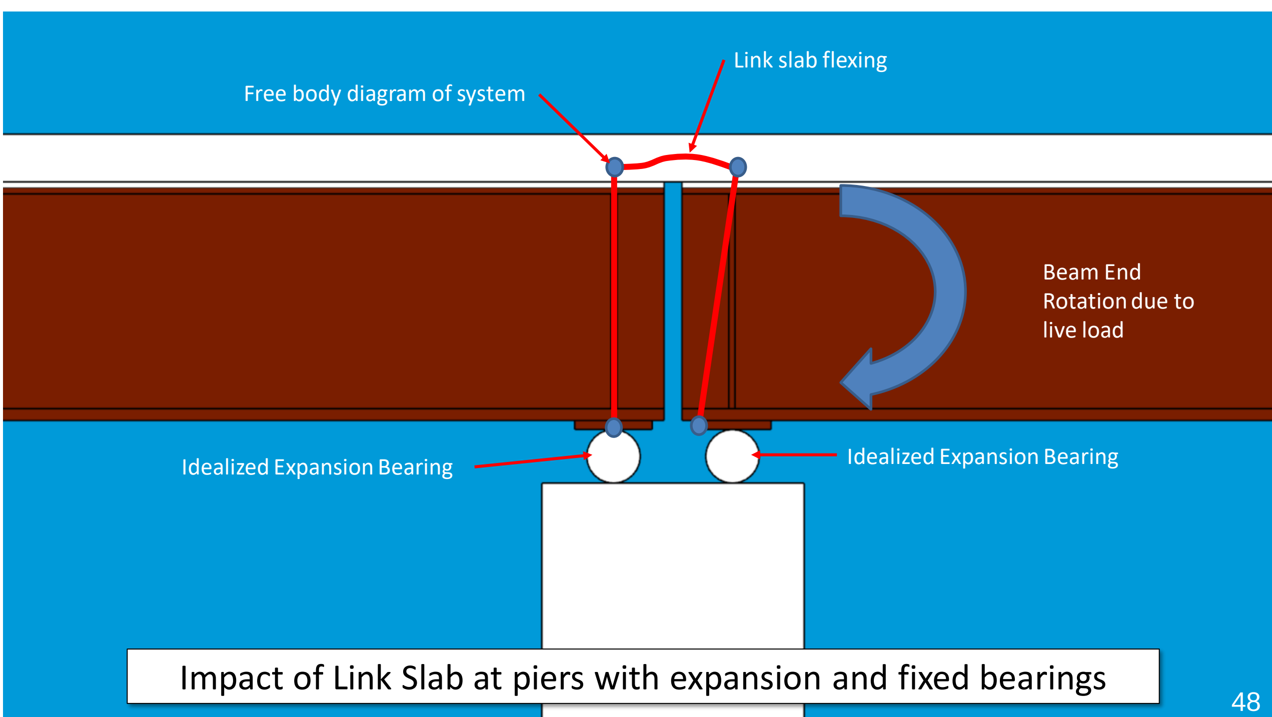
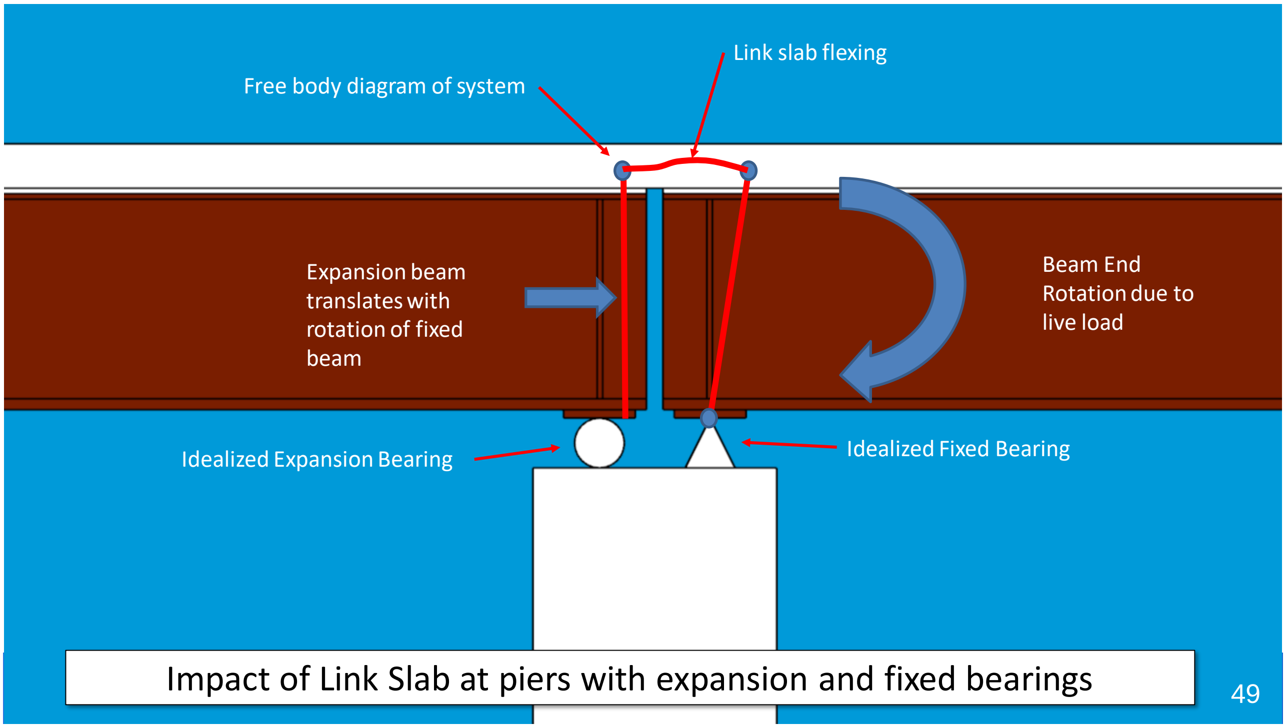


Figure C3.6.9.2-2—Links Slab Free Body Diagram




Free body diagram of system
Link slab flexing
Expansion beam translates with rotation of fixed beam
Beam End Rotation due to live load
Idealized Expansion Bearing
Idealized Fixed Bearing
Impact of Link Slab at piers with expansion and fixed bearings
49

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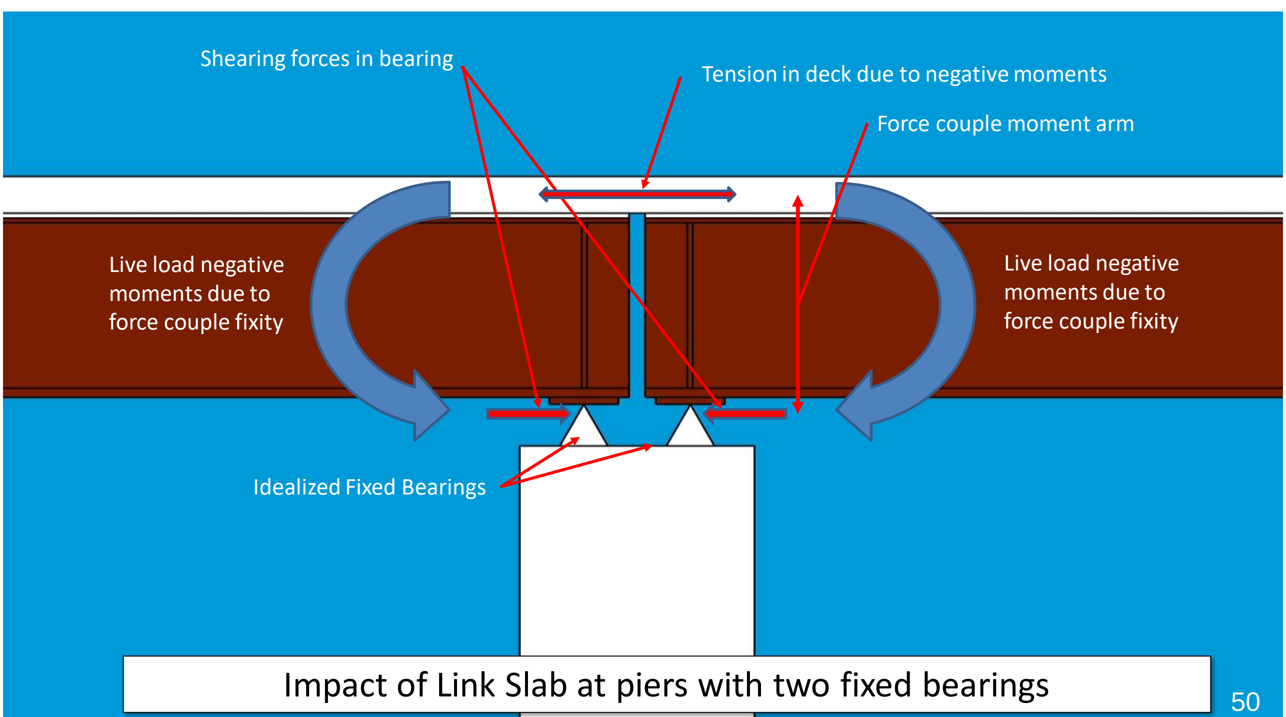
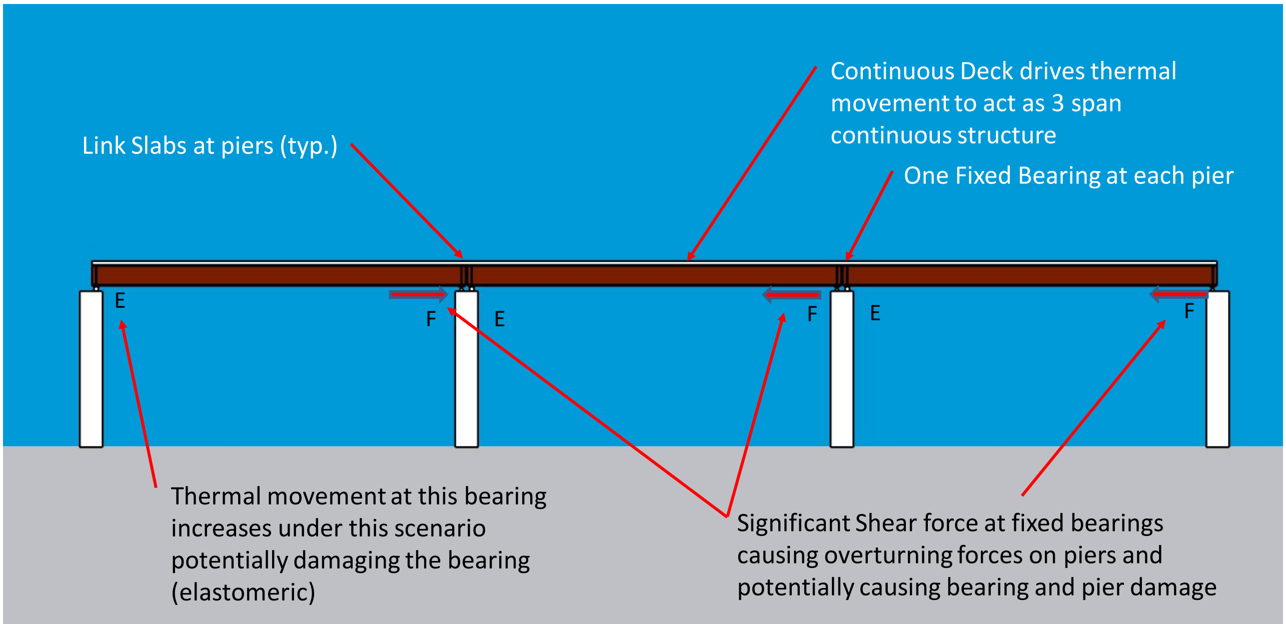

Shearing forces in bearing
Tension in deck due to negative moments
Force couple moment arm
Live load negative moments due to force couple fixity
Live load negative moments due to force couple fixity
Idealized Fixed Bearings
Impact of Link Slab at piers with two fixed bearings
50

50



Impact of Link Slab on Multiple Spans with fixed Bearings

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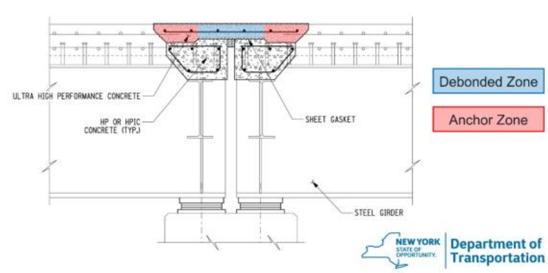
What about UHPC Link Slabs?

- UHPC link slabs have been developed and built since the publishing of the AASHTO LRFD Guide Specifications for ABC
- Another durable deck-level solution
- For more information, see FIU IBT/ABC webinar archive
  - Go to: https://abc-utc.fiu.edu/
  - Locate April 18, 2024 webinar recording



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5.5.3.1 DECK CONNECTIONS

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

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

C5.5.3.1

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

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

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

AASHTO LRFD Guide Specifications for ABC

5.5.3.2 BARE DECKS

Special care shall be exercised for the design and detailing of bare concrete decks. Only high quality materials shall be used for closure joints. Designers shall specify sealing of any potential shrinkage cracks in the decks and closure joints after installation.

Bare concrete decks should be ground to a profile after installation to improve ride quality and reduce wheel impact loading.

C5.5.3.2

Minor cracking in closure joints and deck panels can occur, even with good materials and workmanship. Liquid deck sealers can be used to eliminate the potential ingress of water into cracks.

An improved riding surface will result in less impact, which should improve the long-term performance of the deck.

5.5.3.3 OVERLAYS

Overlays should be used to provide a secondary protection system for prefabricated bridge decks. Overlays can be made with high performance concrete, asphalt, or thin polymer impregnated with aggregate. Asphalt overlays shall be combined with high quality waterproofing systems.

C5.5.3.3

Thin high performance concrete overlays, such as latex modified concretes, have been successfully used. Asphaltic overlays require the inclusion of a waterproofing system under the asphalt due to the fact that most asphalts are porous. Lack of a waterproofing system can lead to rapid deterioration of the deck concrete. Asphalt overlays with waterproofing systems have proven to provide excellent long-term performance.

AASHTO LRFD Guide Specifications for ABC

5.5.4—CORROSION PROTECTION OF POST-TENSIONING TENDONS AND ANCHORAGES

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

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

- concrete cover for ducts and anchorages equal to or greater than that used for element reinforcing steel
- corrosion protected ducts with quality duct splice devices
- high quality duct grout

C5.5.4

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

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

The basis for these recommendations is from the Post-Tensioning Institute (PTI) Committee M-50/ASBI Joint Task Group. This group has identified standardized Tendon Protection Levels (PL) for Post-Tensioning Systems and can be found in the PTI document entitled *Guide Specifications for Grouted Post-Tensioning* (2013).

AASHTO LRFD Guide Specifications for ABC

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

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

AASHTO LRFD Guide Specifications for ABC

10.4.4—HIGH EARLY STRENGTH CONCRETE

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

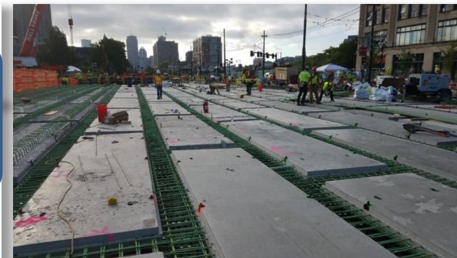
The concrete shall be designed to meet the following goals:

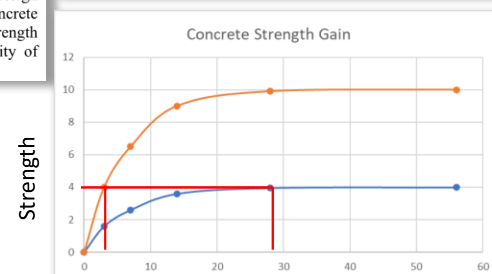
- Meet the required compressive strength (both interim and final) in an accelerated manner;
- Reduce the cure time for the concrete;
- Provide durable (low permeability) concrete; and
- Provide low shrinkage properties to reduce cracking in the field

C10.4.4

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

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





Reasons for cracking in joints

- Concrete closure Joints
  - Concrete in closure joints is highly restrained during curing
    - Autogenous shrinkage leads to tensile stress and cracking
    - To minimize cracking, the concrete tensile resistance gain needs to outpace the shrinkage tensile strength gain during curing

If: Internal shrinkage tensile stress $\geq$ Mod. Of Rupture = Crack

<u>This varies over time</u>

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

1. Portland cement shall be Type II, IIA or III conforming to AASHTO M 85 or M 240, as appropriate.
2. Portland cement shall conform to AASHTO M 85 with compatible admixtures and air entraining agent.
3. Water-cementitious material ratio shall not exceed 0.4 by weight, including water in the admixture solution and based on saturated surface dry condition of aggregates.
4. The amount of entrained air shall be 6.0 +/– 1.5 percent when tested according to AASHTO T 152.
5. The early strength characteristics of the concrete shall be commensurate with the intended construction procedure that is developed by the Contractor in the Assembly Plan.

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

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

The Designer may allow loads to be applied to high early strength concrete prior to the achievement of the specified final strength. See Article 8.5.4.

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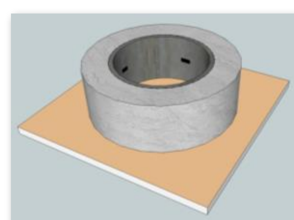
6. A shrinkage reducing admixture shall be tested by an approved testing lab and meet the requirements of ASTM C494 Type S, except that in Table 1 length change shall be measured as: Length Change (percent of control) shall be a minimum of 35 percent less than that of the control. Table 1 Length Change (increase over control) shall not apply. Shrinkage reducing admixtures shall not contain expansive metallic materials.
7. It is recommended that the mix be tested for restrained shrinkage according to AASHTO T 334. The test is considered successful if there is no cracking in the sample at 14 days maturity.
8. The maximum allowable total chloride content in concrete shall not exceed 0.1 percent by weight of cement.
9. The concrete mix design shall have a maximum rapid chloride ion permeability of 1,500 coulombs at not more than 56 days when tested according to AASHTO T 277. Multiple samples shall be tested using the intended curing methods in order to establish the required cure times for the mix.

The cited restrained shrinkage test is a test that can be used to compare one mix against another during mix development. The cited recommended age to cracking is approximate based on typical test results of successful High Early Strength concretes. Cracking of a sample before 14 days does not necessarily mean that the mix is inadequate. Likewise, cracking of a sample after 14 days does not guarantee that the material will not crack in the field. The field test specified in Article 10.4.4.2 is a more accurate test of intended field performance. The AASHTO T 334 test can be used to compare the High Early Strength mix to normal deck concrete. If the two mixes perform in a similar manner, the Owner and Contractor may elect to move forward with the field test noted in Article 5.4.4.2.



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10.4.4.1—Mix Design

The Contractor shall design and submit for approval the proportions and test results for a concrete mix which shall attain the minimum final design compressive strength and the early compressive strength as defined by the Assembly Plan.

Should a change in sources of material be made, a new mix design shall be established and approved prior to incorporating the new material. When unsatisfactory results or other conditions make it necessary, the Owner will require a new mix design.

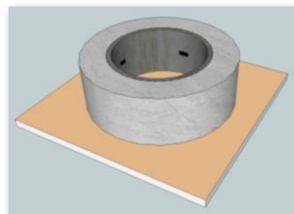
All tests necessary to demonstrate the adequacy of the concrete mix shall be performed by the Contractor, including, but not limited to: strength, slump, air content, temperature, initial set, and final set (AASHTO T 197). Additionally, a confined shrinkage test should be performed in accordance with AASHTO T 334.

Compressive strength tests shall be determined on field cured cylinders (6×12-inch cylinders) at intervals that are commensurate with the intended construction schedule.

The Contractor may make use of specialized concrete strength gain measurement equipment such as maturity testing in order to gain a better understanding of the concrete strength gain characteristics. The development of this system shall be made part of the mix design development process.

C10.4.4.1

Owners may elect to develop prescriptive specifications for high early strength concrete. In this case, these specifications would be superseded.



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10.4.4.2—Field Trial Placement

A trial placement should be undertaken before the intended date of the initial closure pour placement. Sufficient time shall be allotted to allow for modifications of the mix and re-testing if the trial placement does not meet the specified requirements. The Contractor shall demonstrate proper mix design, batching, placement, finishing and curing of the high early strength concrete. The trial placement shall simulate the actual job conditions in all respects including plant conditions, transit equipment, travel conditions, admixtures, forming, use of bonding compounds, restraint of adjacent concrete, placement equipment, and personnel.

The trial placement shall be a full size mock-up of the actual conditions that are to be encountered in the completed structure. Restraint of the closure joint by

C10.4.4.2

The purpose of the trial placement is to demonstrate the effectiveness of the high early strength concrete in the actual structure. A timeframe of 90 days before the actual placement is recommended to allow sufficient time to adjust the mix and re-test. Restrained shrinkage is a common cause of cracking in closure joints. The trial placement is intended to establish the potential for cracking in the joint. The goal is to have minimal cracking with acceptable crack widths. The restrained shrinkage testing may be waived by the Owner for closure joints that are not exposed to the environment in the final structure (i.e., footing closure joints).

An example of a trial placement test is shown in Figure C10.4.4.2-1. This trial pour was for a closure joint between full-depth precast concrete panels. The reinforcement and joint configuration depicted represents the same conditions in the completed

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adjacent concrete or steel elements shall be included in the mock-up.

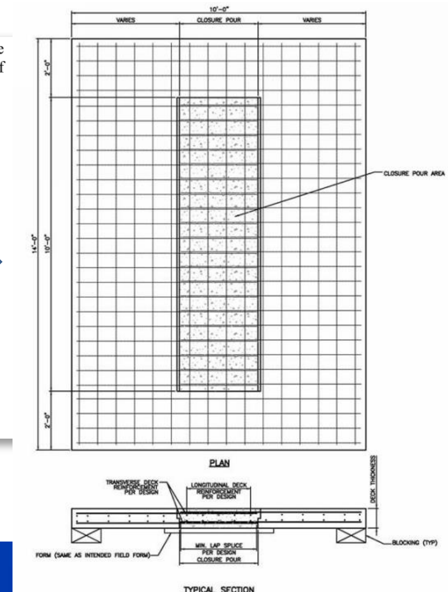
Acceptance requirements for the trial placement shall be as follows:

- The trial placement concrete shall not exhibit cracking or separation from the adjacent element in excess of 0.010 inches wide.
- There shall be no more than one visible transverse crack per 10 feet of joint length. A visible crack is defined as a crack between 0.010 and 0.016 inches wide.
- The evaluation of the trial placement shall take place 14 days after placement.

If the trial placement fails these requirements, the Contractor will be required to submit a corrective action plan on how to prevent the cracks and/or how to repair the cracks. If significant cracking occurs, the Owner may require the Contractor to conduct more trial batches and trial placements using corrective measures.

structure. Other types of closure joints may require different details that more closely match the details of the actual joint.

Example of a field trial placement



Conclusions and Findings

- Full depth precast concrete deck panels
  - Longitudinal PT connections:
    - Excellent performance
  - CIP closure joints made with high early strength concrete
    - Excellent performance
    - Mix design is critical (low shrinkage)
- Deck expansion joints are problematic
  - Minimize and maintain joints
- Link slabs
  - They work
  - Excellent performance
  - Extremely durable

CHA

Conclusions and Findings (continued)

- Overlays can be a viable means of providing a durable deck
  - High quality membrane is critical
  - The performance is excellent
  - This has been the European model for decades
- 2018 AASHTO LRFD Guide Specifications for ABC
  - Contain design provisions for everything covered today
  - Contain construction provisions that can produce a durable bridge deck
- It is reasonable to expect that ABC decks can provide a 75-year service life.