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1. INTRODUCTION

The design of full-depth precast concrete (FDPC) deck panels involves the design of several different components: (1) FDPC deck panel, (2) transverse joint, (3) longitudinal joint, and (4) shear pocket. The major components of the FDPC deck panel deck are highlighted Figure 1.1.

![Figure 1.1: Components of FDPC deck panel deck system](image)

The design and detailing of FDPC deck panels and the connection between panels will be discussed in this guide. Additional details on construction of these systems can be found in the PCI State-of-the-Art Report on Full-Depth Precast Concrete Bridge Deck Panels [1].

2. OVERVIEW OF DESIGN OF FDPC DECK PANEL DECKS

In design of FDPC deck panels system, designers can choose among a variety of choices. Panels can be designed with pre-tensioning or conventional reinforced concrete design. They can be designed for composite action with girder by providing shear stud connections or be designed as a non-composite member. The designer can also consider post-tensioning for minimizing cracks in joints along with several types of panel to panel joints. Additionally, designers can use a combination of FDPC deck panels and CIP details whenever it is needed based on geometry and project requirements.

The following parameters must be determined during the design process of FDPC deck panel systems:

- Panel dimension and configuration
- Shear pocket configuration for achieving full composite action
- Panel reinforcement detail (including prestressing design if used)
- Concrete mix design for precast panel
- Joint geometry and connection type (including post-tension details if used)
- Filling material for joints and shear pockets
- Type of overlay materials
- Parapets and connection detail to panels
• Handling and transportation

Successful construction and long-term performance of FDPC deck panel systems is dependent on successful design and detailing of each of these parameters.

3. PANEL DESIGN

3.1. DIMENSIONS AND CONFIGURATION

There are not any standard dimension for FDPC deck panels; the maximum dimensions are typically limited by shipping and handling requirements [1]. The shorter panel dimension is typically between 8 and 12 feet, as shown in Figure 3.1. The short panel dimension is primarily controlled by the maximum width allowed for shipment of the panels from the precaster to the construction site. The long panel dimension is typically less than 40 to 50 feet and equal to the full bridge width when possible [1], [2]. The long panel dimension is primarily controlled by the tensile stress that develops during lifting and placement of the panels. The thickness of the panels is generally governed by the minimum thickness requirements and minimum cover requirements. These requirements typically result in a minimum deck thickness of 7 inches for typical decks and 8.5 inches for post-tensioned decks [1].

![Figure 3.1: Typical maximum dimensions of FDPC deck panels](image-url)

There are two typical panel configurations that are primarily based on the width of the bridge, shown in Figure 3.2. The FDPC deck panel can be equal to the bridge width for bridge widths less than 40 to 50 feet, which eliminates the need for a longitudinal joint, shown in Figure 3.2 (b). Multiple FDPC deck panels will be needed across the width for bridge widths greater than 40 to 50 feet, shown in Figure 3.2 (a). Both configurations run the precast panels transverse to the girder lines.
Figure 3.2: Conventional deck panel configurations for (a) bridge widths greater than 40 to 50 feet and (b) bridge widths less than 40 to 50 feet

Alternatively, panels can be run in the longitudinal direction, the same direction of the girders, shown in Figure 3.3. Running panels in the longitudinal direction can help to eliminate the need for shear pockets in most of the panels, although some type of shear pocket will be required for the overhang panels. Because of the elimination of the shear pockets, the designer has flexibility with placement of shear studs, so this detail can be used for situations where a large number of shear studs are required. A similar panel configuration was used on Boston’s Commonwealth Avenue Bridge [3].

Figure 3.3: Alternate panel configuration, based on [3]

Skew can be handled in FDPC deck panels either by creating skewed panels or using square panels to create a skewed configuration, as shown in Figure 3.4. Skewed panels can be utilized in any of the three configurations described above for light skewed bridges (where reinforcement would be run in the direction of the skew). Creating a skew with square deck panels would primarily be an option when using the longitudinally configured deck panels for larger skewed bridges (where reinforcement would be run perpendicular to the girder lines).
Figure 3.4: Two ways to handle skew in FDPC deck panels: (a) skewed panels and reinforcement for light skews and (b) offset rectangular panels with reinforcement perpendicular to girder lines for larger skews

Utah DOT allows for up to 15-degree skew with skewed panels and up to 45-degree skew with square panels [1].

3.2. **Precast Panel Reinforcement Detail**

Design of deck panels includes transverse design (perpendicular to traffic flow) and longitudinal design (parallel to traffic flow). There are not any specific design provision for FDPC deck panels in the AASHTO LRFD Bridge Design Specification [4]. The LRFD Guide Specification for Accelerated Bridge Construction [5] specifies that FDPC deck panels themselves should be designed using the provisions for CIP concrete decks as specified in the AASHTO LRFD Bridge Design Specifications.

The procedure which normally is used for design purposes is the strip design method, which considers a small transverse strip of the deck as a continuous beam supported on the girders, as shown in Figure 3.5. Design can be done by using non-prestressed, prestressed or combination of them for transverse direction [1].

Figure 3.5: Basics of strip design method for designing transverse reinforcement
Reinforcement running in the direction of the girders should be detailed to control shrinkage cracking and distribute live load. Both transverse and longitudinal design should satisfy all requirements on AASHTO LRFD specification. Using smaller bar sizes at a closer spacing is typically preferred to using larger bars at larger spacing. Closer spaced reinforcement helps to control cracking. Most states have a cap for maximum bar size in their design approach, which is typically #6 rebar [1].

Panel reinforcement must be detailed for lifting and placement of the panels. Different lifting procedures will impact the moments and stresses generated during lifting, shown in Figure 3.6. In many cases additional reinforcement will need to be provided to resist stressed during lifting.

![Possible lifting points for precast panels](image)

**Figure 3.6:** Possible lifting points for precast panels (a) with spreader beam and (b) without spreader beam

A sample reinforcement detail obtained through the survey from this project is shown in Figure 3.7. Reinforcement is distributed around the shear pockets. Other sample reinforcement details can be found in the PCI State-of-the-Art Report on FDPC Bridge Deck Panels [1] and the ABC Project Database [6].

![Standard FDPC deck panel reinforcement detail](image)

**Figure 3.7:** Standard FDPC deck panel reinforcement detail (obtained through DOT survey from NHDOT)
Note that the empirical design method is not allowed for FDPC deck panels [5].

3.3. **Overhang and Barrier Design**

The provisions for deck overhangs for CIP decks in the AASHTO LRFD Bridge Design Specification [4] can be used for FDPC deck panels. Care should be taken to properly check the stresses in these overhangs and the development of any prestressing strands or reinforcement.

Design of overhangs is often controlled by the forces due to a vehicle impact on the barrier, which will generate large stresses in the deck at the base of the barrier [5], shown in Figure 3.8. The overhang design is based on the assumption that the barrier should fail before the deck overhang fails, so the deck needs to be able to hold the force transferred during impact. Because large stresses can develop near the base of the barrier, the development length of prestressing strands and reinforcement will often control the strength.

![Figure 3.8: Barrier impact often controls design of overhang](image)

An additional challenge in overhang design can result from the presence of shear pockets over the exterior girders. The weight of barriers on the external edge of an overhang can lead to additional reinforcement required over the exterior girder and may lead to congestion around the shear pockets in the panels [2]. A sample detail for barrier and overhang is shown in

![Figure 3.9: Example detail of barrier and overhang (obtained through DOT survey from PennDOT)](image)
Barriers can be cast-in-place after placement of the deck panels, precast separate from the panels and connected to them during construction, or integrally cast with the deck panels (for bridges with a single panel used for the bridge width). An example of a barrier cast integrally with the precast deck panel is shown in Figure 3.10 for a deck panel project in Utah. The added weight of the barriers can increase the demand during lifting and handling, but integrally casting the barriers can further reduce the construction time [2].

A recent project was completed by Iowa State University supported by the Accelerated Bridge Construction University Transportation Center (ABC-UTC) [7]. A detail was developed through static testing for the connection between a deck and a precast barrier, shown in Figure 3.11. Further testing on this connection through impact loading is being planned for the near future through a pooled-fund study.

Figure 3.10: Precast Concrete Deck Panel with Integral Barrier [2]

Figure 3.11: (a) Recommended detail for connection with precast barriers and (b) inclined blockout required in deck element [7]
3.4. **Shear Pockets and Horizontal Shear Connectors**

Shear pockets and shear studs create composite action between the supporting girders and the deck by preventing any horizontal and vertical movement. Design of the shear studs and composite design should be done similar to conventional concrete decks [5]. Based on LRFD specification, the maximum distance of studs should be less than 2 feet for steel girders and 4 feet for concrete girders for non-welded studs to provide complete composite action between the concrete panel and girder [1], [2], [4]. Typically, shear pockets spaced at 2 feet on center, as shown in Figure 3.7, with groups of three shear studs will be sufficient to create a composite connection between the girder and precast deck.

The typical shear pocket geometry and details are shown in Figure 3.12. Welded stud connectors are used for steel girders. Projecting reinforcement from the top of concrete girders or using welded stud connectors attached to an embedded steel plate are two common transfer methods for concrete girders [5].

![Figure 3.12: Examples of shear pocket and connector details for (a) steel plate girders and (b) prestressed concrete girders, based on [1]](image)

A sample shear pocket and shear connector detail obtained from the DOT survey is shown in Figure 3.13.
Either partial depth or full depth pockets can be used. Previous researchers [8], [9] have found that partial depth pockets can successfully transfer the interface shear between the girder and precast deck. A sample partial depth pocket detail is shown in Figure 3.14. This detail utilizes ultra-high performance concrete (UHPC) to provide a full-composite connection between the deck and girder although the shear studs do not extend past the reinforcement in the precast panels.

Shear connectors can also be placed in the longitudinal joints between panels for panel orientations where multiple panels are used across the bridge width. A sample detail of the shear connector extending from a precast beam into the longitudinal joint between two precast panels obtained from the DOT survey is shown in Figure 3.15.
3.5. **Panel Leveling System**

A panel leveling system is required during panel installation to ensure that panels are properly placed and can evenly transfer panel dead loads into the girders prior to joint and shear pocket casting. One common system is to use the bolt and a pipe sleeve which is cast into the panel, shown in Figure 3.15. Several other states have used a similar detail, shown Figure 3.16. Note that it is recommended to consider a minimum 1-inch haunch between panels and girders for tolerances, which is usually provided by using forming at the bottom of the panel in a way that the grout can easily move from one shear pocket to another to be sure that the haunch is fully grouted.

![Diagrams of panel leveling systems](image)

**Figure 3.15:** Example detail of shear connector extending into longitudinal joint between two precast panels (obtained through DOT survey from PennDOT)

**Figure 3.16:** Leveling screw detail from (a) Connecticut River Bridge and (b) PCI New England Recommendation [1]

Different details for forming the haunch have been used. One practical and cost-effective detail utilizes steel angles along the edge of the girders is shown in Figure 3.17.
3.6. **Concrete Mixture for FDPC Deck Panels**

Typical high performance concrete mixtures used for precast concrete construction can be used for casting FDPC deck panels. Adequate concrete strength for service and strength checks can usually be achieved within the first day or two. It is recommended to wait at least 28 days after panel casting before placement of the panels [1]. Waiting this extra time will help to ensure that shrinkage and creep deformations occur prior to placement of the panels.

4. **Joint Design**

As previously discussed, there are two primary types of joints in FDPC deck panel systems: transverse and longitudinal, shown in Figure 3.2. Longitudinal joints are only required in bridges where multiple panels are required across the bridge width. All joints should be designed and detailed as full moment connections [5].

The most commonly used joint details for bridges contained in the FDPC Deck Panel Database are summarized in Table 4.1 with the percentage of bridges having the detail shown. The longitudinal post-tensioning detail was only used for transverse joints. The grouted shear key without post-tensioning detail was primarily used for off-system bridges in Alaska, although there is one bridge in New York and one in Washington with this detail.

**Table 4.1: Joint detail usage for bridges in FDPC Deck Panel Database**

<table>
<thead>
<tr>
<th>Joint Type</th>
<th>Percent of Bridges in FDPC Deck Panel Database with Joint Type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Transverse Joint</td>
</tr>
<tr>
<td>Longitudinal post-tensioning</td>
<td>34.2%</td>
</tr>
<tr>
<td>Conventional concrete with hooped or straight bar detail</td>
<td>11.4%</td>
</tr>
<tr>
<td>UHPC with straight bar detail</td>
<td>15.8%</td>
</tr>
<tr>
<td>Grouted shear key without post-tensioning</td>
<td>24.1%</td>
</tr>
</tbody>
</table>

*39.5% of bridges did not have a longitudinal joint*
Many bridges utilized deck panels that were the full width of the bridge (around 40 percent), so no longitudinal joint was required. When multiple panels were required across the width of the superstructure, the most common longitudinal joint details with good long-term performance are:

- Conventional concrete with hooped or straight bar details and
- UHPC with straight bar detail.

Schematics of these basic details are shown in Figure 4.1. The convention concrete with straight bar detail is similar to the hooped bar detail only with a slightly wider joint region to provide the additional required splice length.

![Figure 4.1: Most popular joint details with good long-term performance](image)

There are several configurations of these joints that have been observed through the state survey and development of the FDPC Deck Panel Database. The three most common combinations of transverse and longitudinal joints used in FDPC deck panel systems are shown in Table 4.2.

<table>
<thead>
<tr>
<th>#</th>
<th>Transverse Joint</th>
<th>Longitudinal Joint</th>
<th>Percent of Bridges*</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>UHPC with straight bar detail</td>
<td>UHPC with straight bar detail</td>
<td>25.3%</td>
</tr>
<tr>
<td>2</td>
<td>Longitudinal post-tensioning</td>
<td>Conventional concrete with hooped or straight bar detail</td>
<td>24.2%</td>
</tr>
<tr>
<td>3</td>
<td>Conventional concrete with hooped or straight bar detail</td>
<td>Conventional concrete with hooped or straight bar detail</td>
<td>13.7%</td>
</tr>
</tbody>
</table>

*Percent of bridges with a longitudinal joint; bridges without a longitudinal joint were not included.

The two most common combinations of transverse and longitudinal joints are shown in Figure 4.2.
The four most common joint details will be discussed in more depth in the following sections.

4.1. **LONGITUDINAL POST TENSIONING WITH GROUTED SHEAR KEY**

The longitudinal post tensioning with grouted shear key detail has been the most commonly used transverse joint detail. While post-tensioning can increase the construction cost of the bridge deck, it is an effective method for improving the durability of the system. Post-tensioning is generally placed at the mid-depth of panels and runs along the entire length of the bridge. Longitudinal post tensioning for FDPC deck panel systems is typically done using high strength threaded rods, monstrands, or flat multi-strand tendons. For simple spans, sufficient post-tensioning should be provided to provide a minimum prestress level of 0.250 ksi after all prestress losses [4]. Waiting at least 28 days after panel casting for placement and tensioning of the panels will help to decrease the shrinkage and creep losses. Additional details on post-tensioning design and details for FDPC deck panels can be found in PCI’s “State-of-the-Art Report on Full-Depth Precast Concrete Bridge Deck Panels” [1].

The joint geometry for longitudinal post-tensioned systems can either be a female-to-female joint with a small grouted section between panels or a male-to-female match-cast joint with epoxy or grout between panels, shown in Figure 4.3.
Figure 4.3: Typical longitudinal PT joints: (a) female-to-female and (b) male-to-female match cast

A blockout is created at the location of the longitudinal post-tensioning ducts to house the splice sleeve connector, shown in Figure 4.4. Grout tubes or vents should be installed to ensure proper grouting of the duct.

Figure 4.4: Typical blockout detail for splice sleeve connector between PT ducts in adjacent panels

A sample detail for the shear key and blockout detail are shown in Figure 4.5

Figure 4.5: Example detail of (a) longitudinal PT keyway and (b) blockout for PT duct coupler (obtained through DOT survey from UDOT and WYDOT, respectively)
4.2. **CONVENTIONAL CONCRETE WITH HOOPED OR STRAIGHT BARS**

The conventional concrete joint detail with hooped or straight bars has been used for both transverse and longitudinal joints. This joint requires a larger width closure pour, so it makes the most sense to be used in the longitudinal direction, where the joint between panels runs along the girder line. The top of the girder can be used as the bottom form for the joint and the larger joint width gives more flexibility with the placement of shear studs and joint reinforcement, as shown in Figure 4.6.

![Figure 4.6: Schematic of conventional concrete longitudinal joint over girder](image)

The splice length of the straight bars must satisfy the development length, splice length, and non-contact splice dimension requirements of the AASHTO LRFD Bridge Design Specification [4], [5]. The splice length of hooked bars is only required to be equal to the development length of the hooked bar, $l_{hb}$, as specified by AASHTO LRFD [5]; previous research [10]–[12] has shown this length to be sufficient. As shown in Figure 4.6, at least one transverse bar of equal size should be set within the inside radius and in contact with each hook [5], which will result in at least two bars contained within the hooked splice. Typical details include four bars included in a hooped bar splice. The joint reinforcement can be staggered for hooked and hooped bars to improve constructability. Hooked and hooped bars can be staggered such that the distance between spliced bars does not exceed 4 inches [5], as shown in Figure 4.7.
A sample detail for the conventional concrete with hooped bar detail obtained through the DOT survey is shown in Figure 4.8.

4.3. **UHPC with Straight Bar**

The UHPC with straight bar connection has been used in both longitudinal and transverse joints between FDPC deck panels. Several research projects have been conducted on UHPC connections with straight bars [13], [14] and guidance on the connection is provided by Graybeal [9] and AASHTO [5]. A summary of the basic reinforcement requirements for this splice connection are shown in Figure 4.9. There are modifications for higher strength reinforcement and less cover provided by both resources. Additionally, Graybeal [9] allows for the same $8d_b$ required development length for #5 bars with minimum cover greater than or equal to 1.25 inches; this special provision is due to additional research on this specific bar size. The strength of UHPC is typically required to be greater than 14 ksi before the application of construction or live loads [5].
Figure 4.9: Summary of basic recommendations for UHPC connections between panels, based on [5], [9].

A sample detail for the UHPC with straight bar detail obtained through the DOT survey is shown in Figure 4.10.

For:
- $f_y \leq 75$ ksi
- Bar size $\leq \#8$
- $f'_c \geq 14$ ksi
- Fiber content $\geq 2\%$

$\text{embedment length} \geq l_{dhu} = 8d_b$

$\text{splice length}, l_s \geq 0.75l_{dhu} = 6d_b$

$\text{min. cover} \geq 3d_b$

Figure 4.10: Example detail of UHPC with straight bar detail between panels (obtained through DOT survey from NYDOT)

4.4. **Grouted Shear Key without Post-Tensioning**

The grouted shear key without post-tensioning detail has been primarily used for the transverse joint in off-system bridges in Alaska. The majority of these bridges with this joint detail (about 90 percent) have an ADT less than 500, although there are two bridges with ADT of 12,000 and 20,000 that have performed well since the earliest date inspection records were obtained (2004). A sample detail for the grouted shear key without post-tensioning detail obtained through the DOT survey is shown in Figure 4.11.
4.5. **Surface Preparation for Joints**

An exposed aggregate finish is recommended for the interface surface of connections requiring resistance against water infiltration [5], which is generally all joints between FDPC deck panels. For joints that will be post-tensioned, the joint surface of the precast element can simply be cleaned of all laitance and other deleterious materials prior to grout casting [5]. The joint face of the FDPC deck panel should be prepared to a saturated surface dry (SSD) condition prior to casting [5], where the surface is dry but the voids in the concrete matrix are saturated with water.

An exposed aggregate finish with a ¼” surface roughness is recommended to achieve adequate bond between the precast concrete of the panel and the UHPC in the joint [9]. This can be achieved by doing the following:

1. Paint a paste retarder on the formwork of the joint of the precast element within 24 hours before casting. Paste retarders are generally moisture activated and are less effective if applied earlier than this.
2. Strip the joint formwork within 24 hours after casting and use a pressure washer to wash away the soft paste. Adjust the pressure and distance so that the paste is removed without damaging the aggregate.
3. Before casting the UHPC, prewet the joint surface for several hours to achieve an SSD condition.

Sandblasting will generally not be adequate for getting a ¼” surface roughness, although it may be used for lower surface roughnesses. Mockups can be requested prior to casting of the precast panels to verify that the precaster’s procedure will produce a sufficiently rough finish in the joints.

5. **Materials for Joints, Shear Pockets, and Post-Tensioning Ducts**

5.1. **Grouts**

5.1.1. **Grout for haunches, pockets, voids, and joints**

A low shrinkage, durable material with high early strength and high freeze/thaw resistance is desirable for filling all voids, haunches, and pockets in FDPC deck panel systems [1]. Non-shrink cementitious grouts are the most common grout used for these applications as they are economical.
and have the desired properties listed above. The recommended grout properties suggested by Nottingham [15] and presented in [1] are shown in Table 5.1.

<table>
<thead>
<tr>
<th>Property</th>
<th>Typical Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive Strength</td>
<td>1.2 ksi @ 6 hours</td>
</tr>
<tr>
<td></td>
<td>4.5 ksi @ 1 day</td>
</tr>
<tr>
<td></td>
<td>6.5 ksi @ 28 days</td>
</tr>
<tr>
<td>Flexural Strength</td>
<td>0.55 ksi @ 1 day</td>
</tr>
<tr>
<td></td>
<td>0.60 ksi @ 28 days</td>
</tr>
<tr>
<td>Slant Shear Bond</td>
<td>2.5 ksi @ 28 days</td>
</tr>
<tr>
<td>Freeze-Thaw Resistance</td>
<td>RDF of 80%</td>
</tr>
<tr>
<td>Scaling Resistance</td>
<td>0 scaling rating</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>0.03% @ 28 days</td>
</tr>
<tr>
<td>Sulfate Resistance</td>
<td>0.10% @ 28 days</td>
</tr>
</tbody>
</table>

Epoxy grouts are also common, but normally have a lower modulus of elasticity and are relatively more expensive than cementitious grouts. Expansive base grouts are also available, but these lead to excessive expansion and bleeding in most cases [16]. Other grout products can be used, but they should satisfy ASTM C1107 (Standard Specification for Packaged Dry, Hydraulic-Cement Non-Shrink Grout) [17].

Grout can be used for smaller joints (e.g., typical PT joint) but becomes too expensive an option for larger joints. Conventional concrete, high performance concrete, or ultra-high performance concrete are typically used for larger joints.

5.1.2. Grout for post-tensioning ducts

Cementitious grouts are typically used to fill post-tensioning ducts as they are chemically basic and provide a passive environment around the prestressing strands [18]. Details on grouts for post-tensioning ducts can be found in the FHWA manual “Post-Tensioning Tendon Installation and Grouting Manual” [18] and PTI Publication “Specification for Grouting of Post-Tensioned Structures (PTI M55.1-12)” [19].

These cementitious grouts are composed of ordinary Portland cement (Type I or II), supplementary cementitious materials (fly ash, slag cement, or silica fume), chemical admixtures, fine aggregates, and water. SCMs are used to improve the corrosion resistance by creating a less permeable and denser packed concrete matrix. Chemical admixtures are used to improve the workability (high range water reducers), control the set times, entrain air, prevent excessive bleeding, and inhibit corrosion. Fine aggregates are an inert material used as a filler. Note that the water-to-cement ratio should never exceed 0.45. High-range water reducers can be used to improve workability without adding additional water. Pre-bagged grouts are available and commonly used. Pre-bagged grouts should be stored in dry locations and used within a reasonable amount of time.
Complete grouting of the ducts with high quality grout is required to ensure protection of prestressing strands. Voids in the grout or soft or segregated grout materials can lead to accelerated corrosion of the strands [20]. Deficiencies can occur when pre-bagged grout is improperly stored, excessive water is used, or improperly mixed or placed.

5.2. **Conventional Concrete**

Conventional concrete and high-performance concrete mixtures can be used for joints. Using these materials will typically result in larger width joint regions, due to longer required development and splice lengths. Shrinkage reducing admixtures and proper curing of the joints can be used to reduce the likelihood of shrinkage cracking in larger volume closure pours.

5.3. **Ultra-High Performance Concrete (UHPC)**

Ultra-high performance concrete (UHPC) is a cementitious composite material with high compressive and tensile strengths and low permeability. The typical ranges for some of the most relevant mechanical properties are shown in Table 5.2.

<table>
<thead>
<tr>
<th>Property</th>
<th>Typical Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>14-day Compressive Strength</td>
<td>18 to 22 ksi</td>
</tr>
<tr>
<td>Direct Tensile Cracking Strength</td>
<td>0.8 to 1.2 ksi</td>
</tr>
<tr>
<td>Direct Tension Bond Test</td>
<td>0.35 to 0.6 ksi</td>
</tr>
<tr>
<td>Modulus of Elasticity</td>
<td>4,250 to 8,000 ksi</td>
</tr>
<tr>
<td>Long-term Drying Shrinkage</td>
<td>300 to 1,200 με</td>
</tr>
<tr>
<td>Long-term Autogenous Shrinkage</td>
<td>200 to 900 με</td>
</tr>
</tbody>
</table>

Proprietary UHPC mixtures are available from several different vendors. Proprietary UHPC mixtures typically come in three separate components: a pre-bagged cementitious powder, chemical admixtures, and steel fiber reinforcement. Non-proprietary UHPC mixtures have been developed by several states [21]. A research project is also starting by the ABC-UTC investigating non-proprietary UHPC mixtures. Typical UHPC materials contain 2-percent (by volume) steel fiber content [9].

5.4. **Polymer Concrete**

There are alternative materials that are being researched for use in closure joints between FDPC deck panels. Polymer concrete is being researched for use in closure pours in a parallel ABC-UTC project “More Choices For Connecting Prefabricated Bridge Elements and Systems (PBES)” being conducted by the University of Nevada - Reno [22]. This concrete has a high tensile strength (compared to its compressive strength) and good tensile adhesion, as shown in Table 5.3, which makes it a suitable material for joints.
Table 5.3: Mechanical properties of T-17 polymer concrete [22]

<table>
<thead>
<tr>
<th>Property</th>
<th>Typical Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive Strength</td>
<td>8 to 9 ksi</td>
</tr>
<tr>
<td>Flexural Strength</td>
<td>1.8 to 2.5 ksi</td>
</tr>
<tr>
<td>Linear Shrinkage</td>
<td>&lt; 0.2%</td>
</tr>
<tr>
<td>Tensile Strength</td>
<td>1 to 1.2 ksi</td>
</tr>
<tr>
<td>Compressive Modulus</td>
<td>1,100 to 1,200 ksi</td>
</tr>
<tr>
<td>Tensile Adhesion</td>
<td>&gt; 0.25 ksi</td>
</tr>
</tbody>
</table>

6. WEARING SURFACE AND OVERLAYS

Overlays are not required on FDPC deck panel systems, but are generally used as they can improve the long-term performance of the deck system and create a smooth riding surface. The typical wearing surfaces and overlays that are used for FDPC deck panel systems are shown in Figure 6.1. These wearing surface and overlays are discussed in depth in PCI’s “State-of-the-Art Report on Full-Depth Precast Concrete Bridge Deck Panels” [1].

Figure 6.1: Primary types of wearing surfaces and overlays: (a) bonded concrete overlay, (b) epoxy overlay, (c) waterproof membrane with asphalt, (d) monolithic concrete overlay, and (e) low permeability panel with no overlay, based on [1]
The frequency of use for different wearing surface and overlay types for the bridges in the FDPC Deck Panel Database are summarized in Table 6.1.

*Table 6.1: Overlay usage for bridges in FDPC Deck Panel Database*

<table>
<thead>
<tr>
<th>Type of Overlay (from NBI)</th>
<th>Percentage of Bridges</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bituminous (Asphalt)</td>
<td>47.0%</td>
</tr>
<tr>
<td>Monolithic Concrete*</td>
<td>19.5%</td>
</tr>
<tr>
<td>Epoxy Overlay</td>
<td>10.9%</td>
</tr>
<tr>
<td>Bonded Concrete Overlay (Conventional)</td>
<td>7.1%</td>
</tr>
<tr>
<td>Bonded Concrete Overlay (Latex)</td>
<td>6.4%</td>
</tr>
<tr>
<td>No Overlay</td>
<td>3.4%</td>
</tr>
<tr>
<td>Other Overlay</td>
<td>5.6%</td>
</tr>
</tbody>
</table>

*majority of monolithic concrete bridges are used with the grouted shear key without post-tensioning in Alaska*
7. AVAILABLE RESOURCES

There are many available resources with additional information on the design and fabrication of FDPC deck panels. Several of these resources are listed in this section.

7.1. GENERAL RESOURCES


7.2. UHPC MATERIALS AND JOINT DESIGN


7.3. POST-TENSIONING


7.4. PAST PERFORMANCE

- D. Garber and E. Shahrokhisnasab, “Performance Comparison of In-Service, Full-Depth Precast Concrete Deck Panels to Cast-in-Place Decks,” Accelerated Bridge Construction University Transportation Center (ABC-UTC), ABC-UTC-2013-C3-FIU05-Final, Mar. 2019.
8. PAST PERFORMANCE

A database was developed for bridges with FDPC deck panels through a DOT survey. Inspection information and details from the NBI were gathered for each bridge in the database from the LTBP InfoBridge [23] resource and combined with information gathered from the DOT survey. Comparison projects with cast-in-place (CIP) concrete decks were selected for most of the bridges in the database. More details about the database and comparison projects can be found in Garber and Shahrokhinasab [24].

The performance of the bridges in the database were determined based on the deck condition rating obtained from the NBI. The deck condition ratings were used to determine a deterioration rate and an estimated service life. A deck rating of 4 was used as the threshold for deck repairs to determine the estimated service life. A summary of the overall performance of bridges with FDPC deck panels compared with CIP decks is shown in Table 8.1. The average performance of bridges with FDPC deck panel decks is similar to the performance of bridge with CIP based on this analysis.

<table>
<thead>
<tr>
<th>Deck Type</th>
<th>FDPC</th>
<th>CIP</th>
</tr>
</thead>
<tbody>
<tr>
<td>$n_{bridges}$</td>
<td>206</td>
<td>178</td>
</tr>
<tr>
<td>Avg. $n_{inspections}$ per bridge</td>
<td>12.6</td>
<td>13.0</td>
</tr>
<tr>
<td>Avg. Year of 1st Inspection</td>
<td>2004</td>
<td>2005</td>
</tr>
<tr>
<td>Deterioration Rate (per year)</td>
<td>-0.12</td>
<td>-0.09</td>
</tr>
<tr>
<td>Estimated Service Life (year)</td>
<td>33</td>
<td>35</td>
</tr>
</tbody>
</table>

There are limitations to this approach as this analysis was based on just the deck condition rating from the NBI. The database produced from this project and the proposed CIP deck comparison projects could be used as a starting point for a more in-depth evaluation of the bridges using non-destructive evaluation techniques.

Despite the limitations of this approach, these results may suggest that there is room for improvement in the design and construction of bridges with FDPC deck panels. The precast panels themselves offer superior durability to CIP decks (due to better concrete materials and higher quality of construction), so more care should be taken in designing, detailing, and constructing the joints.
9. LIST OF SUCCESSFUL PROJECTS IN ABC PROJECT DATABASE

There are currently 17 completed projects that have utilized FDPC deck panels with and without longitudinal post tensioning in the ABC Project Database [6], shown in Table 9.1. Some of the details available for these projects includes lessons learned, contract plans, specifications, bid tabs, and construction schedules.

Table 9.1: List of projects from ABC Project Database [6]

<table>
<thead>
<tr>
<th>Bridge</th>
<th>State</th>
<th>Year</th>
<th>Trans. Joint Type</th>
<th>Long. Joint Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>TH 53 Bridge over Paleface River</td>
<td>MN</td>
<td>2012</td>
<td>Longitudinal PT</td>
<td>CC with Hooped Bars</td>
</tr>
<tr>
<td>Lake Champlain Bridge</td>
<td>NY</td>
<td>2011</td>
<td>Longitudinal PT</td>
<td>CC with Hooped Bars</td>
</tr>
<tr>
<td>I-93 Bridge over Loudon Road (Route 9)</td>
<td>NH</td>
<td>2010</td>
<td>Longitudinal PT</td>
<td>None</td>
</tr>
<tr>
<td>I-70 Bridge over Eagle Canyon (Eastbound)</td>
<td>UT</td>
<td>2010</td>
<td>Longitudinal PT</td>
<td>None</td>
</tr>
<tr>
<td>24th Street Bridge over I-29/I-80</td>
<td>IA</td>
<td>2008</td>
<td>Longitudinal PT</td>
<td>CC with Hooped Bars</td>
</tr>
<tr>
<td>US 131 / Parkview Avenue Bridge</td>
<td>MI</td>
<td>2008</td>
<td>Longitudinal PT</td>
<td>CC with Hooped Bars</td>
</tr>
<tr>
<td>Riverdale Road Bridge over I-84</td>
<td>UT</td>
<td>2008</td>
<td>Longitudinal PT</td>
<td>CC with Hooped Bars</td>
</tr>
<tr>
<td>Mackey Bridge (Marsh Rainbow Arch Bridge)</td>
<td>IA</td>
<td>2006</td>
<td>Longitudinal PT</td>
<td>CC with Hooped Bars</td>
</tr>
<tr>
<td>Nemo Bridge</td>
<td>MO</td>
<td>2004</td>
<td>Longitudinal PT (match-cast)</td>
<td>None</td>
</tr>
<tr>
<td>Illinois Route 29 Bridge over Sugar Creek</td>
<td>IL</td>
<td>2001</td>
<td>Longitudinal PT</td>
<td>None</td>
</tr>
<tr>
<td>Dead Run and Turkey Run Bridges</td>
<td>VA</td>
<td>1998</td>
<td>Longitudinal PT</td>
<td>None</td>
</tr>
<tr>
<td>Burnt River &amp; UPRR Bridge</td>
<td>OR</td>
<td>2012</td>
<td>UHPC with Straight Bars</td>
<td>None</td>
</tr>
<tr>
<td>I-84 Bridge F-114</td>
<td>UT</td>
<td>2011</td>
<td>Grouted Dowel</td>
<td>None</td>
</tr>
<tr>
<td>SH 290 Bridge over Live Oak Creek</td>
<td>TX</td>
<td>2008</td>
<td>Grouted Dowel</td>
<td>None</td>
</tr>
<tr>
<td>Grayling Creek Bridge</td>
<td>AK</td>
<td>2006</td>
<td>Grouted Shear Key w/o PT</td>
<td>None</td>
</tr>
<tr>
<td>Lewis and Clark Bridge</td>
<td>WA</td>
<td>2004</td>
<td>Grouted Shear Key w/o PT</td>
<td>Grouted Shear Key w/o PT</td>
</tr>
<tr>
<td>Kouwegok Slough Bridge</td>
<td>AK</td>
<td>2000</td>
<td>Grouted Shear Key w/o PT</td>
<td>None</td>
</tr>
</tbody>
</table>
REFERENCES


