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Federal Highway Administration

STATE-OF-THE-PRACTICE REPORT

PARTIAL-DEPTH PRECAST CONCRETE DECK PANELS

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
	NOTE: volumes greater than 1000 L shall be shown in m^3			
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

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16. Abstract

Partial-depth precast concrete deck panels (PDDPs) are relatively thin precast prestressed concrete panels that span between girders which combine to act compositely with a cast-in-place (CIP) concrete topping to provide the full structural thickness of a bridge deck. PDDPs may have some advantages including faster and safer construction, ease of design, improved quality, and accelerated bridge construction duration. This Report summarizes the state-of-the-practice for partial-depth precast concrete deck panel design and construction based on standard practices of six States that are longtime and regular users of PDDPs. Design, fabrication, and installation practices are all discussed in this Report.

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EXECUTIVE SUMMARY

Partial-depth precast concrete deck panels (PDDPs) are relatively thin precast prestressed concrete panels that span between girders. When combined with a cast-in-place (CIP) concrete topping, they act compositely with the CIP concrete to provide the full structural thickness of a bridge deck. Panels are placed next to each other along the length of the girders and are generally not connected at the joints between panels which are transverse to the span of the girders. PDDPs may have advantages including faster and safer construction, as well as ease of design, improved quality, and accelerated bridge construction time.

To address the issue of reflective cracking on precast deck panels, the Federal Highway Administration (FHWA) published a memorandum in 1987 to show design, detailing, and construction techniques that had been successfully used by State departments of transportation to construct bridge decks with minimal to no reflective cracking. (A copy of the FHWA memorandum is included in Appendix E.)

The objective of this Report is to provide information about the use of PDDP as a construction option. To help achieve that objective, this Report summarizes the state-of-the-practice for partial-depth precast concrete deck panel design and construction based on a review of standard practices as demonstrated in their published design guidance, standard drawings, and specifications of six States that are longtime and regular users of PDDPs, as well as documents from the Precast/Prestressed Concrete Institute (PCI) and the American Association of State Highway and Transportation Officials (AASHTO) documents. The report discusses design (Chapter 2), fabrication (Chapter 3), installation (Chapter 4), and cast-in-place deck concrete placement (Chapter 5) practices. In addition, a look at some emerging practices (Chapter 0) and international practices (Chapter 7) is provided. Finally, the Report addresses some concerns about PDDPs (Chapter 8) and provides some ideas that agencies may use to deploy PDDPs (Chapter 9).

The first uses of PDDPs in the United States were for the construction of bridges in the 1950s, including a project for the Illinois Tollway in 1956. At least 25 State DOTs have used PDDPs over the following decades, and at least six States use them regularly, as shown in Table 1. Texas and Missouri are two of the States that have used PDDPs the most.

State-of-the-Practice Overview

A summary of the state-of-the-practice of PDDP usage of the six States is provided below. The term state-of-the-practice, as used in this report, refers to the practices generally used by the State DOTs of Colorado, Missouri, New Hampshire, Tennessee, Texas, and Utah, that frequently use PDDPs.

Design

The state-of-the-practice for PDDP design by the six States is to use a panel thickness within the range of 3 inches to 4 inches, with 3/8-inch to ½-inch diameter strands. The maximum panel width transverse to the main longitudinal bridge members varies depending on the panel thickness and girder spacing.

AASHTO and PCI specifications on strand jacking force and minimum concrete release strength generally provide satisfactory performance according to the six States. However, several key States reduce the jacking force to 0.63 f_{pu} and use a lower concrete compressive stress limit at release of 0.19 f'_{ci} to reduce the chances of panel cracking.

The state-of-the-practice for the six States includes providing minimum longitudinal distribution reinforcing of 0.22 square-inches per foot and splitting reinforcing when initial concrete compressive stresses exceed 0.19 f'_{ci} or when using 3.0-inch thick panels that are more than 8-feet wide to minimize cracking.

Permanent panel support is typically provided by the cast-in-place deck concrete, with a 1.5-inch panel support width. A continuous semi-flexible material is often used for temporary panel support and serves as a form for the grout or concrete that will support the panel. Steel support angles mounted on the girder flange are also used on steel girders with narrow girder flanges or tall haunches.

The minimum haunch thickness when using PDDPs is 1.0 inch. If 0.75-inch chamfers are provided on the bottom corners of the panels at the supports, the minimum haunch thickness typically can be reduced to 0.5 inch, provided the CIP concrete maximum aggregate size is less than the haunch thickness, according to the six States.

To ensure composite behavior between the PDDPs and the CIP concrete, the state-of-the-practice is to roughen the top surface of the PDDPs to a minimum amplitude of 1/8-inch. Based on their public design standards and specifications, the six States have found that roughness pattern and orientation is not important.

In the States' experience, partial-depth deck panels may be used with any type of steel or concrete girder. Caution should be used, however, when considering PDDPs with spread box beams and slab beams because the large total deck thickness over the beams (including the CIP concrete, panel support height or haunch, and precast slab or top flange) may result in an inefficient structure. Conventional PDDPs are not used for deck overhangs. Skewed end panels are typically used for skew angles up to 45 degrees.

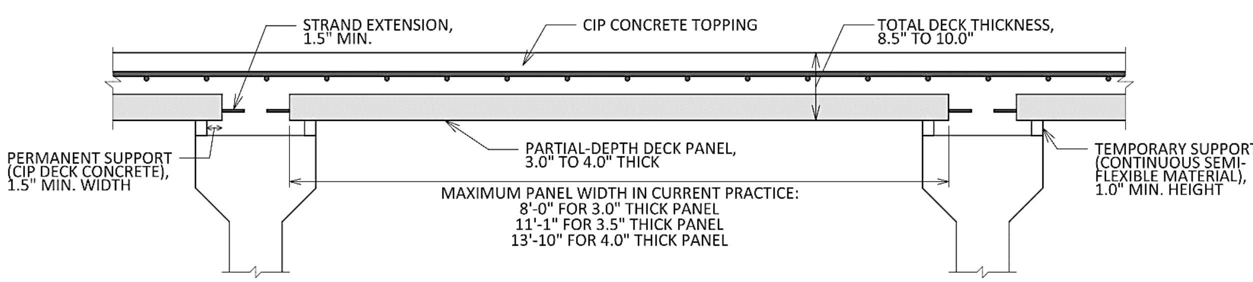

Figure 1. Illustration. Typical transverse cross section of a panel.

The CIP concrete deck above the panels entails only one mat of reinforcement, with the strand or reinforcing bars in the panels serving as the bottom mat of reinforcement.

Fabrication

The States typical specifications for fabrication of prestressed concrete bridge elements are shared. The States generally use the PCI *Manual for Quality Control for Plants and Production of Structural Precast Concrete Products, MNL-116*. For lifting the panels, mild reinforcing bars with inverted U-bends that rise above the deck are embedded in the panels to provide a minimum of four lifting points. For repair of partial-depth deck panels, the state-of-the-practice by the six States is to use the 2006 PCI *Manual for the Evaluation and Repair of Precast, Prestressed Concrete Bridge Products*, PCI-MNL-137 or the 2018 PCI Northeast (PCINE) *Guidelines for Resolution of Non-Conformances in Precast Concrete Bridge Elements*, PCINE-18-RNPCBE.

Panel Installation and Placement of CIP Deck Concrete

The six States generally do not require temporary bracing of PDDPs during erection. Field conflicts can typically be addressed by varying the height of the panel temporary supports and therefore the thickness of the haunch, or, if necessary, by varying the width or adjusting the position of the panel's temporary supports to avoid flange splice plate bolts.

The panels should be in a saturated surface-dry (SSD) state at the time of placing the CIP concrete to minimize the potential for cracking. This can be achieved by continuously wetting the PDDPs for a minimum of 12 hours and up to 24 hours prior to the CIP concrete placement, according to the practices of the six States.

During CIP concrete placement, the contractor should pay attention to the areas under the panel supports. The States indicated that it is important for the deck's long-term performance that the panel supports are fully filled with concrete.

Emerging Concepts

The report discusses several emerging concepts and variations of typical PDDPs. This includes the use of new materials such as ultra-high performance concrete (UHPC) and fiber-reinforced polymers. Research is being conducted for the use of UHPC for either the panel itself to reduce shipping weights, or for the CIP topping to enhance durability. Fiber reinforced polymer reinforcement materials have been extensively researched by the Colorado DOT using carbon fiber reinforced polymer (CFRP) prestressing strands with deformed glass fiber reinforced polymeric (GFRP) temperature reinforcement.

The report also discusses the first generation NUDECK, an existing but not widely used variation on typical PDDPs developed by the University of Nebraska-Lincoln. This system spans the full width of the bridge, eliminating overhang formwork and supports. Finally, the use of the AASHTO empirical design method for the CIP concrete portion of the deck is discussed, along with the studies that have been performed and the States that allow it.

International Practices

As discussed in the Report, other countries review of PDDPs practices outside the United States is also included in this Report. PDDPs are being used in countries including Canada, which has similar practices to the United States. In the United Kingdom, the typical panels are quite different, being only 12-inches wide with a single lattice bar truss per panel in lieu of prestressing. In Spain and Australia, partial-depth precast concrete panels are fabricated for the full width of a bridge deck, with lattice bar trusses extending above the panel with no prestressing.

Perceived Barriers and Solutions to the Use of PDDP Technology

This Report addresses perceived barriers by State DOTs to the use of PDDP technology, including concerns about cracking of the CIP concrete above PDDPs, concerns about panel rejection due to cracking during fabrication, and cost concerns. Each concern is addressed in detail along with the actual experience of the States that are regular users of PDDPs.

Approach and Strategies to Effectively Deploy PDDP Technology

This Report concludes with possible strategies to deploy PDDP. These strategies include developing standard specifications and details for PDDPs, ideally combined with a demonstration project. One suggestion is to undertake an information campaign to improve state agency, designer, and contractor awareness, including activities such as giving presentations at conferences and to contractors or contractor associations. A suggested implementation plan flow chart is provided in Chapter 9.

1. INTRODUCTION

Description

Partial-depth precast concrete deck panels (PDDPs) are relatively thin precast prestressed concrete panels that span between girders. When combined with a cast-in-place (CIP) concrete topping, they act compositely with the CIP concrete to provide the full structural thickness of a bridge deck. Panels are placed next to each other along the length of the girders and are generally not connected at the joints between panels which are transverse to the span of the girders. The prestressing strands in PDDPs are oriented transversely to the longitudinal axis of the bridge's main supporting girders and are located at or near mid-depth of the panels. The panels act as stay-in-place (SIP) forms for the CIP portion of the deck and the prestressing strands and the associated precompression allows the panels to support the panel self-weight, the weight of the fresh CIP topping concrete spanning between the girders, and construction loads. The prestressing strands also act as the bottom layer of reinforcement in the completed composite deck, so the tops of panels are intentionally roughened to ensure bond and composite action with the CIP concrete. The top layer of deck reinforcement is placed above the panels prior to placement of the CIP layer. PDDPs are generally in the range of 3 to 4 inches thick, and the concrete topping normally ranges from 4 to 5 inches thick, to make up the full depth of an 8 to 9 inch structural deck. Figure 2 shows a typical PDDP installation.

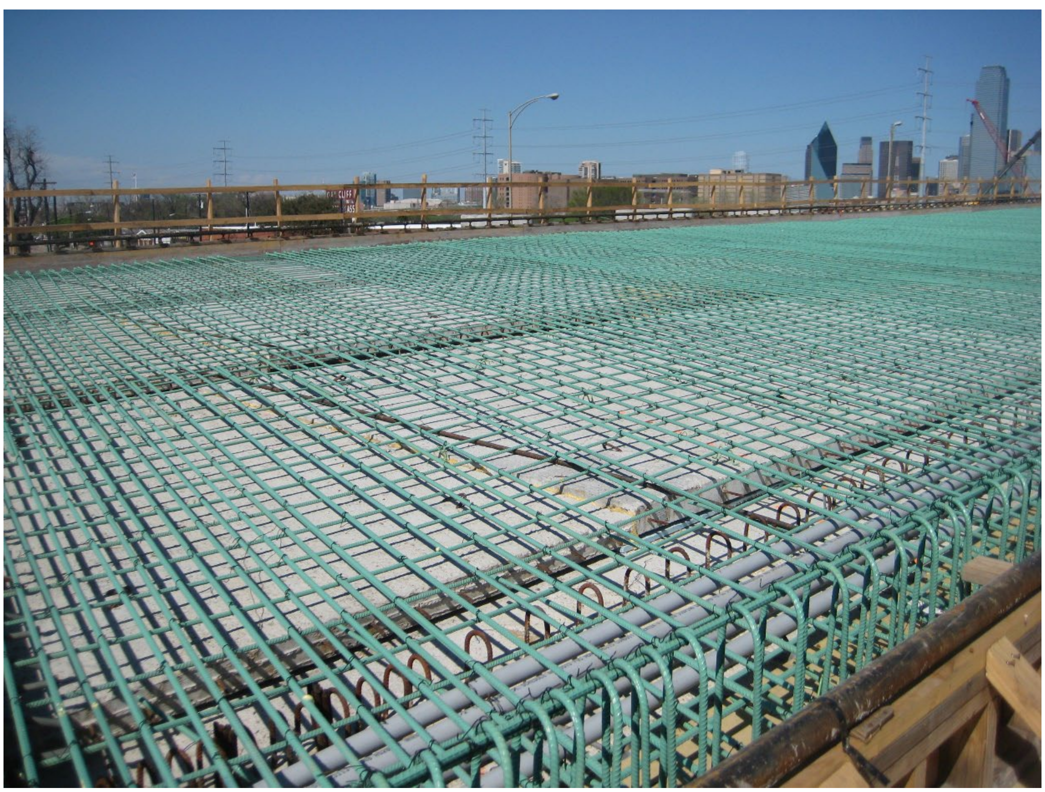

Figure 2. Photo. Typical PDDP installation prior to cast-in-place topping.

History of Use

Partial-depth precast concrete deck panels were first used in the United States for the construction of bridges in the 1950s, including a project for the Illinois Tollway in 1956 (Barker 1975, PCI Committee on Bridges 1987). At least twenty-five State departments of transportation have used PDDPs since then, and at least six States routinely utilize them. For example, Texas first began using this technology in 1963, and as of 2016 approximately 85 percent of bridge construction projects in the State of Texas included PDDPs. This equates to more than 2,000 bridges in the State incorporating PDDPs (Jones et al. 2016). Missouri began using PDDPs in 1973, and now 90 percent of the bridge decks in the State are constructed using partial-depth precast concrete deck panels, with over 1,700 bridges in the State incorporating PDDPs as of 2010 (Sneed et al. 2010). However, some State highway agencies have only used PDDPs in a limited manner, some have never utilized PDDPs, and some used them but stopped after some concerns with performance. Ways in which those concerns have been addressed through improvements in PDDP details and practices are described in Chapter 8.

Table 1 contains information on 26 States that have used the panels in the past and/or are using them. The second and third columns show the results of two surveys over the past decade documenting State usage of PDDPs. The two columns on the right are the result of investigations carried out for this report. The second column from the right indicates whether the State has guidance on PDDPs, either in the form of standard details, special provisions, or design manual guidance. The far-right column indicates whether the State is using PDDPs. A "No" in this column does not indicate that the use of PPDPs is prohibited, but simply that they are not in common usage. Conversely, a "Yes" may indicate that the panels are allowed but are not necessarily in common usage.

In 1987, FHWA published a memorandum on precast concrete deck panels that addressed the issue of reflective cracking that had been observed in the cast-in-place topping. At that time, 20 States were using precast deck panels for bridge deck construction. Nine States had reported reflective cracking but did not consider the tight hairline cracking to be a problem. However, one State had experienced extensive longitudinal and transverse cracking, which led to the issuance of a moratorium on the use of partial depth deck panels in that State. The FHWA memorandum observed that most States were receiving satisfactory performance from their decks with precast panels and highlighted design, detailing, and construction techniques that had been successfully used by other State departments of transportation to achieve decks with minimal to no reflective cracking. (The memorandum is included in Appendix E.)

The purpose of this report is to provide information about this technology.

As can be seen in Table 1, multiple States have been using PDDPs for decades and continue to use them today. In addition to Texas and Missouri, Colorado also uses PDDPs for a large majority of bridge construction projects. New Hampshire, Tennessee, and Utah likewise have an established track record of PDDP use.

Table 1. State usage of partial-depth deck panels.

State	No. Years of Use to 2010*	No. Projects 2005-2015**	State Guidance	Currently Using
Arizona	*NA*	0	No	No
California	*NA*	3	Yes	Yes
Colorado	**16**	**95% of all projects**	Yes	Yes
Florida	40	*NA*	No	No
Georgia	28	*NA*	No	No
Hawaii	14	0	No	No
Illinois	*NA*	1	No	No
Iowa	25	0	Yes	Yes
Kansas	20	*NA*	Yes	Yes
Kentucky	10	0	No	No
Maine	*NA*	*NA*	Yes	No
Minnesota	8	3	No	No
Missouri	**35**	**1,712 as of 2010***	Yes	Yes
Nevada	*NA*	1	No	Yes
New Hampshire	*NA*	**59**	Yes	Yes
New Jersey	*NA*	6	No	No
New Mexico	*NA*	7	Yes	Yes
New York	*NA*	0	Yes	No
North Carolina	*NA*	*NR*	Yes	No
Oklahoma	15	0	Yes	No
Rhode Island	*NA*	*NA*	Yes	No
Tennessee	**33**	**20**	Yes	Yes
Texas	**25**	**2000**	Yes	Yes
Utah	*NA*	**74 as of 2010*****	Yes	Yes
Washington	*NA*	0	Yes	Yes
Wisconsin	*NA*	4	Yes	No

* Sneed et al. 2010
** Jones et al. 2016
*** Lindsey 2010
NA indicates that the information is not available

Benefits

Partial-depth precast concrete deck panels may have some advantages, including faster and safer construction, as well as ease of design, and improved quality. The reduced bridge construction duration may save money, even on non-accelerated projects. Their key characteristics and the possible advantages may include:

- The panels act as stay-in-place forms for the CIP concrete portion of the deck.
    - Eliminating the need for additional deck forming, saving time and money.
    - Eliminating the time and cost of removing deck forming, especially where other stay-in-place forms are not allowed. This is also advantageous where access from below is limited, such as over water, environmentally sensitive areas, or busy roadways.
    - Increasing worker safety since panels are inherently stable and can be safely walked on (Figure 3).
    - Reducing time that workers are exposed to construction hazards since construction is faster when using PDDPs as compared to a traditional CIP deck.
    - Facilitating longer girder spans and eliminating the need to place the CIP portion in phases for continuous steel girder and precast spliced girder superstructures, where the panels allow the deck load to be applied in stages, which helps to balance the deck self-weight moments.
    - Increasing structural efficiencies for curved precast tub girder superstructures, where the panels can be grouted in place, creating closed box girders with torsional rigidity to resist the weight of the CIP concrete.

- The panels become the bottom portion of the completed bridge deck.

    Act compositely with the CIP concrete topping. The prestressed strands in the panel act as the bottom layer of deck reinforcement.

    Reduce the amount of reinforcement placed in the field.

    - Reduce the amount of concrete placed in the field.
    - Provide visibility of the bottom surface of the completed deck to facilitate future condition inspections.

- The panels are typically precast off-site and result in overall better performing bridge decks.
    - Because the PDDPs are precast, they are likely to have higher quality versus field cast concrete due to being fabricated in a controlled environment, with more uniform geometry and material properties. This also boosts the quality of the full composite deck.
    - The higher quality fabrication and the precompression due to prestressing that is typically used to reinforce PDDPs leads to increased durability that can reduce the likelihood of future deck soffit spalling over the deck service life.
    - Precasting the panels off-site can reduce the amount of construction time on site, thereby reducing road closure time for replacement bridges or bridge redecking and the associated user costs.
    - Completed decks using PDDPs with proper design and detailing typically have equal or less overall cracking compared to full-depth CIP bridge decks.

- Design and detailing of PDDPs can be well suited to standardization.
    - Many States that are successfully using PDDPs have standardized details and typical designs or design tables.
    - These standard details and design methods can simplify the deck design process.

- Standard details can be inserted into contract plans reducing drawing production effort.
- Standard details can also help increase fabrication efficiency and reduce costs over multiple projects.

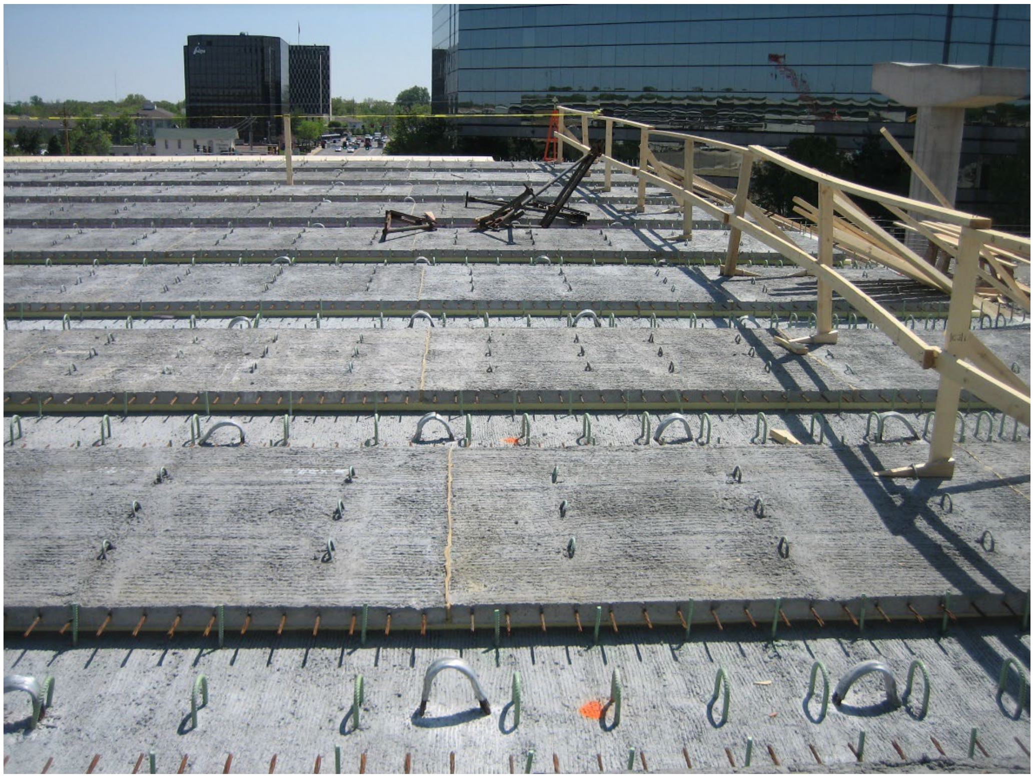

Figure 3.Photo. PDDP's in place providing a safe work zone.
© Reid Castrodale

Prior Experiences

There may be some concerns about cracking of the cast-in-place concrete placed on top of the panels in the form of longitudinal reflective cracking along the ends of the panels resting on the girders, at the transverse joints between panels, and at interfaces between panels and full-depth CIP concrete (Merrill 2002, Bayrak et al. 2013, Sneed et al. 2010). These issues may be mitigated through proper detailing of the support for the panels and proper procedures for placement of CIP concrete on top of the precast panel and below the panel ends, as described in Chapter 2 and Chapter 5. With proper design, detailing, and installation, the amount of cracking that does occur in the CIP is typically the same or less than what is seen with full-depth CIP concrete bridge decks.

Another concern is cracking of panels during fabrication, leading to panel rejection. Panel rejection can be costly. As a result, research has been performed to analyze the performance of cracked panels to determine acceptable levels of cracking (Volle 2002) and to find ways to control it (Bayrak et al. 2013). Because of this research, panel rejection rates are now less than 1 percent in States that routine employ PDDPs. One reason the panel rejection rate has been lowered is the adoption of lower initial strand prestressed force and reduction of the assumed losses during design. Another technique is to add splitting reinforcement in the panels as discussed in Chapter 2.

Localized punching shear failure of panels was observed in the past (Sen et al. 2005). This issue is typically a result of poor bearing of the panels on the superstructure and can be mitigated through details that ensure positive bearing as shown in Chapter 2.

The following chapters review the practices of the six States to show the state-of the practice for PDDP detailing, design, fabrication, and installation and refer to AASHTO, PCI, and PCINE documents. Chapter 8 will discuss in more detail possible concerns about PDDP performance as well as other potential barriers.

2. DESIGN

Design documents from PCI, PCINE, and AASHTO related to PDDPs are reviewed and compared in the following sections. Furthermore, the practices of six State departments of transportation that are longtime and regular users of PDDPs are compared to the AASHTO and PCI documents and to each other. Some of these industry technical documents are not incorporated by reference in the Code of Federal Regulations (CFR) Title 23; therefore, they are not Federal requirements. Where such industry technical document is referenced in this report, a footnote is used to provide additional information.

Design Approach

AASHTO

The two primary AASHTO design documents relating to PDDP design are the following. These standard specifications are incorporated by reference in the CFR Title 23 when this report is being written:

- AASHTO Standard Specifications for Highway Bridges, 17th Edition (2002) are incorporated by reference at 23 CFR 625.4(d)(1)(iii) and referred in this report as 2002 AASHTO Standard Specifications (23 CFR 625.4(d)(1)(iii)).

- AASHTO LRFD Bridge Design Specifications, 8th Edition (2017) are incorporated by reference at 23 CFR 625.4(d)(1)(v) and referred in this report as the 2017 AASHTO LRFD-8 Bridge Design Specifications (23 CFR 625.4(d)(1)(v)).

These two documents include only very short sections devoted explicitly to partial-depth deck panels, although the AASHTO LRFD-8 Bridge Design Specifications (23 CFR 625.4(d)(1)(v)) provide equivalent widths for cast-in-place concrete decks with stay-in-place concrete formwork for deck design using the strip method. The design of the panels in both AASHTO documents falls under the general prestressed concrete design provisions with no specific criteria for partial-depth panels except for some criteria on panel thickness, reinforcement, top surface roughening, and panel support details.

The 2017 AASHTO LRFD-8 Bridge Design Specifications (23 CFR 625.4(d)(1)(v)) is the primary reference for panel design. The 2002 AASHTO Standard Specifications (23 CFR 625.4(d)(1)(iii)) is also included in this review because some of the early PCI documents on PDDP design that are still in use refer to it for existing structures.

PCI

The Precast/Prestressed Concrete Institute (PCI) has developed multiple industry technical documents that are not incorporated in the CFR Title 23; therefore, they are not Federal requirements. These PCI documents discuss the design of PDDPs over the last 35 years. The first two documents were the PCI Special Report *Precast Prestressed Concrete Bridge Deck Panels*[1] (PCI Committee on Bridges 1987) and the subsequent and more comprehensive PCI Special Report JR-343 *Recommended Practice for Precast Prestressed Concrete Composite Bridge Deck Panels*[2](Ross Bryan Associates, Inc. 1988), which provides suggestions for the design, fabrication, shipping and handling, erection, and inspection of PDDPs and includes a design example. The *PCI Bridge Design Manual*[3] (Bridge Design Manual Steering Committee 2014), first published in 1997 and updated in 2014, also contains information regarding the design of PDDPs.

In 2001, the PCI New England Region published the *Precast Deck Panel Guidelines*[4], which included guidelines, details and specifications for design, detailing, fabrication and installation of partial-depth precast deck panels designed to act composite with a cast-in-place deck and issued revisions to a few of the details in 2003. The Bridge Technical Committee of PCI Northeast (PCINE), the successor to the PCI New England Region, published a second edition of the 2001 guidelines in 2017, *Partial Depth Deck Panel Guidelines*[5] (PCI Northeast Bridge Technical Committee 2017) with minor updates.

The 1988 Special Report *Recommended Practice for Precast Prestressed Concrete Composite Bridge Deck Panels*[2] and the PCI 2014 *Bridge Design Manual*[3] give detailed design procedures following the AASHTO bridge design specifications current at the time, and each includes a design example. The 2001 and 2017 documents are more limited for design, suggesting concrete compression and tension limits and some geometric limits, and then simply referring to AASHTO for the design. However, the PCI documents do include sample details and

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[1] Special Report: Precast Prestressed Concrete Bridge Deck Panels, *PCI Journal,* March-April 1987, pp. 26-45, Precast/Prestressed Concrete Institute, is not incorporated in the CFR Title 23. Therefore, this industry technical document is not a Federal requirement.
[2] Special Report: Recommended Practice for Precast Prestressed Concrete Composite Bridge Deck Panels, Report No. JR-343-88, 1988, Prestressed Concrete Institute, is not incorporated in the CFR Title 23. Therefore, this industry technical document is not a Federal requirement.
[3] Precast Prestressed Concrete Bridge Design Manual, Third Edition, Second Release, 2014, Precast/Prestressed Concrete Institute, is not incorporated in the CFR Title 23. Therefore, this industry technical document is not a Federal requirement
[4] Precast Deck Panel Guidelines, 2001, Precast/Prestressed Concrete Institute New England Region, is not incorporated in the CFR Title 23. Therefore, this industry technical document is not a Federal requirement.
[5] Partial Depth Deck Panel Guidelines, Second Edition, Report No. PCINE-17-PDDPG, 2017, Precast/Prestressed Concrete Institute Northeast, is not incorporated in the CFR Title 23. Therefore, this industry technical document is not a Federal requirement.

construction specifications. Despite the difference in the depth of information, the details and design approach are similar across the PCI documents.

One unique set of provisions is included in the PCI 2014 *Bridge Design Manual*[3] covering the first generation NUDECK, developed by the University of Nebraska-Lincoln. This is a partial-depth precast deck panel that spans the full width of the bridge, with reinforcing bars passing through open strips in the panels over the girders. This has the advantage of incorporating deck overhangs in the panels and thereby eliminating overhang formwork and supports. The disadvantage is that the panels are much larger to transport and must be customized for each bridge due to variability in not just deck width but also number of girders and girder locations.

DOT Approaches

The usage of partial-depth precast concrete deck panels varies widely across States. Some States incorporate PDDPs in most of their bridge construction projects, while some States explicitly exclude PDDPs. Other States allow the usage of PDDPs in only specific circumstances or only at the request of a contractor. This section documents the practices of six States that regularly allow the use partial-depth precast concrete deck panels, as a contractor option, and have used them for many years. These States are Colorado, Missouri, New Hampshire, Tennessee, Texas, and Utah. These six States were selected after reviewing surveys of State practices with regards to PDDPs, as shown in Table 1. The surveys included one conducted for the Missouri Department of Transportation (MoDOT) (Sneed et al. 2010); one conducted for the Nevada Department of Transportation (NvDOT) (Jones et al. 2016); and a review of State practices based on design guidelines, memoranda, standard details, and standard specifications.

The 2010 MoDOT survey received responses from 29 States and indicated there were 14 States using partial-depth panels to some degree. The 2016 NvDOT survey received 32 responses and indicated there were 12 States that have used PDDPs between 2005 and 2015. Notably, 8 of those 12 States did not respond to the MoDOT survey 6 years earlier. The NvDOT survey results also indicated the number of bridges built with PDDPs from 2005 to 2015, and 4 States in the MoDOT survey indicated as using PDDPs reported zero PDDP bridges built from 2005 to 2015. However, 5 States stood out for having reported constructing 20 or more bridges with PDDPs from 2005 to 2015. The 2020 review of State practices found 23 States that clearly allow the use of PPDPs and 10 States that explicitly exclude them. Of the 23 States that allow PDDPs, only 9 appear to be actively using them. The review also found that 14 States have design guidance on PDDPs, 13 States have standard PDDP details, and 10 States have construction specifications for PDDPs.

The five States—Colorado, New Hampshire, Tennessee, Texas, and Utah—indicated in the NvDOT survey as having constructed 20 or more bridges with PDDPs from 2005 to 2010 also were found in the 2020 review to be actively using PDDPs, and all had some combination of PDDP design guidance, standard details, and standard specifications.

A sixth State, Missouri, was absent from the 2015 NvDOT survey, but was reported in the 2010 MoDOT survey to have constructed 1,712 bridges with PDDPs up to the year 2010. The 2020 review found that Missouri is still an active user with a full complement of design guidance, standard details, and construction specifications related to PDDPs. As a result, Missouri was added to the list of States whose practices are reviewed in this state-of-the-practice report.

Each of the six States have standard PDDP details that require very little input from the designer.

- Colorado has two standard PDDP drawings, one for placing the panels on concrete girders and one for placing them on steel girders, with very minor differences in the panel details. Each drawing calls for three different panel thicknesses depending on the girder spacing. The strand layout is the same for all panel thicknesses within the allowable girder spacing range, with an option to reduce strand quantity on closely spaced girders.

- Missouri has six standard PDDP drawings, one for placing the panels on six different girder types, with very minor differences in the panel details. Each drawing has only one universal panel design within a range of allowable girder spacings. The Missouri Department of Transportation Engineering Policy Guide states that the PDDPs with a cast-in-place concrete topping is the preferred bridge deck for use on all girder and beam superstructures.

- New Hampshire has two standard PDDP drawings, one for placing the panels on concrete girders and one for placing them on steel girders. Each drawing contains a table that varies the panel concrete strengths and the strand quantity and spacing based on the girder spacing.

  The New Hampshire Department of Transportation mandates that all bridges crossing over interstate highways, turnpikes, divided highways, and statewide corridors use prestressed partial-depth concrete deck panels for the interior deck spans, and contractors are not allowed to substitute a full-depth cast-in-place deck. New Hampshire's reason for using PDDPs over important roads is to decrease the possibility of the concrete deck underside spalling over the travel way from future deck deterioration. Construction speed is also identified as an advantage, as well as the solving of constructability issues where removal of conventional deck formwork would be difficult or hazardous.

- Tennessee has one standard panel drawing set, with a table giving strand size, strand spacing, strand strength, panel thickness, and total slab thickness based on girder spacing, with some varying panel support details based on girder type.

- Texas has one standard PDDP detail with varying strand quantity and size based on the girder spacing. The use of mild reinforcement is an option to replace strands for medium-spaced girders, while mild reinforcement is required in lieu of strands for very closely spaced girders. Uniquely, the Texas Department of Transportation also has standard details for overhang panels, which are part full-depth panels and part partial-depth panels, covering the overhang and the first interior bay. These can be combined with standard partial-depth panels on the remaining interior girder bays to eliminate all temporary deck formwork and overhang brackets.

- Utah has one PDDP panel drawing set with a single universal panel design for the range of allowable girder spacings. In its Structures Design and Detailing Manual, the Utah

Department of Transportation recognizes that partial-depth precast concrete deck panels can decrease construction time and reduce user impacts.

To clearly present the similarities and differences in recommendations and practices, the practices of the six States are presented in the sections that follow alongside provisions from the PCI and AASHTO documents. Finally, a suggested practice is presented for each parameter.

Panel Geometry

Suggested panel geometry based on the practice of the six States is depicted in Figure 4.

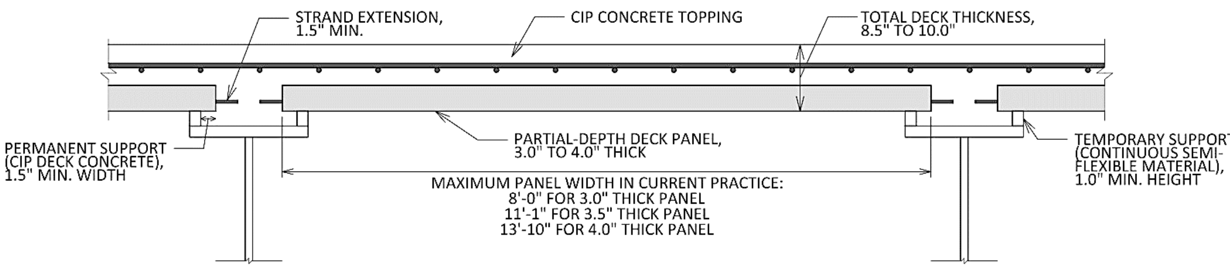

Figure 4. Illustration. Transverse section: panel geometry.

Panel and CIP Thickness

The thickness of partial-depth precast concrete deck panels generally ranges from 3 to 4 inches. There are inconsistencies among the PCI and PCINE design guides regarding the minimum thickness of partial-depth panels. The 2014 PCI *Bridge Design Manual*[3] suggests that panels should be 3- to 4-inches thick, while the 1988 *PCI Special Report JR-343*[2] recommends a 3-inch minimum panel thickness, with an absolute minimum thickness of 2.5 inches. The *PCINE Guidelines*[5] specifies a 3.5-inch minimum thickness. The 2017 *AASHTO LRFD-8 Bridge Design Specifications* (23 CFR 625.4(d)(1)(v)) specifies a 3.5-inch minimum panel thickness, and the 2002 *AASHTO Standard Specifications* 2002 (23 CFR 625.4(d)(1)(iii)) do not specify a minimum thickness.

Table 2 shows the suggested or specified panel thicknesses alongside the corresponding cast-in-place topping thickness. From the table, the thickness of the precast panels is consistently in the range of 3.0-inches to 4.0-inches for the six States, sometimes with choices based on girder spacing or strand size and spacing. However, the total deck thickness used by the six States ranges from 8.5-inches to as much as 10-inches.

Thicker panels should be less fragile, better accommodating unintended deviations in strand position and handling stresses. However, this comes at the expense of increased handling and shipping costs, and increased fabrication cost from more concrete and more prestressing strand. Therefore, the suggested panel thickness based on the practices of the six States is a range of 3 inches to 4 inches; however, the use of a 3-inch panel thickness should be correlated to a maximum panel span.

Table 2. Reference panel, CIP, and total deck thicknesses.

Source	Panel Thickness	CIP Topping Thickness	Total Deck Thickness
PCI Special Report JR-343 (1988)[2]	3.0" min. recommended, 2.5" absolute min.	Not specified; Design curves for 4.0", 5.0", 6.0" toppings	Not specified
PCI Bridge Design Manual (2014)[3]	3.0" to 4.0"	Not specified	Not specified
PCINE Guidelines (2017)[5]	3.5" min.	4.5" min.	8.0" min.
AASHTO Standard Specifications (2002) (23 CFR 625.4(d)(1)(iii))	Not specified	Not specified	Not specified
AASHTO LRFD-8 Bridge Design Specifications (2017) (23 CFR 625.4(d)(1)(v)	3.5" min.; Max. 55% of total deck thickness.	Not specified	Not specified
Colorado DOT	3.0" (8'-0" max panel width) 3.5" (11'-1" max panel width) 4.0" (13'-1" max panel width)	5.0"	8.0" to 9.0" depending on girder spacing
Missouri DOT	3.0"	5.5" min.	8.5" min.
New Hampshire DOT	3.5"	5.0" with asphalt overlay; 6.5" with bare deck	8.5" with asphalt overlay; 10.0" with bare deck
Tennessee DOT	3.5" or 4.0", depending on strand size, type, and spacing	4.75" to 5.5"	8.25" to 9.5"
Texas DOT	4.0"	4.5" min.	8.5" min.
Utah DOT	3.5"	5.5"	8.5" min.

State-of-the-Practice

- Use of panel thickness within the range of 3 inches to 4 inches as shown in Figure 5 have been used by the six States.

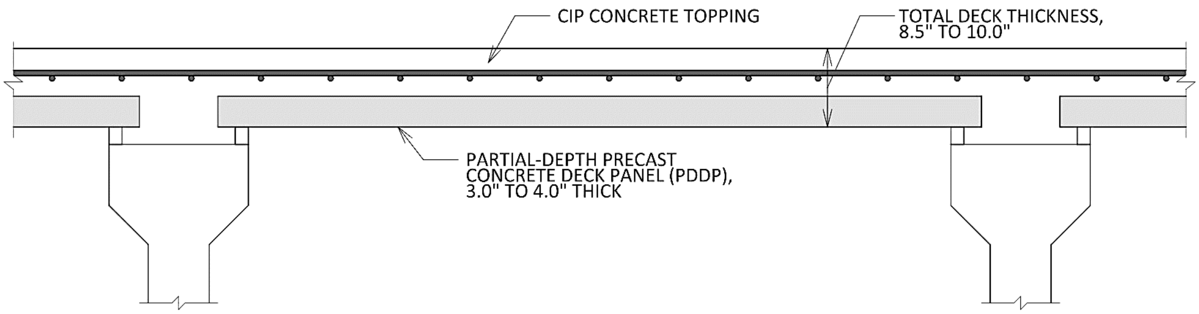

Figure 5. Illustration. Transverse section: total panel and CIP deck thickness.

Panel Length and Width Limits

The 1988 *PCI Special Report JR-343*[2] is the only document that gives a panel length. It suggests4-feet and 8-feet lengths, while noting that the panel width is based on the girder spacing without suggesting any limits. Figure 6 shows how panel length and width are described.

Table 3. Reference panel length and width limits.

Source	Panel Length (parallel to bridge CL and girders)	Panel Width (perpendicular to bridge CL and girders)
PCI Special Report JR-343 (1988)[2]	4’-0” and 8’-0” (preferred)	Based on girder spacing
PCI Bridge Design Manual (2014)[3]	Not specified	Not specified
PCINE Guidelines (2017)	Not specified	Not specified
AASHTO Standard Specifications (2002) (23 CFR 625.4(d)(1)(iii))	Not specified	Not specified
AASHTO LRFD Specifications (2017) (23 CFR 625.4(d)(1)(v))	Not specified	Not specified
Colorado DOT	3’-0” min., 8’-0” max.	13’-10” max.
Missouri DOT	2’-10” min., 8’-0” max.	4’-0” min., 9’-6” max.
New Hampshire DOT	4’-0” min., 8’-0” max.	Based on girder spacing
Tennessee DOT	2’-0” min., 8’-0” max.	2’-10” to 11’-2”
Texas DOT	2’-10” min., 8’-0” max.	9’-6” max.
Utah DOT	8’-0” max.	11’-6” max.

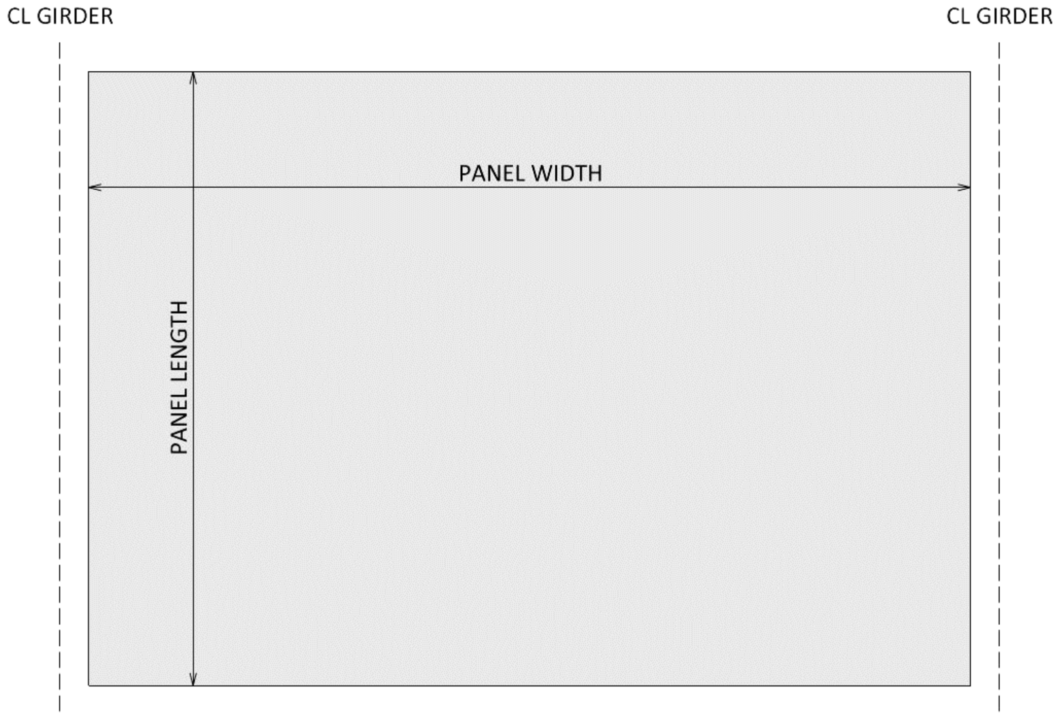

Figure 6. Illustration. Typical plan view: panel length and width.

Five of the six States each provide minimum and maximum panel lengths, with Utah only providing a maximum panel length. All the States use 8 feet as the maximum panel length, but the minimum varies between 2 feet and 4 feet. A maximum length of 8 feet is a practical limit for shipping of panels.

The more important dimension is the panel width since this determines the span length of the panel and the design span length of the completed slab. The panel width relates directly to the girder spacing. Four States explicitly state maximum panel widths ranging from 9-ft-6 inches to 13-ft-10-inches. Notably, Missouri and Colorado are the only States that use a 3-inch panel thickness. Missouri has a universal panel design and limits the panel width to 9 ft-6-inches, and Colorado limits the use of a 3-inch-thick panel to a maximum panel width of 8 feet and requires a 4-inch-thick panel when the panel width exceeds 11-ft-1-inch.

State-of-the-Practice

- Maximum panel widths relative to panel thickness, based on State practices, are shown in Table 4. The designer should verify the actual panel width based on the specific panel design. Larger widths may be possible based on the practices of a few States.

Table 4. Maximum panel width.

Panel Thickness	Maximum Panel Widths Allowed in Current Practice
3.0”	8’-0”
3.5”	11’-1”
4.0”	13’-10”

Design Parameters

Deck Design Method, Span Configuration, and Loading

Partial-depth precast concrete panels are designed to act compositely with the cast-in-place concrete topping. The panels are designed as simple-span prestressed concrete elements acting independently to carry the weight of the fresh cast-in-place deck concrete plus a construction load. The panels then become part of the composite deck system resisting live loads and superimposed dead loads. The method to determine the live load design moment in the PCI documents follows the AASHTO standard in effect at the time of publication. The 2002 *AASHTO Standard Specifications for Highway Bridges* (23 CFR 625.4(d)(1)(iii)) (as well as previous and outdated versions) provide an approximate method of determining the live load bending moments per foot of deck width based on the Westergaard Theory using the wheel load from a HS-15 or HS-20 truck. The 2017 *AASHTO LRFD-8 Bridge Design Specifications* (23 CFR 625.4(d)(1)(v))gives an approximate strip method where equations are provided to determine the width of deck to be used and then the HL-93 design truck wheel loads are applied to the strip and the live load bending moments are determined from classical beam theory. The 2017 *AASHTO LRFD-8 Bridge Design Specifications* (23 CFR 625.4(d)(1)(v)) also allows refined methods of analysis, but explicitly excludes the use of the empirical design method for situations other than full cast-in-place concrete decks.

The 2017 *AASHTO LRFD-8 Bridge Design Specifications* (23 CFR 625.4(d)(1)(v)) approximate strip method and the 2002 *AASHTO Standard Specifications* (23 CFR 625.4(d)(1)(iii)) approximate method for determining deck live load moments can both give moments based on simple spans or continuous spans. They do not dictate whether to assume continuity or not for decks with partial-depth precast deck panels, however, they do not prohibit assuming continuity with the panels so it can be assumed that it is permissible. The *1988 PCI Special Report JR-343*[2] and the *2014 PCI Bridge Design Manual*[3] explicitly suggest assuming continuity for live loads and superimposed dead loads applied to the completed, composite deck section.

For construction loads, the 2017 *AASHTO LRFD-8 Bridge Design Specifications* (23 CFR 625.4(d)(1)(v)) and the 2002 *AASHTO Standard Specifications* (23 CFR 625.4(d)(1)(iii)) call for a minimum of 50 lb/ft^2, as do the 1988 *PCI Special Report JR-343*[2] and the 2014 *PCI Bridge Design Manual*[3]. The 2017 *PCINE Guidelines*[5], however, only call for a construction load of 40 lb/ft^2.

Table 5. Reference design live load, span configuration, and construction load.

Source	Deck Design Method and Live Load	Deck Span Configuration	Construction Load
PCI Special Report JR-343 (1988)[2]	AASHTO approximate live load moment for simple spans (Westergaard Theory); HS-20	Continuous	50 psf
PCI Bridge Design Manual (2014)[3]	Approximate Strip method or Refined method; HL-93	Continuous	50 psf
PCINE Guidelines (2017)[5]	Standard AASHTO design of composite sections	Not specified	40 psf
AASHTO Standard Specifications (2002) (23 CFR 625.4(d)(1)(iii))	Approximate live load moment for simple spans (Westergaard Theory); HS-20	No restriction	50 psf specified for metal stay-in-place forms
AASHTO LRFD-8 Bridge Design Specifications (2017) (23 CFR 625.4(d)(1)(v))	Approximate Strip method or refined methods; HL-93	No restriction	50 psf min.
Colorado DOT	Design Tables; Empirical method allowed; HS-25 and Military Load	Implicitly specified	117 psf*
Missouri DOT	Approximate Strip method; HL-93	No restriction	50 psf
New Hampshire DOT	Approximate Strip method; HL-93 or HS-25	Continuous	40 psf or 50 psf
Tennessee DOT	AASHTO current edition	Not specified	100 psf
Texas DOT	Empirical method; HL-93	No restriction	50 psf
Utah DOT	Approximate Strip method; HL-93	No restriction	117 psf*

* Value represents the maximum construction load and also the maximum total load including wet deck concrete during construction.

For the design of the composite deck, four of the six States follow the 2017 AASHTO LRFD-8 Bridge Design Specifications (23 CFR 625.4(d)(1)(v)) for the design method. Despite AASHTO's exclusion, however, Colorado and Texas allow the use of AASHTO's empirical method. Research performed for the Colorado Department of Transportation demonstrated that decks with PPDPs designed with the empirical method performed just as well as those designed with conventional methods, and even recommended the empirical design method for decks with PPDPs because it results in a 40 percent reduction in the amount of deck reinforcing steel which in turn makes the deck less prone to reinforcement corrosion problems leading to a longer service life (Shing and Xi 2003). Colorado also provides a deck design table in its Bridge Design Manual that applies equally to monolithic concrete decks and composite decks with partial-depth precast concrete panels, making the design of decks with these panels very simple.

All the six States have standard details for partial-depth precast concrete deck panels. But the full deck section still needs to be designed by the designer. Colorado and Texas, by virtue of allowing the 2017 *AASHTO LRFD-8 Bridge Design Specifications* (23 CFR 625.4(d)(1)(v)) empirical design method, implicitly assume continuous spans for the completed deck. Other States such as New Hampshire and Utah explicitly recommend assuming continuity for completed decks incorporating partial-depth precast panels, and the remaining States do not explicitly restrict it.

For construction live loads, three of the States follow AASHTO specifications of 50 pounds per square foot while the remaining three specify double the value or more, up to 117 pounds per square foot. The construction load value, however, should be reflective of local construction practices.

State-of-the-Practice:

- Allowing the use of the AASHTO empirical design method to simplify deck design with partial-depth precast concrete deck panels based has been done by both TxDOT and CDOT.
- Assuming transverse continuity for live loads and superimposed dead loads when designing the completed deck section has been used by the six States.
- Using a 50 psf minimum construction load that is adjusted upward as necessary according to local practices has been shown to be a successful practice.

Design Criteria for Prestress

Strand Diameter, Spacing, Eccentricity, and Vertical Placement Tolerance

Limits for strand diameter and horizontal spacing are discussed in the 2017 *AASHTO LRFD-8 Bridge Design Specifications* (23 CFR 625.4(d)(1)(v)), the 2002 *AASHTO Standard Specifications* (23 CFR 625.4(d)(1)(iii)), the *1988 PCI Special Report JR-343*[2], the *2017 PCINE Guidelines*[5], and the *PCI 2014 Bridge Design Manual*[3]. For the strand vertical position, the three PCI documents[2, 3, 5] are generally in agreement that the strand should be centered in the panel, while the two AASHTO specifications do not address this. The 1988 *PCI Special Report JR-343*[2] varies the tolerance for the strand vertical position based on the panel thickness, while the other two PCI documents[3,5] simply state a 1/8-inch tolerance. The 1988 *PCI Special Report JR-343*[2] and the 2017 *PCINE Guidelines*[5] indicate that the average tolerance for the strand group is only in the downward direction, but the PCI 2014 *Bridge Design Manual*[3] does not make this clarification. While some States have recommendations for 250 ksi and 270 ksi strands, the values shown below are only for 270 ksi strands, for consistency among the States and with typical current practices.

Table 6. Reference strand diameter, spacing, eccentricity, and vertical tolerance.

Source	Strand Diameter	Strand Horizontal Spacing	Strand Eccentricity	Strand Vertical Position Tolerance
PCI Special Report JR-343 (1988)[2]	Approx. 1/8 of panel thickness up to 0.5” (Note: 0.6” strands not in wide use at time of publication)	Uniform spacing	“majority of panels have strands located at centroid of section although eccentric prestressing can result in significant cost savings”	Average tolerance of +0”, -1/8” for <3.0” thick panels, and tolerance of +0”,-1/4” for ≥3.0” thick panels Individual strand tolerance of ±1/8” for <3.0” thick panels, ±1/4” for ≥3.0” thick panels
PCI Bridge Design Manual (2014)[3]	Not specified	Not specified	Not specified, but design example has zero eccentricity	±1/8” per PCI-MNL-116 for bridge deck unit
PCINE Guidelines (2017)[5]	3/8”	Not specified	Strands shall be centered vertically in the panel	+0”, -1/8” for strand group ±1/8” for individual strands
AASHTO Standard Specifications (2002) (23 CFR 625.4(d)(1)(iii))	Not specified	1.5 times total slab thickness or 18.0” max.	Not specified	Not specified
AASHTO LRFD-8 Bridge Design Specifications (2017) (23 CFR 625.4(d)(1)(v))	3/8”	1.5” min.	Not specified	Not specified
	7/16”-1/2”	1.75” min.		
	9/16”-0.6”	2.0” min.		
	All	1.5 times total slab thickness or 18.0” max.		

Source	Strand Diameter	Strand Horizontal Spacing	Strand Eccentricity	Strand Vertical Position Tolerance
Colorado DOT	3/8” max.	4.75” typical; 9.5” for panel width < 5’-7”	Constant 1.3” from bottom, leading to the following eccentricities: -0.2” in 3.0” thick panel (-13.3%); -0.45” in 3.5” thick panel (-25.7%); -0.7 in 4.0” thick panel (-35.0%)	±1/4”
Missouri DOT	3/8” min.	4.5”	0”	±1/8” for strand group and for individual strands
New Hampshire DOT	3/8”	8.0”, 6.0”, 5.0”	0”	+0”, -1/8” for strand group; ±1/8” for individual strands
Tennessee DOT	3/8”, 7/16”, or 1/2” (with 4.0” thick panels only)	3.0”, 6.0”, 9.0”, or 12.0”	0”	+1/8”, -1/4”
Texas DOT	3/8” or 1/2”	6.0”	0”	±1/8”
Utah DOT	3/8”	4.75”	-0.4375” in 3.5” thick panel (-25.0%)	±1/4”

All six States use 3/8-inch diameter strand, with Missouri, Tennessee, and Texas allowing for larger diameter strands up to 1/2-inch diameter. Horizontal spacing criteria varies, but the lowest value used by one State is 3.0-inches, which is twice the minimum value specified by the 2017 *AASHTO LRFD-8 Bridge Design Specifications* (23 CFR 625.4(d)(1)(v)). The other five States use a minimum horizontal spacing between 4.5-inches and 6.0-inches. Note that a smaller spacing will result in more total strand and more prestress force in the panel.

Four of the six States center the strand vertically in the panel. Two States, however, specify a downward eccentricity on the strand. Colorado specifies three strand eccentricities based on panel thickness, whose average is -25 percent (meaning it is located 25 percent of the distance between the center of gravity and the bottom of the panel), which is also very close to the middle value specified for a 3.5-inch thick panel. Utah specifies a single strand eccentricity of -25 percent for a 3.5-inch thick panel.

Colorado and Utah have the most permissive tolerance on strand vertical position, allowing the positions to vary by ±1/4-inch. The other four States place the strands at the level of the panel center of gravity and generally use a tolerance of 1/8-inch, with Tennessee allowing +1/8-inch and -1/4-inch, New Hampshire allowing +0-inch and -1/8-inch on the strand group, and the remaining two States using ±1/8-inch.

Upward strand deviation may be contraindicated, as it will deflect the panel downward, exacerbating deflections from the panel self-weight and the weight of the cast-in-place concrete, and creating the possibility of bottom cracking that will be permanently exposed to the elements. Conversely, the effects of downward strand deviation may be generally benign, as the upward deflection will be counterbalanced by the deflection from panel self-weight and the cast-in-place concrete weight, and any cracking that occurs will be on the top of the panel which will be covered by the cast-in-place concrete. Furthermore, placing the strand at the mid-height of the panel and only allowing a -1/8-inch tolerance does not allow much room for fabrication placement deviations. By placing the strand below the panel center of gravity, downward deflections and the risk of bottom cracking are greatly reduced when the panel is lifted or set in place, and the strand placement tolerance can be increased, reducing precasters' risk of having panels rejected and therefore reducing the cost of the panels.

The practices of Colorado and Utah indicate that a -0.44-inch to -0.45-inch strand eccentricity on a 3.5-inch thick panel (-25 percent eccentricity) will result in a satisfactory condition, with the dead load deflections and stresses counterbalancing the stress and deflection from the strand eccentricity. Colorado uses less eccentricity for 3.0-inch thick panels (-0.2-inch or -13 percent), and more eccentricity for 4.0-inch thick panels (-0.7-inch or -35 percent) which is logical for Colorado's practices. The panel thickness increases with girder spacing, and the total composite deck thickness increases for girder spacings above 10'-0". This means that thicker panels will experience larger stresses and deflections due to self-weight and due to the cast-in-place deck concrete dead load, making larger counteracting stresses due to strand eccentricity beneficial. Of course, the design needs to ensure that the strand eccentricity does not overstress the panel before the cast-in-place deck concrete is applied.

State-of-the-Practice:

- Six States have used 3/8-inch minimum and 1/2-inch maximum strand diameters.
- A minimum horizontal strand spacing of 3.0 inches has been used, but this should be correlated with strand size and panel size so that panels are not overstressed
- Negative strand eccentricity (strand positioned below the panel center of gravity) has also been used, combined with a ±1/4-inch tolerance on the strand vertical position to lower fabrication costs and reduce the risk of downward panel sag and bottom cracking, and varying strand eccentricity with panel thickness and width, as shown in Table 7 and Figure 7.

Table 7. Strand eccentricity.

Panel Thickness	Strand Eccentricity*
3.0”	-0.20”
3.5”	-0.45”
4.0”	-0.70”

* Positive eccentricity measured up from the panel center of gravity

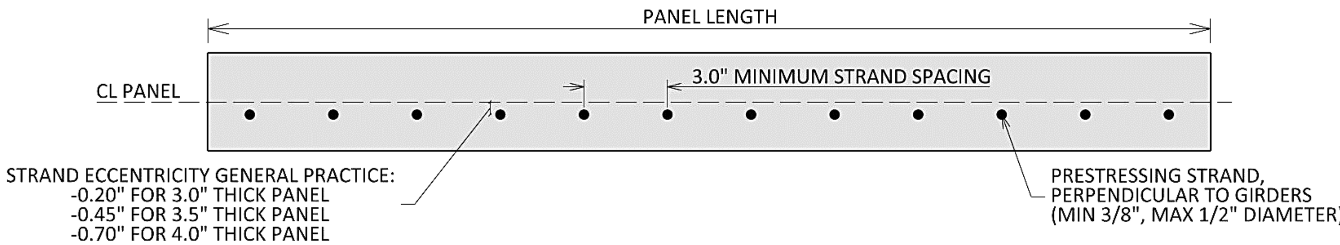

Figure 7. Illustration. Longitudinal section: strand spacing and eccentricity.

Strand Prestressing Force and Concrete Strength at Release and Final

The 2017*AASHTO LRFD-8 Bridge Design Specifications* (23 CFR 625.4(d)(1)(v)), the 2002 *AASHTO Standard Specifications* (23 CFR 625.4(d)(1)(iii)), the *1988 PCI Special Report JR-343*[2], and the *PCI 2014 Bridge Design Manual*[3] are generally consistent for concrete stress limits at release and for the initial strand prestressing force, with the exception of the 2017 *PCINE Guidelines*[5] which does not specify a tension limit and specifies a maximum concrete compressive release stress of 0.75 ksi instead of 0.60f’ci. For a concrete release strength of 4.0 ksi, this is equivalent to 0.19f’ci. The 2017 *PCINE Guidelines*[5] also do not specify a maximum strand jacking force, whereas the other PCI and AASHTO documents agree on a maximum initial strand force equivalent to a stress of 75 percent of the strand ultimate strength. For a 3/8-inch diameter strand with a 270 ksi strength, this is an initial strand force of 17.2 kips per strand.

Table 8. Reference concrete strengths and initial prestressing force.

Source	Maximum Concrete Stress at Release	Concrete Strength at Release / Final	Initial Prestressing Force per Strand
PCI Special Report JR-343 (1988)[2]	Compr. = $0.60f'_{ci}$ Ten. = $3\sqrt{(f'_{ci})}$ (f'_{ci} in psi)	4.0 ksi / 5.0 ksi	$0.75f'_s$ max. ($f'_s = f_{pu}$)
PCI Bridge Design Manual (2014)[3]	Compr. = $0.60f'_{ci}$ Ten. = 0.2 ksi or $0.0948\sqrt{(f'_{ci})}$ (f'_{ci} in ksi);	Not specified	$0.75f_{pu}$ max.
PCINE Guidelines (2017)[5]	Compr. = 0.75 ksi Ten. = Not specified	4.0 ksi min. / 6.0 ksi	Not Specified
AASHTO Standard Specifications (2002) (23 CFR 625.4(d)(1)(iii))	Compr. = $0.60f'_{ci}$ Ten. = 0.2 ksi or $3\sqrt{(f'_{ci})}$; (f'_{ci} in psi)	4.0 ksi / final not specified	$0.75f'_s$ max. (f'_s = fpu)
AASHTO LRFD-8 Bridge Design Specifications (2017) (23 CFR 625.4(d)(1)(v))	Compr. = $0.65f'_{ci}$ Ten. = 0.2 ksi or $0.0948\sqrt{(f'_{ci})}$ for normal weight concrete (f'_{ci} in ksi);	Not specified	$0.75f_{pu}$ max.
Colorado DOT	Compr. = $0.65f'_{ci}$	4.5 ksi / 6.0 ksi	17.2 kips min. jacking force; 14.2 kips est. final force
Missouri DOT	Not specified for panels	4.0 ksi / 6.0 ksi	17.2 kips
New Hampshire DOT	Compr. = $0.19f'_{ci}$ (per drawings design manual)	4.0 ksi / 6.0 ksi 4.8 ksi / 6.0 ksi 6.0 ksi / 8.0 ksi (Based on girder spacing)	17.2 kips
Tennessee DOT	Compr. = $0.60f'_{ci}$ Ten. = $3.35\sqrt{(f'_{ci})}$ (f'_{ci} in psi)	4.0 ksi / 5.0 ksi	16.1 kips (3/8” diam.); 22.1 kips (7/16” diam); 28.9 kips (1/2” diam.); Values shown for 270 ksi strand, lower for 250 ksi strand.
Texas DOT	Not specified for panels	3.5 ksi / 5.0 ksi	14.4 kips
Utah DOT	Not specified for panels	4.5 ksi / 6.0 ksi	17.2 kips min. jacking force; 14.3 kips est. final force

The practices of the six States vary with respect to concrete stress limits at release. Only three of the six States give a concrete compressive stress limit at release, with two following AASHTO (either $0.6f'_{ci}$ or $0.65f'_{ci}$ in the more recent editions of the 2017 *AASHTO LRFD-8 Bridge Design Specifications* (23 CFR 625.4(d)(1)(v))), and with New Hampshire specifying a much lower limit of only $0.19f'_{ci}$. For the concrete tension stress limit at release, only Tennessee specifies a value, which is about 10 percent greater than AASHTO of $3\sqrt{f'_{ci}}$ (psi) or $0.0948\sqrt{f'_{ci}}$ (ksi).

The required minimum concrete strength at release for the six States ranges from 3.5 ksi to 4.5 ksi, with New Hampshire requiring higher strengths up to 6.0 ksi for wider panels and girder spacings.

For initial tendon force, Utah and Colorado state a *minimum* jacking force of 17.2 kips and an estimated final strand force of 14.2 and 14.3 kips. Jacking force will typically be higher than initial strand force due to losses during transfer, and final stand force will typically be lower than the initial strand force due to long term losses such as creep and shrinkage of the concrete. For the purposes of comparison, it is assumed that the practices of Utah and Colorado are equivalent to an initial strand force of 17.2 kips, since long-term losses often amount to about 15 percent of the force. This makes their practices like New Hampshire and Missouri. Tennessee uses a lower jacking force of 16.1 kips for a 3/8-inch strand, equal to 0.70fpu (used for all strand sizes). Texas uses a jacking force of only 14.4 kips, which is $0.63f_{pu}$ for a 3/8-inch strand and an even lower stress value on a 1/2-inch strand.

In practice, compressive stresses at release due to prestressing (including allowable tolerances) and panel self-weight are generally in the 0.4 ksi to 0.5 ksi range for all the States except New Hampshire and Texas, whose compressive stresses are limited to $0.19f'_{ci}$. New Hampshire accomplishes this by increasing the initial required concrete strength while decreasing the strand spacing on wider (longer span) panels. Texas accomplishes the same thing with a single panel design that is slightly thicker and with a lower jacking stress, all the while with a lower concrete release strength. Notably, both the 2006 PCI *Manual for the Evaluation and Repair of Precast, Prestressed Concrete Bridge Products*, PCI-MNL-137[6] (PCI Bridge Committee 2006) and the 2018 PCINE *Guidelines for Resolution of Non-Conformances in Precast Concrete Bridge Elements*, PCINE-18-RNPCBE[7] (PCI Northeast Bridge Technical Committee 2018) suggest limiting the average prestress force over the end faces of panels to 750 psi, which is equivalent to $0.19f'_{ci}$ for a 4,000 psi release strength, as a means to prevent panel cracking.

Lower initial concrete compressive stresses can reduce the likelihood of cracking due to the prestressing, and Texas' method of doing this by not fully stressing the strands is interesting. Not only does Texas have a single panel design for all girder spacings versus New Hampshire's three designs but bursting and splitting stresses around individual strands will be reduced due to the

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[6] *Manual for the Evaluation and Repair of Precast, Prestressed Concrete Bridge Products*, 2006, Report No. MNL-137-06-E, Precast/Prestressed Concrete Institute, is not incorporated in the CFR Title 23. Therefore, this industry technical document is not a Federal requirement.

[7] *Guidelines for Resolution of Non-Conformances in Precast Concrete Bridge Elements, 2018,* Report No. PCINE-18-RNPCBE, Precast/Prestressed Concrete Institute Northeast, is not incorporated in the CFR Title 23. Therefore, this industry technical document is not a Federal requirement.

lower jacking force, and a lower concrete release strength can allow for faster panel production. And the full area of the strand can still be used for the composite deck reinforcing, even if the jacking force is less than the maximum allowable.

Only Colorado and Utah impart tensile stress in the tops of the panels for certain panel widths due to the strand eccentricity, staying within the AASHTO and PCI limit of $0.0948\sqrt{f'_{ci}}$ (ksi).

State-of-the-Practice

- AASHTO and PCI strand jacking force and minimum concrete release strength appear to provide satisfactory performance based on the experience of the six States. To further reduce the chances of panel cracking, consider:
  - Reducing the jacking force to $0.63f_{pu}$ and using a lower concrete compressive stress limit at release of $0.19f'_{ci}$.
  - Providing splitting reinforcing as described in the following section.

Panel Reinforcement

Mild reinforcing bars are typically used to reinforce partial-depth deck panels in the direction perpendicular to the strands (parallel to the bridge axis), generally as a form of distribution reinforcement. The *AASHTO Standard Specifications 2002* (23 CFR 625.4(d)(1)(iii)) recommends providing 0.11 square-inches of reinforcing bar area per foot of panel length, as does the 1988 *PCI Special Report JR-343*[2]. The 2017 *AASHTO LRFD-8 Bridge Design Specifications* (23 CFR 625.4(d)(1)(v)), though, do not include a minimum amount of reinforcing perpendicular to the strands, nor do the 2014 *Bridge Design Manual*[3] *or* the 2017 *PCINE Guidelines*[5].

However, the 2017 *AASHTO LRFD-8 Bridge Design Specifications* (23 CFR 625.4(d)(1)(v)) do state that bottom distribution reinforcing may be placed directly on top of the panels, in which case the secondary reinforcing in the panels would not need to meet the requirements for general deck distribution reinforcing that appears in both the 2002 *AASHTO Standard Specifications* (23 CFR 625.4(d)(1)(iii)), and the 2017 *AASHTO LRFD-8 Bridge Design Specifications* (23 CFR 625.4(d)(1)(v)).

Another potential use of mild reinforcing bars parallel to the span of the supporting girders is to function as splitting reinforcing near the panel ends. In this application, the splitting reinforcing is placed both above and below the strands, or only on the opposite side of the strand from the distribution reinforcing, in order to place bars perpendicular to the strand both above and below the strands ends.

Table 9. Reference mild reinforcement quantities.

Source	Longitudinal* Mild Reinforcement and position relative to strand	Longitudinal* Mild Reinforcement for Splitting and position relative to strand
PCI Special Report JR-343 (1988)[2]	0.11 in^2/ft.	Not specified
PCI Bridge Design Manual (2014)[3]	Not specified	Not specified
PCINE Guidelines (2017)[5]	Not specified	Not specified
AASHTO Standard Specifications (2002) (23 CFR 625.4(d)(1)(iii))	0.11 in^2/ft.	Not specified
AASHTO LRFD-8 Bridge Design Specifications (2017) (23 CFR 625.4(d)(1)(v)	Amount not specified for panels, but bottom distribution reinforcement may be placed directly on the top of the panels. For decks in general, $220/\sqrt{S} \leq 67\%$ of transverse reinforcement.	Not specified
Colorado DOT	#3 bar at 12.0" max. (0.11 in^2/ft.), above strands	None
Missouri DOT	#3 bar at 6.0" (0.22 in^2/ft.), above strands	2-#3 bars at each end, below strands
New Hampshire DOT	#4 bar at 6.0" (0.40 in^2/ft.), above strands	None
Tennessee DOT	#3 bar at 9.0" max. (0.15 in^2/ft.), alternating above and below strands	3-#3 bars at each end, alternating above and below strands
Texas DOT	#3 bar at 6.0" (0.22 in^2/ft.) max; or 3/8" unstressed strand at 4.5" (0.23 in^2/ft.) max; 1/2" unstressed strand at 6" (0.31 in^2/ft.) max.; or Deformed Welded wire 0.22 in^2/ft, D11 max Placed above or below strands	None
Utah DOT	#4 bar at 12.0" (0.20 in^2/ft.) max., above strands	None

* Oriented parallel to girder span

Colorado requires a minimum area of distribution reinforcing of 0.11 square-inches per foot, matching the 2002 *AASHTO Standard Specifications* (23 CFR 625.4(d)(1)(iii)). Tennessee requires 33 percent more steel, three States require 100 percent more steel, and New Hampshire requires almost four times as much steel. Four of the States place the distribution reinforcing above the strands, while Texas allows it to be placed above or below, and Tennessee alternates each distribution reinforcing bar above and below the strand. Therefore, to avoid having to place additional distribution reinforcing on top of the panels, it is suggested that the longitudinal steel meet the AASHTO Specifications provisions for distribution steel. This can be accomplished by adjusting the longitudinal reinforcing depending on the strand size and spacing, or by using a single value likely to cover all situations. Notably, three States require 0.22 square-inches per foot, which meets the AASHTO distribution steel provisions for all the six States. Tennessee requires slightly more (0.228 square-inches per foot) due to the very close spacing (3 inches) of the strands.

Given that typical practice is to not place a bottom mat of distribution steel on top of the panels in the field, it may be useful to size the secondary reinforcing in the panels to meet the AASHTO specification for distribution reinforcing. Because the distribution reinforcing relates directly to the primary reinforcing (67 percent, or slightly less for panels wider than 10'-9⅜"), the longitudinal reinforcing meets the AASHTO provisions for distribution reinforcing.

Only two of the six States specify splitting reinforcement. Missouri places two #3 reinforcing bars at 3-inch spacing under the strand at the panel ends, on the opposite side of the strands from the distribution reinforcement. Tennessee calls for an additional three #3 reinforcing bars spaced at 1.5-inches alternating above and below the strand at the panel ends.

While only two of the six States provide splitting tensile reinforcement, they are not the two States that use a lower initial concrete compressive stress limit. Tennessee, which provides splitting tensile reinforcement, does use a lower jacking force, but the resulting initial concrete stresses are some of the highest among the six States (relative to the initial concrete strength), about 92 percent of the AASHTO provision, partly due to it being the only State that allows a -0.25-inch tolerance without any strand eccentricity. Missouri's initial concrete stresses are only about 70 percent of the AASHTO provisions, but Missouri is the only State using a single panel design that is only 3.0-inches thick. Colorado uses a 3.0-inch thick panel but limits the width of that panel to 8.0-feet.

State-of-the-Practice:

- Providing minimum longitudinal distribution reinforcing in the panels of 0.22 square inches per foot may be a useful practice.
- Providing splitting tensile reinforcement when initial concrete compressive stresses exceed $0.19f'_{ci}$, and when using 3.0-inch thick panels that are more than 8-feet wide to minimize chances for formation of panel cracking colinear with the strands (Figure 8) may be a useful practice.

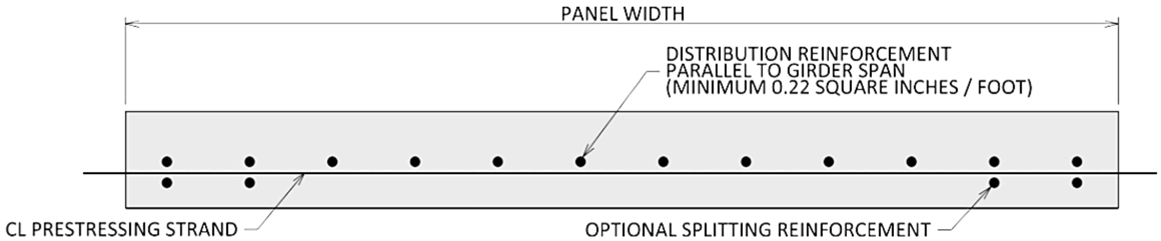

Figure 8. Illustration. Transverse section: panel distribution and splitting reinforcement.

Detailing Considerations

Strand or Rebar Extensions

Inconsistency on extending the strands or reinforcing bars beyond the panel ends exists. The 1988 *Special Report JR-343*[2] does not suggest extending the strands due to the added cost of not being able to produce panels by long line casting and saw cutting, citing research by Bieschke and Klingner published in 1982 and 1988 showing that strand extensions provided no benefit. However, the 2017 *PCINE Guidelines*[5] state that the strands should extend 4 inches minimum outside the panel ends.

The 2002 *AASHTO Standard Specifications* (23 CFR 625.4(d)(1)(iii)) do not discuss strand extensions, while the 2017 *AASHTO LRFD-8 Bridge Design Specifications* (23 CFR 625.4(d)(1)(v)) state that strand or reinforcing bar extensions are not needed, referencing the same research as the 1988 *PCI Special Report JR-343*[2]. However, the commentary notes that a lack of extended reinforcing may affect transverse load distribution due to a lack of positive moment continuity over the beams or may result in reflective cracking at the panel ends.

Four of the six States require strand or reinforcing bar extensions and a fifth State prefers them, while only one explicitly does not provide for any extensions. Hence, the majority of these States indicate there is a benefit in extending the reinforcing outside the panel ends over the girders. Because of this and the risk of reflective cracking at panel ends mentioned by the 2017 AASHTO LRFD-8 Bridge Design Specifications commentary (23 CFR 625.4(d)(1)(v)), strand or reinforcing bar extensions are presented as the state of the practice. While Texas only requires a 1.5-inch minimum extension, the other three States that specify the extension length call for a minimum of 3.0-inches (Figure 9). Even though New Hampshire and the 2017 *PCINE Guidelines*[5] call for 4-inch minimum extensions, the combined experiences of Missouri and Tennessee indicate that a 3-inch extension is enough.

Short extensions are not likely to provide significant development of strands or reinforcing bars. Projects have been constructed without them with no CIP cracking observed at the panel ends. There is no detriment to structural performance, but there is a lack of concurrence on the benefits of the extensions, and extensions could potentially add fabrication cost.

Table 10. Reference strand or reinforcing bar extension length.

Source	Strand or Reinforcing Bar Extension Length
PCI Special Report JR-343 (1988)[2]	Not recommended to extend strands or reinforcing bars
PCI Bridge Design Manual (2014)[3]	NA
PCINE Guidelines (2017)[5]	4.0” minimum strand extension
AASHTO Standard Specifications (2002) (23 CFR 625.4(d)(1)(iii))	Not specified
AASHTO LRFD-8 Bridge Design Specifications (2017) (23 CFR 625.4(d)(1)(v)	Not required, but commentary states that a lack of extended reinforcing may cause reflective cracking at panel ends.
Colorado DOT	Strand extension preferred; length not specified
Missouri DOT	3.0” min. strand extension
New Hampshire DOT	4.0” min. strand extension
Tennessee DOT	3.0” strand extension
Texas DOT	1.5” min. to 3.5” max. strand and/or bar extension
Utah DOT	No strand or bar extensions

Note that girder shear studs or shear stirrups may interfere with strand or reinforcing bar extensions, possibly making panel placement difficult. Coordination between the girder fabricator or shear stud installer and the panel fabricator can help avoid conflicts. Some conflicts can be avoided by bending the extended panel reinforcement or the girder shear studs or stirrups, or by selective removal of individual panel strand or reinforcement extensions.

Figure 9. Photo. Typical strand extension from PDDP panels.

State-of-the-Practice:

- Extending strands or reinforcing bars a minimum 1.5-inch (Figure 10) is the most common practice. Designers should specify adequate tolerance to avoid conflict with shear studs or stirrups.

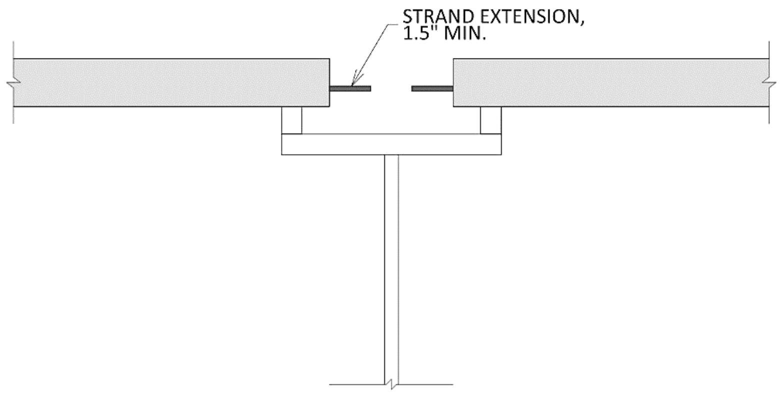

Figure 10. Illustration. Transverse section: strand extensions.

Chamfer Detail, Haunch Height, and Permanent Support Width

None of the PCI or AASHTO documents explicitly call for a bottom chamfer on the supported edges of the panels. All the PCI documents suggest permanently supporting the panels with 1-inch minimum thick grout, with the earlier documents, the 1988 PCI *Special Report JR-343*[2] and the 2014 PCI *Bridge Design Manual*[3] also allowing for the panels to be supported by the cast-in-place deck concrete with a minimum height of 1.5-inches. The 2002 *AASHTO Standard Specifications* (23 CFR 625.4(d)(1)(iii) is silent on how to permanently support the panels, while the 2017 *AASHTO LRFD-8 Bridge Design Specifications* (23 CFR 625.4(d)(1)(v) simply call for a continuous mortar bed or the cast-in-place deck concrete to permanently support the panels.

For the permanent support width, the 2017 AASHTO LRFD-8 Bridge Design Specifications (23 CFR 625.4(d)(1)(v) and the 2014 *PCI Bridge Design Manual*[3] do not provide information while the 1988 *PCI Special Report JR-343*[2] calls for a minimum support width of 1.5-inches (Figure 11) and the 2017 *PCINE Guidelines*[5] call for a permanent support width of 2.0-inches (Figure 12).

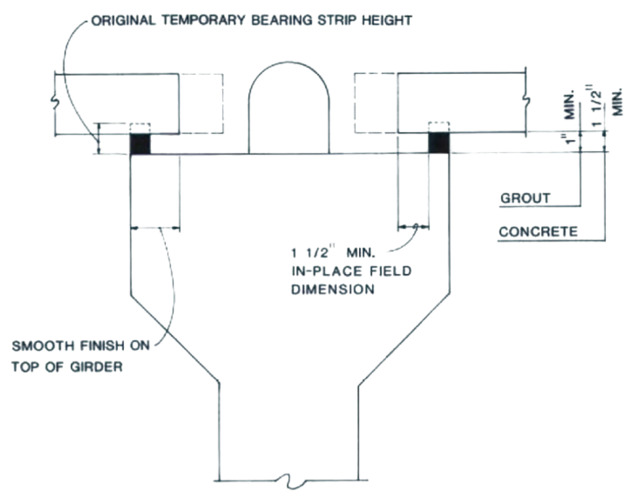

Figure 11. Illustration. Permanent panel support detail in 1988 PCI Special Report *JR-343*[2].

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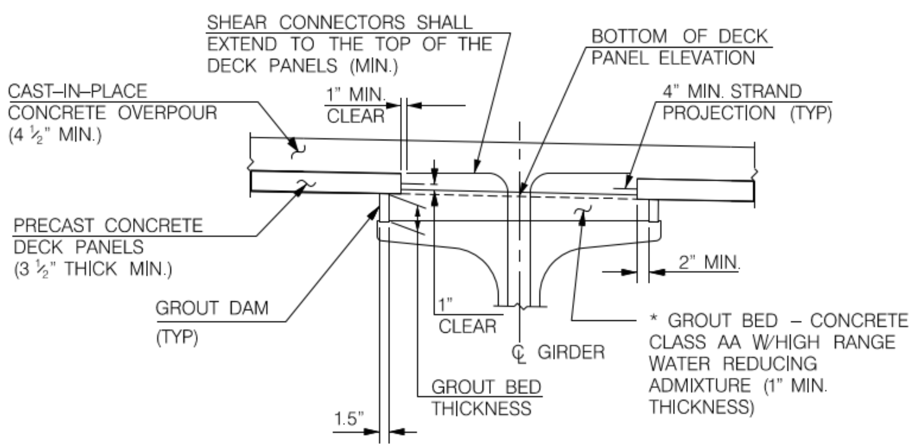

Figure 12. Illustration. Permanent support detail in 2017 PCINE Guidelines[5].

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Table 11. Reference chamfer, haunch, and support width dimensions.

Source	Deck Panel Bottom Chamfer	Haunch Height* and Permanent Support Material	Permanent Support Width**
PCI Special Report JR-343 (1988)[2]	Not specified	1.0” min. grout, or 1.5” min. deck concrete	1.5” min.
PCI Bridge Design Manual (2014)[3]	Not specified	1.0” min. grout, or 1.5” min. deck concrete	Not specified
PCINE Guidelines (2017)[5]	Not specified	1.0” min. grout bed	2.0” min. (details), 2.25” min. (guidelines)
AASHTO Standard Specifications (2002) (23 CFR 625.4(d)(1)(iii))	Not specified	Not specified	Not specified
AASHTO LRFD-8 Bridge Design Specifications (2017) (23 CFR 625.4(d)(1)(v)	Not specified	Continuous mortar bed or CIP deck concrete	Not specified
Colorado DOT	None	1.0” min. deck concrete; 1.5” min. haunch with concrete girders (1.0” vertical clearance with 0.5” tolerance)	2.0” min.
Missouri DOT	Optional ½” chamfer	1.0” min., 4.0” max. deck concrete	1.5” min.
New Hampshire DOT	None	1.0” min. grout bed	2.0” min.
Tennessee DOT	None	1.0” min. deck concrete	1.5” min. to 2.5” max.
Texas DOT	¾” chamfer	0.5” min., 6.0” max. deck concrete	1.5” min.
Utah DOT	None	1.75” min., 3.75” max deck concrete	2.0” min.

* Clearance beneath panel; **Panel extension beyond temporary support

The practice among the majority of the six States is to not provide a chamfer at the supported panel edges. Five of the six States use the cast-in-place deck concrete to permanently support the panels, with only New Hampshire using grout. Four of the States use a minimum haunch thickness of 1-inch, and three States use a minimum support width of 2-inches. However, Texas, the only State that requires a chamfer, uses a minimum haunch height of only 1/2-inch. The chamfer facilitates the shallower haunch by helping to ensure the CIP concrete flows under the panel edge. Given the majority of States using a 1-inch minimum haunch, this is considered to be the state-of-the-practice. However, given the large number of panels produced in Texas each year, providing a ¾-inch chamfer and 0.5-inch minimum has also been shown to be a successful practice.

Grout beds are used by several States. Other States, such as Texas, specify grout beds where there are very tall haunches, such as at the ends of girders with significant camber and where the deck profile varies from the girder profile. Nonetheless, using grout beds adds an extra construction step and an extra material to be placed. A majority of the States use the cast-in-place deck concrete to permanently support the deck panels, so this is considered the state-of-the-practice.

Finally, based on the practices of Missouri, Tennessee, and Texas, a minimum support width of 1.5-inches is considered as the state-of-the-practice. Support details as shown in each of the six States' respective standard details are included below.

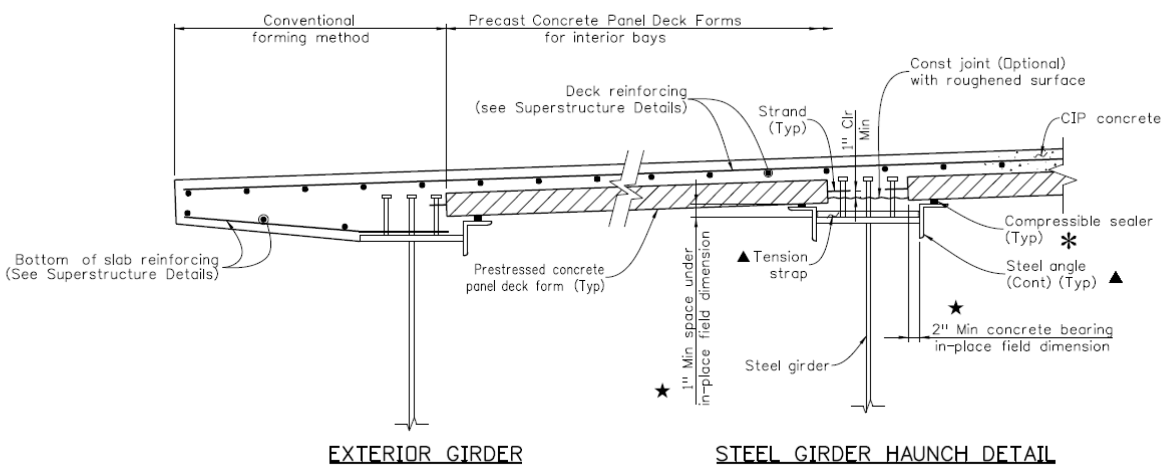

Figure 13. Illustration. Colorado panel support detail.
Source: Colorado Department of Transportation

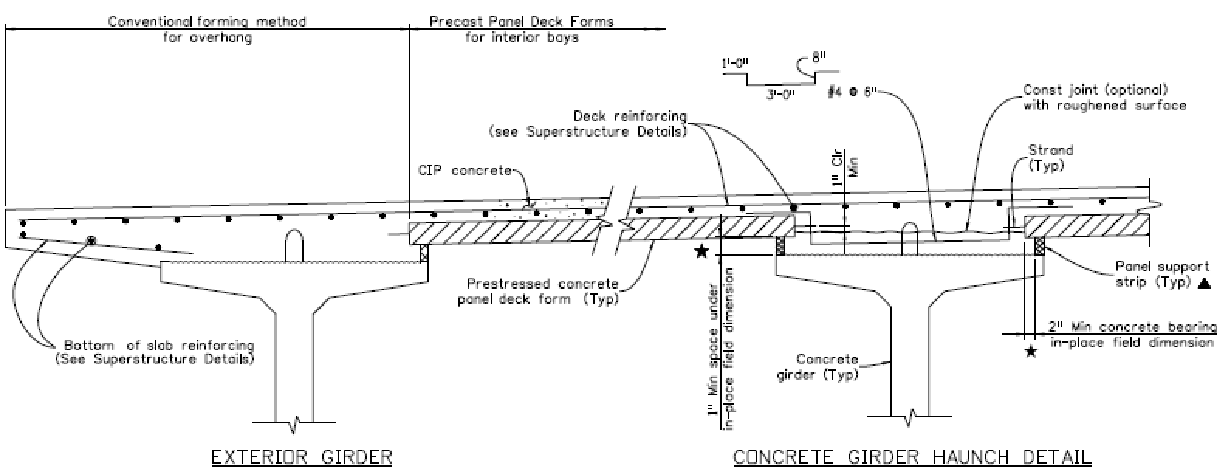

Figure 14. Illustration. Colorado alternate panel support detail.
Source: Colorado Department of Transportation

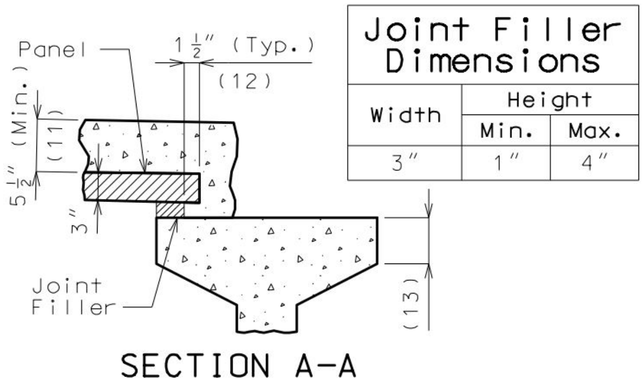

Figure 15. Illustration. Missouri panel support detail.
Source: Missouri Department of Transportation

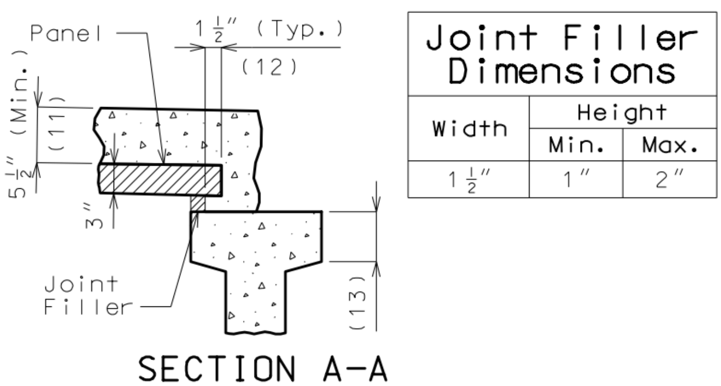

Figure 16. Illustration. Missouri alternate panel support detail.
Source: Missouri Department of Transportation

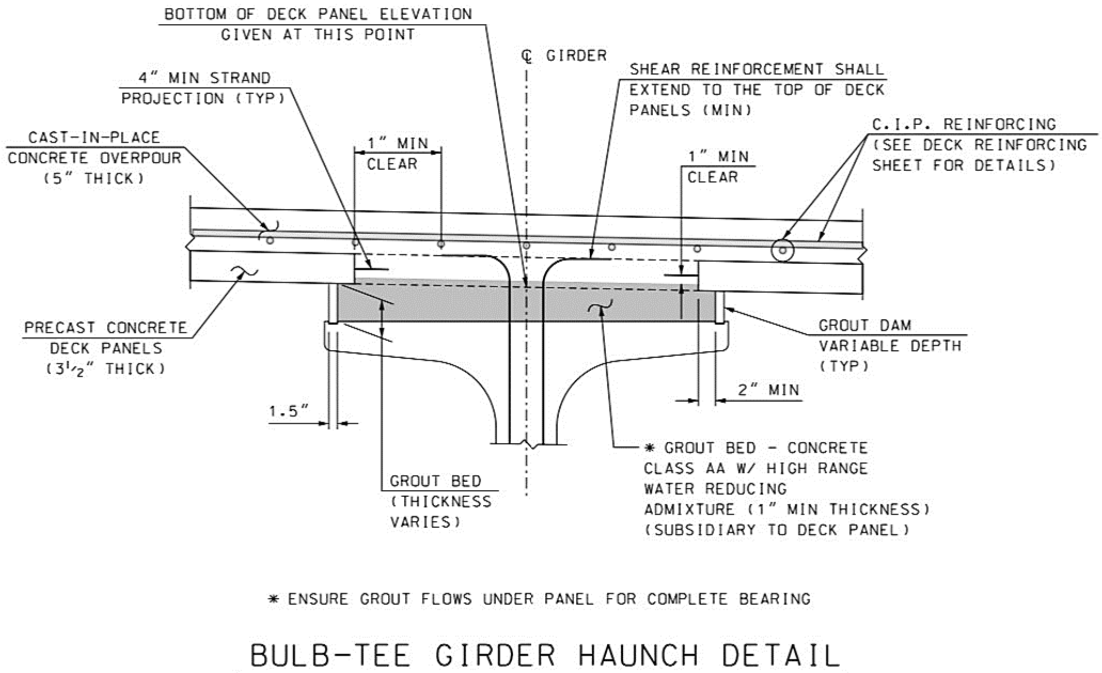

Figure 17. Illustration. New Hampshire panel support detail.
Source: New Hampshire Department of Transportation

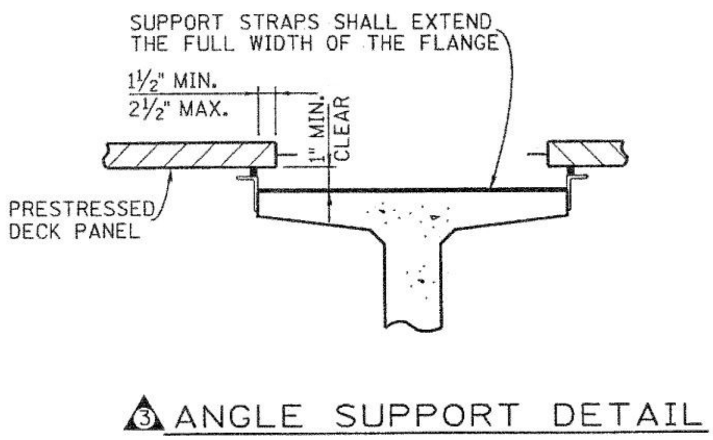

Figure 18. Illustration. Tennessee panel support detail.
Source: Tennessee Department of Transportation

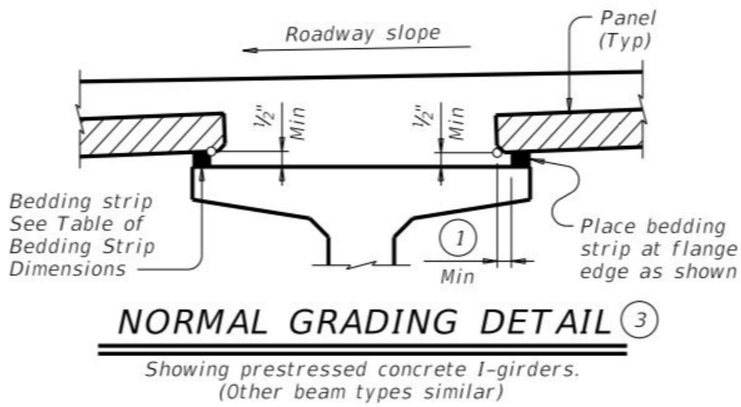

Figure 19. Illustration. Texas panel support detail.

Source: Texas Department of Transportation

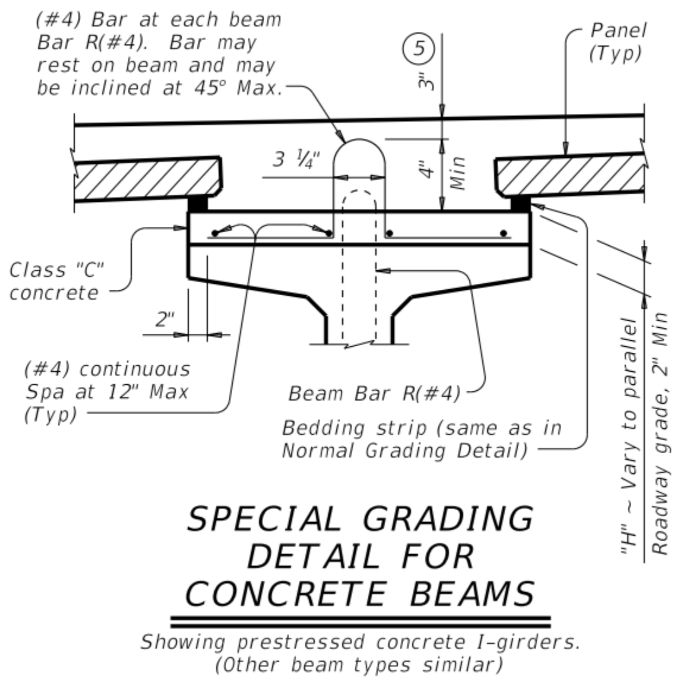

Figure 20. Illustration. Texas alternate panel support detail.

Source: Texas Department of Transportation

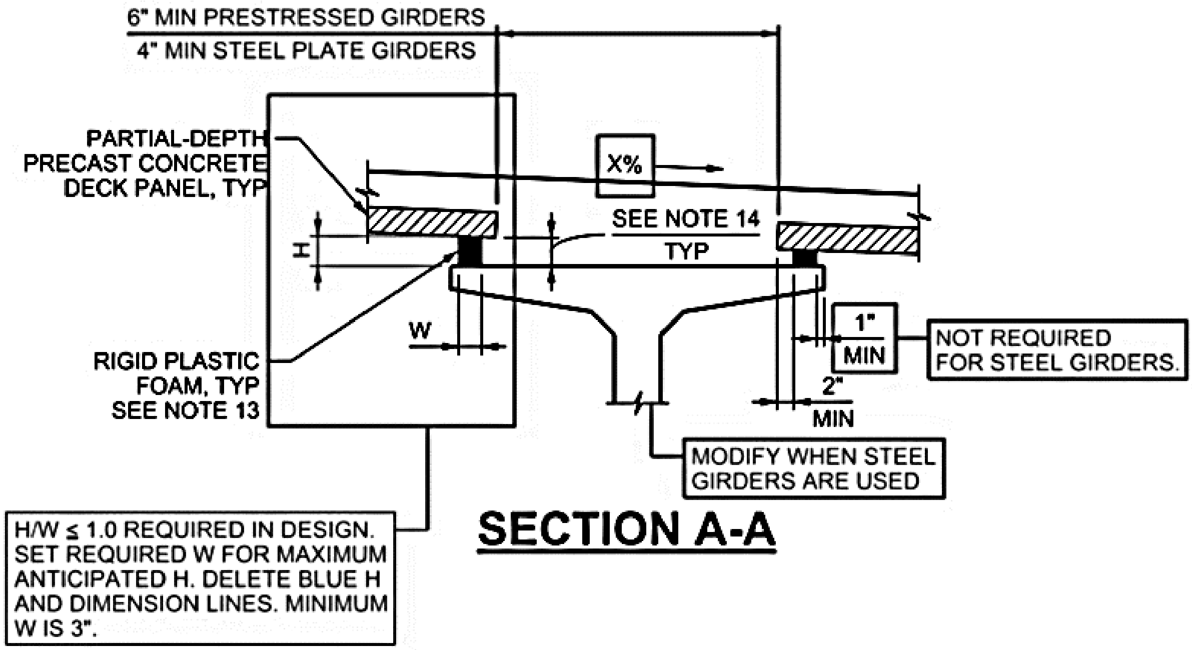

Figure 21. Illustration. Utah panel support detail.

Source: Utah Department of Transportation

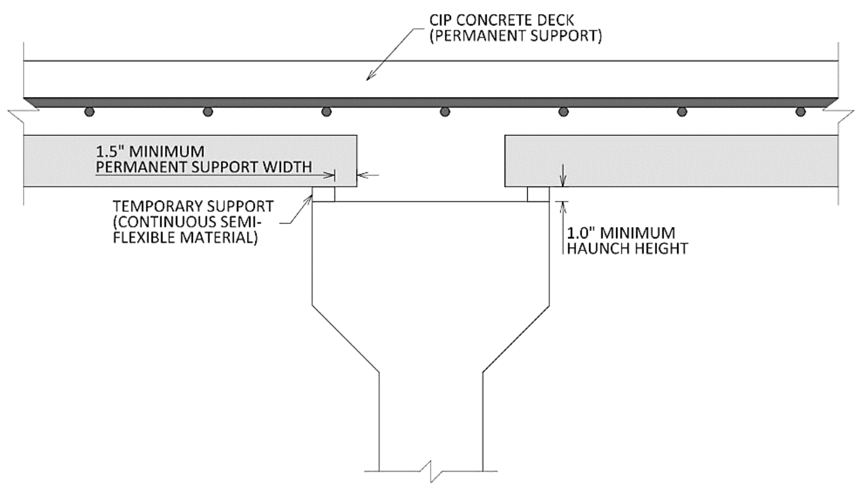

Figure 22. Illustration. Transverse section: haunch detail, no chamfer.

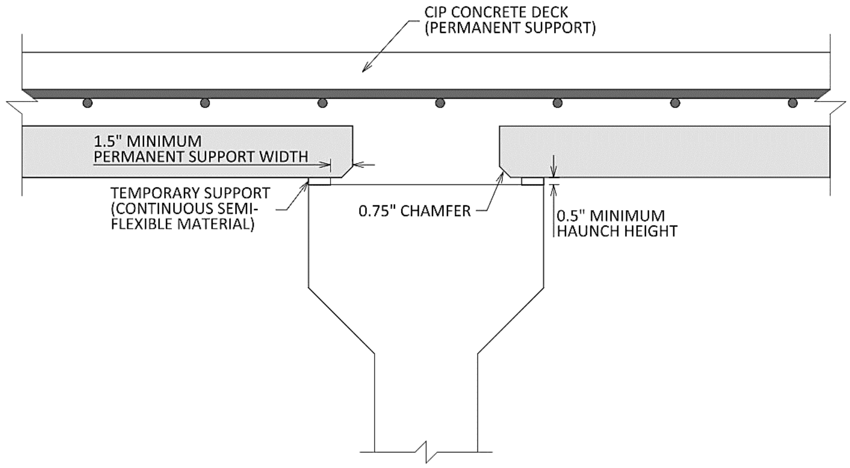

Figure 23. Illustration. Transverse section: haunch detail with chamfer.

State-of-the-Practice:

- Use of the cast-in-place deck concrete for the permanent support of the panels has been used by the six States.

- Minimum haunch heights, based on the practices of the six States, should be one of the following:
  - 1.0-inch (Figure 22), or
  - 0.5-inch with 0.75-inch chamfer on panel bottom corners at supports (Figure 23) as long as the size of the coarse aggregate in the CIP concrete is compatible with this small haunch height.

- Minimum support width should be 1.5 inches.

Temporary Panel Support Type and Width

The PCI and AASHTO documents provide various methods of temporarily supporting the panels, with the exception of the 2002 *AASHTO Standard Specifications* (23 CFR 625.4(d)(1)(iii) which is silent on the topic. Methods include compressible materials such as polystyrene, fiber board, neoprene, as well as steel setting or levelling screws (that are to be removed after the panels are permanently supported).

Like the PCI and AASHTO suggestions, the practices of the six States vary and often include options. All six States specify a flexible material for temporary support, while New Hampshire also allows levelling screws and requires them to be removed after the permanent support is in place. Tennessee uses a flexible temporary support material, but places that on top of steel structural support angles mounted to the edges of the girder flanges. This accommodates narrow girder flanges and facilitates taller haunches and allows a single detail to be used on steel and concrete girders. Colorado also provides a similar support detail. The objective of AASHTO, PCI, and all six States is to ensure that the temporary supports do not carry any load after the permanent support is in place, either because they are more flexible than the permanent support, or because they are removed.

Semi-flexible temporary supports can be advantageous because they do not need to be removed, saving on-site labor. They can also serve as grout dams or forms for the permanent support grout or concrete if they are made continuous (Figure 24). Conversely, setting or levelling screws should only be used in conjunction with grout beds since they should be removed, and the holes grouted prior to placement of CIP concrete topping.

Refer to the Panel Geometry section in Chapter 2 for permanent panel support details for each of the six States.

Table 12. Reference temporary support details.

Source	Temporary Panel Support Type	Temporary Panel Support Width
PCI Special Report JR-343 (1988)[2]	Removable: wood planks and steel angles or channels attached to side of girders Stay in place: strips of compressible material such as expanded polystyrene, fiberboard, or bituminous fiberboard	Not specified
PCI Bridge Design Manual (2014)[3]	Four screw-jack embedment (leveling screws) near panel corners	Not specified
PCINE Guidelines (2017)[5]	High-density expanded polystyrene strips with 55 psi min. compressive strength Leveling screws with a 1.7 pcf polyethylene foam seal	1.5”
AASHTO Standard Specifications (2002) (23 CFR 625.4(d)(1)(iii)	Not specified	Not specified
AASHTO LRFD-8 Bridge Design Specifications (2017) (23 CFR 625.4(d)(1)(v)	Setting screws, bituminous fiber boards, neoprene glands, etc., may be appropriate	Not specified
Colorado DOT	Steel angles with resilient material mounted to sides of girders High density extruded polystyrene foam insulation boards	2.0” min., height/width = 1.0
Missouri DOT	Preformed fiber expansion joint material or expanded or extruded polystyrene bedding with 60 psi min. compressive strength	1.5”
New Hampshire DOT	High density expanded polystyrene strips with 55 psi min. compressive strength Leveling screws with a 1.7 pcf polyethylene foam seal	Not specified
Tennessee DOT	Preformed fiber expansion joint filler or 50 durometer elastomeric bearing material, sitting on steel support angle mounted to edge of girder flange.	1.0”
Texas DOT	Extruded polystyrene with 40 psi compressive strength bedding strip	1.0” min., 3.0” max.
Utah DOT	Rigid plastic foam with 50 psi min. compressive strength	3.0” min., height/width ≤ 1.0

State-of-the-Practice:

- Continuous semi-flexible material has been used for the panel temporary support and can also serve as a grout dam or form to reduce labor (Figure 25) Material options include expanded polystyrene, rigid plastic foam, preformed fiber expansion joint material, or 50 durometer elastomeric rubber. Steel support angles mounted on girder flange can provide additional flexibility with narrow girder flanges or tall haunches.

- Grout beds for temporary support in situations with very tall haunches where typical temporary supports would not be stable (Figure 26) have been used.

Figure 24. Photo. Semi-flexible temporary supports.
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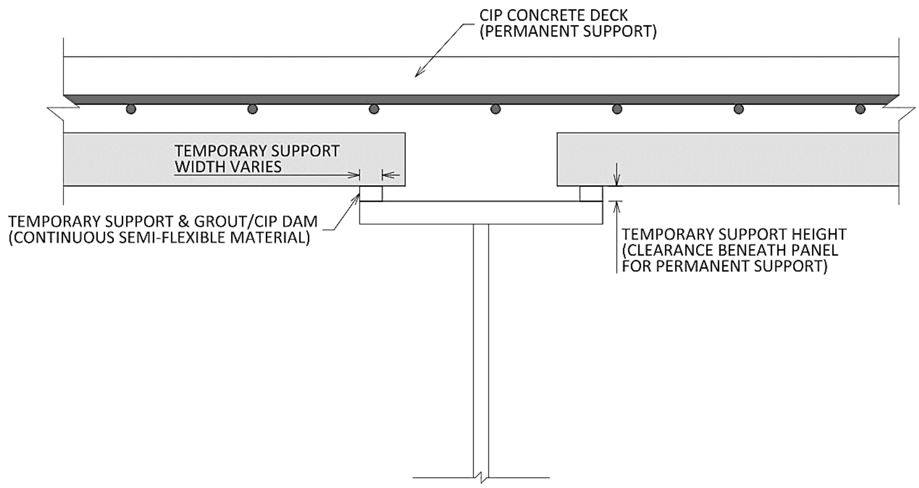

Figure 25. Illustration. Transverse section: temporary and permanent panel support.

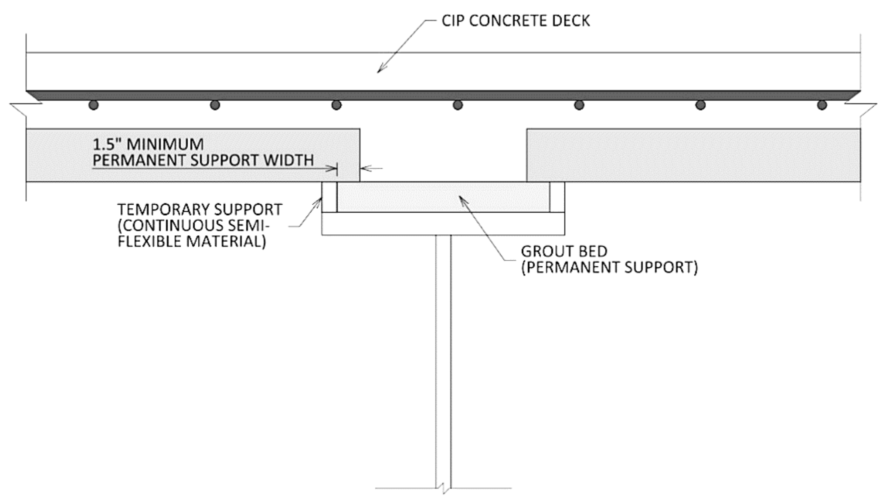

Figure 26. Illustration. Transverse section: haunch detail, optional grout dam.

Panel Concrete Mixture and Top Surface Finish

The 2017 AASHTO LRFD-8 Bridge Design Specification (2017) (23 CFR 625.4(d)(1)(v) does not specify the concrete mixture to be used in precast partial-depth panels, nor does the 2017 *PCINE Guidelines*[5]. The 1988 *PCI Special Report JR-343*[2] and the 2014 PCI *Bridge Design Manual*[3] only address the maximum aggregate size, with the 1988 PCI *Special Report JR-343*[2] specifying a maximum aggregate size of 0.75 inch and the 2014 *Bridge Design Manual*[3] stating that the maximum aggregate size should be smaller than the tightest space the concrete is to fill.

For the finish of the concrete on the top of the panel, all the AASHTO and PCI documents call for a roughened finish to ensure composite action between the panel and the cast-in-place deck concrete above. The only documents to specify a roughness amplitude are the 2017 *PCINE Guidelines*[5] (0.06 inch) and the 2014 *PCI Bridge Design Manual*[3] (0.05 inch to 0.075 inch), with both calling for the surface to be raked or broomed in the direction parallel to the strands. The Bridge Design Manual notes that the 0.05-inch to 0.075-inch amplitude roughness recommendation comes from research by Kumar (1996) and will achieve full composite action provided the nominal horizontal shear stress is less than 0.116 ksi.

The practices of the six States have varying criteria for the concrete mix for precast partial-depth panels, with several States limiting aggregate size to 0.75 inch and others allowing up to 1.0 inch. Texas, which has the 1.0-inch maximum aggregate size, places both the strand and the distribution reinforcing at a 6.0-inch spacing. Missouri uses the 0.75-inch maximum aggregate size and places the distribution reinforcing at a 6.0-inch spacing, but the strands are placed at a 4.5-inch spacing. Three States specify their standard precast concrete mix, one State specifies its standard deck concrete mix, and two States do not specify any mix, which would probably default to the States' precast concrete mixes.

All six States require roughening of the panel top surface to facilitate composite behavior with the cast-in-place deck concrete (Figure 27). However, whereas the PCI documents call for a roughness amplitude of approximately 1/16 inch, five of the six States require twice the amplitude, with Texas calling for an amplitude from 1/8 inch up to 7/32 inch. Only New Hampshire follows the 1/16-inch roughness amplitude.

Additionally, three States specify that the roughness shall be obtained via raking, brooming, or grooving, with specific orientations of the profile. Missouri and Tennessee require the linear ridges and grooves to be oriented perpendicular to the prestressing strand, while New Hampshire requires the profiling to be oriented parallel. The former might encourage higher composite behavior within the deck system for flexure of the girders with which the deck system is acting compositely. The later might encourage higher composite behavior of the slab system spanning between the girders. The 1998 *PCI Special Report JR-343*[2] suggests a broomed finish oriented parallel to the strands to minimize reductions in the panel section modulus.

It appears that the most important aspect is simply having roughness, with the pattern or orientation of the roughness being unimportant. However, it is worth noting that New Hampshire is the only State using a roughness of only 1/16-inch and the only State requiring a linear roughness pattern that is oriented parallel to the prestressing strand. This suggests that the concerns New Hampshire is addressing by the roughness pattern and orientation requirements can be addressed simply by using a larger amplitude without any specific pattern or orientation.

While reinforcing bars or strand extending above the panel top surface for lifting the panels can contribute to the interface shear capacity and composite behavior, only two States, Colorado and Texas, specify the lifting reinforcing size and placement. Missouri specifies it but allows it to be bent over if it interferes with the placement of the top reinforcing. Of the remaining States, Tennessee and Utah show lifting reinforcing but do not size it, and New Hampshire only shows lifting inserts, with all three requiring the contractor to design the lifting system. And again, New Hampshire only requires 1/16-inch surface roughness, unlike the other States that require twice as much. With most of the States not designing the lifting system, it appears that the surface roughness alone is regarded as enough to achieve composite behavior.

Table 13. Reference concrete mixture and top surface finish.

Source	Concrete Mixture	Top Surface Finish
PCI Special Report JR-343 (1988)[2]	0.75 » max. aggregate	Full composite action is achieved if the deck panel surface is roughened. Panels should be raked in the direction parallel to the strands to minimize the reduction in section modulus.
PCI Bridge Design Manual (2014)[3]	Max. aggregate should be smaller than tightest space concrete is to fill	0.05" to 0.075" amplitude broom finish to achieve full composite action
PCINE Guidelines (2017)[5]	Not specified	Broom roughened to an amplitude of approximately 0.06" parallel to strands
AASHTO Standard Specifications (2002) (23 CFR 625.4(d)(1)(iii)	Not specified	Roughened in such a manner as to ensure composite action
AASHTO LRFD-8 Bridge Design Specifications (2017) (23 CFR 625.4(d)(1)(v)	Not specified	Roughened in such a manner as to ensure composite action
Colorado DOT	Standard precast concrete mix (Class PS, No. 8 or larger aggregate); No entrained air required.	Rough finish 1/8" amplitude
Missouri DOT	Standard precast concrete mix (Class A-1, max 3/4" aggregate)	Scored finish with a depth of scoring 1/8" perpendicular to the prestressing strands
New Hampshire DOT	Not specified.	Broomed to a surface roughness of 0.06" parallel to the strands.
Tennessee DOT	Not specified.	Rake finish transverse to strands, 1/8" max depressions at 1" spacing
Texas DOT	Standard precast concrete mix (Class H), 1.0" maximum aggregate size	ICRI no. 6 to no. 9 surface profile = 125 mils (1/8") to 214 mils (7/32")
Utah DOT	Standard deck concrete mix (Class AA(LS)) per std. specs); General structural concrete mix (Class AAA(AE) [AA(AE)?] per drawings.	Leave concrete rough (1/8" amplitude)

State-of-the-Practice:

- Limiting maximum aggregate size to 1.0-inch has been a practice. Consider using a smaller maximum aggregate size where space between strands, reinforcing bars, and forms is limited to ensure the concrete can fully fill the forms.

- Roughening the top surface of the panel to a minimum amplitude of 1/8 inch to ensure composite behavior with the cast-in-place concrete is done by all six States. If nominal horizontal interface shear stresses are anticipated to exceed 0.116 ksi, consider a larger roughness amplitude and/or incorporation of projecting reinforcing to increase the interface shear capacity.

Figure 27. Photo.Roughened surface of PDDPs and top of girder.

Superstructure Type

All the PCI documents on partial-depth precast panels indicate that the panels can be used with concrete girders, steel girders, as well as other precast concrete sections such as spread box beams. Neither AASHTO documents mentions girder type for the panels.

All six States have details for using partial-depth precast concrete panels with steel girders and concrete girders, sometimes with separate panel support details for different types of precast concrete girders. Missouri also has details for using the panels with precast prestressed concrete spread box beams. Texas has details for using the panels on a host of additional precast prestressed concrete sections, including U beams, spread box beams, and slab beams. Tennessee also has details for using the panels with precast prestressed concrete box beams and with cast-in-place concrete beams and box girders.

Because the precast panels sit on top of the beams, using panels with precast spread box beams or slabs will result in a large amount of concrete on top of the slab or box beam top flange. Whereas a 4-inch or 5-inch topping might normally be used on top of these beams, with precast panels the depth of concrete on top of the beams will be the full deck thickness of 8-inches or more plus the panel support height or haunch.

Table 14. Reference girder types used with panels.

Source	Steel Girders	Concrete Girders	Other Types Specified
PCI Special Report JR-343 (1988)[2]	Yes	Yes	Steel box sections
PCI Bridge Design Manual (2014)[3]	Yes	Yes	Spread box beams
PCINE Guidelines (2017)[5]	Yes	Yes	Not specified
AASHTO Standard Specifications (2002) (23 CFR 625.4(d)(1)(iii)	Not specified	Not specified	Not specified
AASHTO LRFD-8 Bridge Design Specifications (2017) (23 CFR 625.4(d)(1)(v)	Not specified	Not specified	Not specified
Colorado DOT	Yes	Yes	None
Missouri DOT	Yes	Yes	Prestressed concrete spread box beams
New Hampshire DOT	Yes	Yes	None
Tennessee DOT	Yes	Yes	Prestressed concrete box beams, CIP concrete beams and box girders,
Texas DOT	Yes	Yes	Prestressed concrete U-beams, spread box beams (X-beams), and spread slab beams
Utah DOT	Yes	Yes	None

State-of-the-Practice

- Partial-depth precast concrete deck panels may be used with any type of steel or concrete girder. Use caution when considering partial-depth precast concrete deck panels with spread box beams and slab beams as the large total deck thickness over the beams (including the cast-in-place concrete, panel support height or haunch, and precast slab or top flange) may result in an inefficient structure.

Deck Geometry Considerations

Overhangs

Both the 2017 *AASHTO LRFD Specifications* (23 CFR 625.4(d)(1)(v)) and the 2002 *AASHTO Standard Specifications* (23 CFR 625.4(d)(1)(iii)) exclude the use of partial-depth precast concrete deck panels on overhangs. The 1988 *PCI Special Report JR-343*[2] and the 2017 *PCINE Guidelines*[5] do not discuss overhangs, since the typical panels are designed to be supported on two sides. The 2014 *PCI Bridge Design Manual*[3] indicates that conventional partial-depth panels should not be used on overhangs, but presents the first generation NUDECK panels, which are continuous across the full width of a bridge deck, including the overhangs. The reinforcing is continuous across the width, but the concrete is interrupted at the girders to allow for the cast-in-place topping to connect the panels with the girders. The first generation NUDECK panel, however, has not been widely used. This may be due to the increased complexity and the large size and weight of the panels.

Table 15. Reference panel overhang limitations.

Source	Panel Overhang	Precautions
PCI Special Report JR-343 (1988)[2]	Not discussed.	None
PCI Bridge Design Manual (2014)[3]	No overhangs with conventional panels. First-generation NUDECK panels are continuous across deck width and include overhangs.	None
PCINE Guidelines (2017)[5]	Not discussed.	N/A
AASHTO Standard Specifications (2002) (23 CFR 625.4(d)(1)(iii)	Panels span between girders or stringers only.	N/A
AASHTO LRFD-8 Bridge Design Specifications (2017) (23 CFR 625.4(d)(1)(v)	Overhangs not permitted.	N/A
Colorado DOT	No	N/A
Missouri DOT	No	N/A
New Hampshire DOT	No	N/A
Tennessee DOT	No	N/A
Texas DOT	Precast overhangs allowed, to 3.33 ft beyond design section for negative moment or 1.3 times girder depth.	Screed rail not allowed on precast overhang.
Utah DOT	No	N/A

All six States also prohibit the use of the partial-depth panels on overhangs. However, Texas has developed a special panel design for overhangs (Moyer 2017). The overhang panel sits on the overhang and the first interior bay, with the majority of the panel being full-depth except for a 1'-6" band around the panel that is only partial-depth, to allow for lap splices of the top reinforcing layer with cast-in-place concrete. Like the NUDECK, this panel has not been widely adopted because of the increased complexity and the size and weight.

Consequently, the overhangs of most decks constructed with partial-depth precast panels are constructed using full-depth cast-in-place concrete with traditional forming methods.

State-of-the-Practice

- Conventional partial-depth panels for deck overhangs are typically are not used. Constructing the overhangs using traditional full-depth cast-in-place construction methods is most common. Alternatively, the use existing, or new precast deck overhang products can be considered where they can provide advantages such as faster construction, safer construction, or lower cost.

Geometry Limitations or Precautions

Most, if not all, standard details and design tables for PDDPs assume that the girders are parallel to one another, and hence the panels have a constant width along the bridge length. This is an important consideration to obtain many of the economic benefits by allowing the mass production of the panels. While many of the principles for PDDP design apply to panels with non-parallel ends (not perpendicular to the panel strands or reinforcement), designers cannot rely on existing details and tables for such panels. Similarly, designers cannot rely on existing details and tables for curved bridges. Accordingly, the information presented is intended for bridge decks with straight and parallel girders.

Skewed bridges, however, are quite common and generally partial-depth precast concrete panels can be used on those bridges. Assuming the girders are straight and parallel, modifications to accommodate skew are only warranted at the bridge ends and possibly at intermediate piers. Typically, on skewed bridges, rectangular panels are placed across the bridge, with triangular or trapezoidal panels or full-depth cast-in-place deck only placed adjacent to the abutments and adjacent to intermediate piers with expansion joints or where access to pier diaphragms is needed for concrete placement. Both the 2017 *AASHTO LRFD Specifications* (23 CFR 625.4(d)(1)(v)) and the 2002 *AASHTO Standard* Specifications (23 CFR 625.4(d)(1)(iii)) allow sawing end panels to fit the skew. The 1988 *PCI Special Report JR-343*[2] suggests using trapezoidal or triangular panels without discussing how they would be fabricated. Using partial-depth panels on skewed bridges is not discussed in the 2014 *PCI Bridge Design Manual*[3]. While skewed end panels are not recommended by the 2017 *PCINE Guidelines*[5], some direction is given on skewed end panel design regarding panel support and strand shielding to prevent cracking on the acute end. None of the AASHTO and PCI documents discuss a limit on the skew angle for end panels when using partial-depth panels.

All six States allow the use of partial-depth precast deck panels on skewed bridges, with five of the six States permitting the use of skewed end panels. Three States impose limits on the use of skewed end panels based on the skew angle. New Hampshire limits the panel skew angle to 15 degrees maximum, Colorado limits the skew angle to 30degrees maximum, and Texas uses a maximum panel skew angle of 45degrees. Missouri and Tennessee do not impose a maximum skew angle for end panels.

Texas, Missouri, and Colorado allow skewed end panels to be saw cut from rectangular panels. However, Colorado only allows it where projecting reinforcing is not required. Saw cut end panels are not permitted in New Hampshire or Tennessee because projecting reinforcing at the skewed ends are required.

Three States also have requirements for support and/or additional reinforcing along the skewed panel edge to limit panel cracking. Missouri requires support on the skewed end until the cast-in-place concrete strength reaches 3,000 psi while Tennessee requires permanent support of the skewed panel ends. Texas does not require support of the skewed ends but requires that the panel transverse reinforcing be placed parallel to the skewed end with supplemental reinforcing placed parallel to the square end (Figure 28 and Figure 29).

Table 16. Reference panel skew limitations.

Source	Skew End Panel Limits*	Precautions
PCI Special Report JR-343 (1988)[2]	None	None
PCI Bridge Design Manual (2014)[3]	None	None
PCINE Guidelines (2017)[5]	None	If skewed end panels used, the designer should account for the skewed support of the panel and its effect on the reinforcing. Shielding of the strand in the acute end of the panel may be used to control potential cracking. Additional mild steel placed adjacent to shielded strands or along the skew may be needed to meet the design capacity of the deck. The use of non-prestressed skewed end panels may also be considered.
AASHTO Standard Specifications (2002) (23 CFR 625.4(d)(1)(iii)	None	None
AASHTO LRFD-8 Bridge Design Specifications (2017) (23 CFR 625.4(d)(1)(v)	None	None
Colorado DOT	Skewed end panels allowed for skews ≤ 30° at abutments only. Panels may be saw cut where projecting reinforcing is not required.	None
Missouri DOT	Skewed end panels allowed, no skew limit. Panels may be saw cut.	Support from diaphragm forms is required under skewed panel ends until CIP concrete has reached 3,000 psi compressive strength.
New Hampshire DOT	Skewed end panels allowed for skews ≤ 15°	None
Tennessee DOT	Skewed end panels allowed, no skew limit.	Skewed ends of panels are supported on end walls and bent caps.
Texas DOT	Skewed end panels allowed for skews ≤ 45°. Panels may be saw cut.	Primary (transverse) panel reinforcing (strand or reinforcing bar) placed parallel to skewed end panel edge, with supplemental mild reinforcing parallel to square edge in rectangular portion.
Utah DOT	Skewed end panels not allowed	None

* Skew angle measured relative to transverse line

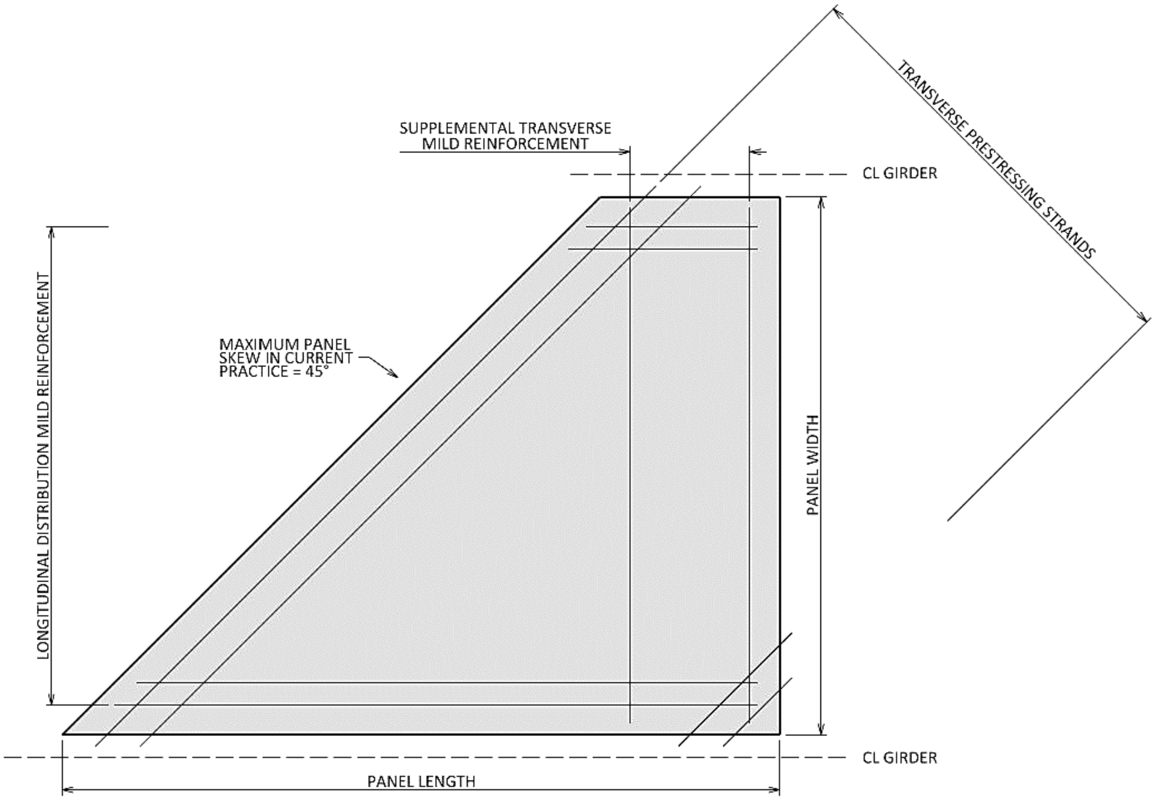

Figure 28. Illustration. Plan view: skewed end panel prestressing and reinforcement layout.

Figure 29. Photo. Skewed end panel with strands parallel with skewed end.

State-of-the-Practice:

- To maximize the benefits of partial-depth panels, use skewed end panels for skew angles up to 45 degrees. Skewed end panels with skew angles up to 30 may be saw cut from rectangular panels, however projecting transverse reinforcing into any adjacent cast-in-place concrete is preferred. For skews in excess of 30 degrees up to 45 degrees, support should be provided for the skewed edge of the panel on the pier or end diaphragm; alternatively orient the primary transverse reinforcing (strands or bars) with the skew and provide additional transverse mild reinforcing parallel to the square edge in the rectangular portion of the panel to limit cracking.

3. FABRICATION

Forming

Forms for PDDPs should be rigid to avoid deforming due to the weight of the wet concrete. The 1988 *PCI Special Report JR-343*[2] suggests using steel forms, and the 2002 *AASHTO Standard Specifications* (23 CFR 625.4(d)(1)(iii)) says the forms should be simply unyielding (Figure 30). The 1988 *PCI Special Report JR-343*[2] also suggests cleaning the forms after each use and spraying water to cool the forms, while the 2002 *AASHTO Standard Specifications* (23 CFR 625.4(d)(1)(iii)) note that several PDDP panels can be cast in a continuous line and suggests space be left between the panels to permit the cutting of the tendons. The other PCI and AASHTO documents do not address forms for PDDPs.

Figure 30. Photo. Forming of PDDPs

The practices of the six States generally follow the AASHTO and PCI provisions. Three States specify the use of oil or form releasing agents to facilitate removing the panels from the forms.

Missouri requires metal forms but prohibits the use of aluminum, due to the potential for concrete to corrode aluminum.

Table 17. Reference precast panel forming details.

Source	Precast Panel Forming
PCI Special Report JR-343 (1988)[2]	Steel forms, clean forms after each use.
PCI Bridge Design Manual (2014)[3]	Not specified
PCINE Guidelines (2017)[5]	Not specified
AASHTO Standard Specifications (2002) (23 CFR 625.4(d)(1)(iii)	Cast on unyielding beds or pallets. Several units may be cast in one continuous line and stressed simultaneously. Sufficient space shall be left between units to permit cutting of tendons
AASHTO LRFD-8 Bridge Design Specifications (2017) (23 CFR 625.4(d)(1)(v)	Not specified
Colorado DOT	Forms shall be kept clean, smooth, and adjusted to minimize form finish irregularities. Treat forms with form release agent
Missouri DOT	Exterior forms shall be metal other than aluminum.
New Hampshire DOT	Side forms shall be steel. Forms shall be cleaned before each use. Several units may be cast in one continuous line and stressed simultaneously. Sufficient space shall be left between units to permit cutting of tendons
Tennessee DOT	Metal forms. Treat forms with oil before placing concrete.
Texas DOT	Clean forms thoroughly before each use. Treat forms with form-release agent.
Utah DOT	Clean forms thoroughly before each use.

Information for forming precast concrete can also be found in the 1999 PCI *Manual for Quality Control for Plants and Production of Structural Precast Concrete Products,* MNL-116, Fourth Edition[8] (PCI Plant Certification Committee 1999).

State-of-the-Practice:

- Consider the suggestions of the 1999 *PCI Manual for Quality Control MNL-116*[8], in addition to the following based on the experience of the six States:
    - The use of steel forms on the panel sides and supporting the bottom forms with steel will provide a smooth and unyielding platform.

[8] *Manual for Quality Control for Plants and Production of Structural Precast Concrete Products, Fourth Edition,* Report No. MNL-116-99, Precast/Prestressed Concrete Institute, is not incorporated to the CFR Title 23. Therefore, this industry technical document is not a Federal requirement.

- The use of form releasing agents in the forms will facilitate removal of the cured panels and will simplify cleaning of the forms prior to each use.
- When casting multiple panels in a line, consideration should be made for leaving enough space between individual panels to cut the strands and allow for strand extensions. If not using strand extensions, panels can be sawcut from a continuous pour.

Placement of Prestressing and Reinforcing Elements & Lifting Hooks

The 1988 PCI *Special Report JR-343*[2] and the 2014 PCI *Bridge Design Manual*[3] provide some provisions for the placement of strand and reinforcing bars in the panels, while the 2017 AASHTO LRFD-8 Bridge Design Specifications (23 CFR 625.4(d)(1)(v)) and the PCINE *Guidelines*[5] do not provide information on the topic. The 1988 *Special Report JR-343*[2] states that the steel headers should be securely attached to the forms before concrete placement and strand chairs should be located at sufficient intervals to maintain the strand position. The 2014 PCI *Bridge Design Manual*[3] states that the mild reinforcing should be placed after the strands are tensioned. The six States are generally silent on these issues, except for New Hampshire which echoes the 2014 PCI *Bridge Design Manual*[3] by stating that the mild reinforcing shall be placed after the strands have been stressed.

All the PCI and AASHTO documents have recommendations about lifting devices. The 2017 *PCINE Guidelines*[5] simply call for four lifting devices per panel, while the *1988 PCI Special Report JR-343*[2] adds that the devices are typically located near the four corners. The 2014 *PCI Bridge Design Manual*[3] states that the locations of the lifting devices should ensure that the panel concrete stresses remain within allowable limits during handling. The 2002 *AASHTO Standard Specifications* (23 CFR 625.4(d)(1)(iii)) has a very similar provision, while the 2017 *AASHTO LRFD-8 Bridge Design Specifications* (23 CFR 625.4(d)(1)(v)) simply state that any State requirements or restrictions on lifting devices should be shown in the contract documents.

Among the six States, Colorado and Texas specify two #3 reinforcing bars placed parallel to and at the same level as the strands, with two inverted U-bends rising above the top of the panel a distance of 2.0 to 2.25 inches (Figure 31). Missouri specifies a similar system, but with greater flexibility for lifting options, placing multiple #3 reinforcing bars at a 3-feet-0-inches maximum spacing, and with three inverted U-bends in each bar. The remaining three States leave the design and placement of lifting devices to the responsibility of the contractor, with Utah simply specifying that there should be four lifting devices per panel.

The advantages of using embedded rebar with inverted U-bends, versus methods that rely on embedded lifting attachments or simply holes in the panel, are two-fold. First, there are no holes that need to be grouted after the panels are set in place. Secondly, the lifting rebar provides interface shear capacity, increasing the factor of safety on composite behavior with the cast-in-place topping concrete. One disadvantage of the rebar with the inverted U-bends is that the bends can interfere with the field-placed top reinforcing bar mat. However, if the top reinforcing cannot be adjusted to avoid interference, the U-bends can be selectively bent or cut without any negative consequences.

Table 18. Reference strand and reinforcement placement details and lifting devices.

Source	Placement of Strands and Reinforcement	Lifting Devices
PCI Special Report JR-343 (1988)[2]	-Steel headers securely fastened to soffit forms for proper geometry control -Chairs located at sufficient intervals	Lifting devices near the four corners are generally used
PCI Bridge Design Manual (2014)[3]	-Mild reinforcement should be detailed for placement after strands have been placed and tensioned	Location of lifting devices should ensure that concrete stresses remain within allowable limits during handling
PCINE Guidelines (2017)[5]	Not specified	Four lifting devices per panel
AASHTO Standard Specifications (2002) (23 CFR 625.4(d)(1)(iii)	Not specified	Location of lifting devices should be such that there will be no damaging bending or torsional forces during handling
AASHTO LRFD-8 Bridge Design Specifications (2017) (23 CFR 625.4(d)(1)(v)	Not specified	Requirements or restrictions for lifting devices shall be shown in the contract documents
Colorado DOT	Not specified	Mandatory installation of two #3 bars with two inverted U-bends each for lifting; bars placed at same elevation as strands and inverted U-bends to extend 2" above top of panel; placed 1'-6" from edge of panel
Missouri DOT	Not specified	#3 bars with three inverted U-bends each for lifting; bars spaced at max 3'-0"; bars placed at same elevation as strands and inverted U-bends to extend 2.25" above top of panel; placed min 8.5" & max 1'-6" from edge of panel
New Hampshire DOT	Mild reinforcement placed after stressing of strands	Lifting methods and locations to be determined by fabricator
Tennessee DOT	Not specified	Design of lifting straps to be responsibility of Contractor
Texas DOT	Not specified	Two #3 bars with two inverted U-bends each for lifting; bars placed at same elevation as strands and inverted U-bends to extend 2" above top of panel; placed min 6" & max 1'-6" from edge of panel
Utah DOT	Not specified	Mandatory installation of lifting devices, four devices per panel; lifting methods and locations to be determined by fabricator

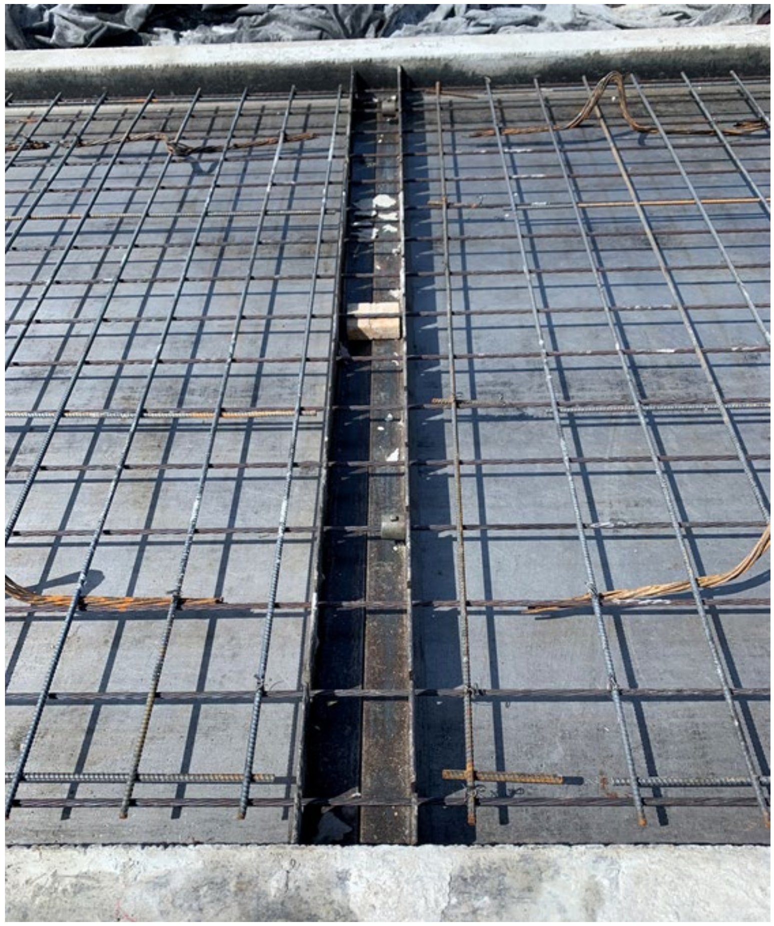

Figure 31. Photo. Placing of reinforcement and lifting loops for PDDPs

State of the Practice:

- The use of steel prestressing headers attached to the panel forms and regularly spaced strand chairs will securely maintain the strand position prior to panel concrete placement.
- Consider detailing the panel mild reinforcing to be placed after the strands are placed and tensioned.
- The use of embedded mild reinforcing bars with inverted U-bends that rise above the deck to provide a minimum of four lifting attachments is advantageous (Figure 32).
- Lifting attachments and locations should be designed to ensure that the panel concrete stress limits are not exceeded during lifting and handling.

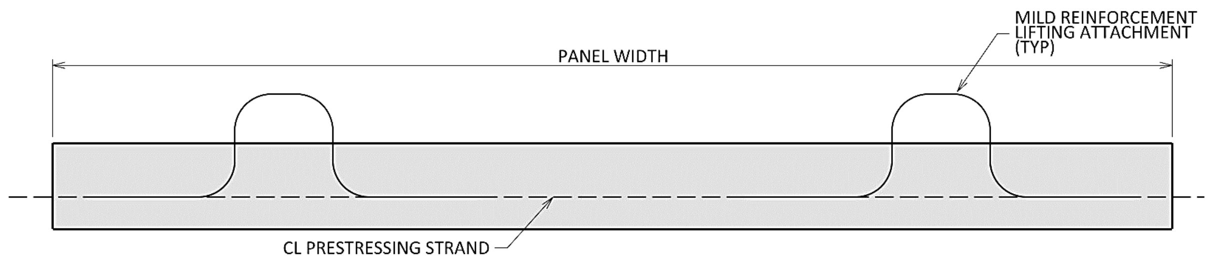

Figure 32. Illustration. Transverse section: lifting attachments.

Placement of Concrete

Neither the 2017 AASHTO LRFD-8 Bridge Design Specifications (23 CFR 625.4(d)(1)(v)) or the 2002 AASHTO Standard Specifications (23 CFR 625.4(d)(1)(iii)) includes provisions for the placement of the panel concrete. The non-binding 1988 *PCI Special Report JR-343*[2] suggests cooling the forms just prior to concrete placement by spraying them with water. However, the 1999 *PCI Manual for Quality Control MNL-116*[8] contains many provisions specific to placement of precast concrete.

All six States place limits of temperature on the concrete or the formwork during concrete placement. Five States limit the concrete or form temperature during concrete placement to 90 degrees F, while Texas follows the 1999 *PCI Manual for Quality Control MNL-116*[8] with a 95 degree F limit. Colorado places additional limits on the form temperature prior to placing the concrete, and Utah prohibits concrete placement when the ambient temperature is 40 degrees F and decreasing.

Five of the States specify mechanical vibration of the concrete to ensure good consolidation. Texas also limits the time between batching and placement to 30 minutes if the concrete is not agitated during transport, or 60 minutes if it is agitated during transport.

Table 19. Reference panel concrete placement details.

Source	Placement of Panel Concrete
PCI Special Report JR-343 (1988)[2]	Cool forms by spraying with water
PCI Bridge Design Manual (2014)[3]	Not specified (no special procedures are recommended, refer to 1999 PCI *Manual for Quality Control* MNL-116[8] for concreting procedures)
PCINE Guidelines (2017)[5]	Not specified
AASHTO Standard Specifications (2002) (23 CFR 625.4(d)(1)(iii)	Not specified
AASHTO LRFD-8 Bridge Design Specifications (2017) (23 CFR 625.4(d)(1)(v)	Not specified
Colorado DOT	-Place concrete within 90 minutes after batching -Min 50°F, max 90°F concrete temperature during placement -Inner form temperature within 40°F of concrete temperature at time of placement -Min 32°F, max 130°F inner form temperature at time of placement -Inner form free of ice at time of placement -Consolidate through mechanical vibrating
Missouri DOT	-Max 90°F concrete temperature during placement -Max 90°F forms and reinforcing steel temperature during placement
New Hampshire DOT	-Max 90°F concrete temperature during placement -Consolidate through mechanical vibrating
Tennessee DOT	-Place concrete immediately after mixing -Max 90°F form temperature during placement -Do not begin concrete placement when ambient temperature is below 26°F -Consolidate concrete with vibrators and spading tools
Texas DOT	-Place concrete within 30 minutes after batching if non-agitated delivery, or within 60 minutes after batching if agitated delivery -Min 50°F, max 95°F concrete temperature during placement -Consolidate through mechanical vibration
Utah DOT	-Min 50°F, max 90 F concrete temperature during placement -Min 36°F, max 95° contact surfaces during placement -Do not begin concrete placement when ambient temperature is 40°F and decreasing -Consolidate through mechanical vibration

State-of-the-Practice:

- Consider the suggestions of the 1999 *PCI Manual for Quality Control MNL-116*[8], in addition to the following based on the experience of the six States:
    - Limit the temperature of the concrete during placement to 90 degrees F maximum.
    - Consolidate the concrete with mechanical vibration.

Top of Panel Surface Finish (Texture)

See the Design Parameters section in Chapter 2 for information and suggestions on the panel top surface texture.

Curing

The 1988 PCI *Special Report JR-343*[2] and the 2014 PCI *Bridge Design Manual*[3] note that PDDPs, and precast structural products in general, can be cured by normal methods (Figure 34 & Figure 34), typically meaning without applied heat, or by various accelerated methods involving added heat, such as steam, convection, or radian heat. The 2002 *AASHTO Standard Specifications* (23 CFR 625.4(d)(1)(iii)) contains similar language. The decision to accelerate curing typically is based on the initial concrete strength, the ambient temperature, and the rate at which a precaster wishes to reuse the forms. Given that PDDPs typically need relatively low initial concrete compressive strengths of around 4 ksi, normal curing may be enough for a precaster who wants to turn over the forms once per day. However, during colder ambient temperatures, or for higher strengths or a more rapid turnover, precasters may elect to use accelerated curing methods.

When using accelerated curing, the temperature of the concrete should be monitored to prevent it from rising too high or too rapidly. The 2014 PCI *Bridge Design Manual*[3] states maximum temperatures between 150 degrees F and 180 degrees F. The reason for the 150 degrees F temperature limit is to avoid delayed ettringite formation (DEF). DEF occurs when excess sulfate accumulates in the concrete mix leading to the eventual formation of the mineral ettringite in the mature concrete. The ettringite can fill entrained air pockets and exert internal expansive forces that damage the concrete. The formation of DEF is also dependent on the chemistry of the cementitious materials and is less likely to occur in concrete made with pozzolans such as fly ash and slag cement that typically contain silica, in which case the 2014 PCI *Bridge Design Manual*[3] permits a maximum concrete curing temperature of 170 degrees F. Furthermore, if the precast elements are to be installed in a dry environment or only subject to infrequent wetting, curing temperatures of up to 180 degrees F are permitted. The 2017 *AASHTO LRFD-8 Bridge Design Specifications* (23 CFR 625.4(d)(1)(v)) specifies a maximum curing temperature of 160 degrees F to prevent DEF.

The 1999 *PCI Manual for Quality Control MNL-116*[8] provides a limiting concrete curing temperature of 180 degrees F, noting that if the potential for DEF exists, the temperature should be limited to 158 degrees F. Criteria for the rate of increase of heat is also given, as a rapid rise in temperature can also be damaging to the concrete.

Figure 33.Photo. Curing of PDDPs using plastic sheeting.

Figure 34.Photo.PDDPs after removal of plastic sheeting.

Source: Texas Department of Transportation

Table 20. Reference panel concrete curing method and temperature limits.

Source	Precast Panel Concrete Curing Methods	Concrete Curing Temperature Limits
PCI Special Report JR-343 (1988)[2]	Accelerated heat curing methods or natural curing	Not specified
PCI Bridge Design Manual (2014)[3]	Normal curing or accelerated curing by convection, with radiant heat, with steam, and with electric heating elements	Max: 150°F to 180°F
PCINE Guidelines (2017)[5]	Not specified	Not specified
AASHTO Standard Specifications (2002) (23 CFR 625.4(d)(1)(iii)	Curing by water, steam, or radiant heat method	Not specified
AASHTO LRFD-8 Bridge Design Specifications (2017) (23 CFR 625.4(d)(1)(v)	Not specified	Max: 160°F
Colorado DOT	Keep concrete surfaces moist and cure in enclosure that retains heat and moisture	Min: 50°F Max: 160°F
Missouri DOT	Curing by steam or complete submersion in water	Max: 160°F
New Hampshire DOT	Curing by saturated cover or low-pressure steam	Min: 68°F Max: 160°F
Tennessee DOT	Water curing or steam curing	Do not use water curing when ambient temperature is expected to drop below 45°F.
Texas DOT	Water curing, moisture retention curing, membrane curing, accelerated steam curing, and other accelerated curing methods upon approval	Min: 50°F Max: 150°F to 170°F
Utah DOT	Water method, waterproof cover method, or steam or radiant heat method	Min: 50°F Max: 160°F

The curing practices of the six States are like the PCI suggestions. All six States allow normal curing, where moisture loss is minimized by a membrane, a continuous application of water, or submersion in water; and allow accelerated curing methods involving the application of heat or steam. Four of the States specify a maximum temperature of 160 degrees F, in accordance with the 2017 AASHTO LRFD-8 Bridge Design Specifications (23 CFR 625.4(d)(1)(v)). Texas limits the maximum temperature to 150 degrees F, unless various mix design options including pozzolans are used, in which case the maximum temperature limit is increased to 170 degrees F, in accordance with the 2014 *PCI Bridge Design Manual*[3].

Three States specify a minimum concrete temperature of 50 degrees F. New Hampshire uses a minimum of 68 degrees F and Tennessee simply prohibits the use of water curing when the ambient temperature is expected to drop below 45 degrees F.

State-of-the-Practice

- Cure panels using normal or accelerated curing methods depending on the desired time for removing panels from the forms.

- Consider following the suggestions of the 1999 *PCI Manual for Quality Control MNL-116*[8] for curing, with the following modifications:

    - Maximum concrete curing temperature should be 160 degrees F
    - If enough pozzolans are incorporated in the concrete mix to make the occurrence of delayed ettringite formation (DEF) unlikely, the maximum concrete curing temperature may be increased to 170 degrees F

Strand Detensioning Considerations

The *1988 PCI Special Report JR-343*[2] and the 2014 *PCI Bridge Design Manual*[3] each stipulate that strand de-tensioning or release should be done gradually and can be done by hydraulic transfer of the force or by flame cutting of the strands. If the strands are to be detensioned by saw cutting, the 1988 PCI Special Report *JR-343*[2] suggests monitoring strand slippage. The 2014 *PCI Bridge Design Manual*[3] notes that releasing the prestressing force gradually typically leads to less cracking of the panel ends. The 2017 AASHTO LRFD-8 Bridge Design Specifications (23 CFR 625.4(d)(1)(v)), the 2002 AASHTO Standard Specifications (23 CFR 625.4(d)(1)(iii)), and the 2017 *PCINE Guidelines*[5] do not discuss strand detensioning.

The strand detensioning practices of the Six States generally follows the PCI provisions with additional criteria generally found in the 1999 *PCI Manual for Quality Control MNL-116*[8]. In addition to calling for a gradual release of the prestressing, three States specify that the prestressing release should be done symmetrically to minimize the amount of prestressing eccentricity. Two States specify that if accelerated curing is used, the prestressing should be released while the concrete is still warm and moist. Three States give parameters for heat or flame cutting. New Hampshire and Texas require both ends of a strand to be cut simultaneously, with New Hampshire further specifying that a minimum length of 5-inches of strand should be heated at each end to prevent the strand from suddenly breaking all at once and applying a shock to the panel. Tennessee calls for a minimum length of 3-inches to be heated to prevent strand snapping. Finally, two States require that the sequence of strand release be shown on the shop drawings.

Table 21. Reference strand detensioning details.

Source	Precast Panel Strand Detensioning
PCI Special Report JR-343 (1988)[2]	-Release prestress force gradually -Cut strands slowly with acetylene torches -Monitor strand slippage when strands detensioned by saw cutting
PCI Bridge Design Manual (2014)[3]	-Hydraulic transfer or transfer by flame cutting -Gradual release of force (hydraulic transfer) often leads to less end cracking
PCINE Guidelines (2017)[5]	Not specified
AASHTO Standard Specifications (2002) (23 CFR 625.4(d)(1)(iii)	Not specified
AASHTO LRFD-8 Bridge Design Specifications (2017) (23 CFR 625.4(d)(1)(v)	Not specified
Colorado DOT	Not specified
Missouri DOT	-Cut or release strands in manner that produces the least eccentricity of the load -If steam curing used, release strands when members still warm -Sequence of release to be performed as indicated on shop drawings
New Hampshire DOT	-Gradual release of jack pressure or cut individual strands -If individual strands released by heat cutting, heat both ends of strand simultaneously and minimum length of 5" heated to prevent shock or snap -If heat curing used, release strands when concrete is still warm and moist
Tennessee DOT	-Multiple strand release or single strand release method -If multiple strand release, gradually and simultaneously release symmetrical group or all strands -If single strand release, heat with low oxygen flame along minimum length of 3"
Texas DOT	-Multiple strand detensioning or single-strand flame detensioning -Flame-release strand at both ends simultaneously using symmetrical sequence
Utah DOT	-Sequence of release to be performed as indicated on shop drawings

State-of-the-Practice

- Consider the suggestions of 1999 *PCI Manual for Quality Control MNL-116*, in addition to the following based on the experience of the six States:

    - If concrete is heat-cured, detension prestressing while the concrete is still warm and moist.
    - For single-strand detensioning, release both ends of a strand simultaneously. Release strands in a symmetric pattern about the panel axis. For heat or flame cutting, heat a length of 3-inches to 5-inches in the cutting area to prevent a sudden snapping of the strand.
    - For multiple-strand detensioning, release prestressing force gradually by hydraulic jacks.
    - Show strand detensioning sequence on the panel shop drawings.

Lifting, Handling and Shipping Considerations

Lifting and handling of precast panels is not detailed in the three PCI documents (1988 *Special Report JR-343*[2], 2014 *Bridge Design Manual*[3], and 2017 *PCINE Guidelines*[5]) or the 2017 *AASHTO LRFD-8 Bridge Design Specifications* (23 CFR 625.4(d)(1)(v)) or the 2002 *AASHTO Standard Specifications* (23 CFR 625.4(d)(1)(iii)), other than broad statements to handle the panels only with approved devices at designated locations and taking care to prevent damage. Once again, the 1999 PCI *Manual for Quality Control* MNL-116[8] provides more complete lifting and handling criteria.

The lifting and handling requirements of the six States have a similar level of detail as the three PCI documents and two AASHTO specifications, with general statements to handle in a manner to not cause damage, handling panels only with approved devices at designated support locations and maintaining support from beneath at approximately the same locations as the permanent support.

Table 22. Reference lifting, handling, and shipping considerations.

Source	Precast Panel Lifting, Handling and Shipping
PCI Special Report JR-343 (1988)[2]	-Handle panels only with approved devices at designated locations
PCI Bridge Design Manual (2014)[3]	-Handle panels only with approved devices at designated locations
PCINE Guidelines (2017)[5]	Not specified
AASHTO Standard Specifications (2002) (23 CFR 625.4(d)(1)(iii)	Care shall be taken to prevent damage
AASHTO LRFD-8 Bridge Design Specifications (2017) (23 CFR 625.4(d)(1)(v)	Not specified
Colorado DOT	-Contractor to handle such that damage does not occur -Lifting of multiple panels to be as shown in shop drawings
Missouri DOT	Maintain supports in approximately same position as designed for the final position
New Hampshire DOT	-Handle panels only with approved devices at designated locations -Maintain supports in approximately same position as designed for the final position
Tennessee DOT	-Handle panels only at designated locations
Texas DOT	-Handle panels only with approved devices at designated locations
Utah DOT	-Use lifting devices such that bending or torsional forces will not occur in the panel

State-of-the-Practice

- Consider the suggestions of 1999 *PCI Manual for Quality Control MNL-116*[8], in addition to the experience of the six States:
  - Only lift panels using approved devices at designated locations
  - When shipping panels, support them from below in approximately the same position as the permanent support locations

Storage

The 1988 PCI *Special Report JR-343*[2] and 2014 PCI *Bridge Design Manual*[3] provide some provisions for storing partial-depth precast concrete panels including suggestions to support stacks of panels by continuous dunnage perpendicular to the strands, where the panels will be supported by the girders in the final position (Figure 35). Intermediate dunnage can be full length or near all four corners and should be located directly above dunnage beneath. The 1988 PCI *Special Report JR-343*[2] also advises to store panels for as little time as possible, and to be aware of the potential for frost heaving of soil in the winter which can affect the levelness of the panel stack. The 1999 PCI *Manual for Quality Control MNL-116*[8] contains similar suggestions with additional detail. The practices of the six States are similar, although with only one State explicitly calling for continuous dunnage.

Table 23. Reference panel storage considerations.

Source	Precast Storage
PCI Special Report JR-343 (1988)[2]	-Stacks of panels should be supported by continuous dunnage perpendicular to strands -Intermediate dunnage should be full length or should provide support near all four corners -Store for minimum amount of time possible -Special consideration are given during winter months (freeze/thaw cycles can cause soil heaving and loss of levelness)
PCI Bridge Design Manual (2014)[3]	-Stacks of panels supported by continuous dunnage perpendicular to strands -Intermediate dunnage shall be directly above line of dunnage beneath
PCINE Guidelines (2017)[5]	Not specified
AASHTO Standard Specifications (2002) (23 CFR 625.4(d)(1)(iii)	Care shall be taken to prevent damage
AASHTO LRFD-8 Bridge Design Specifications (2017) (23 CFR 625.4(d)(1)(v)	Not specified
Colorado DOT	Store panels such that damage does not occur
Missouri DOT	-Store such that panels remain level -Stack panels only with approval of the engineer
New Hampshire DOT	-Stacks of panels supported by continuous dunnage perpendicular to strands -Intermediate dunnage shall be directly above line of dunnage beneath
Tennessee DOT	Not specified
Texas DOT	-Stacks of panels supported by dunnage or blocking -Stack so lifting devices are visible and undamaged -Use dunnage or blocking that will not damage the panel
Utah DOT	Not specified

Figure 35.Photo. PDDP storage.

State-of-the-Practice

- Consider the suggestions of the 1999 *PCI Manual for Quality Control MNL-116*[8], in addition to the experience of the six States:
    - Support stacks of panels with dunnage oriented perpendicular to the strands, with intermediate dunnage located directly over the dunnage below

<u>Panel Acceptance Criteria</u>

Dimensional Tolerances

The 1988 PCI *Special Report JR-343*[2] and the 2017 PCINE *Guidelines*[5] provide suggested tolerances for partial-depth precast concrete deck panels. The 1999 PCI *Manual for Quality Control MNL-116*[8], also provides a set of tolerances specific to PDDPs, with slightly different values. The 1988 PCI *Special Report JR-343*[2] and the 1999 PCI *Manual for Quality Control MNL-116*[8] agree on panel thickness tolerance (+1/4 inch, -1/8 inch), with the 2017 PCINE *Guidelines*[5] being stricter (+0 inch, -1/8 inch). The 2017 PCINE *Guidelines* and the 1999 PCI *Manual for Quality Control MNL-116*[8] agree on length and width tolerances (±1/4 inch), with the 1988 PCI *Special Report JR-343*[2] being more permissive on the length tolerance (+3/4 inch, -1/4 inch).

For strand horizontal placement tolerances, the 2017 PCINE *Guidelines*[5] and the 1999 PCI *Manual for Quality Control MNL-116*[8] specify a tolerance of ±1/4 inch, while the 1988 PCI Special Report *JR-343*[2] allows a tolerance of ±1/2 inch. For strand vertical placement tolerances and suggestion, see the Design Parameters chapter. The 1999 PCI *Manual for Quality Control MNL-116*[8] has been added to the following table because of its relevance to precast fabrication tolerances.

Table 24. Reference panel dimensions and strand placement tolerances.

Source	Panel Dimensions	Horizontal Strand Placement Tolerance	Vertical Strand Placement Tolerance*
PCI Special Report JR-343 (1988)[2]	Length: +3/4”, -1/4” Width: ±1/4” Thickness: +1/4”, -1/8”	Horizontal: ±1/2”	Vertical position of strand group for panel < 3” thick: +0”, -1/8” Vertical position of strand group for panel ≥ 3” thick: +0”, -1/4” Vertical position of individual strand for panel < 3” thick: ±1/8” Vertical position of individual strand for panel ≥ 3” thick: ±1/4”
PCI Bridge Design Manual (2014)[3]	Not specified	Not specified	Not specified
PCINE Guidelines (2017)[5]	Length: ±1/4” Width: ±1/4” Thickness: +0”, -1/8”	Horizontal: ±1/4”	Vertical position of strand group: +0”, -1/8” Vertical position of individual strands: ±1/8”
PCI Manual for Quality MNL-116 (1999)	Length: ±1/4” Width: ±1/4” Thickness: +1/4”, -1/8”	Horizontal: ±1/4”	Vertical position of individual strands: ±1/8”
AASHTO Standard Specifications (2002) (23 CFR 625.4(d)(1)(iii)	Not specified	Not specified	Not specified
AASHTO LRFD-8 Bridge Design Specifications (2017) (23 CFR 625.4(d)(1)(v)	Not specified	Not specified	Not specified
Colorado DOT	Length: +3/4”, -1/4” (per JR-343-88) Width: ±1/4” (per JR-343-88) Thickness: ±1/4”	Not specified	Position of individual strands: ±1/4”
Missouri DOT	Length: +1/8”, -1/2” Width: ±1/4” Thickness: ±1/8”	Not specified	±1/8” for center of gravity of strand group and for individual tendons

Source	Panel Dimensions	Horizontal Strand Placement Tolerance	Vertical Strand Placement Tolerance*
New Hampshire DOT	Length: ±1/4” Width: ±1/4” Thickness: +1/4”, -1/8”	Horizontal: ±1/4”	Vertical position of strand group: +0”, -1/8” Vertical position of individual strands: ±1/8”
Tennessee DOT	Length: +1/4”, -1/8” Width: +1/4”, -1/8” Thickness: +1/4”, -1/8”	Not specified	Vertical position of individual strands: +1/8”, -1/4”
Texas DOT	Length: ±1/2” Width: ±1/2” Thickness: +1/4”, -1/8”	Horizontal position of individual strands: ±1/2”	Vertical position of individual strands: ±1/8”
Utah DOT	Length: Not specified Width: Not specified Thickness: +1/4”, -1/8”	Not specified	Position of individual strands: ±1/4”

* Vertical strand placement tolerances included in table to show all tolerances together. See the Design Parameters, Design Criteria for Prestress section in Chapter 2 for related discussion and recommendation

The panel dimensional tolerances very among the six States. For thickness, four States use a tolerance of +1/4-inch, -1/8-inch, with the remaining two States using a tolerance of ±1/4-inch and ±1/8-inch. Panel length tolerances range from -1/2-inch to +3/4-inch, and panel width tolerances ranges within ±1/2-inch. Only two States specify a horizontal tolerance on the strand position, using either ±1/4-inch or ±1/2-inch.

The consequences of panel thickness tolerances are probably more significant than length and width tolerances, as a reduction in panel thickness can both reduce its bending capacity and increase the concrete stress. This is reflected by the fact that five of the six States only allow an under-size tolerance of -1/8-inch, and all six States allow an oversize tolerance of +1/4-inch. Tolerances on the panel width (perpendicular to the bridge axis), however, can also be important, with an undersizing reducing the available support width and an oversizing increasing the risk of interfering with girder shear connectors. An undersizing of the panel length (parallel to the bridge axis) can also increase the concrete stress, while the risks of oversizing any one panel is minimal, although an oversizing of many panels could result in the total length of panels being greater than intended.

State-of-the-Practice:

- Consideration may be given to the following fabrication tolerances and the experience of the six States:

  - Fabrication Tolerance on Panel Length (parallel to bridge axis): +3/4-inch, -1/2-inch

  - Fabrication Tolerance on Panel Width (perpendicular to bridge axis): ±1/2-inch

  - Fabrication Tolerance on Panel Thickness: ±1/4-inch

  - Fabrication Tolerance on Strand Horizontal Position: ±1/2-inch

  - Fabrication Tolerance on Strand Vertical Position: ±1/4-inch

Cracking Limits

Of the PCI and AASHTO documents reviewed, three documents contain limits on cracking. These documents are the 1988 PCI *Special Report, JR-343*[2]; the 2006 PCI *Manual for the Evaluation and Repair, PCI-MNL-137*[6]; and the 2018 PCINE *Guidelines for Resolution of Non-Conformance in Precast Concrete Bridge Elements, PCINE-RNCPBE*[7], which is an update of the 2003 *PCINER Bridge Member Repair Guidelines, PCINER-BMRG*[9]. They provide the following general crack limits:

- Two adjacent strands with colinear cracks are cause for rejection.

- Colinear cracks are cause for an engineering assessment when along more than 6 percent of strands (PCINE-RNCPBE[7]) or 12 percent strands (MNL-137[6]) and colinear cracks along more than 25 percent of the strands are cause for rejection (JR-343[2]).

- Transverse and diagonal cracks that cross more than 6 percent of strands (PCINE-RNCPBE[7]) or 12 percent of strands (MNL-137[6]) are cause for an engineering assessment.

- Corner cracks or breaks involving two or more stands (JR-343[2])

- Shrinkage cracks and cracks not in proximity to the strands are of less significance.

The 2014 *PCI Bridge Design Manual*[3] simply contains generic language stating that cracks are not a cause for rejection unless the panel is structurally or aesthetically impaired beyond repair.

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[9] *Bridge Member Repair Guidelines, 2003 Revision,* Report No. PCINER-01-BMRG, Precast/Prestressed Concrete Institute New England, is not incorporated in the CFR Title 23. Therefore, this industry technical document is not a Federal requirement.

Table 25. Reference panel cracking limits.

Source	Cracking Limits
PCI Special Report JR-343 (1988)[2]	Cracks that are cause for rejection include: -Two cracks, each occurring within 1 inch of two adjacent strands -Corner cracks or breaks involving two or more strands -Cracks parallel to and along more than 25 percent of the strands -Shrinkage cracks and cracks not in proximity to the strands are of less significance.
PCI Bridge Design Manual (2014)[3]	Cracks are not cause for rejection unless panel is structurally or aesthetically impaired beyond repair
PCINE Guidelines (2017)[5]	Not specified
PCI-MNL-137 (2006)[6]	Cracks that warrant engineering assessment include: -More than 12 percent of strands with colinear cracks -Transverse and diagonal cracks that cross more than 12 percent of strands Cracks that are cause for rejection include: -Two adjacent strands with colinear cracks
PCINE-RNCPBE (2018)[7]	Cracks that warrant engineering assessment include: -More than 6 percent of strands with colinear cracks -Transverse and diagonal cracks that cross more than 6 percent of strands Cracks that are cause for rejection include: -Two adjacent strands with colinear cracks
AASHTO Standard Specifications (2002) (23 CFR 625.4(d)(1)(iii)	Not specified
AASHTO LRFD-8 Bridge Design Specifications (2017) (23 CFR 625.4(d)(1)(v)	Not specified
Colorado DOT	Bottom flexural cracks may be cause for rejection
Missouri DOT	Not specified
New Hampshire DOT	Cracks that are cause for rejection include: -Any crack transverse or diagonal to the strand pattern and crossing more than one strand -Any crack parallel to a strand and longer than 1/3 of the panel length -Cracks shorter than 1/3 of the panel length and present at more than 12 percent of the total number of strands in the panel
Tennessee DOT	Cracks that are cause for rejection include: -Any crack that comes within 1 inch of a strand -Corner cracks or breaks that involve one strand Any crack that appears in a panel after the deck has been placed will be considered to extend to the mid-depth of the panel or to the strands.

Source	Cracking Limits
Texas DOT	Cracks that are cause for rejection include: Any crack extending to the reinforcing plane and running parallel and within 1 inch of a strand for at least 1/3 of the strand length; Any transverse or diagonal crack intersecting at least two adjacent strands
Utah DOT	Bottom flexural cracks may be cause for rejection

Of the six States, three States do not have specific criteria for cracking limits, with two of the three simply stating that bottom flexural cracks may be cause for rejection. The remaining three States--New Hampshire, Tennessee, and Texas—have more specific criteria for panel rejection due to cracking.

Tennessee has the strictest criteria, taking the 1988 PCI *Special Report JR-343*[2] limits of cracks relating to two strands, but using only one strand for rejection. Tennessee rejects panels with any crack that comes within one inch of a strand or with corner cracks or breaks that involve one strand. Furthermore, Tennessee specifies that any cracks that appear after the cast-in-place deck has been placed will be considered to extend to the strand and thus will require remedial action.

Both New Hampshire and Texas require that cracks that cross or are diagonal to the strands must cross two or more strands to reject a panel. For cracks that are parallel to the strands, the cracks must be within one inch of a strand and extend for at least one-third of the panel length to warrant panel rejection. New Hampshire adds that longitudinal cracks shorter than one-third of the panel length but present at more than 12 percent of the strands in the panel are also cause for panel rejection.

Tennessee is alone among the six States with its very strict cracking criteria, whereas Texas and New Hampshire have similar criteria that are less strict, with New Hampshire having the most severe climate of the three States. Therefore, it is suggested to base panel cracking limits on New Hampshire's practices.

State-of-the-Practice

- Based on the experience of the six States, causes for panel rejection may include:
    - Any crack that is oriented diagonally or perpendicular to the strands and that crosses a minimum of two strands
    - Any crack oriented parallel to the strands that is longer than one-third of the panel length
    - Cracks oriented parallel to the strands that are shorter than one-third of the panel but present at more than 12 percent of the strands. Note that recent research indicates that if splitting reinforcement is provided at panel ends, colinear cracks are not a problem (Bayrak et al. 2013).

Repair/Rejection

The only reason for panel rejection mentioned by the 1988 PCI *Special Report JR-343*[2] and the *Bridge Design Manual* is cracking. However, both the 2006 *Manual for the Evaluation and Repair of Precast, Prestressed Concrete Bridge Products*, PCI-MNL-137 and the 2018 PCINE *Guidelines for Resolution of Non-Conformance in Precast Concrete Bridge Elements*, PCINE-18-RNPCBE, identify honeycombing more than one-inch deep as an additional reason for partial-depth precast concrete deck panel rejection.

For crack repairs, the four above mentioned PCI and PCINE documents all suggest epoxy injection, depending on the crack size. The 1988 PCI *Special Report JR-343*[2] suggests epoxy injection in general but says that “small” cracks should be painted or covered with epoxy. The *Bridge Design Manual* suggests epoxy injection for “narrow structural” cracks, and a rubbing of mortar into plastic shrinkage cracks. The 2006 PCI *Manual for Evaluation and Repair*[6] and the 2018 PCINE *Guidelines for Resolution of Non-Conformance*[7] suggest epoxy injection specifically for cracks that are parallel but not co-linear with strand and that exceed 0.006 inch (MNL-137) or 0.007 inch (PCINE-18-RNPCBE) in width, in addition to stating that honeycombing not exceeding one-inch in depth be repaired.

Only two of the six States specify a reason for rejection other than cracking, and both suggest that panels with sags or cambers greater than ½-inch be rejected. Notably, these are the same two States that simply said bottom flexural cracking may be cause for rejection, without any more detail on the size or distribution of the cracking.

Three of the six States also do not specify any crack repair methods, and a fourth State simply says that repair procedures are to be submitted for approval. Of the remaining two States, one calls for epoxy injection or gravity-fed sealant, and one calls for repairs to be carried out in accordance with the PCINER Bridge Member Repair Guidelines, PCINER-01-BMRG[9], originally published in 2001. This document has been superseded by the 2018 PCINE *Guidelines for Resolution of Non-Conformance in Precast Concrete Bridge Elements*, PCINE-18-RNPCBE[7], although with no changes to the partial-depth panel repair suggestions. The suggestions are also very similar to those in the 2006 PCI *Manual for the Evaluation and Repair, PCI-MNL-137*[6].

Repair methods are in PCINE-18-RNPCBE[7] and PCI-MNL-137[6], either of those methods can be followed. Since only two States require ½-inch sag or camber as an automatic reason for rejection, and in certain circumstances excess camber could be beneficial, this is not considered part of the state of the practice by the research team.

Table 26. Reference panel rejection criteria and repair methods.

Source	Panel Rejection Criteria	Panel Repair Methods
PCI Special Report JR-343 (1988)[2]	See cracking limits above	-Epoxy injection -Painted/covered with epoxy for small cracks
PCI Bridge Design Manual (2014)[3]	See cracking limits above	-Plastic shrinkage cracks: rub full using mortar -Narrow structural cracks: epoxy injection
PCINE Guidelines (2017)[5]	Not specified	Not specified
PCI-MNL-137 (2006)[6]	Honeycombs deeper than 1 inch may compromise strand bond; See cracking limits above	Epoxy inject longitudinal cracks that exceed 0.006 inch in width that are not colinear with strand; Honeycombing not exceeding one inch in depth at either face should be repaired
PCINE-RNCPBE (2018)[7]	Honeycombing exceeding one-inch depth at either face may be cause for rejection; See cracking limits above	Epoxy inject longitudinal cracks that exceed 0.007 inch in width that are not colinear with strand; Honeycombing not exceeding one inch in depth at either face should be repaired
AASHTO Standard Specifications (2002) (23 CFR 625.4(d)(1)(iii)	Not specified	Not specified
AASHTO LRFD-8 Bridge Design Specifications (2017) (23 CFR 625.4(d)(1)(v)	Not specified	Not specified
Colorado DOT	Sags greater than ½", cambers greater than ½"; See cracking limits above	Not specified
Missouri DOT	Not specified	Not specified
New Hampshire DOT	See cracking limits above	Repair methods per PCINER-01-BMRG (superseded by PCINE-18-RNPCBE)
Tennessee DOT	See cracking limits above	Epoxy grout for small damage/honeycombed areas that are not greater than 1" in depth
Texas DOT	See cracking limits above	Epoxy injection or gravity-fed sealant
Utah DOT	Sags greater than ½", cambers greater than ½"; See cracking limits above	Repair procedures to be submitted for approval

State-of-the-Practice:

- For repair of partial-depth panels, and for additional reasons for panel rejection besides cracking, such as honeycombing, consideration may be given to the 2006 PCI *Manual for the Evaluation and Repair of Precast, Prestressed Concrete Bridge Products*, PCI-MNL-137[6] or the 2018 PCINE *Guidelines for Resolution of Non-Conformances in Precast Concrete Bridge Elements*, PCINE-18-RNPCBE[7].

Aging of Panels

None of the PCI or AASHTO design documents gives information on the aging of panels prior to removal from forms or prior to shipping, besides the initial concrete strengths noted above. Only two of the six States have aging requirements, with both New Hampshire and Texas requiring a minimum of seven days between casting and shipping. This delay may allow the concrete to gain additional strength to make the panels more robust to resist the stresses of shipment without cracking and will also lower the amount of creep and shrinkage remaining in the panel that can contribute to panel cracking once it is fixed in place.

Table 27. Reference panel aging criteria.

Source	Aging of Panels
PCI Special Report JR-343 (1988)[2]	Not specified
PCI Bridge Design Manual (2014)[3]	Not specified
PCINE Guidelines (2017)[5]	Not specified
AASHTO Standard Specifications (2002) (23 CFR 625.4(d)(1)(iii))	Not specified
AASHTO LRFD-8 Bridge Design Specifications (2017) (23 CFR 625.4(d)(1)(v)	Not specified
Colorado DOT	Not specified
Missouri DOT	Not specified
New Hampshire DOT	Minimum 7 days between casting and shipping
Tennessee DOT	Not specified
Texas DOT	Minimum 7 days between casting and shipping
Utah DOT	Not specified

Figure 36.Photo.Aging of panels in storage yard.
Source: Texas Department of Transportation

State-of-the-Practice:

- Waiting a minimum of seven days after panel casting before shipping may be a useful practice.

Transporting

The 1988 PCI *Special Report JR-343*[2]provides detailed suggestions for shipping of partial-depth precast concrete deck panels. The recommendations include using dunnage in the same manner and positions as during panel storage, placing tie-down straps over the lines of dunnage, banding panels together in both directions to prevent panel shifting, and reducing the height of the panel stacks on the last 20-feet of the trailer to half the height of the rest of the panel stacks.

The 2014 PCI *Bridge Design Manual*[3] suggests loading the panels with supports located as close as possible to the lifting devices, while also providing typical flatbed trailer weight restrictions of 50 to 60 kips.

Three of the States do not specify any requirements for transporting partial-depth precast panels. Texas simply requires that the panels be placed in the truck in manner that avoids excessive bending stresses or damage. Colorado has a similar requirement and adds that the panels are to be shipped in a horizontal position.

New Hampshire has the most detailed requirements, closely matching the 1988 PCI *Special Report JR-343*[2]. These requirements include blocking perpendicular to the strands and with intermediate blocking between panels located over blocking below (both as required for storage). Tie-down straps are required at the lines of blocking, along with a requirement for shock-absorbing materials at all panel bearing points.

Only one of the six States has strict shipping criteria which may indicate that the industry is generally capable of safely shipping the panels without causing much damage. This is most likely due to the fact that the repair or replacement of panels damaged during shipping remains the responsibility of the contractor team, including the general contractor, precaster, and/or shipper.

Table 28. Reference panel transportation criteria.

Source	Transportation of Panels
PCI Special Report JR-343 (1988)[2]	-Loaded on trailers with dunnage same as during storage -Tie-down straps over lines of dunnage -Band panels together in both directions to prevent shifting -Reduce panel stacks to half height for last 20' of trailer
PCI Bridge Design Manual (2014)[3]	-Loaded on trucks with supports as close as possible to lifting devices -Flat-bed trailer shipping weight restrictions generally vary from 50 to 60 kips
PCINE Guidelines (2017)[5]	Not specified
AASHTO Standard Specifications (2002) (23 CFR 625.4(d)(1)(iii))	Not specified
AASHTO LRFD-8 Bridge Design Specifications (2017) (23 CFR 625.4(d)(1)(v)	Not specified
Colorado DOT	-Panels to be shipped in a horizontal position -Panels to be stacked in manner such that damage does not occur
Missouri DOT	Not specified
New Hampshire DOT	-Blocking perpendicular to strands, intermediate blocking between members directly over blocking below -Shock-absorbing cushioning materials at all bearing points -Tie-down straps at lines of blocking
Tennessee DOT	Not specified
Texas DOT	-Panels placed in truck in manner to avoid excessive bending stresses or damage
Utah DOT	Not specified

State-of-the-Practice

- Panels should be transported in a manner to avoid damage or excessive bending stresses. The means and methods to achieve this are to be determined by the precaster and/or shipper. Possible means to prevent damage include the following:

    - Ship panels in the horizontal position.
    - Load panels on trailers with dunnage the same as during storage, including intermediate dunnage located over dunnage below.
    - Install tie-down straps over lines of dunnage
    - Band panels together in both directions to prevent shifting
    - Reduce panel stacks to half height for last 20' of trailer
    - Place shock-absorbing cushioning materials at all bearing points

4. INSTALLATION

Grading of Deck Panels

Panel Support/Bearing Details and Materials

Refer to the Panel Geometry section in Chapter 2 for permanent panel support details, and to Design Parameters section in Chapter 2 for temporary panel support details.

Precautions to Take

Construction Load Limits

See the Design Parameters section in Chapter 2 for details and suggestions on construction load limits.

Keeping Panels Secured During Erection

Securing the precast panels in their position during erection is important for worker safety and for the proper construction of the composite deck. The only design guide addressing this is the 2017 PCINE *Guidelines*[5], which suggests installing temporary bracing between panel ends as necessary to prevent transverse panel movement.

Only three of the six States mention the need to secure the panels in place. Colorado simply states that the stability of the panels is the contractor's responsibility. New Hampshire says to install temporary bracing between panel ends as required to prevent transverse panel movement, like the suggestion in the 2017 PCINE *Guidelines*[5]. Utah expands the reasons for bracing, mentioning the need to resist wind or other loads, but also does not specify how to do it.

This is another area where the means and methods are not typically given to avoid shifting responsibility to the project owner or engineer. There are also likely many possible ways to secure the panels in place, minimizing the need for specific suggestions on how to do it. Consequently, the best approach may be to inform the contractor team of the potential need for bracing but leave it up to them to decide how to do it.

Table 29. Reference criteria for securing panels to bridge.

Source	Keeping Panels Secured
PCI Special Report JR-343 (1988)[2]	Not specified
PCI Bridge Design Manual (2014)[3]	Not specified
PCINE Guidelines (2017)[5]	Install temporary bracing between panel ends to prevent transverse panel movement
AASHTO Standard Specifications (2002) (23 CFR 625.4(d)(1)(iii))	Not specified
AASHTO LRFD-8 Bridge Design Specifications (2017) (23 CFR 625.4(d)(1)(v)	Not specified
Colorado DOT	Stability of panels is contractor's responsibility
Missouri DOT	Not specified
New Hampshire DOT	Install temporary bracing between panel ends as required to prevent transverse panel movement
Tennessee DOT	Not specified
Texas DOT	Not specified
Utah DOT	Temporarily support, anchor, and brace erected superstructure members as necessary for stability and to resist wind or other loads until they are permanently secured to the structure

State-of-the-Practice:

- Temporary bracing during erection is generally the responsibility of the contractor's team, and the specific means to achieve this are not specified. For certain unique situations, such as with tall haunches, it may be prudent to provide temporary bracing until the panels are permanently incorporated into the structure.

Deck Forming Options to Address Areas of Conflict or Challenges

The 1988 PCI *Special Report JR-343*[2] provides some provisions on addressing areas of conflict where there is a varying elevation of the top of the framing with respect to the finished top of deck, such as where there are localized steel girder top flange splice plates, or variable girder camber. The suggestions are simply to either vary the thickness of the cast-in-place concrete topping, or to vary the thickness of the panel bearing. Varying the thickness of the cast-in-place concrete, however, will result in additional dead load on the panels and on the substructure, so varying the thickness of the panel bearing, and by association the haunch, is preferred. Variations in the distance between the top of the girders and the finished top of deck should be accounted for in the design, including an allowance for girder camber deviation from predicted values, to avoid a field conflict where there is not enough space available.

Only two of the six States provide specific guidance on this situation, while the remaining four States simply give minimum haunch heights that can be increased as necessary to avoid conflicts, with Texas giving a special detail with a reinforced grout bed for tall haunches.

Missouri is one of the two States that gives specific guidance. The standard drawing for panels on steel girders contains a table giving the height and width of the joint filler when seated on girder flanges, and when seated on splice plates. Changes to the height of the joint filler, which is used to temporarily support the panels, will modify the haunch height. The joint filler height may be reduced to as little as 0.25-inch on a splice plate, whereas 1.0-inch is the minimum thickness when supported on the flange. Additionally, the width of the joint filler may be reduced from 1.5-inch minimum to 0.75-inch minimum, along with a corresponding reduction in the width of the panel that overlaps the girder flange to avoid splice plate bolts. For all types of concrete girders, the joint filler thickness can vary from 1.0-inch minimum to as much as 4.0-inches to accommodate girder camber.

New Hampshire, which uses a grout bed for the haunch and permanent girder support, states that the thickness of the haunch should be adjusted to account for additional girder camber, additional deck thickness, field splice plates, and any other detail that might impact the 1.0 inch minimum haunch thickness requirement, with the intent to make up any changes in deck thickness within the haunch.

Table 30. Reference deck forming options to address areas of conflict.

Source	Deck Forming Options to Address Areas of Conflict
PCI Special Report JR-343 (1988)[2]	Two options to account for varying elevation: -Uniform thickness bearing with variable thickness CIP topping -Variable thickness bearing with uniform thickness CIP topping
PCI Bridge Design Manual (2014)[3]	Not specified
PCINE Guidelines (2017)[5]	Not specified
AASHTO Standard Specifications (2002) (23 CFR 625.4(d)(1)(iii))	Not specified
AASHTO LRFD-8 Bridge Design Specifications (2017) (23 CFR 625.4(d)(1)(v)	Not specified
Colorado DOT	Not specified
Missouri DOT	Joint filler thickness may be decreased to minimum of ¼” over steel flange splice plates. Joint filler thickness may be varied on concrete and steel structures to account for camber and cross-slope.
New Hampshire DOT	Vary haunch thickness as required to account for camber, splice plates, or other details impacting minimum haunch requirement. Changes in deck thickness to be accounted for within haunch.
Tennessee DOT	Not specified
Texas DOT	Special detail provided with minimum 2.0-inch tall reinforced grout bed for tall haunches.
Utah DOT	Not specified

State-of-the-Practice

- Anticipate and avoid conflicts between the panels and the girders during the design phase. Methods contractors may use to avoid conflicts in the field include:
    - Vary the thickness of the panel temporary supports and therefore the thickness of the haunch
    - Vary the width of the panel temporary supports if necessary, to avoid flange splice plate bolts

Staged Construction Considerations

None of the PCI or AASHTO design documents provides provisions for staged construction with partial-depth precast concrete deck panels. Only two of the six States provide guidance.

The New Hampshire Bridge Design Manual states that partial-depth deck panels are not allowed in the bay adjacent to the previously placed deck, as they require a full-depth cast-in-place concrete closure pour between stages.

Conversely, Texas allows partial-depth deck panels in the bay adjacent to the previously placed deck, provided that the stage construction line falls over a girder flange and leaves more than 3.0-inches of the girder flange exposed to support the second stage panels and includes a detail for this on their partial-depth panel standard drawings.

Table 31. Reference staged construction considerations.

Source	Staged Construction Considerations
PCI Special Report JR-343 (1988)[2]	Not specified
PCI Bridge Design Manual (2014)[3]	Not specified
PCINE Guidelines (2017)[5]	Not specified
AASHTO Standard Specifications (2002) (23 CFR 625.4(d)(1)(iii))	Not specified
AASHTO LRFD-8 Bridge Design Specifications (2017) (23 CFR 625.4(d)(1)(v)	Not specified
Colorado DOT	Not specified
Missouri DOT	Not specified
New Hampshire DOT	Panels not allowed in the bay adjacent to the previously placed deck due to requirement of CIP closure
Tennessee DOT	Not specified
Texas DOT	Panels allowed in bay adjacent to previously placed deck as long as distance between previously placed deck and adjacent newly placed panel is greater than 3”.
Utah DOT	Not specified

As the Texas detail shows, to use precast concrete deck panels in the bay adjacent to the stage line, the stage line will have to be located over a girder. This results in the girder at the stage line only carrying half of its final deck load, with a corresponding deflection. Installing the second stage then adds the remaining deck load to the girder at the stage line, increasing its deflection and creating the potential for cracking in the deck in the bay between the girder at the stage line and the adjacent first stage girder. This concern can be largely eliminated by locating the stage line in the center of a girder bay and placing the second stage deck first before stitching the two stages together with a closure pour. This will necessitate full-depth cast-in-place concrete deck overhanging the two girders adjacent to the stage line and for the closure pour.

The divergent practices between New Hampshire and Texas suggests that New Hampshire has concerns about cracking due to the two stages of dead load on the girder at the stage line, or has experienced it in the past, while Texas does not see much cracking.

State-of-the-Practice

- For staged construction, the use of partial-depth precast concrete deck panels in the bay adjacent to the stage line can maximize all the benefits of using partial-depth panels, but caution needs to be exercised to avoid the potential to cause cracking in the deck of the first stage due to the two phases of dead load applied to the girder at the stage line. To use partial-depth panels in the bay adjacent to the stage line, the stage line should be located over a girder, with enough space on the girder flange left for proper seating of the second stage panels.

- Where large changes in the stage line girder deflection between stages are anticipated, consider placing the stage line near the center of a girder bay with full-depth cast-in-place deck overhangs adjacent to the stage line and a full-depth cast-in-place concrete closure pour after the second stage deck is complete (Figure 37).

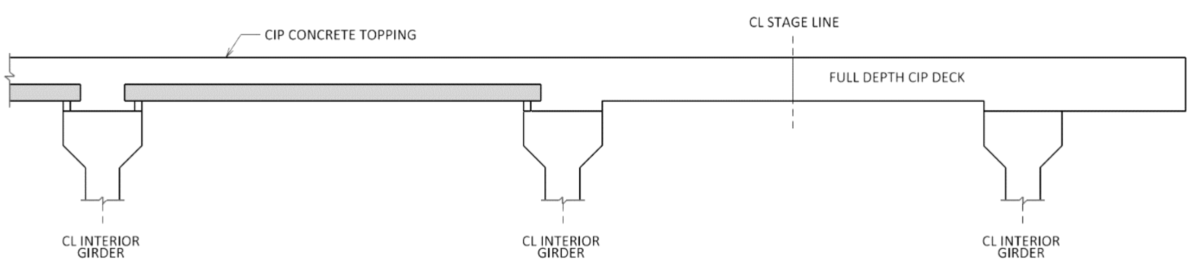

Figure 37. Illustration. Staged construction: stage line near center of girder bay.

5. FIELD PLACEMENT OF CIP DECK CONCRETE

Prior to placing the cast-in-place deck concrete, as the 1988 *PCI Special Report JR-343*[2] indicates, the precast panels should be inspected while in-place to confirm that no damage occurred during shipping and erection. Additionally, it is advised to confirm that the panels are properly supported and that, if designed to rely on the cast-in-place concrete for permanent support, the panel ends have sufficient horizontal and vertical clearance to allow the CIP concrete to flow and fill the space under the panel ends. The supporting material or formwork should be continuous to prevent CIP concrete from spilling through the panel support area.

Once the panels and their supports are confirmed to be acceptable (Figure 38), the next step before placing the top reinforcing steel mat is to inspect and, if necessary, remove any laitance or contaminants from the top surface that could negatively affect the bond to the CIP concrete. Finally, the panel surfaces should be thoroughly wetted so that the surfaces in contact with the CIP concrete are in a saturated surface-dry state (SSD) at the time of the concrete pour, with no standing water, to prevent the dry precast concrete from drawing water out of the fresh CIP concrete leading to increased shrinkage and drying cracking of the CIP concrete. This step is essential for the structural integrity of the bridge deck, although it can be difficult to confirm that a SSD condition has been achieved. In addition, because the CIP concrete placed on top of partial-depth panels is typically only 4 inches to 5 inches thick, it is more sensitive to cold, hot, or drying conditions, and shrinkage restraint cracking than a typical full-depth CIP deck. Some steps that may be taken to reduce the likelihood or severity of shrinkage and drying cracking of the CIP concrete are:

- Establish a significant minimum duration of continuous prewetting, normally between 12 and 24 hours, immediately prior to placing the CIP concrete to ensure the precast deck panels are in an SSD state. It is suggested that this be a part of every project, with strong and detailed language in the specifications.

- Place concrete in accordance with standard deck placement procedures.

When placing the CIP concrete, the concrete should be vibrated to provide proper consolidation, especially at the precast panel ends to ensure the concrete fully fills the spaces under the panels ends to provide permanent support (where the panels are detailed to do so) (Figure 39). Finally, the CIP concrete should be cured in accordance with standard deck curing procedures. For placing and curing the CIP deck concrete in cold weather, typical cold weather placement procedures would apply. However, with PDDP construction, the precast panels should be preheated and maintained at the same temperature of the CIP concrete to avoid cracking due to differential thermal movements.

Figure 38.Photo. Deck ready for CIP concrete.

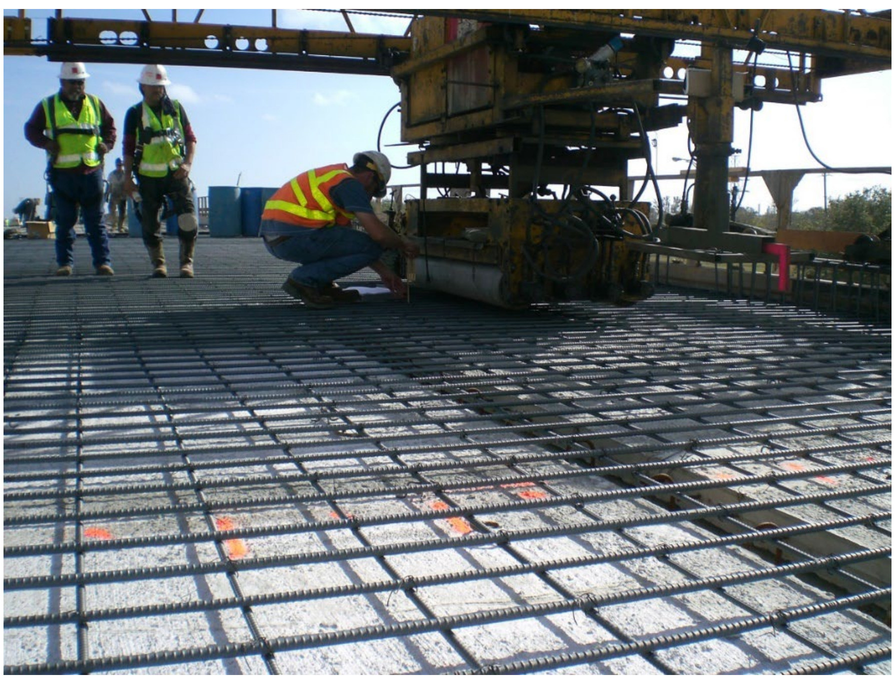

Figure 39.Photo.Screed used to strike off CIP concrete.

6. EMERGING CONCEPTS FOR PDDPS

Ultra-High Performance Concrete

To date, 33 States and the District of Columbia have used ultra-high performance concrete (UHPC) UHPC in at least one bridge. New York has used UHPC in at least 90 bridges – the most of any State. UHPC is an attractive material for bridge construction because of its high compressive and tensile strengths, ductility, strong bond to conventional concrete, and extremely low permeability due to its discontinuous pore structure.

UHPC Panels

While UHPC has seen extensive use as connections between precast conventional concrete deck panels, due to its ability to create very short lap splices on reinforcing bars, using UHPC for the deck panels themselves is much less common. The only constructed example in North America is the bridge carrying Dahlonega Road over Little Cedar Creek in Wapello County, Iowa, constructed in 2011. This example, however, consists of full-depth precast UHPC panels, albeit panels that only use half of the material of a conventional full-depth deck due to the exceptional material mechanical properties and a waffle shape on the underside of the deck (Aaleti et al. 2013).

The two biggest potential advantages of UHPC in partial-depth deck panels is the ability to significantly reduce the thickness of the panels and therefore the shipping and handling weight, leading to potential overall cost savings, and the enhanced durability provided by UHPC which could potentially lead to a longer deck service life and life-cycle cost savings.

Research

Multiple studies have been performed on precast partial-depth panels fabricated from UHPC, including at the Polytechnique Montreal Technical University (Lessard 2009) and at the Missouri University of Science and Technology (Venancio 2016). In both of those studies the mechanical performance of the UHPC panels was found to be superior to that of panels fabricated from conventional concretes. The UHPC panels were as thin as 2 inches and were reinforced with mild reinforcing bars or wire fabric. The Canadian study also found that the performance of a bilayer panel fabricated from UHPC and high-performance fiber-reinforced concrete performed just as well as a panel fabricated entirely from UHPC, indicating a possible means to obtain the benefits of UHPC for less cost.

Research is being performed in the United States on partial-depth precast UHPC deck panels by Dr. Maher Tadros in partnership with the University of Nebraska-Lincoln. These panels incorporate features of the first-generation NUDECK, described in the following section, and of international practices, described in Chapter 7. The geometry is like the first-generation NUDECKI, spanning the entire width of the deck, which allows the panels to cantilever beyond the exterior girders, thereby eliminating the need for temporary deck forms, but with a panel thickness of only 1.5 inches. In lieu of the internal prestressing employed in the NUDECK panels, rebar trusses are used with the truss bottom bars embedded in the UHPC and the truss top bars exposed above the top of the panel. The truss bars stiffen the panels for handling and shipping and help support the cantilevers.

International Examples

Several international examples exist of UHPC stay-in-place forms that are reinforced only with steel fibers and are not designed to act compositely with the cast-in-place concrete placed above. One example is a road bridge constructed in Australia in 2003, where 1.0-inch-thick (25-mm-thick) UHPC panels were used as stay-in-place forms between girders (notably also fabricated from UHPC) spaced at 4ft-3inches (1.3 meters) (Cavill and Chirgwin 2003). In France, precast UHPC stay-in-place forms were designed and installed on an historic bridge as part of a deck replacement project in 2016 (Amanjean 2015). The panels span 6ft-2inches (1.88 meters) between girders and have a thickness of only 1.57-inches (4.0 cm). The thinness of these non-composite UHPC panels and the lack of any prestressing or reinforcing bars could make them an economically viable alternative to traditional deck forming solutions, with enhanced durability as compared to panels fabricated from conventional concrete or metal or compared to full-depth cast-in-place concrete.

Cast-in-Place UHPC Topping over Conventional PDDPs

UHPC has not yet been used or studied as a topping over partial-depth precast conventional concrete deck panels. However, UHPC is being used in two other bridge applications.

First, UHPC is used to connect full-depth precast concrete deck panels, with the ability to fully lap splice the reinforcing bars from adjacent panels in very narrow closure pour – as narrow as 6-inches. By placing UHPC over the girders and between the panels, extended panel reinforcing would become fully lap spliced, creating bottom deck reinforcing continuity across an entire bridge deck. While testing has shown that the performance of panels with and without extended reinforcing is not very different, this was with conventional CIP concretes which are generally not capable of creating a full lap splice connection over the width of a typical girder flange. Furthermore, as noted in the 2017 AASHTO LRFD-8 Bridge Design Specifications, Commentary C9.7.4.3.2 (23 CFR 625.4(d)(1)(v)), extending the reinforcing does have some benefit by reducing the potential for reflective cracking at panel ends. With the bottom reinforcing made structurally continuous across the full deck width using UHPC, the potential for reflective cracking would likely be even further reduced.

The second current UHPC application is a bridge deck overlay. These overlays have been shown to bond to conventional concrete with a strength that makes them composite with the underlying concrete, and the high compressive strength of the UHPC is able to increase the capacity of the deck. Applying UHPC on top of partial-depth precast conventional concrete deck panels is like an overlay, bringing many of the same benefits. The impermeable nature of the UHPC topping would also allow for smaller top cover over the top reinforcing mat, perhaps as thin as 1/2-inch, while still providing superior durability. A reduced cover thickness and the superior mechanical properties of the UHPC could lead to an overall thinner composite deck section, resulting in cost savings by using less material and applying less weight to the bridge foundations.

NUDECK

The first generation NUDECK is a full-width, partial-depth precast deck panel system developed jointly by the University of Nebraska-Lincoln (UNL) and Nebraska Department of Roads

(NDOR) and first implemented in Nebraska in 2004 (Hanna et al. 2010, Morcous et al. 2013, Morcous et al. 2015). No proprietary materials are used in producing the panels. These panels are pretensioned transversely and are formed with open channels along the girder lines for longitudinal post-tensioning and to allow the cast-in-place concrete topping to connect the panels to the girders. Placement of the post-tensioning strands in the girder haunch eliminates the need for threading through ducts and grouting. The NUDECK panel system spans the full width of the bridge, eliminating overhang formwork and supports. The cast-in-place concrete overlay creates a composite system and allows for field adjustments to achieve the required roadway profile. The 2014 PCI Bridge Design Manual[3] (Bridge Design Manual Steering Committee 2014) contains a detailed chapter on the NUDECK system.

Although the first generation NUDECK system has the benefits of faster construction and higher durability than cast-in-place monolithic decks, it has not been widely implemented. Threading of post-tensioning tendons can be time-consuming and frequent conflicts between panel reinforcement and shear studs welded to girders have been reported. Additionally, these panels must be customized for each bridge based on bridge width and girder layout, and the panels are large and heavy to transport. The second generation NUDECK system was developed to improve upon the first generation by simplifying constructability and was first implemented in 2015. One of the major updates made was to eliminate the cast-in-place concrete portion by making the second-generation NUDECK panels full-depth.

GFRP and CFRP Reinforcing

The Colorado Department of Transportation (CDOT) has performed research investigating the feasibility of using carbon fiber reinforced polymeric (CFRP) prestressed partial-depth deck panels with deformed glass fiber reinforced polymeric (GFRP) temperature reinforcement (Zylstra et al. 2001, Shing et al. 2003, Shing and Xi 2003). CDOT designed and constructed a 2/3-scale bridge using both seven-wire steel prestressed and CFRP prestressed panels, and then tested the bridge under static and fatigue loading. The CFRP prestressed panels also had GFRP bars as temperature reinforcement, whereas the steel prestressed panels had standard steel temperature reinforcement. The purpose of the project was to evaluate the performance of CFRP panels as compared to steel-prestressed panels, and also to compare different design methods, as discussed further in the following section.

Carbon fiber reinforced polymeric tendons are comparable to steel tendons with respect to the modulus of elasticity and they also have a very high tensile strength. CFRP tendons are non-corrosive, which can mitigate deck corrosion and thereby extend the service life of a bridge. However, CFRP is more costly than steel, necessitating further life-cycle cost analysis for implementation. Additionally, CFRP is more fragile than steel and involves careful handling during construction. GFRP bars were investigated as temperature reinforcement for consistency with the non-corroding nature of the CFRP tendons. While the GFRP bars have a higher bond strength to concrete than steel reinforcing bars or tendons, the quantity was increased to partially compensate for the low elastic modulus of GFRP compared to steel.

The performance of the CFRP prestressed panels with GFRP temperature reinforcement was compared to the seven-wire steel prestressed panels with steel temperature reinforcement after the 2/3-scale bridge was exposed to fatigue load cycles. Results of the testing showed that the

CFRP prestressed panels performed the same as the steel prestressed panels, confirming that CFRP is a viable alternative to steel tendons. However, panels with alternative reinforcement exhibited several drawbacks. The testing showed that the CFRP tendons may be more susceptible than steel tendons to freeze-thaw induced tensile strength reduction. Moment analysis demonstrated that the steel reinforced slab better distributed loading in the transverse direction, implying that steel is more effective at distributing load than GFRP. The behavior of the CFRP-reinforced panels in the non-composite state was brittle and weaker than the steel-reinforced panels even though the CFRP was nominally significantly stronger than the steel reinforcement. Finally, the anchorage details for the CFRP tendons were found to be expensive and awkward compared to the anchorage details for steel tendons, leading to a conclusion that epoxy coated steel strand or stainless steel strand was a better choice for corrosion resistance. Nonetheless, the benefits of utilizing CFRP tendons and GFRP distribution reinforcement in partial-depth concrete deck panels should continue to be investigated, especially as improved CFRP and GFRP products and details are developed.

Using Empirical Design Method

In the CDOT research project discussed in the previous section, bridge decks utilizing partial-depth deck panels and a CIP topping were designed using various methods and their performance was evaluated under fatigue load cycles (Shing et al. 2003, Shing and Xi 2003). The cast-in-place portions of the composite decks were designed using the conventional AASHTO method, the empirical design method, and a new limit-state design method developed by CDOT for the research project. The 2017 AASHTO LRFD-8 Bridge Design Specifications (23 CFR 625.4(d)(1)(v)) allows refined methods of deck analysis but prohibits the use of the empirical design method for decks with partial-depth precast concrete deck panels.

The empirical design method eliminates the need for any analysis by specifying the minimum amount of deck reinforcement. Reinforcement is significantly reduced in decks designed using the empirical method as compared to decks designed by the traditional method. The traditional method uses approximately 70 percent more negative moment reinforcement than the empirical method. Reducing the amount of negative moment reinforcement reduces both labor and material costs and can also minimize future deck reinforcement corrosion.

As part of CDOT's research project, a limit-state deck design method was developed that accounts for the effects of girder deflections and arching action. Specifically, this method significantly reduces the deck reinforcement by considering the reduced negative moment due to girder deflection as well as the increased moment resistance due to arching action. Like the empirical method, this limit-state deck design method uses approximately 30 percent of the reinforcement required by the traditional method.

The performance of the composite decks designed using the three design methods was evaluated after being exposed to fatigue load cycles. Test results showed that the portions of the deck designed using the empirical method and limit-state method performed the same as the portion of the deck design using the traditional method. Portions of the deck designed using the empirical and traditional methods showed no cracks in the bottom of the panels, whereas the portion of the deck designed using the limit-state approach exhibited bottom cracks in the positive moment region. The research report concluded that the limit-state design approach can be further investigated for adoption into design practice and suggested that the empirical design approach be utilized for the design of the cast-in-place slab atop partial-depth deck panels.

7. INTERNATIONAL PRACTICES

In Canada, the use of partial-depth precast concrete deck panels is not that different from U.S. practices. One notable difference is the long extensions of prestressing strands used in some Canadian provinces, as shown in Figure 40. The extended strands are draped over the top of the adjacent panel with the objective of increasing the development of the strands to reduce the risk of cracking in the CIP topping.

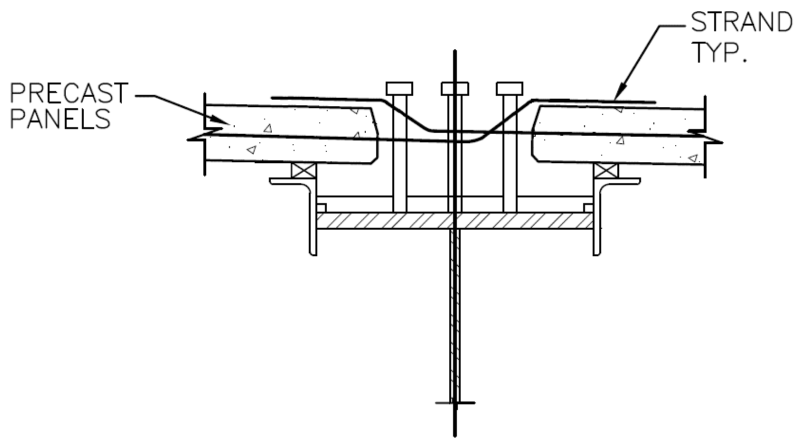

Figure 40. Illustration. Long strand projections used in some Canadian provinces.
Source: Government of Saskatchewan Ministry of Highways and Infrastructure

In Europe and Australia, partial-depth precast concrete panels are often fabricated with mild reinforcing bars only, and with lattice bar trusses that extend above the top of the panel. The lattice bar trusses stiffen the panels, providing resistance to cracking during shipping and handling that, in North America, is typically provided by prestressing the panel concrete. The lattice top bars also support the final top deck reinforcing and can serve to supplement the top deck reinforcement

In the United Kingdom, it is common to have panels that are only 12-inches wide with a single lattice bar truss per panel. These panels are used between girders and are not used for deck overhangs.

In Spain and Australia, partial-depth precast concrete panels are fabricated for the full width of a bridge deck, similar to the NU DECK, but with lattice bar trusses extending above the panel, as shown in Figure 41. The panel reinforcing is continuous across the entire deck width, with gaps in the precast concrete over the girder flanges. This allows the panels to support the deck overhangs, eliminating the need for deck falsework.

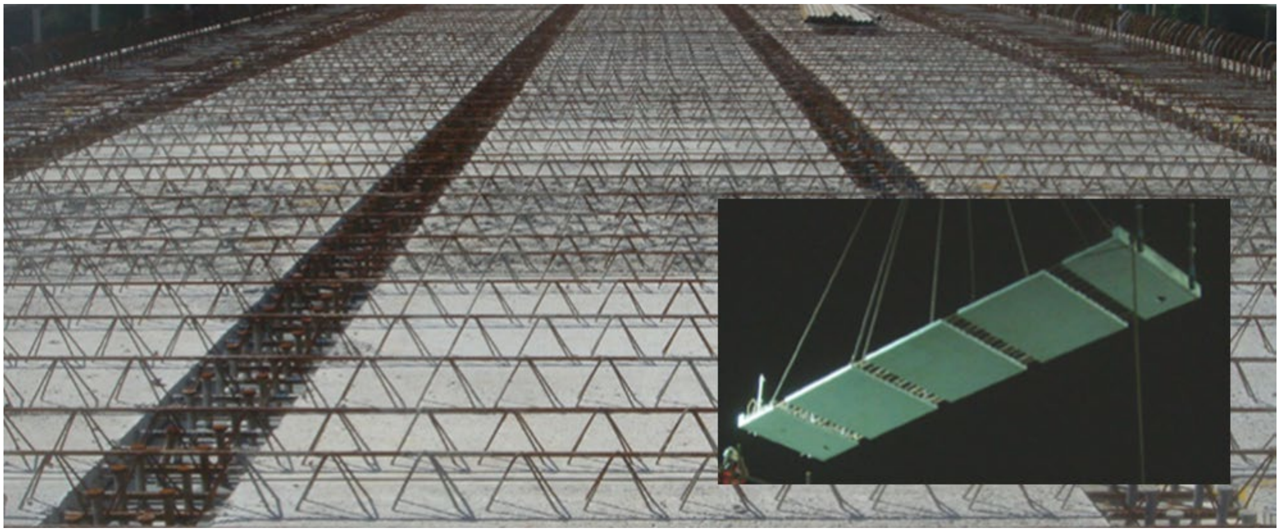

Figure 41. Photo. Full-width panels used in Spain and Australia.

8. PERCEIVED BARRIERS AND SOLUTIONS TO THE USE OF PDDP TECHNOLOGY

The following is a discussion of some perceived barriers to PDDP use, along with information and potential solutions.

Cracking of CIP Deck Topping

Perceived Barrier: A potential concern regarding partial-depth concrete deck panels is that the CIP deck will experience substantial amounts of cracking, typically exhibited as reflective cracking of the panel edges.

Experience: A survey of State practices completed for the Nevada DOT identified reflective cracking as a concern with PDDPs (Jones et al. 2016). However, only a few of the twenty responding States reported it as a frequent occurrence. Other studies have concluded that many States, including New Hampshire, do not typically see reflective cracking (Whittemore et al. 2006).

Solutions: To reduce cracking it is important to ensure that the CIP concrete has adequate moisture during curing to reduce shrinkage and drying cracking, as discussed in Chapter 5. This is consistent with current construction practice to reduce cracking in any CIP concrete deck. Useful practices include:

- Proper panel edge support (ensuring CIP completely fills space under panel edges)
- Having a saturated surface-dry condition of the panels prior to placement of CIP topping
- Increasing the amount of longitudinal distribution reinforcement in the CIP concrete
- Allowing panels to age before placing the CIP concrete

There is also some indication based on experiences in Colorado that the stiffness of diaphragms between girders at the time of the CIP deck pour can play a role in longitudinal cracking. Relatively light diaphragms, intended primarily for construction stability, combined with placing concrete diaphragms at piers and abutments at the same time as the CIP deck concrete, provide relatively little restraint to transverse creep and shrinkage, thereby reducing the potential for longitudinal cracking. Because of this practice, Colorado has relatively little trouble with longitudinal cracking of the CIP concrete topping over PDDPs.

It should be noted that some cracking of the CIP is normal, and some cracking may appear as reflective cracking, but the amount of cracking has not been shown to be any more than what is seen with full-depth cast-in-place concrete decks when the PDDP deck system is properly designed and detailed. The amount of cracking can even be less than that of full-depth cast-in-place concrete decks. When cracking of the CIP concrete does occur, concrete sealers can be applied periodically to reduce or prevent moisture intrusion as with any full depth cast in place concrete deck, if desired. Another method to address serviceability concerns related to deck cracking, whether on decks with PDDPs or full-depth CIP concrete deck, is to use glass fiber reinforced polymer (GFRP) reinforcing bars for the top reinforcement mat. GFRP bars are being used in the panhandle of Texas as well as in some Canadian provinces.

Panel Rejection Rates

Perceived Barrier: High rates of panel rejection may increase overall project costs and could delay project schedules. This might make some agencies and designers hesitant to specify or allow PDDPs as well as make contractors hesitant to select PDDPs if given the option.

Experience: Estimates of the rate of panel rejection due to cracking during fabrication range from less than 1 percent to a maximum of 2 percent of all panels fabricated. Because the panels are prestressed and will be covered with CIP concrete, cosmetic cracking is generally not a concern and typically only cracks that pose a structural concern would be cause for rejection.

Solutions: Notwithstanding the low panel rejection rates, research performed by Texas DOT has shown that panel cracking during fabrication can be mitigated by reducing the initial strand prestressing force (from 16.1 kips/strand to 14.4 kips/strand) and by reducing the assumed lump-sum losses (from 45 ksi to 25 ksi) during design (Bayrak et al. 2013). The use of smaller strands can also be used to reduce the required force per strand and provides the additional benefit of reducing the transfer and development length.

Another approach to mitigating panel rejection is to load test panels that do not meet cracking limits to determine if they are acceptable based on providing adequate strength. Illinois carried out a similar study on two bridges and the results were the basis for lifting a previous moratorium on the use of PDDPs (Volle 2002).

Either of these methods may be used to reduce the rate of panel cracking during fabrication and mitigate the potential for panel rejection. However, it should be noted that it is relatively simple and inexpensive to reject or repair partial depth panels, whereas rejection of a full depth CIP concrete deck is difficult, expensive, and not necessarily effective without replacing the entire contiguous pours of CIP concrete.

Cost Concerns

Perceived Barrier: Initial startup cost must be a consideration When first introducing partial-depth concrete deck panels in an area with limited or no previous usage, the capacity to fabricate PDDPs would need to be developed if it is not cost-effective to ship panels from existing facilities in neighboring areas. The establishment of new PDDP fabrication facilities involves an initial startup cost.

Additionally, depending on the bid item payment for deck construction outlined in the State specifications, contractors may anticipate a CIP deck to be more cost-effective than a PDDP system, even though using PDDPs eliminates the need to install forms, place a bottom mat of rebar in the field, accelerates the construction schedule, and significantly reduces the volume of CIP concrete.

Experience: Contractors who become familiar with the use of PDDPs may select them over full-depth CIP decks when given the option as evidenced by the experience of TXDOT, MDOT, and CDOT.

Solutions: Modifying payment terms for deck construction may be useful. For example, in 1983 TxDOT updated its specifications to change the bid item measurement for reinforced concrete decks from cubic yard of deck concrete to square feet of deck. This change and the development of standard panel details resulted in increased use of PDDPs.

When using PDDPs for the first time, startup costs may be reduced by first introducing it in a large project or spreading it over multiple projects through project bundling.

9. APPROACH AND STRATEGIES TO DEPLOY PDDP TECHNOLOGY

If an agency decides to use PDDPs an implementation plan should be developed. This Chapter presents a possible approach as shown in Figure 42.

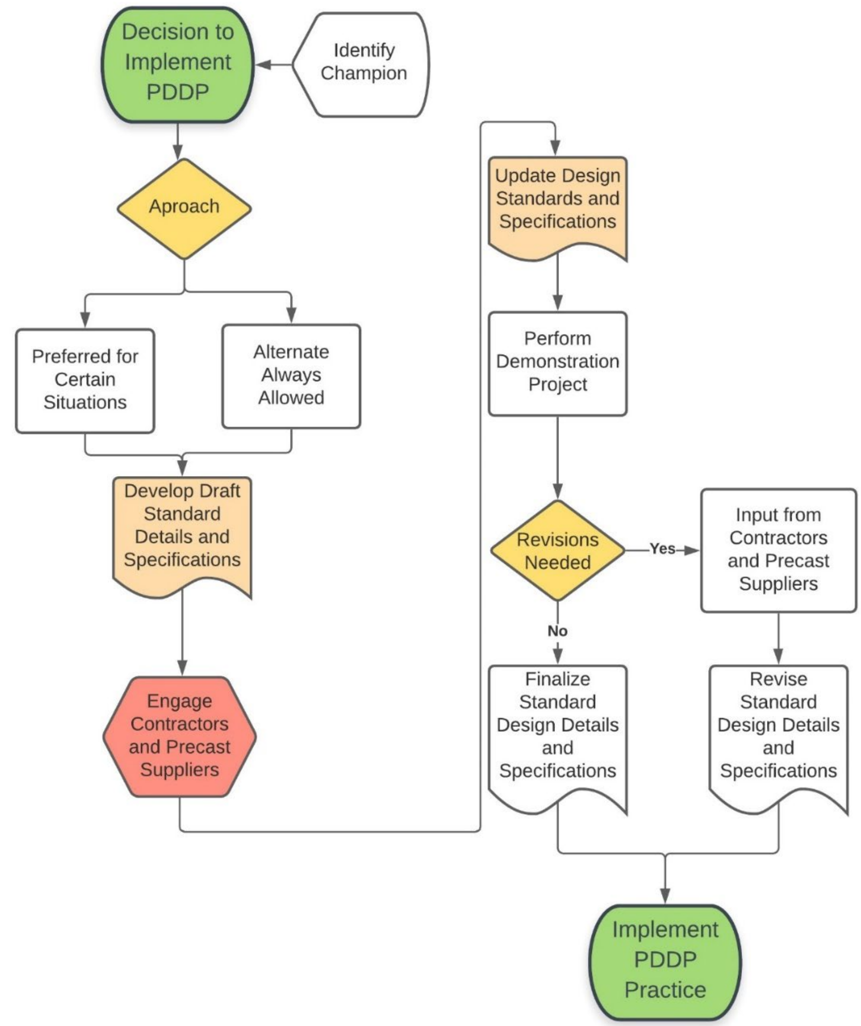

Figure 42. Illustration. Example PDDP implementation plan.

The implementation of any new or underutilized technology benefits from a champion who will work with others within the agency to take on the responsibility of overcoming internal and external hurdles. Visiting with and talking to peers at agencies that have used this technology may also help.

The initial decision involves how PDDPs may be deployed within the agency. Most States that are regular users allow PDDP as an alternate to cast in place decks in many situations. In this approach, it is up to the contractor to decide which deck type is more cost effective for each project since the agency considers them as technically equivalent. Once local contractors are familiar with the use of PDDP they may choose them.

In some instances, an agency may decide that it will only allow or require PDDP in certain situations. An example is using PDDP in coastal areas where SIP forms will corrode or where aesthetics are an important consideration. PDDPs are significantly more cost effective then removeable formwork in these situations in addition to the benefits for their use in general.

One of the major benefits of partial-depth precast concrete deck panels is their repeatability and standardization for typical bridges. As such, developing State-specific standard specifications and details is an important step in implementing PDDPs. As referenced throughout this report, several States have robust details and specifications for PDDPs. This information, in conjunction with the contents of this report, could be utilized in the development of new State-specific details and specifications. Some sample project specifications are included in Appendix C.

Once the agency has developed draft design standards and specifications, it should engage the contractor and precaster community to understand potential hurdles or sources of resistance. The agency can then address those concerns to the extent possible prior to putting out a project with PDDPs to bid. The agency could review the proposed PDDP details and specifications with the contractor community. This would provide an opportunity for the agency to receive feedback and make adjustments that might decrease contractor resistance and lower costs. The agency also should identify at least one precaster, located within or near the area served by the agency, that is willing to make the investments to produce PDDPs to ensure that reasonably priced bids are received.

If desired, a demonstration project could be carried out with PDDPs using the newly developed design standards and specifications. This is an opportunity to evaluate all aspects of the PDDP deployment and determine if any revisions are needed to the standard design details and specifications prior to full scale deployment. If any revisions are needed, they can be incorporated into the final design details and specifications that will be used for final full deployment.

APPENDIX A – CASE STUDIES

STUDIES ON THE USE OF HIGH-PERFORMANCE CONCRETE AND FRP REINFORCEMENT IN PARTIAL-DEPTH PRECAST CONCRETE DECK PANELS FOR THE I-225/PARKER ROAD BRIDGE IN COLORADO

INTRODUCTION:

The Colorado Department of Transportation Research Branch report titled *Studies on the Use of High-Performance Concrete and FRP Reinforcement for the I-225/Parker Road Bridge* details the investigation of innovative construction materials on the I-225/Parker Road Bridge as well as on a 2/3-scale model bridge deck. This study was conducted to evaluate the performance of innovative construction materials and design methodologies in response to concerns regarding cracking of typically-reinforced concrete bridge decks. Materials that were studied include high performance concrete (HPC), fiber reinforced polymeric (FRP) reinforcement, carbon fiber reinforced polymeric reinforcement (CFRP), and glass fiber reinforced polymeric reinforcement (GFRP). These materials were used in conjunction with partial-depth precast concrete deck panels with a concrete overlay. Also investigated as part of this study was the performance of the AASHTO Empirical Design Method and a proposed limit-state design method for designing the cast-in-place (CIP) concrete overlay.

CONSTRUCTION MATERIALS EVALUATED:

As part of the study conducted by the Colorado Department of Transportation (CDOT), various innovative construction materials were investigated. High performance concrete (HPC) was used in conjunction with FRP reinforcement for part of the deck on the I-225/Parker Road Bridge. HPC was studied because it has a higher resistance to temperature and shrinkage cracks than traditional concrete, and therefore has the potential to minimize deterioration of bridge decks that is caused by deterioration of reinforcement resulting from chemicals penetrating through cracks. Additionally, FRP tendons and reinforcement were studied because FRP is a noncorrosive material and can therefore also reduce deck deterioration. Specifically, CFRP prestressing tendons and GFRP temperature reinforcement in partial-depth precast concrete deck panels were analyzed as part of this study.

DESIGN METHODOLOGIES EVALUATED:

In addition to studying innovative construction materials, this study also investigated the performance of decks designed using various design methodologies. The AASHTO empirical design method and a proposed limit-state design method were both studied for the design of the cast-in-place topping over partial-depth concrete deck panels. Currently, the AASHTO LRFD specifications prohibits the use of the empirical design method for decks utilizing partial-depth precast concrete deck panels. The empirical design method specifies the minimum area of deck reinforcement and therefore eliminates the need for additional analysis. This method also significantly reduces the amount of reinforcement as compared to the AASHTO traditional deck design method. Similarly, the limit-state design method proposed by CDOT results in significantly reduced deck reinforcement by accounting for the effects of girder deflections and arching action.

PROJECT DESCRIPTION:

This report summarized the study performed on the I-225/Parker Road Bridge, as well as testing performed on a 2/3-scale model deck. The I-225/Parker Road Bridge consists of “post-tensioned cast-in-place reinforced concrete box girders” with 3.5” thick precast concrete deck panels topped with a 5” thick cast-in-place (CIP) concrete slab. HPC and CFRP tendons within the partial-depth panels was used for part of the bridge deck, and traditionally reinforced concrete was used for the remaining part of the deck.

The University of Colorado at Boulder conducted studies on the Bridge to validate the design of the deck. A 2/3-scale model of the bridge was constructed and subjected to static and fatigue load cycles to test the performance of the innovative materials and the design methodologies used. The model deck was constructed in 4 unique segments:

- Segment A: Partial-depth precast concrete deck panels with a composite CIP slab designed using the AASHTO empirical deck design method,
- Segment B: Partial-depth precast concrete deck panels with a composite CIP slab designed using the AASHTO conventional deck design method,
- Segment C: Partial-depth precast concrete deck panels with a composite CIP slab designed using a limit-state deck design method, and
- Segment D: Full-depth CIP concrete slab designed using the AASHTO empirical design method.

Additionally, panels on the north side of the model deck were constructed with standard steel prestressing strands and steel bars for mild reinforcement, whereas panels on the south side of the model deck had CFRP prestressing tendons and GFRP temperature and shrinkage reinforcement. The 2/3-scale model deck was subjected to both static and fatigue load cycle testing, and strains and deflections during testing were measured using strain gages and linear variable differential transformers (LVDT's) which were installed along the model deck.

TESTING RESULTS AND CONCLUSIONS:

Based on the results of the study two HPC mixes were suggested for use, one for summer and one for winter construction. For the suggested mixes, "the desired range of cement content is from 465 to 485 lb/yd3, water/cementitious material ratio from 0.37 to 0.41, and Class F fly ash from 20 percent to 25 percent." The HPC mixes were suggested to contain Class F fly ash as it results in more durable HPC than when using Class C fly ash.

Testing on the model deck showed that the partial-depth concrete deck panels with CFRP prestressing tendons performed equally to the panels with steel prestressing with respect to load resistance. Per the conclusions of the report, "CFRP bars seem to be a viable alternative to steel tendons for precast panel construction." The tests showed that GFRP reinforcement may not be as effective as traditional steel reinforcement in transferring load, and therefore "may not be adequate for distribution reinforcement." Additionally, freeze-thaw exposure may impact the fatigue properties of FRP tendons and reinforcement.

Both the empirical design method and proposed limit-state design method use only 30 percent of top reinforcement as calculated by the conventional AASHTO deck design method. The reduced reinforcement saves material and labor costs. Sections designed using both empirical method and limit-state method performed equally to the deck portion designed using the traditional method. The portions designed using the empirical method did not exhibit any cracking, while the portion designed using the limit-state method exhibited limited bottom cracking in the positive moment. However, these cracks originated in adjacent full-depth CIP portion of the deck. The study concluded, "it is recommended that the empirical method be allowed for the design of the topping slab for precast panel decks."

REFERENCE:

Colorado Department of Transportation (CDOT), *Studies on the Use of High-Performance Concrete and FRP Reinforcement for the I-225/Parker Road Bridge*

https://www.codot.gov/programs/research/pdfs/2003/hpcparker.pdf

CONSTRUCTION OF BRIDGE DECKS WITH PARTIAL-DEPTH PRECAST CONCRETE DECK PANELS IN JERSEY-GREENE COUNTY AND LOGAN COUNTY, ILLINOIS

INTRODUCTION:

The Illinois Department of Transportation (IDOT) report titled *Construction of Bridge Decks with Precast, Prestressed Concrete Deck Planks* details the construction of two demonstration projects in 1999 and 2000 which utilized partial-depth precast concrete deck panels (PDDPs). IDOT first began using PDDPs between 1980 and 1985 but placed a moratorium on their use in 1985 after experiencing issues with longitudinal cracking of decks. In the 1990's research was performed to address the concerns with the previously constructed PDDP bridge decks, and in 1997 IDOT decided to undertake two demonstration projects to incorporate the design changes recommended by the research and to evaluate the performance of the redesigned PDDPs. Between 1999 and 2000 two projects were constructed utilizing PDDPs: The Jersey-Greene County (District 8) Bridge, and The Logan County (District 6) Project which encompassed two separate bridges.

DESCRIPTION OF PROJECT:

The five-span Jersey-Greene County (District 8) Bridge was constructed first and utilized 3" thick PDDPs in the first three spans and metal stay-in-place (SIP) forms in the last two spans. The Logan County (District 6) Project consisted of two bridge replacements. The three-span Kickapoo Creek bridge was constructed using 3.5-inch thick PDDPs, and the three-span Kickapoo Creek Overflow bridge was constructed using 3-inch thick PDDPs. All of the bridges in the demonstration project were bridge replacement projects, and thus were each constructed in two stages.

IMPROVEMENTS TO PANEL DESIGN:

As part of the research performed in the 1990's to investigate the concerns with the PDDP bridge decks from 1980-1985, design modifications were recommended and incorporated into this project. The panels from the 1980's were generally only 2.5-inch thick, and thus tight tolerances were needed for the vertical placement of the prestressing strands. Panels experienced warping and cracking when tolerances were not met, contributing to deck cracking. For all three bridges constructed as part of this project, panel thickness ranged from 3.0 to 3.5 inches". This increased panel thickness minimized the impact of any strands not placed within tolerances and reduced panel warping and cracking.

Another aspect of the panel design that was modified from the 1980-1985 construction was the panel seating. In the 1980's, the panels were usually set on mortar beds or polystyrene. Mortar bed cracking was a commonly reported issue, as well as deterioration of the deck concrete. For the demonstration project, the panels were designed with leveling jacks which were inserted during fabrication and adjusted in the field for proper seating height. Polystyrene strips were used as a temporary support during installation and acted as a dam for the deck concrete to flow beneath the panels.

In the originally constructed PDDP bridge decks, there was no age limit for the PDDPs. Panels "were often shipped as soon as they met the strength limit and used immediately upon shipment." Per the recommendations of the 1990's research, a minimum PDDP age at the time of deck pouring of 28-days for Stage I and 60-days for Stage II was used for the demonstration project. Additionally, a minimum PDDP age of 4-days was needed prior to shipment.

LOAD TESTING REJECTED PANELS:

For the demonstration project, panels were to be rejected "if cracks were visible from arm's length and greater than 3 inches (75 mm) along the plank." Based on this limitation, a large portion of the panels fabricated for the three bridge decks were initially rejected. To avoid costly delays due to material ordering and re-fabrication of panels, a load testing procedure was developed to approve the previously rejected PDDPs. PDDPs were grouped in sets of ten based on similar cracking, and one representative panel was randomly selected for load testing. Deflection was measured during and after the load tests, and a visual inspection was performed afterwards to identify any lengthening of existing cracks or new damage cause by the load tests. The results of the tests were then sent to the Illinois Bureau of Bridges and Structures for approval. Per this load testing procedure, many of the previously rejected panels were approved and utilized in the three bridge decks.

PERFORMANCE OF DECK PANELS:

A series of distress surveys were performed on all three bridge decks for a period of approximately one year after construction. Both the Jersey-Greene County Bridge and the Logan County Bridges exhibited transverse and longitudinal cracks, but the majority were minor hairline cracks. Surveys were performed on similar bridges with full-depth cast-in-place (CIP) concrete decks, and it was noted that for the CIP bridge deck that also exhibited longitudinal cracking, there were more longitudinal cracks than seen on either of the PDDP bridge decks constructed for this project.

CONCLUSIONS AND RECOMMENDATIONS:

The contractors that were involved in the construction of the two demonstration projects noted that deck form construction time was reduced, and form removal time was eliminated. One of the contractors also noted that efficiency of panel placement will increase as the process is refined through more projects.

The construction of bridge decks utilizing partial-depth precast concrete deck panels is now allowed in Illinois. Per the results of this project, the following recommendations were made for future construction projects using PDDPs:

- The load testing procedure that was used to approve previously rejected deck panels, or a similar method for accepting panels, should be adopted for future PDDP bridge deck construction projects.
- Leveling screws should be installed in the panels during fabrication.
- The polystyrene strips used for temporary support of the panels and grout dam for the CIP deck concrete should be ASTM C 578 Type IV or higher.

REFERENCE:

Illinois Department of Transportation (IDOT), *Construction of Bridge Decks with Precast, Prestressed Concrete Deck Planks*

https://idot.illinois.gov/Assets/uploads/files/Transportation-System/Research/Physical-Research-Reports/139.pdf

STUDIES ON PARTIAL-DEPTH PRECAST CONCRETE DECK PANEL PERFORMANCE ON A LONG-SPAN, HIGH TRAFFIC VOLUME BRIDGE: I-393 OVER THE MERRIMACK RIVER IN CONCORD, NEW HAMPSHIRE

INTRODUCTION:

The New Hampshire Department of Transportation (NHDOT) report titled *Precast Concrete Deck Panel Performance on Long Span, High Traffic Volume Bridges* details a project undertaken to study the performance of partial-depth precast concrete deck panels (PDDPs) on long span, high traffic volume bridges. New Hampshire first began utilizing PDDPs in the early 1990's, but at that time there was not clear guidance on the implementation of this technology. As contractors implemented the panels on shorter, low traffic volume bridges, they began requesting to use them on longer span, higher traffic volume bridges as well. Without clear guidance, some of these longer bridges began to exhibit deck cracking. NHDOT placed a temporary halt on the use of PDDPs until guidance on panel implementation could be developed. As part of the new implementation plan that was developed, panels were prohibited on high volume roadways and long span and multi-span structures with high truck traffic. The purpose of this project was to investigate whether PDDPs and the cast-in-place (CIP) concrete overlay acted compositely and whether there was adequate transfer of loads between panels to allow the use of this technology on long span, high traffic volume bridges.

DESCRIPTION OF PROJECT:

The NHDOT chose the I-393 bridge over the Merrimack River as the test bridge for this project because it is a "long span, multi-span bridge with high traffic volume" and needed a deck replacement. The Merrimack River bridge is a 520-ft-0-inches-long, three span structure with a span length arrangement of 160-200-160 feet. The bridge carries four lanes of traffic and has an average daily traffic (ADT) of 38,000 vehicles per day and an average daily truck traffic (ADTT) of over 2,000 trucks per day. The existing full-depth CIP deck was replaced with 3.5-inch thick PDDPs and a 5-inch thick CIP concrete overlay. To investigate the composite action between the panels and CIP overlay and the transfer of loads between the panels, cores were taken from the positive and negative moment regions of the bridge deck both before and after traffic loading and were subjected to shear and tension testing. The deck was also visually inspected for cracking both before and after traffic loading.

TESTING PROGRAM:

Cores were taken from the bridge deck in two phases: Phase 1 cores were taken before application of vehicular loading, and Phase 2 cores were taken from the same approximate locations 10 months later after the application of vehicular loading. In total, 24 cores were taken from the Merrimack River bridge deck. Cores were taken from 4 specific locations: "near the inflection points of Span 1 and Span 2, centered over Pier 1, and near mid-span of Span 2." It was estimated that these locations would experience the greatest live loads and therefore be the most impacted by the vehicular loading. Phase 1 cores were DC-1 through DC-12 and Phase 2 cores were DC-13 through DC-24.

Four different tests were performed on the deck cores:

- Parallel Shear Test: testing shear strength of the composite deck system parallel to deck scoring,
- Perpendicular Shear Test: testing shear strength of the composite deck system perpendicular to deck scoring,
- Compressive Strength Test: testing compressive strength of the CIP overlay, and

- Straight Tension Test: testing tensile/bond strength between the PDDP and CIP overlay.

TESTING RESULTS:

Compressive Strength:

The CIP overlay was designed with a targeted compressive strength of 4000 psi. The compressive strength of the cores was tested at an age of 66 days. The average recorded compressive strength was 5270 psi, exceeding the targeted compressive strength. The lowest measured compressive strength was 4900 psi, and this value was used to determine the targeted shear and tensile strengths of the composite PDDP and CIP overlay system.

Tensile Strength:

The equation $4–6\sqrt{f'_c}$ was used to generate a tensile strength target range of 280 to 420 psi. The average tensile strength of the cores measured during testing was 263 psi for Phase 1 and 337 psi for Phase 2. For Phase 1, the tensile failure occurred at the interface between the PDDP and CIP overlay in approximately half of the cores, whereas the failure occurred in the CIP overlay for all of the Phase 2 cores. Therefore, the tensile strength testing results show that the bond strength between the PDDP and CIP overlay increases over time and falls within the target range after aging.

Shear Strength:

The equation $2–4\sqrt{f'_c}$ was used to generate a shear strength target range of 140 to 280 psi. The average shear strength of the cores measured during testing was 502 psi for Phase 1 and 567 psi for Phase 2. Both the Phase 1 and Phase 2 shear strengths exceed the shear strength target, and the shear strength increased over time similarly to the tensile strength.

Visual Inspections:

Visual inspections were performed on the composite deck both before and after vehicular traffic was introduced. No visible cracks were identified during either inspection.

CONCLUSIONS:

The test results showed that there is composite action and a strong bond between PDDPs and the CIP overlay system. Additionally, the testing showed that the bond strength continues to increase over time, even with vehicular traffic. NHDOT now allows PDDPs as a contractor option for long span/multi-span, high traffic volume bridges.

REFERENCE:

New Hampshire Department of Transportation (NHDOT), *Precast Concrete Deck Panel Performance on Long Span, High Traffic Volume Bridges*

https://www.nh.gov/dot/org/projectdevelopment/materials/research/projects/documents/FHWA-NH-RD-13733D.pdf

LESSONS LEARNED FROM TEXAS' USE OF PARTIAL-DEPTH PRECAST CONCRETE DECK PANELS FOR BRIDGE DECKS

INTRODUCTION:

The paper titled *Texas' Use of Precast Concrete Stay-in-Place Forms for Bridge Decks* describes the use of partial-depth precast concrete deck panels (PDDPs), including the advantages of this approach. Texas first began utilizing PDDPs in 1963. In 1983, the Texas Department of Transportation (TxDOT) updated its standard specifications and modified the pay item for reinforced concrete bridge decks "from cubic yards to square feet of bridge deck area with the specification allowing either removable forms, stay-in-place metal deck forms, or precast prestressed concrete panels at the contractor's option." At the same time, TxDOT developed standard details for PDDPs. After these two changes, the use of PDDPs for bridge decks greatly increased throughout the State, and this is now the main system for bridge deck construction in Texas.

ADVANTAGES:

Speed:

Texas has seen a reduction in construction time for bridge decks utilizing PDDPs. Contractors that are experienced with PDDP bridge decks can set an entire bridge deck's worth of panels in just a few days. Additionally, the time to tie the deck reinforcing steel in the field is reduced approximately by half since the PDDPs contain the bottom layer of deck reinforcement. It has also been shown that large bridge decks utilizing PDDPs can be poured in one operation since the cast-in-place (CIP) concrete volume is reduced approximately by half in comparison to a full-depth CIP concrete deck.

Cost Savings:

Although deck panels have an associated fabrication cost, contractors in Texas have still seen cost savings when using PDDPs as compared to other deck forming methods. Significant time can be saved during the deck form setting and grading process as well as during the form removal process, which results in decreased labor and equipment rental costs. Additional equipment rental costs can be saved during the CIP deck pouring process due to the decreased CIP concrete volume needed. Another potential cost benefit to contractors who use PDDPs is a reduced insurance premium due to the safety benefits of PDDP deck construction.

Safety:

Partial-depth deck panels are inherently safer as a construction material because their heavy weight, as compared to metal or plywood forming, creates a stable and study working platform. Additionally, since the panels are designed to extend beyond the girder flanges, there is a significantly decreased risk of the panels falling between the girders, as can be seen with metal or plywood deck forms. The safety risks associated with form removal are also eliminated. Also, workers are exposed to construction risks for a shorter period since the use of PDDPs accelerates bridge construction.

Deck Durability:

Deck panels also have been seen to increase the durability of Texas' bridge decks. PDDPs allow for the incorporation of prestressed steel into the positive moment regions of bridge decks, thus increasing long-term durability. The prefabrication of deck panels also ensures that they are high quality.

PANEL DESIGN:

Deck designs in Texas are highly standardized, and the standard drawings for PDDPs produced by TxDOT can be inserted directly into plan sets. Texas decks are 8 inches thick, and when PDDPs are utilized, the panels are to be 4-inch thick with a 4-inch thick CIP concrete overlay. The standard details specify the reinforcement (both prestressed and mild), concrete strength, and panel forming needs.

CONSTRUCTION USING PANELS:

Fabrication:

Most contractors in Texas elect to have their deck panels fabricated at a prestressing plant, although some choose to fabricate their own panels. Deck panels are usually cast in prestressing beds up to 500-ft long. Panels are formed approximately 6-inches apart to allow for panel movement upon strand release, as well as to provide a 3-inch strand extension. It has been reported that the largest prestressing plant in Texas can produce 300 panels in one day, whereas a smaller plant can fabricate approximately 50 panels in one day.

Setting Panels:

TxDOT standard details stipulate that panels be supported a minimum of ¼-inch above the tops of girders, and the panels must extend a minimum of 1½-inches beyond the temporary support to allow for the CIP concrete to flow beneath the panel. The CIP concrete beneath the panel edges provides the permanent support for live loads. Texas uses a high-density extruded polystyrene foam as the temporary support as it "does not deflect under the weight of the panels, does not absorb water, and can be glued to the girders for additional stability."

POTENTIAL CONCERNS AND SOLUTIONS:

Longitudinal Cracking:

Longitudinal cracks are the most significant potential concern with the use of PDDPs because they "can result in a reduction in deck stiffness over the girders that could compromise the deck's load-transfer mechanism." Texas has seen that these cracks are most often a result of improper bearing of the panels on the girders. A lack of CIP concrete beneath the panels leads to a flexible bearing that can lead to cracking in the deck. Even with proper panel bearing, longitudinal cracks can still form due to drying shrinkage of the CIP concrete. However, these cracks are generally minor, and Texas has not experienced significant deck problems due to these cracks.

Transverse Cracking:

Transverse cracking, caused by concrete shrinkage and gaps between panels, has also been seen on bridge decks in Texas constructed using PDDPs. However, these cracks typically do not impact the structural performance of the deck and TxDOT has not experienced any issues with decks that exhibit this type of cracking. To minimize the appearance of these cracks, TxDOT updated its standard drawings to reduce the spacing of longitudinal reinforcement in the CIP concrete overlay.

CONCLUSIONS:

Texas reported success with using partial-depth deck panels in bridge deck construction. The primary advantages of this technology have been enhanced construction speed and safety, cost savings, and long-term deck durability. Although Texas has encountered some concerns with PDDPs, the State has developed solutions through the extensive use of this technology.

REFERENCE:

2002 Concrete Bridge Conference, Brian D. Merrill, P.E. (Texas Department of Transportation), *Texas' Use of Precast Concrete Stay-in-Place Forms for Bridge Decks*

http://www.dot.state.tx.us/BRG/Publications/CBC_Merrill.pdf

APPENDIX B – DESIGN EXAMPLES

B.1 INTRODUCTION

The following example is based on the design example originally presented in the PCI Bridge Design Manual, 3rd Ed, 2014 and has been updated to reflect the state-of-the-practice presented in this report and the *AASHTO LRFD Bridge Design Specifications, 9th Edition* (2020), which is not incorporated by reference in the CFR Title 23 , and therefore these AASHTO specifications are not Federal requirements. It demonstrates the design of a partial-depth precast concrete deck panel (PDDP) with a cast-in-place (CIP) concrete slab. This example is not intended to be mandatory and binding and is provided for information purposes only.

Per the state-of-the-practice listed in this report, the PDDP is 3.5- inches thick and the CIP slab is 5.5-inches thick inclusive of a 1/2-inch thick sacrificial layer. The design also accounts for a 2-inch thick future wearing surface. The PDDP's in this example are designed for a bridge with two 12-foot lanes, a 10-foot inside shoulder and a 5-foot inside shoulder, for a curb to curb width of 39-ft-0-inches. Standard New Jersey barriers, 1-ft-10-inches wide and 2-ft-10-inches high, are included. The PDDP's are supported on four precast concrete girders spaced at 11-ft-6-inches center-to-center, as shown in Figure B.1-1. The design strength of the CIP concrete is 4.0 ksi. The design strength of the PDDP concrete is 4.5 ksi at the time of transfer and 6.0 ksi at the time of casting the CIP slab. The strip design method is used. The design of the CIP deck overhang is not included in this example.

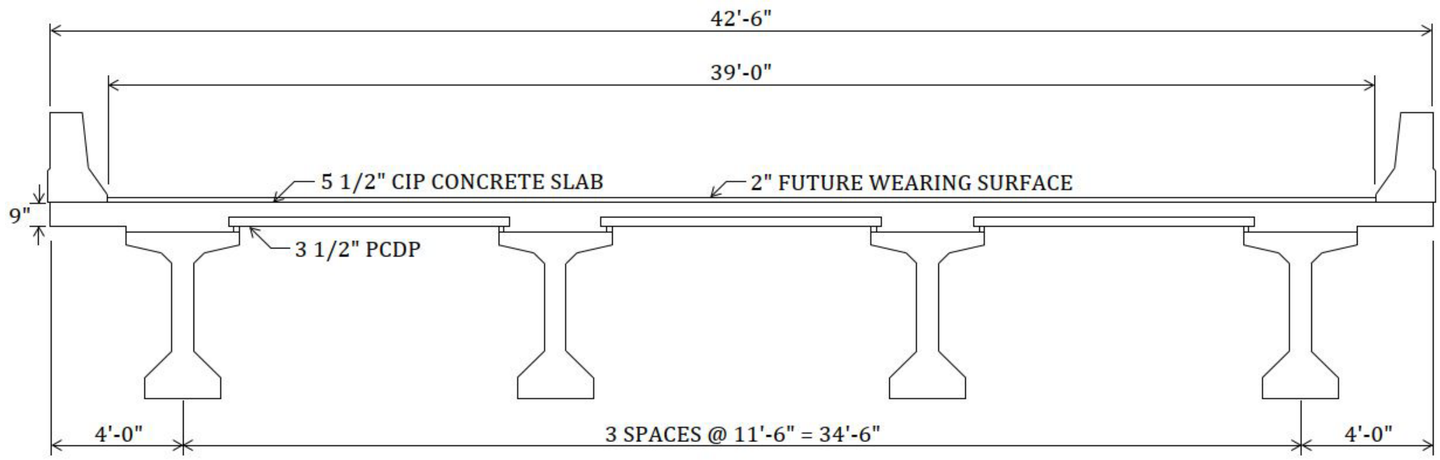

Figure B.1-1.Illistration. Bridge Cross Section

B.1.1 Terminology

The following terminology is used throughout this example:

- noncomposite section - the PDDP cross section only
- noncomposite nontransformed section - the PDDP cross section without the strands transformed
- noncomposite transformed section - the PDDP cross section with the strands transformed
- composite section - the PDDP cross section plus the CIP concrete slab cross section
- composite nontransformed section - the PDDP cross section plus the CIP concrete slab cross section transformed but without the strands transformed
- composite transformed section - the PDDP cross section plus the CIP concrete slab cross section transformed with the strands transformed
- The term "composite" implicitly includes the transformation of the CIP concrete slab
- The term "transformed" generally refers to transformation of the strands

B.2 MATERIALS

Cast-in-place concrete composite slab:

➢ Actual thickness = t_{CIP} =		5.5	in	
➢ Structural thickness,	ts =	5.0	in	
➢ Sacrificial layer thickness,	$tsac$ =	0.5	in	
➢ Specified concrete compressive strength for use in design,	$f'c$ =	4.0	ksi	
➢ Concrete unit weight,	wc =	0.150	kcf	

Superstructure beams:

➢ Beam spacing,		11.5	ft	
➢ Top flange width,		42.0	in	
➢ Web thickness,		8.00	in	

Partial-depth precast concrete deck panels:

➢ Required concrete compressive strength at transfer,	f'_{ci} =	4.5	ksi	
➢ Specified concrete compressive strength for use in design,	f'_c =	6.0	ksi	
➢ Concrete unit weight,	w_c =	0.150	kcf	
➢ Panel thickness,	t_p =	3.5	in	
➢ Panel width (perpendicular to girders),	W_p =	8.67	ft	
➢ Panel length (parallel to girders),	L_p =	8.00	ft	
➢ Centroid of bottom prestressing (from bottom of PDDP),	C_{bp} =	1.30	in	

Prestressing strands: 3/8-in.-dia., low-relaxation	d_b =	0.375	in	
➢ Area of one strand,	A_s =	0.085	in^2	
➢ Specified tensile strength,	f_{pu} =	270.0	ksi	***LRFD Table 5.4.4.1-1***
➢ Yield strength,	$f_{py} = 0.9f_{pu}$ =	243.0	ksi	***LRFD Table 5.4.4.1-1***
➢ Modulus of elasticity,	E_p =	28,500	ksi	***LRFD Art. 5.4.4.2***

Stress limits for prestressing strands:

➢ before transfer,	$f_{pi} \leq 0.75f_{pu}$ =	202.5	ksi	***LRFD Table 5.9.2.2-1***
➢ at service limit state (after all losses),	$f_{pe} \leq 0.80f_{py}$ =	194.4	ksi	***LRFD Table 5.9.2.2-1***

Reinforcing steel:

➢ Yield strength,	f_y =	60.0	ksi	
➢ Modulus of elasticity,	E_s =	29,000	ksi	***LRFD Art. 5.4.3.2***
➢ Top reinforcement clear cover,	cc_t =	2.5	in	***LRFD Table 5.10.1-1***
➢ Bottom reinforcement clear cover,	cc_b = 1.0 in > 0.8 in. OK			***LRFD Table 5.10.1-1***

Future wearing surface:

➢ Thickness of future wearing surface,	t_{FWS} =	2.0 in. additional concrete		
➢ Unit weight of future wearing surface,	w_c =	0.150	kcf	

New Jersey-type barrier:

➢ Unit weight of New-Jersey-type barrier,	w_{NJb} =	0.543	kips/ft/side	

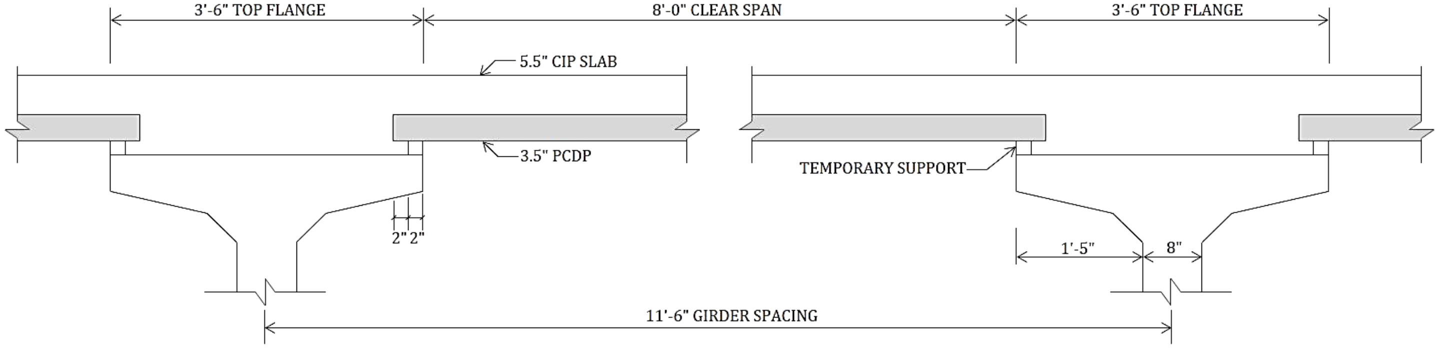

Figure B.2-1.Illistration. Details of the Partial-Depth Precast Deck Panel on Supports

B.3 MINIMUM SLAB THICKNESS

Total deck thickness (panel + CIP),	t_d =	9.0 in	
Structural deck thickness:	$t_s = t_d - t_{sac}$ = 8.5 in > 7.0 in	**OK**	***LRFD Art. 9.7.1.1***
Panel thickness should be ≤ 55% t_d,	t_p = 3.5 in < $0.55t_d$ =	4.95 in **OK**	***LRFD Art. 9.7.4.3.1***

B.4 LOADS

PDDPs support dead loads and construction loads. Superimposed dead and live loads are assumed to be supported by the composite PDDP and CIP system.

B.4.1 Dead Loads

Weight of PDDP:	$W_{PDDP} = (t_p)(w_c)$ =	0.044	ksf
Weight of CIP slab:	$W_{CIP} = (t_{CIP})(w_c)$ =	0.069	ksf
Weight of New Jersey-type barrier:	W_{NJb} =	0.543	kips/ft/side

B.4.2 Wearing Surface and Construction Loads

Weight of future wearing surface:	$W_{FWS} = (t_{FWS})(w_c)$ =	0.025	ksf
Construction load (applied to PDDP only)	W_{const} =	0.050	ksf

B.4.3 Live Loads

Per LRFD Article 3.6.1.3.3, where the spans of primary strips are transverse and do not exceed 15 ft, only the 32-kip axle loads of the design truck should be applied.

Multiple Presence Factor: ***LRFD Table 3.6.1.1.2-1***

- Single Truck = 1.2
- Two trucks = 1.0
- Three trucks = 0.85

Dynamic Load Allowance = 33% ***LRFD Table 3.6.2.1-1***

LRFD Table A4-1 tabulates maximum positive and negative live load moments per unit width for varying span lengths. Multiple presence factors and the dynamic load allowance are included in the tabulated values. The moments are applicable for decks supported on at least three girders and having a width of not less than 14.0 ft between the centerlines of the exterior girders.

For the deck under consideration, the maximum positive bending moment,

M_{LL+I} = 7.74 ft-kips/ft ***LRFD Table A4-1***

B.4.4 Load Combinations LRFD Article 3.4

The total factored force effect shall be taken as: $Q = \Sigma\eta_i\gamma_iQ_i$ ***LRFD Eq. 3.4.1-1***

where:

- η_i = a load modifier relating to ductility, redundancy, and operational classification ***LRFD Article 1.3.2.1***
- γ_i = load factor ***LRFD Table 3.4.1-1***
- Q_i = force effects from specified loads

Investigating different limit states given in LRFD Article 3.4.1, the following limit states are applicable:

Service I: check compressive stresses in prestressed concrete components,

Q = 1.00(DC+DW)+1.00(LL+IM) ***LRFD Table 3.4.1-1***

Service I is the general combination for service limit state stress checks and applies to all conditions other than Service III.

Service III: check tensile stresses in prestressed concrete components,

Q = 1.00(DC+DW)+0.80(LL+IM) ***LRFD Table 3.4.1-1***

Service III is the load combination for longitudinal analysis relating to tension in prestressed concrete superstructures with the objective of crack control. Since this load combination is for longitudinal girders, the 0.8 factor for live load has been replaced with a factor 1.0 for this example.

Strength I: check ultimate strength ***LRFD Tables 3.4.1-1 and 2***

- Maximum Q = 1.25(DC)+1.50(DW)+1.75(LL+IM)
- Minimum Q = 0.90(DC)+0.65(DW)+1.75(LL+IM)

Strength I is the general load combination for strength limit state design.

Fatigue: Fatigue need not be investigated for concrete slabs in multi-beam bridges ***LRFD Article 5.5.3.1 & 9.5.3***

B.5 CROSS-SECTION PROPERTIES FOR A TYPICAL PANEL

B.5.1 Noncomposite, Nontransformed Panel Section

Area of cross section of the precast panel,

Ag = (tp)(12") = 42.0 in^2/ft

Moment of inertia about the centroid of the noncomposite precast panel,

$$I_g = \frac{(12")(t_p)^3}{12} = \quad 42.9 \text{ in}^4\text{/ft}$$

Section modulus for the extreme bottom fiber of the noncomposite precast panel,

$$S_b = \frac{(12")(t_p)^2}{6} = \quad 24.5 \text{ in}^3\text{/ft}$$

Section modulus for the extreme top fiber of the noncomposite precast panel,

$$S_t = \frac{(12")(t_p)^2}{6} = \quad 24.5 \text{ in3/ft}$$

Modulus of elasticity, ***LRFD Eq. 5.4.2.4-1***

$$E_c = 33{,}000K_1(w_c)^{1.5}\sqrt{f'_c}$$

where:

K_1 = correction factor for source of aggregate taken as 1.0

LRFD Table 3.5.1-1 provides unit weights of concrete to be used in the absence of more precise information. For simplicity, the concrete unit weight for both the PDDP and CIP slab is taken as 0.150 ksf for this example.

Therefore, the modulus of elasticity at transfer,

$$E_{ci} = 33{,}000(1.0)(0.150)^{1.5}\sqrt{4.5} = 4{,}067 \text{ ksi}$$

And the modulus of elasticity at service loads,

$$E_c = 33{,}000(1.0)(0.150)^{1.5}\sqrt{6.0} = 4{,}696 \text{ ksi}$$

B.5.2 Composite Section

The pretensioning reinforcement is ignored in the initial calculations of the composite section properties.

For the PDDP,

$$E_c = 33{,}000(1.0)(0.150)^{1.5}\sqrt{6.0} = 4{,}696 \text{ ksi}$$

For the CIP Slab,

$$E_c = 33{,}000(1.0)(0.150)^{1.5}\sqrt{4.0} = 3{,}834 \text{ ksi}$$

B.5.2.1 Modular Ratio between CIP and PDDP Concrete

Modular ratio between CIP slab and PDDP concrete,

$$n = \frac{E_c(\text{slab})}{E_c(\text{PDDP})} = 0.816$$

B.5.2.2 Transformed Composite Section Properties

Transformed width of CIP slab,

$$w_{t,CIP} = (n)(12") = 9.80 \text{ in/ft}$$

Transformed area of CIP slab,

$$A_{t,CIP} = (n)(12")(t_s) = 48.99 \text{ in}^2$$

Transformed moment of inertia of CIP slab,

$$I_{t,CIP} = n\frac{(12")(t_s)^3}{12} = 102.06 \text{ in}^4/\text{ft}$$

Figure B.5.2.2-1 shows the dimensions of the composite section. The 5.5" CIP slab is inclusive of a 1/2" thick sacrificial layer, so only the 5" structural thickness is considered as part of the composite section.

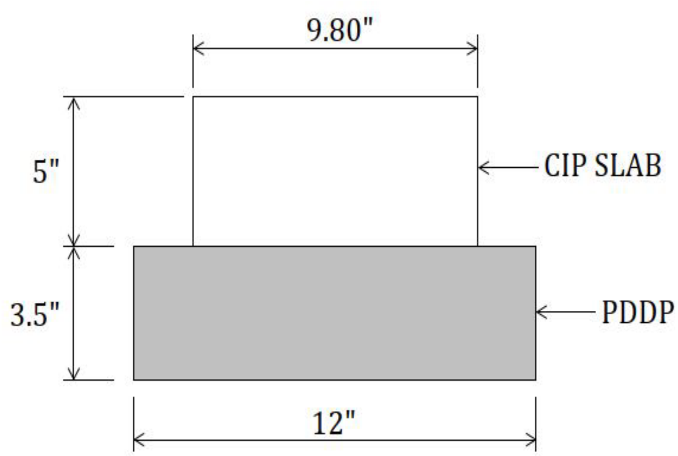

Figure B.5.2.2-1.Illistration.Transformed Composite Section

Total area of composite section,

$A_c = (12")(t_p)+(w_{t,CIP})(t_s) = 90.99\ in^2$

Distance from centroid of composite section to PDDP extreme bottom,

$$y_{bc} = \frac{A_g(t_p/2)+A_{t,CIP}(t_p+t_s/2)}{A_c} = 4.04\ in$$

Distance from centroid of composite section to extreme top fiber of PDDP,

$y_{tg} = t_p - y_{bc} = 0.54\ in$

Distance from centroid of composite section to extreme top fiber of CIP slab,

$y_{tc} = t_p + t_s - y_{bc} = 4.46\ in$

Moment of inertia of composite section,

$$I_c = \frac{A_g t_p{}^2}{12} + A_g\left(y_{bc} - {}^{t_p}/_{2}\right)^2 + \frac{A_{t,CIP} t_s{}^2}{12} + A_{t,CIP}\left(t_p + {}^{t_s}/_{2} - y_{bc}\right)^2 = 553.39\ in^4/ft$$

Composite section modulus for the extreme bottom fiber of the PDDP,

$S_{bc} = I_c/y_{bc} = 137.04\ in^3/ft$

Composite section modulus for the extreme top fiber of the PDDP,

$S_{tg} = I_c/y_{tg} = 1028.14\ in^3/ft$

Composite section modulus for the extreme top fiber of the CIP slab,

$$S_{tc} = \left(\frac{1}{n}\right)(I_c/y_{tc}) = 151.90\ in^3/ft$$

B.6 ESTIMATE REQUIRED PRESTRESS

B.6.1 Service Load Stresses at Midspan

Bottom tensile stress due to applied dead and live loads, using the modified Service III load combination:

$$f_b = \frac{M_{PDDP}+M_{CIP}}{S_b} + \frac{M_{FWS}+M_{NJb}+M_{LL+I}}{S_{bc}} = \quad 1.237 \text{ ksi}$$

where:

- f_b = concrete tensile stress at bottom fiber of panel, ksi
- M_{PDDP} = unfactored bending moment due to PDDP self-weight, ft-kips/ft
- M_{CIP} = unfactored bending moment due to CIP slab weight, ft-kips/ft
- M_{FWS} = unfactored bending moment due to future wearing surface, ft-kips/ft
- M_{NJb} = unfactored bending moment due to New Jersey barrier weight, ft-kips/ft
- M_{LL+I} = unfactored bending moment due to live load plus impact, ft-kips/ft

The span length is conservatively taken as the width of the PDDP for moment due to self-weight of the PDDP and CIP topping.

- $M_{PDDP} = (W_{PDDP})(W_p)^2/\,8 = 0.411$ ft-kips/ft
- $M_{CIP} = (W_{CIP})(W_p)^2/\,8 = 0.645$ ft-kips/ft

Per LRFD Article 4.6.2.1.6, strips shall be treated as continuous beams and the span length shall be taken as the center-to-center distance between the supporting components. Supporting components shall be assumed to be infinitely rigid. Computer software for continuous beam analysis was utilized to determine bending moments due to the future wearing surface and New Jersey barrier. Moments are shown in Figure B.6.1-1.

To arrive at maximum effects, consider the interior span, where:

- M_{FWS} = 0.095 ft-kips/ft
- M_{NJb} = 0.380 ft-kips/ft

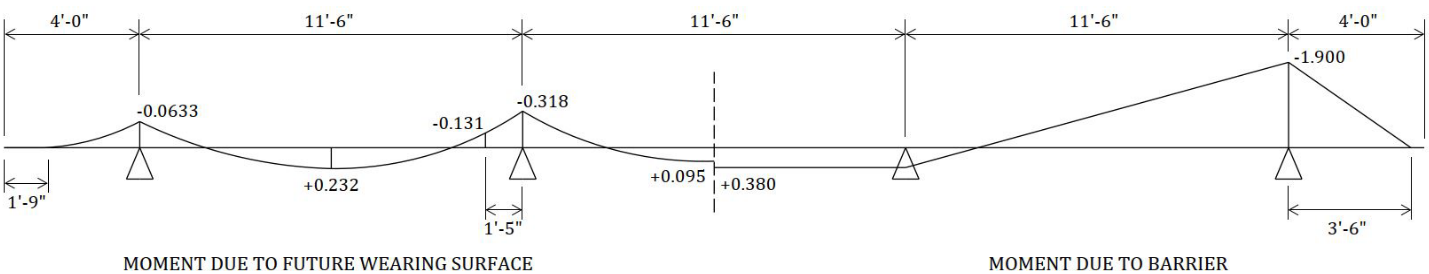

Figure B.6.1-1.Illistration.Bending Moments in ft-kips/ft

B.6.2 Stress Limits for Concrete

Concrete tensile stress limit at service loads,

$f_t = 0.19\sqrt{f'_c} \leq 0.6 =$ -0.465 ksi **LRFD Table *5.9.2.3.2b-1***

B.6.3 Required Number of Strands

The required precompressive stress at bottom fiber of the panel is the difference between bottom tensile stress due to the applied loads and the concrete tensile stress limit:

Required precompressive stress at PDDP bottom fiber,

$f_{pb} = f_b - f_t = 0.771$ ksi

If P_{pe} is the total effective prestress force after all losses:

$F_{pb} = \frac{P_{pe}}{A_g} + \frac{P_{pe} e_c}{S_b} \rightarrow P_{pe} = \frac{A_g f_{bp} S_b}{e_c A_g + S_b} =$ 18.29 kips/ft $\times L_p$ = 146.30 kips/panel

Final prestress force per strand,

$P_{pf} = f_{pi}(A_s)$(1-assumed final losses) = 14.63 kips (with assumed final losses = 15%)

Required number of strands,

P_{pe} / P_{pf} = 10.00 strands/panel **Therefore, try 12 strands/panel**

B.6.4 Strand Pattern

- Distance between cg of bottom strands and bottom concrete fiber of panel, $y_{bs} = c_{bp} =$ 1.30 in
- Distance from centroid of PDDP to extreme bottom fiber of noncomposite panel, $y_b = t_p/2 =$ 1.75 in
- Strand eccentricity, $e_c = y_b - y_{bs} =$ 0.45 in

B.6.5 Steel Transformed Section Properties

The prestressing steel area is multiplied by (n-1) to calculate the transformed section properties, where n is the modular ratio between prestressing strand and concrete. Since the modulus of elasticity is different at transfer and final time, the transformed section properties are calculated separately in the two stages. The transformed section properties are calculated as shown in Table B.6.5-1.

- At transfer, $n-1 = E_p/E_{ci} - 1 = 6.008$
- At final, $n-1 = E_p/E_c - 1 = 5.069$

Table B.6.5-1. Properties of Composite Transformed Section at Final

Location	Transformed Area, in^2	y_b in	Ay_b in^3	$A(y_{btc} - y_b)^2$ in^4	I in^4	$I+A(y_{btc} - y_b)^2$ in^4
Panel	42.00	1.75	73.5	216.2	42.88	259.1
Slab	48.99	6.00	293.9	192.3	102.06	294.3
Row 1	0.65	1.30	0.84	4.8	-	4.8
Σ	91.6	-	368.3	-	-	558.2

B.6.5.1 Noncomposite Transformed Section at Transfer:

Area of transformed section at transfer, $A_{ti} =$ 42.8 in^2/ft

Moment of inertia of the transformed section at transfer, $I_{ti} =$ 43.0 in^4/ft

Eccentricity of strands with respect to transformed section at transfer, $e_{ti} =$ 0.44 in

Distance from centroid of transformed section to extreme bottom fiber of beam at transfer, $y_{bti} =$ 1.74 in

Section modulus for extreme bottom fiber of transformed section at transfer, $S_{bti} =$ 24.7 in^3/ft

Section modulus for extreme top fiber of transformed section at transfer, $S_{tti} =$ 24.5 in^3/ft

B.6.5.2 Noncomposite Transformed Section at Final:

Area of transformed section at final,	A_{tf} =	42.6	in^2/ft
Moment of inertia of the transformed section at final,	I_{tf} =	42.9	in^4/ft
Eccentricity of strands with respect to transformed section at final,	e_{tf} =	0.44	in
Distance from centroid of transformed section to extreme bottom fiber of beam at final,	y_{btf} =	1.74	in
Section modulus for extreme bottom fiber of transformed section at final,	S_{btf} =	24.6	in^3/ft
Section modulus for extreme top fiber of transformed section at final,	S_{ttf} =	24.4	in^3/ft

B.6.5.3 Composite Transformed Section at Final:

Area of transformed composite section at final,	A_{tc} =	91.6	in^2/ft
Moment of inertia of the transformed composite section at final,	I_{tc} =	558.2	in^4/ft
Eccentricity of strands with respect to transformed composite section at final,	e_{tc} =	2.72	in
Distance from centroid of transformed composite section to extreme bottom fiber of beam at final,	y_{btc} =	4.02	in
Section modulus for extreme bottom fiber of transformed composite section at final,	S_{btc} =	138.9	in^3/ft
Section modulus for extreme top fiber of transformed composite section at final,	S_{ttc} =	1075.7	in^3/ft
Section modulus for extreme top fiber of deck of transformed composite section at final,	S_{dtc} =	124.6	in^3/ft

B.7 PRESTRESS LOSSES

Total prestress loss,

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT}$$ ***LRFD Eq. 5.9.3.1-1***

where:

- Δf_{pT} = total loss
- Δf_{pES} = sum of all losses or gains due to elastic shortening or extension at the time of application of prestress and/or external loads
- Δf_{pLT} = losses due to long-term shrinkage and creep of concrete, and relaxation of the steel

Note that the PDDP will be considered as the "girder" and the CIP composite slab will be considered as the "deck" in the analysis.

B.7.1 Elastic Shortening

Loss due to elastic shortening,

$$\Delta f_{pES} = \frac{E_p}{E_{ct}} f_{cgp} = \quad 5.02 \text{ ksi}$$ ***LRFD Eq. 5.9.3.2.3a-1***

where:

- E_p = modulus of elasticity of prestressing steel
- E_{ct} = modulus of elasticity of concrete at transfer or at time of load application
- f_{cgp} = concrete stress at the center of gravity of prestressing tendons due to the prestressing force immediately after transfer and the self-weight of the member at the section of maximum moment

When transformed section properties are used to calculate concrete stress, the effects of loss and gains due to elastic deformations are implicitly accounted for.

Force per strand before transfer,

$F_{str,i} = (A_s)(f_{pi}) = 17.21$ kips

Concrete stress after transfer,

$$f_{cgp} = \frac{P_{pi}}{A_{ti}} + \frac{P_{pi} e_{ti}^2}{I_{ti}} - \frac{M_{PDDP} e_{ti}}{I_{ti}} \quad = 0.717 \text{ ksi}$$

where:

- e_{ti} =eccentricity of strands with respect to transformed section at transfere$_{ti}$ = 0.44 in
- P_{pi} = total prestressing force before transfer,$(F_{str,i})$(strands) / L_p = 25.8 kips/ft

Per AASHTO LRFD C5.9.3.2.3a, when using transformed section properties, the prestressing strand and the concrete are treated together as a composite section in which both the concrete and the prestressing strand are equally strained. Therefore, the loss due to elastic shortening at transfer need not be added to the time dependent losses to determine total losses.

B.7.2 Time-Dependent Losses between Transfer and Deck Placement

The following construction schedule is assumed in calculating the time-dependent losses:

- Concrete age at transfer, t_i = 1 days
- Concrete age at deck placement, t_d = 90days
- Concrete age at final stage, t_f = 20,000 days

The total time-dependent loss between time of transfer and CIP deck placement is the summation of prestress loss due to shrinkage of precast concrete, creep of precast concrete, and relaxation of prestressing strands.

B.7.2.1 Shrinkage of Precast Concrete

Prestress loss due to shrinkage of concrete between

time of transfer and deck placement:

$\Delta f_{pSR} = \varepsilon_{bid} E_p K_{id} =$ 10.170 ksi **_LRFD Eq. 5.9.3.4.2a-1_**

where:

- ε_{bid} =concrete shrinkage strain of girder between the time of transfer and deck placement
- E_p = modulus of elasticity of prestressing steel
- K_{id} =transformed section coefficient that accounts for time-dependent interaction between concrete and bonded steel in the section being considered for time period between transfer and deck placement

Concrete shrinkage strain,

$\varepsilon_{bid} = k_s k_{hs} k_f k_{td} 0.48 \times 10^{-3}$ = 0.00038 **_LRFD Eq. 5.4.2.3.3-1_**

where:

- k_s = factor for the effect of the volume-to-surface ratio
- k_{hs} = humidity factor for shrinkage
- k_f = factor for the effect of concrete strength
- k_{td} = time development factor

Factor for effect of volume-to-surface ratio,

$k_s = 1.45 - 0.13(V/S) \geq 1.0 = 1.237$ **_LRFD Eq. 5.4.2.3.2-2_**

where:

- PDDP Volume, V = 20.22 ft^3
- PDDP Surface, S = 148.39 ft^2

Humidity factor for shrinkage,

$k_{hs} = (2.00 - 0.014H) = 1.02$ **_LRFD Eq. 5.4.2.3.3-2_**

where:

Average annual ambient relative humidity, H = 70% **_LRFD Figure 5.4.2.3.3-1_**

Factor for effect of concrete strength,

$k_f = \frac{5}{1+f'_{ci}} =$ 0.909 **_LRFD Eq. 5.4.2.3.2-4_**

Time development factor,

$$k_{td} = \frac{t}{12\left(\frac{100-4f'_{ci}}{f'_{ci}+20}\right)+t} = \quad 0.689$$ ***LRFD Eq. 5.4.2.3.2-5***

where:

$t = t_d - t_i = 89$ days

Transformed section coefficient,

$$K_{id} = \frac{1}{1+\frac{E_p}{E_{ci}}\frac{A_{ps}}{A_g}\left(1+\frac{A_g e_{pg}^2}{I_g}\right)[1+0.7\psi_b(t_f,t_i)]} = \quad 0.940$$ ***LRFD Eq. 5.9.3.4.2a-2***

where:

- $A_{ps} = (A_s)(\text{strands})/L_p = 0.128$ in^2/ft
- $e_{pg} = 0.45$ in
- $\psi(t_f,t_i) = 1.9\, k_s\, k_{hc}\, k_f\, k_{td}\, t_i^{-0.118} = 2.131$ ***LRFD Eq. 5.4.2.3.2-1***
- $k_{hc} = 1.56\text{-}0.008H = 1.00$ ***LRFD Eq. 5.4.2.3.2-3***
- $t = t_f - t_i = 19{,}999$ days
- $k_{td} = \frac{t}{12\left(\frac{100-4f'_{ci}}{f'_{ci}+20}\right)+t} = 0.997$ ***LRFD Eq. 5.4.2.3.2-5***

B.7.2.2 Creep of Precast Concrete

Prestress loss due to creep of concrete between transfer and deck placement,

$$\Delta f_{pCR} = \frac{E_p}{E_{ci}} f_{cgp}\psi_b(t_d,t_i)K_{id} = \quad 6.955 \text{ ksi}$$ ***LRFD Eq. 5.9.3.4.2b-1***

were:

$\psi(t_d,t_i) = 1.9\, k_s\, k_{hc}\, k_f\, k_{td}\, t_i^{-0.118} = 1.473$ ***LRFD Eq. 5.4.2.3.2-1***

B.7.2.3 Relaxation of Prestressing Strands

Prestress loss due to relaxation of strands between transfer and deck placement,

$$\Delta f_{pR1} = \frac{f_{pt}}{K_L}\left(\frac{f_{pt}}{f_{py}} - 0.55\right) = \quad 1.729 \text{ ksi}$$ ***LRFD Eq. 5.9.3.4.2c-1***

where:

- f_{pt} =stress in prestressing strands immediately after transfer, taken as not less than $0.55f_{py}$, $f_{pt} = 197.5$ ksi
- K_L =factor accounting for type of steel taken as 30 for low relaxation strands and 7.0 for other prestressing steel, unless more accurate manufacturer's data are available, $K_L = 30$

B.7.3 Time-Dependent Losses between Deck Placement and Final

The total time-dependent loss between time of CIP deck placement and final time is the summation of prestress losses due to shrinkage of precast concrete, creep of precast concrete, relaxation of prestressing strands, and shrinkage of CIP deck concrete.

B.7.3.1 Shrinkage of Precast Concrete

Prestress loss due to shrinkage of concrete between deck placement and final,

$\Delta f_{pSD} = \varepsilon_{bdf} E_p K_{df}$ = 4.585 ksi **_LRFD Eq. 5.9.3.4.3a-1_**

where:

- ε_{bdf} = shrinkage strain of panel between time of deck placement and final
- K_{df} = transformed section coefficient that accounts for time-dependent interaction between concrete and bonded steel in the section being considered for the time between deck placement and final time

Shrinkage strain of panel between transfer and final, $\varepsilon_{bif} = k_s k_{hs} k_f k_{td} 0.48\times10^{-3}$=0.000549 **_LRFD Eq. 5.4.2.3.3-1_**

Shrinkage strain of panel between deck placement and final, $\varepsilon_{bdf} = \varepsilon_{bif} - \varepsilon_{bid}$ =0.000170

Transformed section coefficient between deck placement and final,

$K_{df} = \dfrac{1}{1+\frac{E_p A_{ps}}{E_{ci} A_c}\left(1+\frac{A_c e_{pc}^2}{I_c}\right)[1+0.7\psi_b(t_f,t_I)]}$ = 0.949 **_LRFD Eq. 5.9.3.4.3a-2_**

B.7.3.2 Creep of Precast Concrete

Prestress loss due to creep of concrete between deck placement and final,

$\Delta f_{pCD} = \frac{E_p}{E_{ci}} f_{cgp}[\psi_b(t_f,t_i) - \psi_b(t_d,t_i)]K_{df} + \frac{E_p}{E_c}\Delta f_{cd}\psi_b(t_f,t_d)K_{df} =$ 2.576 ksi **_LRFD Eq. 5.9.3.4.3b-1_**

The transformed section coefficient, K_{id}, is already included in the calculation of the losses between initial time and CIP deck placement. Therefore, gross section properties are used in the calculation of Δf_{cd} for the long-term losses.

B.7.3.3 Relaxation of Prestressing Strands

Prestress loss due to relaxation of prestressing strands in composite section between time of CIP deck placement and final time,

$\Delta f_{pR2} = \Delta f_{pR1}$ = 1.729 ksi **_LRFD Eq. 5.9.3.4.3c-1_**

B.7.3.4 Shrinkage of CIP Concrete

Prestress gain due to shrinkage of CIP deck concrete,

$\Delta f_{pSS} = \frac{E_p}{E_c}\Delta f_{cdf} K_{df}[1 + 0.7\psi_b(t_f,t_d)]$ = 0.758 ksi **_LRFD Eq. 5.9.3.4.3d-1_**

where:

- $\Delta f_{cdf} = \frac{\varepsilon_{ddf} A_d E_{c,deck}}{[1+0.7\psi_d(t_f,t_d)]}\left(\frac{1}{A_c} - \frac{e_{pc} e_d}{I_c}\right)$ = 0.070 ksi **_LRFD Eq. 5.9.3.4.3d-2_**
- $\varepsilon_{ddf} = k_s k_{hs} k_f k_{td} 0.48\times10^{-3}$ = 0.000582 **_LRFD Eq. 5.4.2.3.3-1_**
- A_d = area of CIP deck concrete, A_d = (t_s)12" = 60 in^2/ft
- $E_{c,deck}$ = 3,834 ksi
- e_d = 1.96 in
- $\psi(t_f,t_d)$ = creep coefficient of deck concrete at final time due to loading introduced shortly after deck placement
- $k_s = 1.45 - 0.13(V/S) \geq 1.0$ = 1.000 **_LRFD Eq. 5.4.2.3.2-2_**

- Deck Volume, V = 28.89 ft^3
- Deck Surface, S = 69.33 ft^2 (top of deck only)

➢ $k_f = \frac{5}{1+f'_{ci}}$ = 1.190 ***LRFD Eq. 5.4.2.3.2-4***

➢ $k_{td} = \frac{t}{12\left(\frac{100-4f'_{ci}}{f'_{ci}+20}\right)+t}$ = 0.998 (assuming f'_{ci}=0.8f'_c) ***LRFD Eq. 5.4.2.3.2-5***

- $t = t_f - t_d$ = 19,910 days

➢ $\psi(t_f, t_d) = 1.9k_sk_{hc}k_fk_{td}\times10^{-118}$ = 2.257 ***LRFD Eq. 5.4.2.3.2-1***

B.7.3.6 Total Losses at Transfer

Per AASHTO LRFD C5.9.3.2.3a, the losses due to elastic shortening at transfer need not be added to the time dependent losses to determine total losses. However, the losses due to elastic shortening at transfer shall be included in determining the effective stress in prestressing strands immediately after transfer.

Change in effective prestress in prestressing strands,	$\Delta f_{pi} = \Delta f_{pES}$ =	5.02	ksi
Effective stress in tendons immediately after transfer,	$f_{pt} = f_{pi} - \Delta f_{pi}$ =	197.48	ksi
Force per strand,	$P_{st} = (f_{pt})(A_s)$ =	16.79	kips
Total prestressing force after transfer,	$P_{pt} = (P_s)$(strands) =	201.43	kips/panel
	$P_{pt} = P_{pt} / L_p$ =	25.18	kips/ft
Initial loss,	% = $\Delta f_{pi} / f_{pi}$ =	2.5%	

When determining the concrete stresses using transformed section properties the strand force is that before transfer:

➢ Force per strand, $P_{si} = (f_{pi})(A_s)$ = 17.21kips

➢ Total prestressing force before transfer, $P_{pi} = (P_s)$(strands) = 206.55kips/panel = 25.82 kips/ft

B.7.3.6 Total Losses at Service Loads

Total loss due to elastic shortening at transfer and long-term losses,

$\Delta f_{pt} = \Delta f_{pES} + \Delta f_{pLT}$ = 33.52 ksi

Elastic gain due to deck weight, superimposed dead load, and live load:

$$\Delta f_{pes} = \left(\frac{M_{CIP}e_{tf}}{I_{tf}} + \frac{(M_{NJb}+M_{FWS})e_{tc}}{I_{tc}}\right)\frac{E_p}{E_c} + 0.8\left(\frac{M_{LL+I}e_{tc}}{I_{tc}}\right)\frac{E_p}{E_c} = \quad 2.85 \text{ ksi}$$

Effective stress in tendons after all losses and gains,

$f_{pe} = f_{pi} - \Delta f_{pT} + \Delta f_{pes}$ = 171.83 ksi

Check prestressing stress limit at service limit state:

$f_{pe} \le 0.8f_{py}f_{pe}$ = 171.83 ksi < $0.8f_{py}$ = 194.4 ksi **OK**

Effective stress in strands after all losses and permanent gains,

$$F_{pe} = f_{pi} - \Delta f_{pT} + \left(\frac{M_{CIP}e_{tf}}{I_{tf}} + \frac{(M_{NJb}+M_{FWS})e_{tc}}{I_{tc}}\right)\frac{E_p}{E_c} \quad = 169.63 \text{ ksi}$$

Force per strand without live load losses, $P_{se} = (f_{pe})(A_s)$ = 14.42 kips

Total prestressing force after all losses $P_{pe} = (P_{se})$(strands) = 173.02kips/panel = 21.63 kips/ft

Final loss percentage,

$$\% = \frac{\Delta f_{pT} - \left(\frac{M_{CIP} e_{tf}}{I_{tf}} + \frac{(M_{NJb} + M_{FWS}) e_{tc}}{I_{tc}}\right)\frac{E_p}{E_c}}{f_{pi}} = 16.5\%$$

When determining the concrete stress using transformed section properties, all the elastic gains and losses are implicitly accounted for:

- Force per strand with only time dependent losses, $P_{sLT} = (f_{pi} - \Delta_{fpLT})(A_s)$ =14.85 kips
- Total prestressing force, $P_{pe} = (P_{sLT})$(strands) = 178.25 kips/panel =22.28kips/ft

B.8 CONCRETE STRESSES IN THE PDDP AT TRANFER

B.8.1 Stress Limits for Concrete

Concrete compressive stresses = $0.65f'_{ci}$ = 2.925 ksi ***LRFD Art. 5.9.2.3.1a***

Concrete tensile stresses for normal weight concrete and λ =1.0: ***LRFD Table 5.9.2.3.1b-1***

- without bonded reinforcement = $0.0948\lambda\sqrt{f'_{ci}} \leq 0.2$ = -0.200 ksi
- with bonded reinforcement sufficient to resist tensile force= $0.24\lambda\sqrt{f'_{ci}}$ = -0.509 ksi

B.8.2 Stresses at Midspan

Effective prestress after transfer,	P_{pi} =	25.82	kips/ft
Bending moment due to self-weight of PDDP,	M_{PDDP} =	0.411	ft-kips/ft
Thickness of PDDP,	t_p =	3.5	in
Area of transformed section at transfer,	A_{ti} =	42.8	in^2/ft
Section modulus for extreme bottom fiber of transformed section at transfer,	S_{bti} =	24.7	in^3/ft
Section modulus for extreme top fiber of transformed section at transfer,	S_{tti} =	24.5	in^3/ft
Moment of inertia of the transformed section at transfer,	I_{ti} =	43.0	in^4/ft
Eccentricity of strands with respect to transformed section at transfer,	e_{ti} =	0.44	in
Distance from centroid of transformed section to extreme bottom fiber of beam at transfer,	y_{bti} =	1.74	in

Compute stress in the top of PDDP:

$$f_t = \frac{P_{pi}}{A_{ti}}\left(1 - \frac{e_{ti}(t_p - y_{bti})}{I_{ti}/A_{ti}}\right) + \frac{M_{PDDP}}{S_{tti}} = 0.339 \text{ ksi}$$

Check compressive stress limit: 2.925 ksi > 0.339 ksi **OK**

Compute stress in the bottom of PDDP:

$$f_b = \frac{P_{pi}}{A_{ti}}\left(1 + \frac{e_{ti}y_{bti}}{I_{ti}/A_{ti}}\right) - \frac{M_{PDDP}}{S_{bti}} = 0.864 \text{ ksi}$$

Check compressive stress limit: 2.925 ksi > 0.864 ksi **OK**

B.9 CONCRETE STRESSES IN PDDP AT TIME OF CASTING TOPPING SLAB

B.9.1 Stress Limits for Concrete

Per LRFD Article 9.7.4.1, flexural stresses in stay-in-place formwork due to unfactored construction loads shall not exceed 65% of the 28-day compressive strength for concrete in compression or the modulus of rupture in tension for prestressed concrete form panels.

Concrete compressive stresses for Service I:	$0.65f'_c =$	3.900	ksi	***LRFD Art. 9.7.4.1***
Concrete tensile stresses for Service I:	$f_r = 0.24\lambda\sqrt{f'_c} =$	-0.588	ksi	***LRFD Art. 5.4.2.6***

B.9.2 Stresses at Midspan after all Noncomposite Loads

Effective prestress using transformed section properties and refined losses,	$P_{pe} =$	22.28	kips/ft
Bending moment due to self-weight of PDDP,	$M_{PDDP} =$	0.411	ft-kips/ft
Bending moment due to CIP concrete slab,	$M_{CIP} =$	0.645	ft-kips/ft
Bending moment due to construction load,	$M_{const} = (W_{const})(W_p)^2 / 8 =$	0.469	ft-kips/ft
Thickness of PDDP,	$t_p =$	3.5	in
Area of noncomposite transformed section at final,	$A_{tf} =$	42.6	in^2/ft
Section modulus for extreme bottom fiber of noncomposite transformed section at final,	$S_{btf} =$	24.6	in^3/ft
Section modulus for extreme top fiber of noncomposite transformed section at final,	$S_{ttf} =$	24.4	in^3/ft
Moment of inertia of the transformed section at final,	$I_{tf} =$	42.9	in^4/ft
Eccentricity of strands with respect to transformed section at final,	$e_{tf} =$	0.44	in
Distance from centroid of transformed section to extreme bottom fiber of beam at final,	$y_{btf} =$	1.74	in

Compute stress at top fiber of PDDP:

$$f_t = \frac{P_{pe}}{A_{tf}}\left(1 - \frac{e_{tf}(t_p - y_{btf})}{I_{tf}/A_{tf}}\right) + \frac{M_{PDDP}+M_{CIP}+M_{const}}{S_{ttf}} = 0.868 \text{ ksi}$$

Check compressive stress limit: 3.9 ksi > 0.868 ksi **OK**

Compute stress at bottom fiber of PDDP:

$$f_b = \frac{P_{pe}}{A_{tf}}\left(1 + \frac{e_{tf}y_{btf}}{I_{tf}/A_{tf}}\right) - \frac{M_{PDDP}+M_{CIP}+M_{const}}{S_{btf}} = 0.180 \text{ ksi}$$

Check compressive stress limit: 3.9 ksi > 0.18 ksi **OK**

B.9.3 Elastic Deformation

Per LRFD Article 9.7.4.1, for stay-in-place formwork, the elastic deformations caused by the dead load of the forms, plastic concrete, and reinforcement shall not exceed either the form span length divided by 180 or 0.5" for form span lengths of 10 ft or less, or either the form span length divided by 240 or 0.75" for form span lengths greater than 10 ft.

Allowable elastic deformation, max	$(W_p/180, 0.50") =$	0.578	in	***LRFD Art. 9.7.4.1***

Elastic deformation,

$$\frac{5}{48}\frac{(M_{PDDP}+M_{CIP})W_p^2}{E_cI_g} = 0.071\text{in} < 0.578 \text{ in } \textbf{OK}$$

B.10 CONCRETE STRESSES IN PDDP AT SERVICE LOADS

B.10.1 Stress Limits for Concrete

Concrete compressive stresses for Service I: ***LRFD Table 5.9.2.3.2a-1***

- due to effective prestress and permanent loads:
  - for the PDDP: $0.45f'_c = 2.700$ksi
  - for the CIP slab: $0.45f'_c = 1.800$ksi
- due to effective prestress, permanent loads, and transient loads:
  - for the PDDP: $0.60\Phi_w f'_c = 3.600$ksi
  - for the CIP slab: $0.60\Phi_w f'_c = 2.400$ksi
    - where the reduction factor for web and flange slenderness, $\Phi_w = 1.0$ ***LRFD Art. 5.9.2.3.2a***

Concrete tensile stresses for Service III:

for the PDDP: $0.19\sqrt{f'_c} = -0.465$ ksi

B.10.2 Service Load Stresses at Midspan

Effective prestress using transformed section properties and refined losses, $P_{pe} =$ 22.28 kips/ft

The weight of the PDDP and the CIP concrete act on the noncomposite section:

- Bending moment due to self-weight of PDDP, $M_{PDDP} = 0.411$ft-kips/ft
- Bending moment due to CIP concrete slab, $M_{CIP} = 0.645$ft-kips/ft

The wearing surface, barriers, and live loads act on the composite section:

- Bending moment due to future wearing surface, $M_{FWS} = 0.095$ft-kips/ft
- Bending moment due to New Jersey-type barrier, $M_{NJb} = 0.380$ft-kips/ft
- Bending moment due to live load, $M_{LL+I} = 7.74$ft-kips/ft

B.10.2.1 Composite and Noncomposite Transformed Section Properties

Thickness of PDDP,	$t_p =$	3.5	in
Area of noncomposite transformed section at final,	$A_{tf} =$	42.6	in^2/ft
Section modulus for extreme top fiber of deck of transformed composite section at final,	$S_{dtc} =$	124.6	in^3/ft
Section modulus for extreme bottom fiber of transformed noncomposite section at final,	$S_{btf} =$	24.6	in^3/ft
Section modulus for extreme top fiber of transformed noncomposite section at final,	$S_{ttf} =$	24.4	in^3/ft
Section modulus for extreme bottom fiber of transformed composite section at final,	$S_{btc} =$	138.9	in^3/ft
Section modulus for extreme top fiber of transformed composite section at final,	$S_{ttc} =$	1075.7	in^3/ft
Moment of inertia of the noncomposite transformed section at final,	$I_{tf} =$	42.9	in^4/ft
Eccentricity of strands with respect to noncomposite transformed section at final,	$e_{tf} =$	0.44	in
Distance from centroid of transformed section to extreme bottom fiber of beam at final,	$y_{btf} =$	1.74	in

B.10.2.2 Concrete Stress at Top Surface of the CIP Slab

Due to permanent loads, Service I,

$$f_{tc} = \frac{M_{FWS}+M_{NJb}}{S_{dtc}} \quad = 0.046 \text{ ksi}$$

Check compressive stress limit: 1.8 ksi > 0.046 ksi **OK**

Due to permanent and transient loads, Service I,

$$f_{tc} = \frac{M_{FWS}+M_{NJb}+M_{LL+I}}{S_{dtc}} \quad = 0.791\text{ksi}$$

Check compressive stress limit: 2.4 ksi > 0.791ksi **OK**

B.10.2.3 Concrete Stress at Top Fiber of the PDDP

Due to permanent loads, Service I,

$$F_{tg} = \frac{P_{pe}}{A_{tf}}\left(1 - \frac{e_{tf}(t_p - y_{btf})}{I_{tf}/A_{tf}}\right) + \frac{M_{PDDP}+M_{CIP}}{S_{ttf}} + \frac{M_{FWS}+M_{NJb}}{S_{ttc}} \quad = \quad 0.643 \quad \text{ksi}$$

Check compressive stress limit: 2.7 ksi > 0.643 ksi **OK**

Due to permanent and transient loads, Service I:

$$F_{tg} = \frac{P_{pe}}{A_{tf}}\left(1 - \frac{e_{tf}(t_p - y_{btf})}{I_{tf}/A_{tf}}\right) + \frac{M_{PDDP}+M_{CIP}}{S_{ttf}} + \frac{M_{FWS}+M_{NJb}+M_{LL+I}}{S_{ttc}} \quad = 0.729 \quad \text{ksi}$$

Check compressive stress limit: 3.6 ksi > 0.729 ksi **OK**

B.10.2.4 Concrete Stress at Bottom Fiber of the PDDP

Due to permanent and transient loads, Service III:

$$F_b = \frac{P_{pe}}{A_{tf}}\left(1 + \frac{e_{tf}y_{btf}}{I_{tf}/A_{tf}}\right) - \frac{M_{PDDP}+M_{CIP}}{S_{btf}} - \frac{M_{FWS}+M_{NJb}+M_{LL+I}}{S_{btc}} \quad = -0.301\text{ksi}$$

Check tensile stress limit: -0.465 ksi > -0.301ksi **OK**

B.11 FLEXURAL STRENGTH OF POSITIVE MOMENT SECTION

Strength I total ultimate bending moment:
$Mu = 1.25(DC)+1.5(DW)+1.75(LL+IM) = 1.25(M_{PDDP}+M_{CIP}+M_{NJb})+1.5(M_{FWS})+1.75(M_{LL+I}) = 15.5$ ft-kips/ft

Bending moment due to self-weight of PDDP,	$M_{PDDP} =$	0.411	ft-kips/ft
Bending moment due to CIP concrete slab,	$M_{CIP} =$	0.645	ft-kips/ft
Bending moment due to future wearing surface,	$M_{FWS} =$	0.095	ft-kips/ft
Bending moment due to New Jersey-type barrier,	$M_{NJb} =$	0.380	ft-kips/ft
Bending moment due to live load,	$M_{LL+I} =$	7.74	ft-kips/ft

Average stress in prestressing strand when $f_{pe} \geq 0.5f_{pu}$,

- $f_{ps} = f_{pu}\left(1 - k\frac{c}{d_n}\right) = 260.0$ ***LRFD Eq. 5.6.3.1.1-1***
- $K = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right) = 0.28$ ***LRFD Eq. 5.6.3.1.1-2***

Distance from extreme compression fiber of composite section to centroid of prestressing tendons, $d_p = t_s+t_p-c_{bp} =$ 7.20 in

Distance from extreme compression fiber to neutral axis,

$$c = \frac{A_{ps}f_{pu}+A_sf_y-A'_sf'_y}{\alpha_1 f'_c\beta_1 b+kA_{ps}\frac{f_{pu}}{d_p}} = 0.96 \text{ in}$$ ***LRFD Eq. 5.6.3.1.1-4***

Depth of equivalent rectangular stress block,	$a = \beta_1 c =$	0.81	in	
Area of prestressing steel,	$A_{ps} = (\text{strands})(A_s) =$	1.02	in^2	
Specified tensile strength of prestressing steel,	$f_{pu} =$	270.0	ksi	
Area of mild steel tension reinforcement,	$A_s =$	0	in^2	
Yield strength of nonprestressed tension reinforcement,	$f_y =$	60	ksi	
Area of compression reinforcement,	$A'_s =$	0	in^2	
Yield strength of nonprestressed compression reinforcement,	$f'_y =$	60	ksi	
Stress block factor,	$\alpha_1 = 0.75 \leq 0.85 - 0.02(f'_c - 10) \leq 0.85 =$	0.85		***LRFD Art. 5.6.2.2***
Compressive strength of CIP slab,	$f'_c =$	4.0	ksi	
Stress block factor,	$\beta_1 = 0.65 \leq 0.85 - 0.05(f'_c - 4) \leq 0.85 =$	0.85		***LRFD Art. 5.6.2.2***
Effective width of compression flange,	$b = L_p =$	96.0	in	

Check stress in prestressing strand according to available development length:

Stress in prestressing strands,

$$f_{ps} = \frac{l_d}{Kd_b} + \frac{2}{3}f_{pe} = 251.8 \text{ ksi}$$ ***LRFD Eq. 5.9.4.3.2-1***

For pretensioned panels,	$K =$	1.0		***LRFD Art. 5.9.4.3.2***
Nominal strand diameter,	$d_b =$	0.375	in	
Effective stress in prestressing strands after losses,	$f_{pe} =$	169.63	ksi	
Available development length at midspan of the PDDP,	$l_d = 0.5(W_p) =$	4.33	ft	
Factored flexural resistance,	$M_r = \Phi M_n =$	18.17	ft-kip/ft	***LRFD Eq. 5.6.3.2.1-1***
For tension-controlled prestressed concrete sections,	$\Phi =$	1.00		***LRFD Art. 5.5.4.2***

Nominal flexural resistance,

$M_n = A_{ps} f_{ps} \left(d_p - {}^{a}\!/_{2} \right)$ = 145.4 ft-kip/panel = 18.17 ft-kip/ft ***LRFD Eq. 5.6.3.2.2-1***

Check flexural resistance: M_u = 15.48 ft-kip/ft $< \Phi M_n$ = 18.17 ft-kip/ft **OK**

B.12 LIMITS OF REINFORCEMENT FOR POSITIVE MOMENT SECTION

Per LRFD Article 5.6.3.3, at any section of a non-compression-controlled flexural component, the amount of prestressed and nonprestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, M_r, greater than or equal to the lesser of the following:

- 1.33 times the factored moment required by the applicable strength load combination
- the cracking moment, M_{cr}

Cracking moment at midspan,

$$M_{cr} = \gamma_3 \left[(\gamma_1 f_r + \gamma_2 f_{cpe}) S_c - M_{dnc} \left(\frac{S_c}{S_{nc}} - 1 \right) \right] = 12.63 \text{ft-kips/ft}$$ ***LRFD Eq. 5.6.3.3-1***

where:

- Section modulus for extreme fiber of composite section where tensile stress is caused by externally applied loads, $S_c = S_{btc} = 138.9 in^3/ft$
- Section modulus for extreme fiber of noncomposite section where tensile stress is caused by externally applied loads, $S_{nc} = S_{btf} = 24.6 in^3/ft$
- Modulus of rupture of concrete, $f_r = 0.24\lambda\sqrt{f'_c} = 0.588 ksi$ ***LRFD Art. 5.4.2.6***
- Compressive stress in concrete due to effective prestress only at extreme fiber of section where tensile stress is caused by externally applied loads, $f_{pe} = P_{pe}/A_{tf} = 0.522 ksi$
- Noncomposite dead load moment at the section, $M_{dnc} = M_{CIP} + M_{PDDP} = 1.056$ ft-kips/ft
- Flexural cracking variability factor, $\gamma_1 = 1.6$ ***LRFD Art. 5.6.3.3***
- Prestress variability factor (for bonded tendons), $\gamma_2 = 1.1$ ***LRFD Art. 5.6.3.3***
- Ratio of specified minimum yield strength to ultimate tensile strength of nonprestressed reinforcement (taken as 1.0 for prestressed), $\gamma_3 = 1.0$ ***LRFD Art. 5.6.3.3***

1.33 times factored moment required by applicable load combination, $1.33M_u =$ 20.59 ft-kips/ft

Controlling limit on tensile reinforcement, min of (M_{cr}, $1.33M_u$) = 12.63 ft-kips/ft

Check flexural resistance: min of (M_{cr}, $1.33M_u$) = 12.63 ft-kips/ft < ΦM_n = 18.17 ft-kips/ft **OK**

B.13 NEGATIVE MOMENT SECTION OVER INTERIOR BEAMS

B.13.1 Critical Section

The design section for negative moments and shear forces for precast I-shaped concrete beams is at a distance of 1/3 of the flange width from the centerline of the support, but not exceeding 15".

Critical section (distance from centerline), min of (1/3 x flange, 15") = 14.00 in ***LRFD Art. 4.6.2.1.6***

B.13.2 Bending Moment

Per LRFD Article 4.6.2.1.6, strips shall be treated as continuous beams and the span length shall be taken as the center-to-center distance between the supporting components. Supporting components shall be assumed to be infinitely rigid. Computer software for continuous beam analysis was utilized to determine bending moments due to the future wearing surface. LRFD Table A4-1 was used to determine the maximum negative bending moment:

- DC: conservatively ignored because the weight of the barrier produces positive moment at the interior girders (Figure B.6.1-1)
- DW: due to wearing surface, M_{FWS} = 0.164 ft-kips/ft
- LL+IM: From LRFD Table A4.1-1, M_{LL+I} = 5.867 ft-kips/ft

Negative service bending moment, $M_{service} = M_{FWS} + M_{LL+I}$ = 6.031 ft-kips/ft

Negative factored bending moment, $M_u = 1.5(M_{FWS}) + 1.75(M_{LL+I})$ = 10.513 ft-kips/ft

B.13.3 Design of Section

Assumed CIP slab reinforcing:

- Reinforcing bar, d_b = 0.75 in
- Clear cover, cc = 2.5 in
- Spacing, s = 8 in

Depth from bottom fiber to centroid of top reinforcing, $d_e = t_s+t_p-0.5(d_b)-cc$ = 5.625 in

Negative factored bending moment stress, $R_n = M_u / \Phi b d_e^2$ = 0.369 ksi

Negative bending resistance factor, Φ = 0.90 ***LRFD Art. 5.5.4.2***

$m = f_y / 0.85f'_c$ = 17.65

Ratio of reinforcement to concrete in CIP slab,

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mR_n}{f_y}}\right) = 0.0065$$

Required area of steel in CIP slab, $A_s = \rho(bd_e)$ = 0.441 in^2/ft

Area of steel provided in CIP slab, A_s = 0.663 in^2/ft

Depth of equivalent rectangular stress block,

$$a = \frac{A_s f_y}{0.85bf'_c} = 0.975 \text{ in}$$

Negative moment resistance,

$$\emptyset M_n = \emptyset A_s f_y\left(d - {}^{a}/_{2}\right) = 15.32 \text{ ft-kips/ft}$$

Check negative moment capacity: M_u = 10.513ft-kips/ft < ΦM_n = 15.32ft-kips/ft **OK**

B.13.4 Minimum Reinforcement

For the negative moment section, the LRFD Eq. 5.6.3.3-1 can be reduced to:

$M_{cr} = \gamma_3\gamma_1 S_{tc} f_r = 6.514$ ft-kips

where:

- Modulus of rupture of concrete, $f_r = 0.24\lambda\sqrt{f'_c} = 0.480$ksi ***LRFD Art. 5.4.2.6***
- Composite section modulus for the extreme top fiber of the CIP slab, $S_{tc} = 151.90$ in^3
- Flexural cracking variability factor, $\gamma_1 = 1.6$ ***LRFD Art. 5.6.3.3***
- Ratio of specified minimum yield strength to ultimate tensile strength of nonprestressed reinforcement, $\gamma_3 = 0.67$ ***LRFD Art. 5.6.3.3***

1.33 times factored moment required by applicable load combination, $1.33M_u =$ 13.98 ft-kips/ft

Controlling limit on tensile reinforcement, min of $(M_{cr}, 1.33M_u) =$ 6.51 ft-kips/ft

Check flexural resistance: min of $(M_{cr}, 1.33M_u) = 6.51$ft-kips/ft $< \Phi M_n = 15.32$ft-kips/ft **OK**

B.13.5 Crack Control

Spacing of nonprestressed reinforcement in layer closest to tension face shall satisfy,

$s \leq \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c$ = 12.878 in ***LRFD Eq. 5.6.7-1***

where:

- Exposure factor (1.00 for Class 1 exposure condition), $\gamma_e = 1.00$ ***LRFD Art. 5.6.7***
- Ratio of flexural strain at the extreme tension face to the strain at the centroid of the reinforcement layer nearest the tension face,

  $\beta_s = 1 + \frac{d_c}{0.7(h-d_c)} =$ 1.730 ***LRFD Eq. 5.6.7-2***

- Thickness of concrete cover measured from extreme tension fiber to centroid of flexural reinforcement located closes thereto, $d_c = cc + 0.5(d_b) = 2.875$ in
- Overall thickness or depth of deck, $h = 8.5$ in
- Tensile stress in steel reinforcement at service limit state,

  $f_{ss} = \frac{M_{service}}{jd_e A_s} \leq 0.60f_y$ = 21.72 ksi

where:

- Negative service bending moment, $M_{service} = 6.031$ ft-kips/ft
- Factor relating lever arm to effective depth, $j = 1 - k/3 = 0.894$
- $K = \sqrt{(\rho_a n)^2 + (2\rho_a n)} - \rho_a n$ = 0.318

Actual reinforcement ratio,

$\rho_a = {}^{A_s}/_{12d_e}$ = 0.010

Modular ratio of deck steel to CIP deck concrete, $n = Es/Ec =$ 7.563

Check spacing of reinforcement: $s_{req} =$ 12.878 in > s = 8 in **OK**

B.14 DISTRIBUTION REINFORCEMENT

The LRFD Specifications does not provide specific guidelines for the required distribution reinforcement for a PDDP and CIP slab deck system. However, the following guidance for deck slabs with four layers of reinforcement is given in LRFD Article 9.7.3.2:

Percentage of distribution reinforcement to primary reinforcement,

$$\% = \frac{220}{\sqrt{S}} \leq 67\% \quad = 66.84\%$$ ***LRFD Art. 9.7.3.2***

where:

- Effective span length, S = clear span + extreme flange tip to face of web =10.833ft ***LRFD Art. 9.7.2.3***
- Area of primary reinforcement (prestressing strands), A_{ps} =0.128 in^2/ft
- Required area of distribution reinforcement (prestressed), $A_{dist,req} = (A_{ps})(\%) = 0.085\ in^2/ft$
- Equivalent nonprestressed area, $A_{dist,req} = \max(A_{dist}(f_{py}/f_y), 0.22) = 0.345\ in^2/ft$

Therefore, per Traditional Design method,

- Distribution reinforcement bar, # = 4
- Distribution reinforcement spacing, s = 6 in
- Distribution reinforcement area, A_{dist} = 0.393 in^2/ft > $A_{dist,req}$ =0.345in^2/ft **OK**

However, the distribution reinforcement area is significantly less per the Empirical Design method: ***LRFD Art. 9.7.2.5***

- Required area of bottom layer of distribution reinforcement, $A_{dist,bot}$ =0.27in^2/ft
- Required area of top layer of distribution reinforcement, $A_{dist,top}$ =0.18in^2/ft
- Total required area of distribution reinforcement, $A_{dist,req}$ =0.45in^2/ft

Therefore, per Empirical Design method,

- Distribution reinforcement bar, = #5
- Distribution reinforcement spacing, s = 8 in
- Distribution reinforcement area, A_{dist} = 0.460 in^2/ft > $A_{dist,req}$ = 0.450in^2/ft **OK**

APPENDIX C – EXAMPLE CONSTRUCTION SPECIFICATIONS

Note: The example specifications provided herein are based on the state-of-the-practice from the six Sates and are for informational purposes. It is not binding or mandatory to follow these suggested specifications. The example specifications were developed based on:

- Colorado Department of Transportation Standard Specifications for Road and Bridge Construction, 2019
- Missouri Department of Transportation Standard Specifications for Highway Construction, 2019
- New Hampshire Department of Transportation Standard Specifications, 2016
- Tennessee Department of Transportation Standard Specifications for Road and Bridge Construction, 2015
- Texas Department of Transportation Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges, 2014
- Utah Department of Transportation Standard Specifications for Road and Bridge Construction, 2017

Performance based Sample Specification

PARTIAL-DEPTH PRECAST CONCRETE DECK PANELS (PDDPs)

A. Shop Drawings

Shop Drawings, Working Drawings, and Other Submittals – General: All work shall be performed in accordance with the plans, reviewed shop drawings, working drawings, or other submittals. Specific requirements for the required shop drawings, working drawings, and other submittals for partial-depth precast concrete deck panels are specified below.

The Contractor shall be responsible for the accuracy of all dimensions and quantities shown on the shop drawings, working drawings, and other submittals. The Contractor shall correlate all information in the Contract, in the submittals, and in all revisions at the project site to ensure that there are no conflicts and that the work can be constructed as shown. The Contractor shall be responsible for all information that pertains to the fabrication processes and methods of construction.

Shop drawings, working drawings, and other submittals shall be delivered to the Engineer. The Contractor shall notify the Engineer, in writing, at the time of submittal of shop drawings, working drawings, and other submittals, of any information submitted that deviates from the requirements of the plans and specifications. In addition, specific notation of the deviations or changes from the plans and specifications shall be placed on the shop drawing, working drawing, or other submittal.

The first sheet or page of each set of shop drawings, working drawings, and other submittals shall be stamped "Approved for Construction" and signed by the Contractor. Submittals shall be made in complete packages which will allow the Engineer to properly review them for general compliance with the Contract and to effectively evaluate the proposed methods of construction. The allowed time for review shall not begin until such submittals are complete.

Working Drawings:

1. Detailed shop drawings of fabricated products for review
   a. Include the following:

1) Details and calculations of the method, materials, and equipment to be used in the prestressing operations, including additions or rearrangement of reinforcing steel and revisions to concrete dimensions from those shown.
2) Method and sequence of stressing.
   a) Specifications and details of the prestressing steel, working stresses, and other data pertaining to the prestressing operation.
   b) The proposed arrangement of the prestressing steel in the panels.
3) Locations and details of lifting inserts, hardware, or devices.
4) Type and amount of additional reinforcing required for lifting.
5) Minimum compressive strength attained before handling the precast elements.

b. Provide the seal of a Professional Engineer (PE) or Professional Structural Engineer (SE).
c. Include supporting engineering calculations.

2. Erection Drawings for precast concrete deck panels for review.
   d. Illustrate the proposed method of erection.
   e. Provide details of the process including but not limited to the following:
      1) Crane charts
      2) Crane and pick locations
      3) Cables and lifting devices
      4) Load distribution and panel weights
      5) Panel erection and sequence
      6) Sequence used to level panel
      7) Method, equipment, and sequence for forming the camber strips
      8) Method of forming closure pours at joints between precast panels.

Material Submittals:

1. Certifications
   a. Certification stating the manufacturer's minimum ultimate tensile strength for each sample of prestressing steel.
   b. The certified calibration chart required by Section 03412.
2. Pre-stressing Steel
   a. Three 6 ft long strand samples from each heat or lot that will be used on the project.
   b. Testing can require 14 calendar days after the date of receipt.

Repair Procedures:

1. Written repair procedures for defects and breakage of precast elements for approval.

Casting and Shipping Schedules:

1. A tentative casting schedule for information at least 14 calendar days in advance to make inspection and testing arrangements.
2. A tentative shipping schedule for information at least 14 calendar days before shipping precast substructure elements to the job site

B. Forms

Forms shall be subject to the approval of the Engineer.

Forms shall be made and maintained true to the shapes and dimensions shown on the plans.

The surface of forms shall be smooth, and if necessary, joints shall be treated so that a minimum of joint marks are evident in the finished member.

Forms shall be constructed and end bearing plates placed so as to allow for any shortening of the member due to compressive stresses resulting from transfer of stress and from shrinkage.

Side forms shall be of steel and shall be supported without resort to ties or spreaders within the body of the member. They shall be braced and stiffened so that no deflection or curvature occurs during concrete placement.

Forms shall be cleaned before each use.

C. Casting

Cast panels on beds that are clean, straight, level, and in good repair. Bulkheads and headers are to be of the size and configuration to adequately hold cables in place during casting operations. Keep forms, headers, cables, reinforcing bars or other steel that comes in contact with freshly placed concrete below 90 degrees F during casting operations.

D. Curing:

Cure panels using normal or accelerated curing methods. Follow the recommendations of the 1999 PCI *Manual for Quality Control MNL-116*[8] for curing, with the following modifications.

Maximum concrete curing temperature is 160degrees F. If sufficient pozzolans are incorporated in the concrete mix to make the occurrence of delayed ettringite formation (DEF) unlikely, the maximum concrete curing temperature may be increased to 170degrees F.

E. Finishing:

The top surface of the panel shall be adequately roughened to facilitate bond with the cast-in-place deck.

F. Tolerances:

Allowable fabrication tolerances for the dimensions and configurations shown on the plans or approved shop drawings are shown below.

Panel Length (parallel to bridge axis)	+3/4-inch, -1/2-inch
Panel Width (perpendicular to bridge axis)	±1/2-inch
Panel Thickness	±1/4-inch
Strand Horizontal Position	±1/2-inch
Strand Vertical Position	±1/4-inch

G. Panel Storage, Shipping and Handling:

Precast panel deck forms shall be stored and transported in a horizontal position and the Contractor shall handle the product in such a manner as to prevent cracking or damage.

Use lifting devices that can support the required vertical and horizontal forces with the applicable safety factors as specified in the Component Handling and Erection Bracing requirements in the PCI Design Handbook.

H. Rejection/Repair of Panels:

Repair small damaged or isolated honeycombed areas that are purely surface in nature and not over 1 inch in depth at the fabrication plant with an approved epoxy grout. Replace panels with more extensive damage or honeycomb.

Inspect the panels at the point of delivery to the jobsite for identification, dimensional tolerances, cracks, and structural damage. Replace panels exhibiting excessive cracking or other structural damage.

Prestressed bridge deck panels will be rejected for any of the following conditions:

1. Any crack that is oriented diagonally or perpendicular to the strands and that crosses a minimum of two strands,
2. Any crack oriented parallel to the strands that is longer than one-third of the panel length, or
3. Cracks oriented parallel to the strands that are shorter than one-third of the panel but present at more than 12 percent of the strands.

Prestressed bridge deck panels that sustain damage, cracks not listed above, or surface defects during fabrication, handling, storage, hauling, or erection are subject to review.

I. Temporary Supports

Temporary supports/grout dams for precast deck panels shall consist of continuous, high-density, expanded polystyrene strips (grout dam) with a minimum compressive strength of 55 psi. If leveling screws are used, a 1.7 pound per cubic foot polyethylene foam seal shall be used as a grout dam.

J. Installation:

Erect deck panels as shown on the Plans or reviewed Working Drawings, as applicable.

Fully brace concrete girders or steel girders before placing panels

Maintain responsibility for aspects of panel installation during stages of construction including the protection of precast deck panels, the workers, and the traveling public.

Clean bearing surfaces and surfaces that will be in permanent contact before the panes are installed.

Carefully handle materials so that no parts will be cracked, chipped, broken, or otherwise damaged.

Use lifting devices in a manner that will not cause bending or torsional forces.

Place the precast deck panels as shown in approved Working Drawings. The butt joints between precast panels shall be caulked to prevent excessive grout leakage between panels.

K. Placement of CIP Concrete:

Tops of girders, precast deck panels, pier caps, and abutments that will come into contact with bridge deck concrete shall be heated to raise the temperature above 35 degrees F prior to concrete placement.

Thoroughly wet all forms, prestressed concrete panels, T-beams, slab beams, and concrete box beams on which concrete is to be placed before placing concrete on them. Remove free water from the surface or beam lines before placing concrete. Provide surfaces that are in a moist, saturated surface-dry condition when concrete is placed on them.

Prescriptive Sample Specification

PARTIAL-DEPTH PRECAST CONCRETE DECK PANELS (PDDPs)

L. Shop Drawings:

Shop Drawings, Working Drawings, and Other Submittals – General: All work shall be performed in accordance with the plans, reviewed shop drawings, working drawings, or other submittals. Specific requirements for the required shop drawings, working drawings, and other submittals for partial-depth precast concrete deck panels are specified below.

The Contractor shall be responsible for the accuracy of all dimensions and quantities shown on the shop drawings, working drawings, and other submittals. The Contractor shall correlate all information in the Contract, in the submittals, and in all revisions at the project site to ensure that there are no conflicts and that the work can be constructed as shown. The Contractor shall be responsible for all information that pertains to the fabrication processes and methods of construction.

Shop drawings, working drawings, and other submittals shall be delivered to the Engineer. The Contractor shall notify the Engineer, in writing, at the time of submittal of shop drawings, working drawings, and other submittals, of any information submitted that deviates from the requirements of the plans and specifications. In addition, specific notation of the deviations or changes from the plans and specifications shall be placed on the shop drawing, working drawing, or other submittal.

The first sheet or page of each set of shop drawings, working drawings, and other submittals shall be stamped "Approved for Construction" and signed by the Contractor. Submittals shall be made in complete packages which will allow the Engineer to properly review them for general compliance with the Contract and to effectively evaluate the proposed methods of construction. The allowed time for review shall not begin until such submittals are complete.

Working Drawings:

3. Detailed shop drawings of fabricated products for review
    - f. Include the following:
        - 6) Details and calculations of the method, materials, and equipment to be used in the prestressing operations, including additions or rearrangement of reinforcing steel and revisions to concrete dimensions from those shown.
        - 7) Method and sequence of stressing.
            - c) Specifications and details of the prestressing steel, working stresses, and other data pertaining to the prestressing operation.
            - d) The proposed arrangement of the prestressing steel in the panels.
        - 8) Locations and details of lifting inserts, hardware, or devices.
        - 9) Type and amount of additional reinforcing required for lifting.
        - 10) Minimum compressive strength attained before handling the precast elements.
    - g. Provide the seal of a Professional Engineer (PE) or Professional Structural Engineer (SE).
    - h. Include supporting engineering calculations.
4. Erection Drawings for precast concrete deck panels for review.
    - i. Illustrate the proposed method of erection.
    - j. Provide details of the process including but not limited to the following:
        - 9) Crane charts
        - 10) Crane and pick locations

11) Cables and lifting devices
12) Load distribution and panel weights
13) Panel erection and sequence
14) Sequence used to level panel
15) Method, equipment, and sequence for forming the camber strips
16) Method of forming closure pours at joints between precast panels.

Material Submittals:

3. Certifications
   c. Certification stating the manufacturer's minimum ultimate tensile strength for each sample of prestressing steel.
   d. The certified calibration chart required by Section 03412.
4. Pre-stressing Steel
   c. Three 6 ft long strand samples from each heat or lot that will be used on the project.
   d. Testing can require 14 calendar days after the date of receipt.

Repair Procedures:

2. Written repair procedures for defects and breakage of precast elements for approval.

Casting and Shipping Schedules:

3. A tentative casting schedule for information at least 14 calendar days in advance to make inspection and testing arrangements.
4. A tentative shipping schedule for information at least 14 calendar days before shipping precast substructure elements to the job site

M. Forms:

Forms shall be subject to the approval of the Engineer.

Forms shall be made and maintained true to the shapes and dimensions shown on the plans.

The surface of forms shall be smooth, and if necessary, joints shall be treated so that a minimum of joint marks are evident in the finished member.

Forms shall be constructed and end bearing plates placed so as to allow for any shortening of the member due to compressive stresses resulting from transfer of stress and from shrinkage.

Side forms shall be of steel and shall be supported without resort to ties or spreaders within the body of the member. They shall be braced and stiffened so that no deflection or curvature occurs during concrete placement.

Forms shall be cleaned before each use.

The Contractor may remove side forms as soon after 6 hours as their removal will not cause distortion of the hardened concrete. Do not remove the members from the bottom forms until they have been stressed sufficiently to sustain all forces and bending moments that may be applied during handling.

N. Casting:

Concrete shall not be deposited in the forms until the Inspector has approved the placement of the reinforcing and prestressing strands. Concrete shall be deposited only in the presence of the Inspector and in accordance with the Specifications.

When the average daily temperature falls below 35 degrees F for more than 1 day, protective measures shall be taken to prevent damage to the concrete by freezing. The protective measures shall be included on the shop drawings as required.

All reinforcing and strands shall be free of dirt, rust, oil, grease, and other deleterious substances.

All items encased in the concrete shall be accurately placed in the position shown on the plans and firmly held during the placing and setting of the concrete. Clearance from the forms shall be maintained by supports, spacers, or hangers in accordance with the Specifications and shall be of approved shape and dimension.

The details of all inserts, anchors, and any other items required to be cast into the members (whether detailed on the Contract drawings or provided for the Contractor's convenience) shall be shown on the shop drawings. Members shall not be fired or drilled into for attachment purposes. All hardware shall be galvanized except as otherwise noted.

The temperature of the concrete shall not exceed 90 degrees F when placed in the forms.

Placement of concrete in stages to facilitate box beam fabrication will be allowed. Interval times between concrete placement stages shall be limited to 45 minutes to ensure that a cold joint has not formed between the two placement stages. Placement plans requiring interval times longer than 45 minutes shall be approved prior to use. Interval times extending beyond 45 minutes without prior approval are cause for rejection of the member.

O. Curing:

Members shall be uniformly cured from the time of concrete placement until at least two representative product test specimens achieve an average strength that meets or exceeds 0.7 f 'c, or the specified release strength, f 'ci, whichever is higher.

Where:

f'c = 28 Day Compressive Strength of Concrete

f'c i = Required Concrete Strength at Release of Prestress Force

Additional curing requirements shall be maintained until the above strength requirements are achieved, and are as follows:

1. Exposed concrete surfaces shall be kept moist from the time of concrete placement until the freshly finished concrete is covered with an enclosure that retains heat and moisture. After enclosure, moist curing shall be maintained at a minimum 70 percent relative humidity. The Contractor shall monitor the temperature and humidity conditions from the initial curing period through the end of the accelerated curing stage.
2. Temperature of the concrete shall be maintained above 50 degrees F.
3. The internal and surface temperature of the concrete shall not exceed 160 degrees F. The Contractor shall monitor the internal concrete temperature using thermocouples with concrete temperature recorded at intervals not to exceed 15 minutes. A minimum of two thermocouples shall be installed in the element at a maximum spacing of 75 feet with a maximum distance from either end of 40 feet. Thermocouples shall be installed at the center

of mass of the element as uniformly as practical to provide accurate temperature monitoring information. An element is defined as a single precast prestressed concrete girder or beam or cast-in-place span. When multiple elements are cast simultaneously in a single bed, the temperature monitoring thermocouples shall be at a maximum spacing of 75 feet. Temperature logs shall be submitted to the Engineer prior to transporting the element to the project site. When the internal temperature of the element exceeds 160 degrees F, the Contractor shall submit a mitigation plan to ensure future castings do not exceed the 160 degrees F maximum temperature requirement. The mitigation plan shall also include procedures for sampling and testing the element to identify the potential risk for Delayed Ettringite Formation, and/or waterproofing applications to protect against moisture intrusion. The mitigation plan shall be submitted to the Engineer for review and approval. Acceptance or rejection of the element exceeding the temperature specification will be based on review and assessment of the specific curing temperature logs and the submitted documentation. The element shall not be shipped until the Contractor receives written acceptance from the Engineer.

4. Concrete shall attain initial set prior to application of the accelerated curing cycle. If initial set was not determined in accordance with ASTM C403, accelerated curing shall not be induced for 4 hours, or 6 hours if retarding admixtures are used. While waiting for the initial set period, low cycle heat may be applied to maintain the curing chamber temperature; however, the temperature rise shall not exceed 10 degrees F per hour during the waiting period.
5. The rise in temperature in the curing chamber during accelerated curing cycle shall not exceed 40 degrees F per hour.

P. Finishing:

Roughen the top surface of the panel to a minimum amplitude of 1/8 inch to ensure composite behavior with the cast-in-place concrete.

Q. Tolerances:

Allowable fabrication tolerances for the dimensions and configurations shown on the plans or approved shop drawings are shown below.

Panel Length (parallel to bridge axis)	+3/4-inch, -1/2-inch
Panel Width (perpendicular to bridge axis)	±1/2-inch
Panel Thickness	±1/4-inch
Strand Horizontal Position	±1/2-inch
Strand Vertical Position	±1/4-inch

R. Panel Storage, Shipping and Handling:

Members damaged during handling and storage will be repaired or replaced at the Department’s direction at no cost to the Department.

Members shall be lifted at the designated points by approved lifting devices embedded in the concrete and proper hoisting procedures.

The points of support and the direction of the reactions with respect to the member during handling and storage shall be approximately the same as when the member is in its final position. Members shall be stored plumb.

Storage areas shall be smooth and well compacted to prevent damage due to differential settlement. Stacks of members may be supported on the ground by means of continuous blocking located perpendicular to the strands at the ends. Intermediate blocking between members shall be located directly over the blocking below.

Members shall be protected from freezing temperatures 32 degrees F, for 5 days or until attaining design compressive strength detailed on the plans, whichever comes first.

Members may be loaded on a trailer as described above. Shock-absorbing cushioning material shall be used at all bearing points during transportation of the members. Tie-down straps shall be located at the lines of blocking only.

The members shall not be subject to damaging torsional or impact stresses.

Panels stored prior to shipment shall be inspected by the Contractor prior to being delivered to the site to identify damage that would be cause for repair or rejection. The Contractor shall ensure that sufficient acceptable panels are available for anticipated placement so that unacceptable delays to the project completion can be avoided.

S. Rejection/Repair of Panels:

Repair small damaged or isolated honeycombed areas that are purely surface in nature and not over 1 inch in depth at the fabrication plant with an approved epoxy grout. Replace panels with more extensive damage or honeycomb.

Inspect the panels at the point of delivery to the jobsite for identification, dimensional tolerances, cracks, and structural damage. Replace panels exhibiting excessive cracking or other structural damage.

Prestressed bridge deck panels will be rejected for any of the following conditions:

4. Any crack that is oriented diagonally or perpendicular to the strands and that crosses a minimum of two strands,
5. Any crack oriented parallel to the strands that is longer than one-third of the panel length, or
6. Cracks oriented parallel to the strands that are shorter than one-third of the panel but present at more than 12 percent of the strands.

Prestressed bridge deck panels that sustain damage, cracks not listed above, or surface defects during fabrication, handling, storage, hauling, or erection are subject to review.

T. Temporary Supports

Temporary supports/grout dams for precast deck panels shall consist of continuous, high-density, expanded polystyrene strips (grout dam) with a minimum compressive strength of 55 psi. If leveling screws are used, a 1.7 pound per cubic foot polyethylene foam seal shall be used as a grout dam.

U. Installation:

When precast panels are erected, the fit of mating surfaces shall have no more than a ⅛ inch gap to prevent concrete leakage. If such fit cannot be provided, the joint shall be filled with grout or sealed with an acceptable caulking compound prior to the placing of the cast-in-place portion of the slab.

Precast panels and their accessories, including components to set grade, shall not be attached by welding to steel girders or other structural steel elements or reinforcing steel. Welding, including arc strikes or grounding on any structural steel element is prohibited. The Engineer will inspect all girder flanges for blemishes from arc strikes. All identified blemishes shall be repaired in accordance with AWS D1.5 Section 3.10. Repair of all blemishes shall be at the Contractor's expense.

Support angles or other steel components that will be left in place and exposed to the atmosphere in the final product shall be galvanized in accordance with the Specifications.

V. Placement of CIP Concrete:

Tops of girders, precast deck panels, pier caps, and abutments that will come into contact with bridge deck concrete shall be heated to raise the temperature above 35 degrees F prior to concrete placement.

Remove all visible contaminants from the deck panels and protruding reinforcing steel by abrasive blast cleaning or high pressure (minimum of 8,000 psi) water cleaning, prior to placing the deck concrete.

Thoroughly wet all forms, prestressed concrete panels, T-beams, slab beams, and concrete box beams on which concrete is to be placed before placing concrete on them. Remove free water from the surface or beam lines before placing concrete. Provide surfaces that are in a moist, saturated surface-dry condition when concrete is placed on them.

APPENDIX D – EXAMPLE FABRICATION AND CONSTRUCTION INSPECTION CHECKLISTS

Fabrication Inspection Checklist:

Checklist	Remarks
1. Materials:	
• Aggregate size	
• Concrete mix design	
2. Forming	
• Shop drawings approved	
• Form cleanliness	
• Steel forms on panel sides	
• Form releasing agents	
• Adequate space between panels (to cut strands and for strand extensions)	
• Inspector checking forming before pour	
3. Prestressing Tendons	
• Strands clean and free of dust	
• Strand size and spacing	
• Strand position (horizontal and vertical tolerance)	
• Methods for maintaining strand position	
• Calculated elongation	
• Jacking force	
• Tensioning method	
• Inspector checking prestressing procedures	
4. Reinforcement	
• Reinforcement clean and free of dust	
• Minimum longitudinal distribution reinforcement	
• Bursting reinforcement	
• Bar size and positioning	
• Satisfactory clearances	
• Lifting attachments/devices	
• Inspector checking placement of reinforcement	

5. Concrete Placement and Finishing	
• Batching and mixing procedures	
• Temperature of concrete during placement	
• Temperature of forms during placement (adequately cooled)	
• Consolidation of concrete	
• Inspector checking concrete placement	
• Top surface of panel adequately roughened	
6. Curing	
• Curing method	
• Maximum/minimum concrete temperature during curing	
• Heating/cooling rates	
7. Detensioning	
• Detensioning sequence shown on approved shop drawings	
• Minimum concrete strength for detensioning	
• Detensioning method and procedures (single-strand vs multiple-strand procedures)	
• Inspector checking detensioning procedures and recording camber	
8. Final Inspection	
• Top surface roughening amplitude	
• Panel satisfy dimensional tolerances	
• Inspector check for panel cracking/damage	
9. Storage/Handling	
• Panels handled only with approved devices at designated locations	
• Supports maintained in approximately same position as designed for the final position	
• Panels supported with continuous dunnage perpendicular to strands	
• Intermediate dunnage located directly over the dunnage below	

Construction Inspection Checklist:

Checklist	Remarks
1. Panel Acceptance	
• Panel free of damage	
• Top surface roughening amplitude	
• Panel dimensional tolerances	
2. Handling & Transporting	
• Ship in horizontal position	
• Adequate means to prevent damage during shipping	
• Dunnage during transporting and storage	
• Handle with approved lifting devices	
3. Panel Placement	
• Flexible temporary support	
• Temporary support securely attached to girder	
• Construction load limits	
• Temporary supports to stabilize panels before permanently secured	
4. CIP Concrete Placement	
• Confirm panels properly supported	
• Proper clearance beneath panels for CIP concrete flow	
• Grout dams installed as necessary	
• Laitance/contaminants removed from top of panels	
• Panels in saturated surface-dry state at time of concrete pour	
• CIP concrete vibration	

APPENDIX E – 1987 FHWA MEMORANDUM ON PARTIAL-DEPTH PRECAST CONCRETE DECK PANELS

U.S. Department of Transportation
Federal Highway Administration

Memorandum

Washington, D.C. 20590

Subject: Precast Concrete Deck Panels

Date: FEB 27 1987

From: Chief, Bridge Division
Office of Engineering

Reply to Attn. of: HNG-32

To: Regional Federal Highway Administrators
Regions 1-10

Our survey of the Regions in October 1981 indicated there were 20 States using precast concrete deck panels for bridge deck construction. Except for Florida, the survey indicated that the States were experiencing no significant problems in the use of the deck panels. Nine States reported reflective cracking in the cast-in-place concrete topping, but the tight hairline cracking was not considered a problem. However, Florida was experiencing extensive longitudinal and transverse cracking over the deck panels on many of their bridges. Florida issued a moratorium on the use of deck panels which is still in effect because of their concern about the possibility of excessively high maintenance costs due to crack related deterioration of the decks with time. Florida's problems were related to the use of non-rigid bearing of the panels on the beams and to curing problems.

Over the past 5 years, Virginia has experienced some extensive reflective cracking in the concrete topping on some of their bridges. Tennessee and Illinois have reported problems with fabrication of the deck panels. These types of problems raise questions concerning the durability of the bridge deck, reduction in service life and future maintenance requirements, and whether we are obtaining a cost-effective product comparable to the full depth cast-in- place deck. However, it appears that the majority of the States are receiving satisfactory performance from their decks with precast concrete panels. Experience and research have demonstrated the need for quality construction and proper detailing.

The following are our recommendations for the use of precast concrete deck panels with some discussion:

1. The most significant detail for deck panels is to insure proper positive bearing of the deck panels on the beams. The use of fiber-board or other compressible material as the only support for the deck panels is unacceptable. The extensive cracking problems experienced in the concrete topping on bridge decks in Florida and other States are related to the non-rigid bearing supports. Research projects and good field experience have demonstrated that deck panels must be firmly bedded on grout or concrete on the beams. Two methods of positive support appear to have been used successfully: panels supported on grout or concrete alone; and panels supported on a temporary compressible

bearing used in conjunction with a rigid grout bed or concrete.

Compressible temporary bearings in conjunction with a rigid grout bed or concrete have been used to provide a variable depth bolster over the beams. The panel usually projects a minimum of 3 inches onto the beam. The temporary bearing material should be a minimum of 1 to 1-1/2 inches wide and provide a minimum of 1 inch vertical clearance between the top of the girder and the bottom of the panel after the panel has been set in place. The grout or concrete bedding used as a positive bearing with the temporary bearing should be a minimum of 1-1/2 to 2 inches wide. Also, when concrete is cast under the panels supported on temporary compressible bearings, then bleed holes should be provided in the compressible material or through the panels to prevent air and/or water pockets.

2. The minimum thickness of the deck panels should be 3-1/2 inches to meet the 1-1/2-inch cover requirement of AASHTO Article 9.25.1.1. It is recognized that some States have successfully used panel thick- nesses of 2-1/2 and 3 inches. However, there have been problems in fabricating and handling these thin sections. Use of the 3-1/2-inch or greater thickness panels reduce the possibility of cracking in the panels due to handling and the Hoyer effect (i.e., splitting crack caused by inducing too large a force in too thin a member). Eleven States are currently using 3-1/2-inch or greater thickness panels.

3. Nineteen States use 3/8-inch diameter strands with two of the 19 States also allowing larger diameter strands. Only one State specifies just the use of 1/2-inch diameter strands. Tennessee recently experienced extensive cracking along the path of the 1/2-inch diameter strands in the precast concrete deck panels. The splitting crack was probably caused by inducing too large a force in too thin a member (Hoyer effect) and improper handling. It should be noted that the prestress force induced by a 1/2-inch diameter strand (28.91 kips/strand) is approximately 80 percent greater than for a 3/8-inch diameter strand (16.1 kips/strand). We are not aware of splitting problems at the ends of 3-1/2-inch thick panels when 3/8-inch diameter strands are used. We recommend the prestressing strand be limited to a maximum of 3/8-inch diameter to provide the maximum full effective bond length in the panel; to reduce the creep effect on the panel; and to reduce the Hoyer effect. If 3/8-inch diameter strands should be unavailable, then 7/16-inch or 1/2-inch diameter strands could be substituted with no change in force or spacing from that required for 3/8-inch diameter strand. Larger strands should not be used for 3/8-inch diameter strands, unless there are research studies supporting the use of the larger strands.

4. Strand Projections - A 1982 survey conducted by PCI indicated that 13 States required strand extension and 7 States did not. Research has indicated that deck panels without strand extensions performed satisfactorily when compared with deck panels with strand extensions. However, we feel the positive aspects of strand extensions warrant their use for all construction. There is a positive benefit from the dowel action of the strand extensions in the cast-in-place topping. Continuity of the slab across the beams is improved. Restraint against rotation of panel edges aids in controlling cracking. Also, the strand extensions provide some restraint against separation of the ends of the

panels from the cast-in-place concrete caused by creep of the panels due to prestress and temperature and shrinkage stresses.

5. Reflective Cracking - Some cracking in the cast-in-place topping is inherent for this type of construction. However, measures that can be taken to minimize the cracking should be used to prevent a reduced deck service life. A positive bearing support for the panels and strand extension have previously been discussed as helping to control or prevent some of the cracking in the toping. Some other considerations are flexibility of the structure, amount of truck traffic, and cross-slope. Deck panels have been used successfully on both concrete and steel structures without any significant problem. Nevertheless, it has been observed that the degree of cracking in the deck is directly related to the flexibility of the structure. Fairly flexible steel structures with a large amount of truck traffic have shown a tendency for more extensive cracking. Also, temperature changes and live load stresses increase the tensile stresses in the deck and the degree of cracking. Superelevated structures require careful consideration because on the low side of the panel, there is a tendency for the concrete to slough away from the edge of the panels. This break in bond between the edge of the panel and the cast-in-place topping over the beam increases the probability of reflective cracking in the deck. Reducing the size and decreasing the spacing of the distribution reinforcing steel and temperature and shrinkage reinforcing steel in the top of the cast-in-place topping, will help to control the cracking. Because of the potential cracking in the topping, all the reinforcing steel in the topping should be epoxy coated to prevent potential corrosion of the reinforcing steel.

The use of deck panels requires proper design and detailing and good quality construction. To promote this, the Prestressed Concrete Institute's Bridge Producers Committee has contracted a consultant to develop recommended practices for bridge deck panels. The manual will cover design, fabrication, shipping, handling and erection of the prestressed precast concrete deck panels. A draft manual is currently being reviewed by PCI. We will advise you of our co11111ents on the final report after it is published, and we have had an opportunity to review it.

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