Toward Successful Implementation of Prefabricated Deck Panels to Accelerate the Bridge Construction Process

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Task 1 – Literature Review and NDOT Survey Results

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Abstract
Prefabricated deck panels are an accelerated bridge construction technique that is used to decrease the construction time of the deck. This literature review contains a description of different prefabricated deck panel systems as well as guidelines and specifications that have recently been developed by DOTs and agencies for the use of deck panels. In addition, results are summarized for a survey of state DOTs conducted by Dr. Badie and an updated version of that survey recently conducted by the project team. These surveys collected up-to-date information on the use of deck panels and specifications for different state DOT’s, and identified best or preferred practices for prefabricated deck panels. The final goal of the project was to find details of deck panel systems and connection types for recommendation to the Nevada Department of Transportation (NDOT), based on their successful implementation in other states.
Task 1. Literature Review

1.1 Introduction
The development of accelerated bridge construction (ABC) techniques and connection details has become a national research focus. Several states have successfully standardized the ABC approach with high rates of public satisfaction. Precast concrete deck panels are used in ABC to decrease the construction time of installing the deck. Two different types of panels are primarily used. Full-depth panels are designed to span the full-depth of the deck, and therefore comprise the entire deck upon installation. Partial-depth panels are designed to span only part of the deck depth, and upon installation can act as formwork for a cast-in-place (CIP) pour that completes the deck. Both types of panels decrease construction time as formwork for a CIP pour does not need to be installed along the entire length of the bridge.

Compared to other ABC techniques, the technologies for pre-fabricated bridge decks are relatively mature. For instance, full-depth panels have been in use for over 20 years (Culmo, 2011). Partial-depth panels are extensively used in some states; for example, Texas first developed methods for using precast deck panels as formwork in the 1960’s and now applies such methods to 85 percent of bridges (Merrill, 2002). Due to aging of the national bridge inventory, many of the bridges in the United States have significant deterioration, often centered on the bridge superstructure. Therefore, concrete deck replacement projects are becoming increasingly common, and can be expedited efficiently with minimal disruption to traffic using ABC.

This literature review will discuss various precast panel systems, development of standard connection details and construction methods, state specific guidelines and practices, and representative and innovative bridge projects.

1.2 Overview of Prefabricated Deck Systems

1.2.1 Full Depth Deck Panel Systems
Full depth precast concrete decks are prefabricated deck panels that are installed without needing forms. The main advantage to using full depth decks is the decrease in construction or closure time for the deck installation (Sullivan, 2007). Full-depth precast panels are normally produced in a controlled plant environment, which leads to higher quality of the concrete and therefore better performance of the panels (PCI, 2011b). Generally, full depth decks are designed as one-way slabs between the supporting beams and girders and use either mild reinforcement or post-tensioning (Culmo, 2011). Typically, panels span the width of the bridge and extend 8 to 12 ft in the direction of traffic. Bridges that are wider than 50 ft are normally designed so the panels span half the width of the bridge. Full-depth precast panels have pockets or block-outs that are used to connect the deck panels to the girders. The deck is connected to the girder by placing shear connectors in the pockets and filling the pockets with grout. This connection forces the deck to
act compositely with the girders. An example of a full-depth precast deck with pockets is shown in Figure 1.1. Full-depth precast decks are typically more expensive in material and construction costs than a conventional cast-in-place deck but the extra cost is often offset by decreased construction time and less required maintenance. (PCI, 2011b).

Figure 1.1: Full depth precast concrete deck (Culmo, 2011)

1.2.2 Partial Depth Deck Panel Systems
Partial depth precast concrete decks are a cross between full depth decks and stay in place forms in that a panel is used as a form for concrete but a cast-in-place pour is still required. Partial depth decks are normally 3.5 to 4” thick, 8 ft long and are designed to span between the girders in interior bays. The panels are placed directly on top of the girders or on top of a sealant or backer rod barrier (Figure 1.2). The panels are used to support the cast concrete in the same way as a stay-in-place form, and the remaining deck is cast in place over and around the precast panels (Culmo, 2011).
1.2.3 Corrugated Steel Decks

Corrugated steel deck forms are an alternative to partial depth concrete decks. The steel decks run in the same direction as the girders and are placed to span between girders. CONTECH makes three different sizes of corrugated steel decks: 6”x2”, 9”x3”, and 12”x4-1/4”. These range in thickness from 12 gauge steel to 3/8” steel. Figure 1.3 shows a cross section of a CONTECH corrugated steel deck (CONTECH, 2012). They are similar to partial depth concrete decks in that the deck panels are installed, the reinforcement is placed, and the concrete is poured to complete the deck. These steel decks remain on the bridge for the life of the project (Culmo, 2011).

![Figure 1.3: Cross section of corrugated steel deck (CONTECH, 2012)](image)

1.2.4 Steel Grids

In steel grid deck systems, a steel grid and filler concrete are prefabricated together. Steel grid options include open grid, partially or fully filled grid, and exodermic decks. The partially or fully filled grid and exodermic decks use concrete, while the open grid contains no concrete and uses steel as the riding surface. The open grid has main bars that span both directions and either diagonal or intermediate cross bars in between the main bars. The spacing between the main bars ranges from 2” to 8”. This system is the lightest of the steel grid options. Figure 1.4 shows a steel open grid with diagonal intermediate bars. Partially and fully filled grid decks are steel grids with concrete poured within part or all of the steel portion of the assembly, respectively. These decks are installed in one piece and completed with a CIP pour on the grid to produce the final surface. Figure 1.5 shows a half-filled grid deck and Figure 1.6 shows a fully-filled grid deck. An exodermic deck uses the same concept as a partially filled grid except the top concrete
layer is a reinforced deck that is cast on top of the steel section prior to placement of the panel (Culmo, 2011). Figure 1.7 shows sections of an exodermic deck.

Figure 1.4: Open grid deck (BGFMA, 2015)

Figure 1.5: Half-filled grid deck (BGFMA, 2015)

Figure 1.6: Fully filled grid deck (BGFMA, 2015)
1.3 Terminology

Figure 1.8 shows a bridge with panels and joints labeled, and is used to define terminology used to describe prefabricated deck panels. The longitudinal direction refers to the direction of traffic flow, which is from top to bottom in Figure 1.8, while the transverse direction is normal to traffic flow. Label “a” denotes a precast panel. Line “b” designates the longitudinal joint of the deck and line “c” designates the transverse joint. Labels “e” and “f” designate the floor beams and the longitudinal girders, respectively.

Figure 1.8: Definition of terms for prefabricated deck panels (Badie and Tadros, 2008)
1.4 NCHRP 12-65 Project

1.4.1 Project Overview
The NCHRP 12-65 project was conducted to develop, test, and make design recommendations for full-depth precast panel systems with no overlays and no longitudinal post-tensioning (Badie and Tadros, 2008). Both measures were intended to speed the construction or deck replacement process and cost, by eliminating field work. During this project, the researchers conducted a comprehensive literature review on bridges incorporating full-depth, precast panel systems and a national survey to document available specifications and policies developed by highway authorities experienced with precast panel systems. The main goal of the project was to develop guidelines and LRFD specifications for design fabrication and construction of full-depth, precast-concrete bridge deck panel systems without the use of post-tensioning or overlays and to develop connection details for new deck panel systems. Information was collected on all full depth deck projects, but emphasis was placed on projects that did not use longitudinal post-tensioning.

A 14 question survey was distributed to all state and Canadian provincial DOT’s, members of the PCI Bridge Committee, and members of the TRB A2C03 Concrete Bridges Committee; totaling 110 requests. Respondents were asked whether full depth precast deck panels had been used within the past 10 years, and if so to evaluate their experience with the panels. Survey respondents that indicated no recent use of full-depth deck panels were asked if there were any reasons why they had not been used. Information was requested on project size, reinforcement and connection type, overlay type, and the grouting method used for connections. An evaluation of the panel systems was requested based on the experience of each respondent. The respondent was also asked if guidelines on full-depth precast concrete panels systems had been developed by their organization, and if so a copy was requested. The original survey questions are included in Appendix A. 32 responses were received, of which 10 reported application of a full depth precast deck panel system in the prior 10 years.

1.4.2 Survey Results
In the survey, 22 of 32 total respondents reported not using full-depth precast deck panels. The 22 states/provinces were the following: Alberta, Arizona, California, Florida, Hawaii, Kansas, Massachusetts, Maryland, Minnesota, Mississippi, North Carolina, North Dakota, New Jersey, New Mexico, Nevada, Ohio, Ontario, Oregon, Tennessee, Washington, Wisconsin, and Wyoming all indicated they had not used full-depth prefabricated deck panels but would be interested in the findings of the survey. Reasons for not using full-depth precast deck panels included: cost, questions about construction issues, lack of specifications or guidelines, long term durability questions, riding surface concerns, and concerns about joint issues. Each DOT indicated they were interested in the results of the survey and requested to be informed of the findings.

9 states and 1 province responded that full depth precast deck panel systems had been used in the past 10 years. The respondents were: Alaska, California, Colorado, Illinois, Kentucky, New Brunswick, New York, Texas, Utah, and Virginia. All of the state DOTs reported using full
depth deck panels that were constructed to act compositely with the girder for every bridge project except one. All but one respondent rated the overall performance of the deck panels to be good, and the remaining rated the performance as excellent. Of the states that responded, Alaska, Illinois, New York, Texas, Utah, and Virginia reported that guidelines and specifications for design, fabrication or construction of full depth precast concrete panel systems had been developed.

Alaska used full depth panels on two bridges within the prior 10 years of the study, but had constructed about 20 total bridges using the panels in the prior 20 years. Neither of the two most recent bridges used post-tensioning in the longitudinal direction or had an overlay. Both bridges used a female-to-female shear key for the panel connections and shear pockets with connectors to make the deck composite with the steel girders. Inspections to date indicated that the joints and deck were performing satisfactorily and were in very good condition.

Colorado used full depth panels for a deck replacement and widening project on an arch bridge. The deck was supported by cross piers, which were supported by vertical posts that extended to the arch. The panel design included eight total panels with a thick asphalt overlay applied on the panels. This bridge was post-tensioned in the longitudinal direction. A CIP concrete side barrier was connected to the deck using a shear connector. The transverse joint was created by extending conventional reinforcement and placing a CIP closure pour between the panels.

Illinois completed a full depth project, where the deck panels were made composite with the 6 steel girders by using shear pockets and shear connectors. Conventional reinforcement was used in both directions and post-tensioning was applied in the longitudinal direction. Barriers were connected to each side panel by threading bolts through the panel into nuts seated within the barriers.

Kentucky completed one project that used full depth deck panels. The bridge was conventionally reinforced with no post-tensioning. A unique feature about this bridge was that shear connectors between the deck and girders were only applied at exterior girder locations. This eliminated the need for shear pockets. The sides of the exterior panels had a female shear key that allowed a CIP parapet to be installed.

New York used full depth panels on a deck replacement project for a large interstate bridge with a 32 degree skew. The bridge had 9 spans and six steel open box girders and was constructed in three sections. Each section was constructed and post-tensioned separately so that the bridge could be kept in service. Because of the high skew a CIP pour was made at each abutment.

The report included project details from Missouri, Nebraska, New Hampshire, Texas, Utah, Virginia, Wisconsin, and Ontario. These bridges mentioned above are representative of the projects described in the report.
1.4.3 Conclusion
Many of the example bridges discussed within the report had similar specifications. Almost all bridges developed composite action between the deck and the girders through shear connectors. Some of the bridges took advantage of the deck configuration and connected the deck to the girders along the outside perimeter of the deck panels so that shear pockets were not needed in the deck. This allowed the deck panels and the girder to be connected with one CIP pour. Every bridge used a female-to-female connection for the panel-to-panel connection. Many of the bridges used leveling screws during construction to allow the decks to be centered upon placement.

The results of the survey were compiled to list common practices for prefabricated full-depth deck panels and incorporated into a design guide (Badie and Tadros, 2008). Suggestions for modifications to the AASHTO LRFD code were also developed (Badie and Tadros, 2008). The connection details developed as part of the research from this project are discussed in Section 1.5.4.

1.5 Current Practice, Standards and Specifications for Full Depth Deck Panels

1.5.1 General Guidelines
The PCI NE chapter is one organization that has taken the lead on developing specifications and recommendation for use of full-depth deck panels. Two documents have been developed: (1) “Full Depth Deck Panels Guidelines for Accelerated Bridge Deck Replacement or Construction” (PCI, 2011a) provides general guidelines and specifications, and (2) “State-of-the-Art Report on Full-Depth Precast Concrete Bridge Deck Panels” (PCI, 2011b) is a more extensive reference with background information and commentary. These two documents serve as general guidelines for the use of full-depth deck panels, and are the primary sources for the general information presented in this section.

Full depth precast concrete deck panels are normally made of high strength (f’c>6 ksi), high quality concrete as they originate from a precast concrete plant. Typically, the panels are prestressed in the transverse direction during the fabrication process. PCI recommends that the deck panel width in the longitudinal direction be specified in increments of 2 ft, with a maximum of 12 ft to facilitate shipping (PCI, 2011b). A common method for assembling full depth panels is to post-tension the entire deck once all of the panels are in place during construction. To accommodate the post-tensioning, 2” diameter post-tensioning ducts are commonly included in deck panels. The post-tensioning ducts are normally placed in the center of the cross-section of the panel with no eccentricity to prevent deflection in the panels prior to placement (PCI, 2011a). Bridges with curved geometry or skew can still accommodate prefabricated full depth deck panels. If the bridge has a curved profile, the decks can be fabricated with curved ducts to incorporate post-tensioning. PCI has developed separate recommendations for bridges with both large and small skew. If the bridge is skewed less than 15 degrees, the panels are recommended
to be designed as trapezoids to match the bridge profile. A typical layout for a skewed full depth deck is shown in Figure 1.9. If the skew is greater than 15 degrees, the panels are recommended to be set straight but the panels on both sides have to be trimmed down to meet the abutment. A typical layout for a full depth deck with skew greater than 15 degrees is shown in Figure 1.10.

Figure 1.9: Skewed profile of bridge with skew <15° (PCI, 2011a)
Figure 1.10: Skewed profile of bridge with skew >15° (PCI, 2011a)

1.5.1.1 Deck-to-Girder Connection Details

A deck that is composite with the girder is considered an essential component for a precast deck system to work. Without the composite action, joint leakage occurs commonly (Badie and Tadros, 2008). Section 3.11 of PCI (2011a) titled Composite Deck Design recommends that deck panels should be made composite with the supporting members. Composite action can be achieved by placing steel shear studs or channels into prefabricated pockets, welding the studs/channels to the girder, and filling the pocket with grout as shown in Figure 1.11 (Badie and Tadros, 2008). Non-shrink, flowable, moderate strength (5 ksi), and low permeability grout should be used for the shear connector pockets (PCI, 2011a). Shear pockets should be spaced 2 ft on center when possible to attain full composite action. However research has shown that spacing of up to 4 ft may be used to attain full composite action (Badie and Tadros, 2008). Studs should be spaced a minimum of 2.5” from the edge of the shear pocket, and welded at least 1.5” away from the edge of the girder (PCI, 2011b).
For a steel girder, the studs or channels are welded to the top flange of the girder as shown in Figure 1.11. For a concrete girder, studs welded to a steel plate or hooked reinforcing steel from the top of concrete girders form the shear connectors (Figure 1.12). The deck panel-to-girder connection can also be designed similarly to the steel girder connection by casting a plate in the top of the girder and welding the studs in place (PCI, 2011a). In the case of hooked reinforcing steel, the reinforcement is extended out of the top of the girder and hooked a full 180 degrees in the pocket (Figure 1.12). Since the steel is embedded in the girder and grout is used in the same way as the system with shear studs, composite action is achieved.
1.5.1.2 Transverse Panel-to-Panel Connection Details

Two main types of connections are used for the transverse panel to panel connection: shear keys and shear keys with post-tensioning. The transverse connection must transfer two primary forces: the vertical shear force and the bending moment resulting from the loads applied to the bridge (PCI, 2011b). The shear key connection used most often is a grouted female to female joint. The shear is transferred by the interaction between the grout and panel. The surface of the shear key should be roughened by using sand or water blasting to achieve the maximum interaction between the panel and the grout. A wood form must be installed under the panel to contain the grout during installation but may be removed after curing is complete (Badie and Tadros, 2008). A closed cell polyethylene foam backer rod can also be used to contain the grout as shown in Figure 1.13. The backer rod should be secured firmly at the bottom of the joint. Figure 1.14 shows the appropriate spacing for the joint and backer rod as well as the results of installation errors.
If post-tensioning is used, the shear key detailing is similar but ducts are incorporated into the deck panels to allow post-tensioning strands to be installed after the deck is placed (Sullivan, 2007). Applying post-tensioning is widely recognized as the most reliable way to prevent leakage (Badie and Tadros, 2008). Utah experimented with conventional reinforcement and post-tensioning, and concluded that longitudinal post-tensioning is necessary in all situations. Utah observed no leakage in post-tensioned joints even in negative moment regions of the bridge, while bridges constructed with welded tie connections (no post-tensioning) had some connection
leakage but still performed adequately structurally (Culmo, 2013). Further details about Utah’s experience are provided in Section 1.5.3. Alaska does not use post-tensioning, but relies on the interaction between the shear key and grout for each panel (Badie and Tadros, 2008).

Some states have experimented with ultra-high performance concrete (UHPC) in panel-to-panel connection joints because of the greater resistance UHPC offers against cracking and leakage. By using UHPC in combination with conventional reinforcement, joint lengths can be made smaller than connections that use normal grout and post-tensioning can be eliminated. Elimination of post-tensioning makes fabrication simpler. An FHWA researcher found that UHPC could be used as a direct substitute for traditional joint concrete and grout and the deck would perform as well as or better than a CIP deck (Graybeal, 2010). The most widely available UHPC mix in the United States is a proprietary product sold by a multinational construction materials supplier (Graybeal, 2011).

Male to female joints have been attempted before, but they are difficult to implement because of the tolerances required for installation. Because the tolerances are often not met, leakage has been a problem for many bridges that have used this method (Badie and Tadros, 2008).

1.5.1.3 Longitudinal Panel-to-Panel Connection Details

Many smaller bridges use only one panel in the transverse direction, which eliminates the need for a longitudinal joint. On larger bridges that have a longitudinal joint, the longitudinal connection detailing between panels is similar to the transverse connection details. A female-to-female joint is the most commonly used longitudinal panel-to-panel connection type. Prestressing can be used in the transverse direction to help prevent leakage but is less common. For instance, Utah does not use prestressing in the transverse direction.

An alternate to female-to-female grouted connection is available for bridges with two panels spanned transversely over the width of the bridge where both panels overlap with a center girder. In this case, a CIP concrete pour is used instead of grout and shear pockets. The shear studs used for composite action are connected to the girder, the panels are placed, and a high early concrete pour is applied. The CIP pour creates the longitudinal joint and interaction is still maintained because of the female joints on each panel. Several states have used this technique and Figure 1.15 shows an example of this type of longitudinal joint applied to a bridge in Missouri. This method is advantageous because it consolidates the panel-to-panel and panel-to-girder connections into one, eliminating the time and expense associated with manufacturing the panels with pockets. However, to use this approach the panels must be sized to span between the girders (Badie and Tadros, 2008).
1.5.1.4 Production and Construction Guidelines

PCI recommends that the panels are designed so that the long side is oriented in the transverse direction of the bridge (PCI, 2011b). The panel framing should be designed with a slope to allow the bridge to drain, and a crown can be incorporated by casting a closure pour. Design parameters such as allowable concrete stresses, transverse flexure, post-tensioning, and the panel overhang dimensions should meet the specifications in the AASHTO LRFD Bridge Design Specifications (AASHTO, 2014). Expected losses in the post-tensioning should be accounted for in the design process. Losses that should be factored into the prestress force are elastic shortening, anchorage set, and friction. Creep and shrinkage do not need to be included; small losses in the post-tensioning are considered acceptable since the applied post-tensioning is usually higher than what is required (PCI, 2011b). The panel transportation plan should be specified in the shop drawings.

Several quality control items should be checked during the production of full-depth panels. The following is a list of items from PCI (2011b):

- Location and alignment of post-tensioning ducts
- Deck thickness to satisfy cover requirements
- Positioning and rigidity of the transverse shear key
- Uniformity of the surface finish
- Influence of shrinkage, creep, and camber on the final alignment
- Location of attachments for traffic barrier service
- Location and coordination of the shear pocket positioning with respect to the existing or proposed girder alignment
- Accurate location of lifting hardware for handling of the deck panels
• Conflicts between reinforcement, ducts, anchorages, and local reinforcement around pockets as well as the main transverse and longitudinal reinforcement.

Clearances, dimensions, and tolerances must be addressed in the development of shop drawings and the setup of formwork, and then routinely verified in the pre-pour and post-pour inspection phase of production. Concrete should not be deposited in the forms until the engineer and/or the QA/QC inspector has inspected and approved the placements of ducts, anchorages and all other materials in the panels and marked as approved on each item.

Shear studs can be included on the girders prior to installation. For steel girders, it may be easier to weld the studs onto the girders after the panels are in place. Structural angles may be used to hold panels in place during installation. The angles act as a type of form for the panels and allow the proper elevations to be achieved. However, either structural angles or leveling bolts can be used during installation to keep the panels level. For full-depth decks that incorporate a crown, the crown can be created by screeding the panels down to the desired thickness or creating an internal hinge in the center of the panel that allows the panel to rotate under its own weight. The transverse joints are prepared by installing the backer rod and grinding the panel edges down to create a smooth transition from panel-to-panel. Once the panels are in place, grout should be applied to the panel connections (PCI, 2011b). The construction guidelines state that post-tensioning should be applied after the transverse joints have been grouted, but before forming the composite connection with the girder to prevent inducing any undesirable stresses in the girders.

In summary, a general sequence of construction for full depth panels is outlined below (PCI, 2011a). The construction sequence should be included on the plans.

1. Clean surfaces of shear keys.
2. Preset leveling bolts to anticipated height.
3. Place all precast deck panels on girders in a span.
4. Adjust leveling devices on deck panels to bring panels to grade. (Figure 1.16)
5. All leveling bolts shall be torqued to approximately the same value (20 percent maximum deviation).
6. Install longitudinal post-tensioning strand (un-tensioned) in ducts and seal joints in ducts between deck panels.
7. Place a flowable non-shrink grout in all transverse joints. The grout shall be rodded or vibrated to ensure all voids are filled.
8. After the grout in the transverse joints has attained a strength of 1000 psi (based on grout manufacturers’ recommendations), the longitudinal post-tensioning strands may be stressed. The contractor shall determine the jacking force required to achieve the minimum final post-tensioning force shown on the plans accounting for all losses.
10. Install shear connectors in all blockouts.
11. Form haunches between the top of the girders and the bottom of the deck panels.
12. Grout all haunches and shear connector blockouts with a flowable non-shrink grout.
13. Cast end closure pours.
14. Cast parapets and/or sidewalks.
15. Place overlay (if required) and open bridge.

Figure 1.16: Typical section of leveling device (PCI, 2011b)

1.5.2 Oregon DOT Specifications and Construction Guidelines
Oregon DOT (ODOT) responded to the NCHRP 12-65 survey that full-depth prefabricated decks had not been used, but they would be interested in the findings of the survey (Badie and Tadros, 2008). In 2011, ODOT developed standards and specifications for full-depth prefabricated deck panels. The information presented next is based on a webinar presented by Bruce Johnson, a State Bridge Engineer with ODOT, for the Florida International University ABC series that outlined ODOT’s process in creating guidelines for full-depth prefabricated deck panels (Johnson, 2011).

ODOT aimed to design a concrete mix for prefabricated deck panels that would be abrasion and chemical resistant. Silica fume and slag were used for chloride and wear resistance. The developed concrete mix included 8 ksi concrete, 7% silica fume, 15% slag and a 0.3 water/cement ratio. The standards specified that the panels should be steam cured, use a curing
compound, and preferably be fabricated in a PCI certified plant. Because the panels were created with a concrete mix that emphasized chemical and wear resistance, ODOT opted not to use an overlay and rather let the panels take the wear and weathering directly. If the concrete in the panels was shown to inadequately resist corrosion, an overlay could be used.

ODOT evaluated the potential options for each connection type before specifying a selection. For the transverse reinforcement, ODOT considered pre-tensioning the panels or alternatively using mild steel reinforcement. Pre-tensioning was observed to remove tensile cracks and take advantage of the increased durability of the concrete; as well as making the panels more resistant to damage from lifting and transporting the panels. The longitudinal reinforcement was reviewed similarly by comparing a post-tensioning approach with traditional mild steel reinforcement. ODOT has been a proponent of UHPC, and decided that mild steel longitudinal reinforcement lap spliced within narrow joints and filled with UHPC were the best option for the transverse connection. By using UHPC, post-tensioning work was not needed, which would speed up construction time.

ODOT specified a panel thickness of 8.5” to account for 0.5” of sacrificial topping and an 8” structural component for the deck. The width and length of the deck is controlled by the transportation limits of the fabricator. However, ODOT limited the panel width to 10 ft and the length to 50 ft. ODOT used a lifecycle cost analysis to determine the feasibility of using full-depth deck panels in Oregon. The initial cost of using full-depth deck panels was determined to be higher than a CIP deck, but because of the decreased maintenance costs the panels would be cheaper over the life of the bridge.

ODOT’s completed specifications included a plan set for panel layouts and connections. The panel layout is shown in Figure 1.17, with a maximum panel width of 50 ft and pocket spacing of 2 ft in the longitudinal direction. The longitudinal steel is extended in both directions and connected with a lap splice along the transverse joint. Figure 1.18 shows a plan view of a skewed panel. Figures 1.19 and 1.20 show the girder to deck panel connection for a prestressed concrete girder and steel girder, respectively. The prestressed concrete girder uses steel stirrups for composite action while the steel girder uses welded steel studs. Figure 1.21 shows the longitudinal joint with UHPC and spanning reinforcement.
Figure 1.17: Plan view of panel layout (ODOT, 2015)

Figure 1.18: Plan view of skewed panel layout (ODOT, 2015)
Figure 1.19: Prestressed concrete girder-to-deck panel connection (ODOT, 2015)

Figure 1.20: Steel girder-to-deck panel connection (ODOT, 2015)
1.5.3 Utah DOT Specifications and Lessons Learned
Utah has been a leader in the development of ABC methods. UDOT contracted with CME Associates to develop standards for ABC that were first completed in 2009 (Culmo, 2013). The current standards are integrated into UDOT’s Structures Design and Detailing Manual (UDOT, 2015a) and various specification sheets and design drawings (UDOT, 2015b) that are publicly available on UDOT’s website. Currently, Utah’s policy is to evaluate ABC for all projects, and select ABC when an overall cost benefit is expected, where both direct construction costs and indirect costs such as user delays are considered. UDOT uses a standard rating procedure and decision flowchart to determine if an ABC approach is required. Standard procedures include both offline approaches, where the complete bridge is constructed offsite and moved into place, and online approaches, where prefabricated bridge elements are rapidly assembled onsite (UDOT, 2015b). Guidelines on the construction and placement of prefabricated full-depth deck panels apply generally to the online construction approach, and have been developed in this context.

UDOT’s very early experiences with ABC consisted of several rapid deck replacement projects. As a result, critical assessments of process and structural details were performed for several of these projects and assembled in “Lessons Learned” reports (e.g. URS, 2004; Deloy Dye, 2005; Ackerman, 2007). The first project was to replace the decks on a skewed steel plate girder bridge originally constructed in 1967, located remotely on I-80 in Summit County (URS, 2004). The report described the construction process in detail. The contractor Ralph L. Wadsworth was required to obtain a Prefabricator License, which took six months and cost several thousand
dollars. During the project, the contractor encountered numerous problems with the concrete mix used to construct the panels and the non-shrink grout used to construct the shear stud pockets and to fill the pocket joints onsite. In addition, significant differences between the design and as-built (based on survey) bridge measurements were detected, and lateral distortion of the top girder flanges occurring after deck removal made placement of the prefabricated panels very difficult. Despite the many difficulties, the project was completed in 6.5 days of full closure of the local bridge and partial closures of I-80.

A 4-span steel curved girder bridge over I-15 at 800 N in Salt Lake City, originally built in 1965, necessitated a rapid deck displacement due to a sudden blowout in span 1 (Ackerman, 2007). UDOT had planned to use traditional CIP decks, but opted for experimental use of prefabricated panels because the incremental cost increase (less than 30% threshold) was considered reasonable, traffic impacts could be significantly reduced, and transportation costs would be minimal as the bridge was located very close to Granite Construction’s fabrication yard. Granite Construction, who was the pre-selected contractor due to funding considerations, was also required by UDOT to obtain pre-cast certification at a cost of $20,000. Post-project evaluation suggested this expense could be avoided by adding a site-casting specification that addresses such details as shop drawing submittals, leveling pad, and match casting requirement. Nonetheless, to date UDOT has not indicated any current use or experimentation with site-casting.

The original structure was designed with chorded girders, while the deck and parapets followed the bridge alignment (Ackerman, 2007). To simplify design and construction, the replacement panels were designed to follow the girders. Each deck panel was designed as a stand-alone section to avoid having closure pours, which increased the required reinforcing steel by about 25%. Load transfer was accomplished through a welded tie connection between the panel sections. UDOT encountered constructability issues with this detail. The panel edges were keyed, and steel was cast into the concrete along the bottom of the keyway. Adjacent panels were to be connected by welding a steel rod along the joint, but blockouts at the top of the panels did not allow sufficient room for the welder to access the joint. Thus, the rod was replaced with a steel plate during construction (Fig. 1.22). UDOT concluded that the constructability of the joint needed to be improved. Also, the project motivated the desire to develop standardized details and investigate other load transfer techniques, such as post-tensioning the deck and/or providing composite action with the beams. The deck was designed with expansion joints, but UDOT decided to remove all expansion joints. Lacking adequate time to redesign the deck without joints, the bridge was built with raised expansion joints at all bents and abutments, which made the asphalt paving process difficult.
Figure 1.22: As-designed and as-built welded tie connection for 800 N overpass on I-15 rapid deck replacement project (Ackerman, 2007)

The 800 N project led to several lessons about process and project management. UDOT concluded that a contract managed general contractor (CMGC) process should be used on ABC projects if possible, and the project should have a single project manager with a structures background to coordinate all efforts. Significant difficulties were encountered because the surveyor was not included on the discussions from the beginning of the project, and because inclusion of a chain link fence along the parapets was determined late in the process and was not accounted for in the design and casting of the panels. By using precast panels, the traffic impacts were reduced from 20 days of full and partial closures on I-15 to 11 days of partial lane closures at night. However, during post-project assessment, UDOT projected that if full closure of I-15 were allowed, the entire project could have been completed in less than 48 hours (from late Friday to early Sunday).

UDOT has contracted CME Associates to perform regular inspection of bridges constructed using ABC techniques with the goal of evaluating the performance of ABC details. The last inspection report was completed in 2013 (Culmo, 2013) and reflects the continual assessment since the inception of UDOT’s ABC program. 41 bridges were inspected in 2013, including those built to UDOT’s current standards and those built prior to the completion of the standards. The following general discussion of bridge deck performance based on Culmo (2013) is restricted to bridges constructed using online approaches.

Eight of the bridges inspected incorporated a welded tie plate/grouted shear key detail without post-tensioning, similar to that shown in Figure 1.22, for the transverse panel-to-panel connection. The performance of this connection detail has been poor. Specific issues include widespread leakage through the joints, especially in the negative moment region; and the inability of the joint to transfer moment across the panels. Leakage is more problematic on bridges with polymer overlays compared to those with asphalt. As an extreme case, significant joint deterioration has occurred on Bridge C-325, such that the pavement on the top of the deck has popped out and exposed several of the connections. CME estimates the remaining life of this
type of connections to be 15 years from when the leakage evidence is first observed. Joint performance may be influenced by quality of the grout, which also applies to panel-to-girder connections (see below). Repair of the joints through epoxy injection of grout may be possible, but is expected to be difficult, time consuming and costly. As a result of the poor performance, this connection detail has been retired, and does not reflect UDOT’s current standards.

Ten of the bridges inspected incorporated a grouted shear key detail with post-tensioning for the transverse panel-to-panel connection. This detail has performed well to date, and reflects UDOT’s current standard (Fig. 1.23). A few bridges have isolated areas of leakage, effluorescence, and rust staining near the deck ends; however, such problems are not accelerating quickly and CME estimates that the joints should last through the life of the deck (up to 75 years). Most of these bridges included a CIP concrete closure pour. Shrinkage of the closure pour concrete has led to some cracking and joint leakage; CME estimates that this problem can be reduced by relaxing the existing concrete specification as the high early strength requirements tends to lead to more shrinkage issues with the concrete.

**SECTION B-B**

Figure 1.23: UDOT current standard transverse panel connection detail with longitudinal post-tensioning (UDOT, 2015b)

One of the bridges inspected incorporated the standard transverse connection detail without longitudinal post-tensioning that was developed as a result of the NCHRP 12-65 project (Badie and Tadros, 2008). The detail, which incorporates a reinforcing bar grouted into steel pockets that are cast into the deck, is discussed further in Section 1.5.4. The bridge was first inspected in 2011 almost immediately after construction, and no problems were detected. However, in the 2013 inspection unexpected deterioration of the transverse connection joint was detected. CME estimates that the joint will last 20-30 years, which is less optimistic than NCHRP findings. However, the NCHRP conclusions appeared to be based on simple span bridges with only
positive moments, and therefore CME recommends that the connection detail be avoided in negative moment regions. Furthermore, the detail was found to be costly relative to the standard with longitudinal post-tensioning, and thus likely will not be further pursued.

Nineteen of the bridges inspected incorporated full-depth pockets through the panels to form the girder-to-panel shear connections. The pockets are filled with grout. This detail is used with both steel girder and concrete girders. The performance of these connections has been mixed; some of the bridges show signs of minor leakage through the pockets. CME estimates that the primary cause of the leakage is due to shrinkage of grout in the pockets, and recommends that the grout specifications should be modified to include a prequalification procedure and different grouts should be evaluated against a performance standard. The issues are relatively minor and the shear connectors are expected to last 40-75 years.

Most of the inspected bridges incorporated a 3/8” polymer overlay without a waterproofing membrane. Many of the overlays were observed to have cracks at the deck expansion joints or at the transverse deck panel joints. CME estimates that lack of a waterproofing system allows salts and chlorides to seep through the cracks and exacerbate the leakage problems that are observed in the deck joints. In addition, delamination of the overlays was observed in some bridges. CME estimates that this overlay system will need major maintenance or replacement every 10-15 years, and has recommended that UDOT replace the existing polymer overlay with a waterproofing membrane on the bare concrete deck, covered by a 3” thick bituminous wearing surface (asphalt layer). Instead, UDOT updated the standard in early 2014 to require the polymer overlay provider to provide 5 year warranty against material and installation defects (UDOT, 2014).

In addition to details already mentioned, the following guidelines related to precast decks are provided in UDOT’s Structures Design and Detailing Manual (UDOT, 2015a). Precast deck elements are designed using the strip method based on Articles 9.7.3 and 4.6.2.1, and Table A4-1 in the Appendix to Section 4 of the AASHTO LRFD Specifications (AASHTO, 2014). The design table also specifies the concrete deck reinforcing. Skew is considered in the detailing of deck reinforcing for skew angles greater than 20 degrees, which is slightly more conservative than the LRFD recommendations of 25 degrees. General size guidelines restrict the panel maximum width (including projecting reinforcement) to 14 ft. The minimum panel thickness is 8 ¾”.

The connection between panels is generally provided by post-tensioning. The post-tensioning system should be designed to provide at least 0.25 ksi across the joint after all losses. The losses associated with panel creep can be ignored because the deck-girder interaction tends to restrain the creep. Use of lap splices with a closure pour or other alternative details providing reinforcing across the joint are also permitted.
Deck haunches are used to account for construction variations, tolerance, and beam camber. The haunch can vary along the length of the girder due to flange thickness variation, camber variation, and roadway profile. The minimum haunch thickness is 1 ¾” for full-depth precast deck panels.

The designer is to provide a placing sequence for full-depth panels, and a construction sequence for all activities including connecting the panels to each other and the girder. Transverse construction joints are to be placed parallel to any skew, and avoid the girder field splice locations. Longitudinal construction joints are to be avoided unless dictated by exceptional circumstances, e.g. deck width exceeding 120 ft. Longitudinal construction joints should not be located under a wheel line. Closure pours are not required but can be useful in phased construction projects. Closure pours should be a minimum width of 3 ft, and lap splices of the transverse reinforcing should be located within the closure pour.

1.5.4 NCHRP 584 Connection Designs
The results and findings of NCHRP 12-65 were used to assemble optimal designs for full-depth prefabricated deck panels. The main goal of the research was to develop and validate a system that did not require longitudinal post-tensioning or an overlay. Two different panel systems were developed through the research; a transversely pretensioned system and a transversely conventionally reinforced system.

The transversely pretensioned system used an 8 ft long panel that spanned the entire width of the bridge with a structural thickness of 8”. The transverse prestressing was applied through eight ½” diameter strands that are distributed in two layers. No. 6 bars at 13.3” spacing were used in the longitudinal direction. Figure 1.24 shows the plan view of the panel design.
Figure 1.24: Plan view for transversely pretensioned system (Badie and Tadros, 2008)
Two alternative details were proposed as viable options to splice the longitudinal reinforcement across the transverse connection for use with the transversely pretensioned panel system. The first transverse connection consisted of placing an HSS section in one side of the panel and embedding the reinforcement within the section. Reinforcement from the adjacent panel is extended into the HSS section when the panels are placed during construction and the HSS is filled with grout. Figure 1.25 shows different views of the first transverse connection. The second transverse connection used an extra reinforcing bar that was dropped into the connection through a slot in the top of the panel and then covered in grout. Figure 1.26 shows this connection.

Figure 1.25: First transverse connection for transversely pretensioned panel (Badie and Tadros, 2008)
The transversely conventionally reinforced panel is also 8” thick. The panel uses three layers of reinforcement; a top and bottom layer in the transverse direction and a longitudinal layer. No. 6 bars spaced at 18 inches are used for both the top and bottom transverse layer. The longitudinal reinforcement is 1 No. 8 bar with threaded ends. The longitudinal reinforcement is spliced using HSS tubes. The plan view of the panel is shown in Figure 1.27 and the transverse connection is shown in Figure 1.28.
Figure 1.27: Plan view for transverse conventionally reinforced deck (Badie and Tadros, 2008)
Figure 1.28: Transverse connection for conventionally reinforced panel (Badie and Tadros, 2008)

For steel girders, both the pretensioned and conventionally reinforced panel configurations used the same girder-to-deck panel connection (Figure 1.29). The connections were spaced 48” apart and used eight 1 1/4” studs for each pocket. For concrete girders, the girder-to-deck panel connection differed for the two panel configurations. The transversely prestressed panel used a stud configuration with clusters of three 1 1/4” studs spaced 48” apart (Figure 1.30). Studs were used for this connection to minimize the pocket size required to accommodate the connection. The conventionally reinforced panel configuration used projected shear reinforcement from the girder for the connection (Figure 1.31).
Figure 1.29: Steel girder-to-deck panel connection (Badie and Tadros, 2008)
Figure 1.30: Prestressed concrete girder-to-deck panel connection for transversely pretensioned panels (Badie and Tadros, 2008)
1.6 Current Practice, Standards and Specifications for Partial Depth Deck Panels

1.6.1 General Fabrication and Construction Procedures
Two different fabrication processes are used for partial depth precast panels, and both are considered viable (Hieber and Wacker, 2005). The first method is to place spacers between the panels in the casting bed, and cut the prestressing strands after the concrete has cured. This allows the panels to be separated at the completion of the cure time. The second method is to cast one large panel and to cut out individual panels once the concrete is cured. According to the PCI Precast Deck Panel Guidelines (PCI, 2001), partial depth panels should be at least 3.5” thick and use 6000 psi 28 day strength concrete. Once the panels are in place the CIP portion of the deck should be at least 4.5” thick. Prestressing strands, if incorporated, should be 3/8” diameter and located at least 4” away from the outside of the panels (PCI, 2001).

During construction, partial depth panels should be handled as little as possible to prevent cracking or warping. Panels developing cracks that span across more than one prestressing strand or expanding beyond 1/3 of the total length of the panel should not be used on the bridge. To install the panels, temporary supports are placed on the girders and leveling screws are used to adjust the panels to the correct elevation. Once the temporary supports are constructed, the panels can be placed and grouted to the girders to prevent movement. After the panels are
grouted to the girder, the leveling screws used for the panels should be removed. The deck reinforcement can then be placed and the final CIP deck poured and allowed to set.

PCI (2001) specifies standard drawings for panel placement and installation. Figure 1.32 shows a plan view of the deck and specifies general dimensions. Figure 1.33 illustrates the prestressed concrete girder to deck connection. This connection is similar to the full-depth connection shown in Figure 1.12 except the partial depth connection is made composite with a CIP pour while the full-depth connection uses grout. Figure 1.34 illustrates the steel girder to deck connection. This connection is similar to the full-depth connection shown in Figure 1.11. Figure 1.35 shows how the grout dam should be designed.

Figure 1.32: Partial depth deck detail (PCI, 2001)
Figure 1.33: Partial depth deck with prestressed concrete girder (PCI, 2001)

Figure 1.34: Partial depth deck with steel girder (PCI, 2001)
1.6.2 Texas Specifications and Experience

Texas has been a leader in the development of partial-depth precast panels (Merrill, 2002). Texas first designed a bridge with precast partial depth panels to act as stay in place formwork for the CIP portion of the deck in 1963. The method did not immediately gain popularity because of the difficulty in cantilevering the panels on the exterior edges. However, Texas began using partial-depth panels spanning over the interior girders, and currently uses partial-depth panels on most girder bridges in every part of the state. Standard details and specifications have been iterated based on lessons learned from construction challenges, in-house research projects, and evolution in materials over time. Texas explains the benefit of using a partial-depth panel system as follows. Construction is accelerated because the panels can be placed for the formwork within hours, and the CIP finishing pour takes less time due to the decreased amount of concrete compared to a full CIP deck. The decrease in construction time translates to cost savings, both in decreased work for the contractor and less traffic delay. Texas has also found that partial-depth panels are safer to install than a conventional deck. Since the panels are significantly heavier than wood or steel formwork, the formwork cannot blow away. Form removal is unnecessary as the panels stay in place for the life of the bridge. Texas has also seen positive impacts from the prestressing steel applied to the precast panels in the positive moment region of the deck. High quality concrete is achieved through this method of construction because the panels are fabricated in precast plants (Merrill, 2002). When asked whether Texas has considered implementing fully prefabricated decks for further benefit, current designers responded that full-
depth decks are not used extensively as a widespread need has not been developed. However, full-depth decks have been found useful for certain projects, and the state is working with the precast industry to develop best practices (Holt, 2015).

Texas uses panels that are 4” thick with a 4” thick CIP layer for a total deck thickness of 8”. The precast panels are typically cast at a fabrication plant in casting beds that are 350 to 500 ft long, and prestressed to the appropriate level based on AASHTO Bridge Design Specifications (AASHTO, 2014). The largest producers of partial-depth panels can manufacture about 300 panels per day. The panels are typically cast with approximately 6” gap between panels to allow for panel movement at the release of the prestressing strands. The required concrete strength is 5 ksi. After the panels are placed, #5 bars spaced at 6” are normally placed in the panels that experience negative moment Figure 1.36 shows the typical panel-to-girder section with bedding strips and spacing and does not show reinforcement. Figure 1.37 shows the panel placement over both an interior and exterior girder as well as the projected girder reinforcement. Composite action between the girder and the deck is achieved by projecting #4 bars from the girder with a full closed loop as shown in Figure 1.37. Figure 1.38 shows a transverse section between panel ends; the panels are placed with a maximum 1” gap and sealed to prevent joint leakage (Merrill, 2002).

Figure 1.36: Typical panel placement for Texas partial-depth decks (TXDOT, 2006)
Texas has placed limitations on the use of partial-depth precast deck panels. Precast deck panels are not permitted on curved steel girder bridges due to the complicated interaction between the deck, girder, and the diaphragm. Texas prefers to use a monolithic deck for this scenario. Partial-depth panels are not permitted on deck widening and phased construction projects, because the panels cannot usually be placed properly with the existing or currently built part of the structure. Partial-depth panels are impractical to install on steel girders with flange widths less than 12” because it is difficult to weld the shear studs within such a small opening (Merrill, 2002).

Texas has recently started using precast deck panels for the entire superstructure. Previously partial-depth panels were not applied over the expansion joints because of the geometric requirement for skewed panels. However, Texas experimented with different panel configurations and determined that trapezoidal panels could be used over expansion joints and remain structurally sound. Tests showed that either a parallel or fanned strand pattern would
provide the required strength for the deck to meet required design loads (Wood et. al, 2008). Figure 1.39 illustrates Texas’ specifications for a skewed panel layout.

Texas has historically encountered longitudinal cracking on decks that use partial-depth precast panels. The longitudinal cracking was found to be caused by placement of the bedding strip too far from the edge of the girders, which led to insufficient bearing for the panels, or by placement of the bedding strips too far in advance of the panel placement. The latter approach caused the bedding strips, which prevent the CIP concrete from flowing under the panel edges, to crush. Texas mitigated the longitudinal cracking by ensuring the bedding strips were placed as specified, so that the design panel bearing stresses were achieved. Besides the longitudinal cracking, Texas has had positive experiences with partial depth deck panels and has been happy with the performance they have produced (Merrill, 2002).

1.6.3 Colorado Experience
Colorado DOT has also used precast partial-depth panels for many years. Policies dating from 1991 are listed on the CODOT website (CDOT, 1991). Panels are between 2 and 10 ft in length and no less than 3” in depth. Concrete used for the panels must have a minimum 28 day strength of 6 ksi.

CODOT uses similar specifications to the ones mentioned in Section 1.6.2. Figure 1.40 shows a section view of prestressed concrete deck panels spanning between interior girders. CODOT uses projected reinforcement bent 90° at the top, rather than Texas’ closed hoop configuration (Figure 1.37). Steel studs are used to achieve composite action between the deck and girder for partial-
depth decks with steel girders (Figure 1.41). In general, the transverse joint is formed by fitting the panel ends tight against each other (Figure 1.42). After the panels are placed during construction, the deck panel surfaces are roughened to create more interaction between the panels and the CIP concrete.

Figure 1.40: Colorado prestressed girder-to-deck section (CODOT, 2015)

**PART PLAN**

Figure 1.41: Colorado steel girder-to-deck section (CODOT, 2015)
1.7 Noteworthy Projects

1.7.1 Utah Interstate Exchange Ramp Deck Replacement Project (Skewed Bridge)

In 2014, UDOT replaced the deck of a 4-span 48.5° skewed interstate-to-interstate exchange ramp with full-depth precast concrete deck panels (Scoles et al., 2014). The bridge was built in 1967 and the deck had been repaired several times to extend the life of the bridge. The inspections prior to the most recent deck replacement revealed crumbling of the deck, expansion joint failure, heavily rusted bearings at the abutments, and cracking and concrete spalling around the bearing pedestals at the abutments. UDOT decided to perform a full deck replacement to extend the life of the ramp.

Minimizing the traffic impact was a goal of this project, so the construction closure was desired to take 14 days or less. As a result, three different approaches using full-depth deck panels were considered to optimize construction speed. The selected design used an approach that oriented the deck panels perpendicular to the centerline of the structure. This approach eliminated the need for a skewed joint, and allowed for quicker panel construction and placement than if panel joints had followed the skew of the bridge. The main disadvantage was the lack of a performance history for this type of panel configuration. Figure 1.43 shows the final panel layout that was chosen for the project.
The bridge had a superelevation transition along each span. In the existing deck, the change in superelevation had been built into the panels. For the deck replacement, the panels were made flat to simplify construction and fabrication. Varying haunch heights and large post-tensioning ducts allowed the panels to be assembled in this manner. Composite action was achieved by using shear studs and blockouts in the panels. All of the transverse reinforcement was designed using UDOT’s method for a CIP deck.

Because many of the components of the bridge were unique, several parts of the design were checked to ensure the system would perform as expected. The post-tensioning was analyzed thoroughly, and the transfer of the post-tensioning force from the deck to the girder was examined. Long term creep losses were calculated using a finite element model to validate the results of hand calculations from AASHTO LRFD.

The removal of the existing deck was more difficult than was expected as the girder flanges were 5/8” thick and were prone to damage using traditional removing techniques. The girders also rebounded more than expected after the dead load of the previous deck had been removed. This caused damage to the abutment bearings. However, the replacement was completed in 6 days, much shorter than the 14 day target. No post-construction reports were included with the summary. Figure 1.44 shows the placement of the deck panels during construction.
1.7.2 Missouri Bill Emerson Memorial Bridge

In 2003, the Missouri DOT (MODOT) used full depth precast deck panels in the construction of the Bill Emerson Memorial Bridge (Badie and Tadros, 2008). The bridge was a complete replacement of the original bridge that was built in 1927. The main span of the cable-stay bridge is 4000 ft long and 100 ft wide. The superstructure is supported by three longitudinal girders spaced at about 50 ft and transverse floor beams spaced at 18 ft.

The deck consists of two adjacent 10” deep precast panels spanning the width of the bridge, replicated in the longitudinal direction. The panel face on the interior side of the bridge rests on the center girder and floor beams. Because the bridge was not skewed, straight panels were used and the longitudinal joint was created over the center girder of the bridge. Figure 1.45 is a duplicate of Figure 1.15, repeated here for convenience, and shows the panel layout for the bridge.

Figure 1.45: Deck panel placement of Bill Emerson Memorial Bridge (Badie and Tadros, 2008)
The precast deck panels are conventionally reinforced with top and bottom meshes of epoxy coated bars. CIP concrete was used for the side and median barriers, transverse connections, and longitudinal connections. The transverse connection was a shear key with reinforcement projected out of the panel and a CIP pour used to complete the joint. Because the longitudinal connection was formed over a girder, shear keys were not needed for that connection. The panels were made composite with the girder by welding shear studs on the steel girder and completing the longitudinal connection with CIP concrete. The longitudinal direction also used post-tensioning spaced at 12”. The deck was completed with a 3” silica fume overlay. Figure 1.49 shows an individual panel with post-tension ducts and shear keys for the longitudinal joint.

![Figure 1.49 Panel with reinforcement and shear key (Badie and Tadros, 2008)](image)

1.7.3 Iowa US 6 Over Keg Creek
The Strategic Highway Research Program (SHRP2) and the HNTB engineering firm wanted to demonstrate an ABC modular design concept for common multi-span stream crossings. Designing and demonstrating a successful design for this type of crossing would allow ABC to be used on typical small scale bridge replacements. The US 6 over Keg Creek Bridge was chosen as an example because of the moderate size and simplicity of the project (Iowa DOT, 2014).

The project used a precast modular deck system with steel girders, precast pier columns and bent caps, precast abutment footings and wings, precast approach pavement slabs, semi-integral abutments, and UHPC joints between the deck modules. The contractor decided to use site casting for the bridge components rather than using a precast plant because of the cost-savings for this project.
Bridge construction was begun by closing the highway and demolishing the old bridge. The abutments and columns were installed and the abutments were placed within the first five days of construction. When the abutments and columns were in place the cap beams were installed. The cap beam installation included the largest precast lift Iowa had performed. Girders were placed and the deck modules were then installed. The approach slabs were set and the joint and closure pours were installed using UHPC. The girders were post-tensioned using rods. Final surface work was completed and the bridge was opened to traffic. There were no major problems during the construction of the bridge and the entire construction took 16 days to complete. Minor issues, such as the field casting of UHPC and field welds, will need to be addressed on future ABC projects, but the incorporation of ABC into this bridge was deemed a success (Iowa DOT, 2014).

1.8 Conclusion

Prefabricated deck panels are an innovative way to decrease construction time and reduce maintenance requirements. The panels can be incorporated in several ways, including installation of the whole deck in one piece using full-depth prefabricated panels, or incorporating panels as a stay-in-place form for a concrete pour using partial-depth prefabricated panels. Experience with prefabricated deck panels varies widely state-to-state. Some states, such as Texas, have used prefabricated panels for 30+ years while others are just recently experimenting with different panel types. Because of the diversity of experiences, it is instructive to evaluate the methodologies and detailing that other states have incorporated for their use of deck panels along with corresponding successes and failures. The next part of the project attempts to determine the best practices for the use of deck panels by surveying every DOT and assembling the positive and negative experiences for each panel and connection type.
2. State DOT Prefabricated Deck Panel Survey Results

2.1 Introduction and Prior Research
The literature review on prefabricated panels indicated that a variety of deck panel details have been developed and implemented throughout the United States. As a follow-up to this literature review, additional information was gathered to identify the best options for prefabricated deck panels. Specifically, other state experiences with deck panels were investigated to identify best options for Nevada.

A survey was designed for this project to gather information on prefabricated deck panel systems, specifically full-depth and partial-depth panels. This survey was modeled after the NCHRP 12-65 project survey (Section 1.4), with the goal to update the NCHRP survey results (Badie and Tadros, 2008) and investigate additional issues that were of interest to Nevada DOT. The final 32 question survey asked a variety of questions about the use of prefabricated deck panels. The complete survey as presented to the DOT’s is included in Appendix B for reference. All state DOT’s were invited to complete the survey online. The representatives from each DOT were requested to complete the survey within three weeks. At the end of the survey deadline, 31 states had responded to the survey.

2.2 Total Responses and Number of Decks Constructed
Several questions were asked to identify the volume of bridges that had been constructed using prefabricated deck panels and to differentiate between various DOT’s by their levels of experience. Of the 32 DOT’s, 20 reported they had constructed new bridges or performed deck replacement projects with prefabricated deck panels in the past 10 years. Figure 2.1 shows the total number of new bridges constructed in the last 10 years, differentiated by deck and girder type. Figure 2.2 shows the total number of deck replacements performed in the last 10 years, also differentiated by deck and girder type. Because of the difference in experience levels with prefabricated deck panels across different states, it is important to look at the number of bridges each DOT has designed using deck panels. Figure 2.3 shows the percentage of DOT’s with varying levels of experience using prefabricated deck panels, tabulated from among the 32 states that responded. For example, 37% of DOT’s had no experience, 34% had limited experience (designed between 1 and 5 bridges), 16% had some experience (6 to 15 bridges), and 13% had extensive experience (>20 bridges).
Figure 2.1: Number of new bridges built with prefabricated deck panels in past 10 years

Figure 2.2: Number of bridge deck replacement projects using prefabricated deck panels in past 10 years
Figure 2.3: Percentage of DOT’s with various experience levels using prefabricated deck panels by number of bridges

Figure 2.1 shows that for new bridge construction, both partial depth and full depth panels have been used with both steel and concrete girders. Not included in Figure 2.1 due to scaling issues, Texas reported constructing 2000 new bridges with partial depth panels and prestressed concrete girders in the past 10 years. This number was much larger than other states because Texas uses partial depth panels almost exclusively for new bridge construction. Based on Figure 2.2, both partial depth and full depth deck panels are commonly used for deck replacements on steel girder bridges. However, prestressed concrete girder bridges incorporating prefabricated deck panels are much less common. Figures 2.1 and 2.2 imply that partial depth panels are more commonly used than full-depth panels, however this is misleading. Multiple DOT’s use partial depth panel construction as the default for all bridge construction. Because of this, these DOTs have constructed large numbers of bridges that use partial depth deck panels; at a higher proportion than DOT’s that use full-depth panels. In summary, full-depth panels are used by more DOT’s, but more bridges are constructed using partial depth panels.

2.3 Application Trends for Prefabricated Deck Panels

2.3.1 Longitudinal and Transverse Reinforcement
DOTs were surveyed to identify the most common methods for connecting panels in the longitudinal and transverse direction. Figures 2.4 and 2.5 show how many times each reinforcement type was used. The two most common longitudinal reinforcement types were spliced reinforcement with UHPC and longitudinal post-tensioning with standard grout. Tennessee has applied HSS with epoxy grout and New Jersey has tried a rapid set latex modified concrete on two bridges. For the transverse reinforcement, spliced reinforcement with UHPC or
standard grout was the most commonly used connection type. Bridges that did not have a longitudinal joint were identified as “Not Applicable” in Figure 2.5.

**Figure 2.4: Implementation of longitudinal reinforcement details**

**Figure 2.5: Implementation of transverse reinforcement details**
2.3.2 Connection Details

2.3.2.1 Full Depth Panel-to-Panel Connections
To assess use of full-depth panel-to-panel connections, the survey presented a variety of details and respondents were asked to rate implementation of each detail (usage rating) as: regularly used, used but not standard practice, used but would not use again, and never used. Figure 2.6 shows the panel-to-panel connections that were included in the survey and Figure 2.7 shows the number of DOT’s reporting successful application of each connection detail. DOT’s reporting either regular use or used but not as standard practice were included in the count in Figure 2.7. Figure 2.8 shows the number of DOTs that selected each usage rating in pie chart format.

The general female-to-female shear key detail developed by PCI (2011a) shown in Figure 2.6a was the most commonly used transverse connection type and the longitudinal joint resembling that developed by Oregon DOT (Figure 2.6g) was the most commonly used longitudinal connection. The female-to-female shear key with welded shear plate (Figure 2.6b), transverse shear key with steel plate (Figure 2.6c), female-to-female shear key with HSS (Figure 2.6d) and longitudinal joint with spliced reinforcement (Figure 2.6i) were all never used or seldom used.

Figure 2.6: Various panel-to-panel connection details
<table>
<thead>
<tr>
<th>Panel to Panel Connection Detail</th>
<th>Number of DOT's</th>
</tr>
</thead>
<tbody>
<tr>
<td>i. Longitudinal Joint with Spliced Reinforcement</td>
<td>12</td>
</tr>
<tr>
<td>h. Longitudinal Cast-in-place Joint over Girder</td>
<td>10</td>
</tr>
<tr>
<td>g. Longitudinal Cast-in-place Joint</td>
<td>8</td>
</tr>
<tr>
<td>f. Female to Female Shear Key with Bent Rein.</td>
<td>6</td>
</tr>
<tr>
<td>e. Female to Female Diamond Shear Key</td>
<td>4</td>
</tr>
<tr>
<td>d. Female to Female Shear Key with HSS</td>
<td>2</td>
</tr>
<tr>
<td>c. Transverse Shear Key with Shear Plate</td>
<td>1</td>
</tr>
<tr>
<td>b. Female to Female Shear key with Welded Steel Plate</td>
<td>0</td>
</tr>
</tbody>
</table>

Figure 2.7: Number of DOT’s implementing various panel-to-panel connection details
Figure 2.8: Type of use for each panel-to-panel connection detail, counted as number of states selecting each usage rating.
2.3.2.2 Full Depth Panel-to-Girder Connections

Implementation of various full-depth panel-to-girder connections was surveyed in the same way as the full-depth panel-to-panel connections. Figure 2.9 shows the panel-to-girder connections and Figure 2.10 shows the number of DOT’s indicating successful application of each connection detail. Figure 2.11 shows the number of states that indicated each usage rating in pie chart format.

The steel girder with welded steel studs detail developed by PCI (Figure 2.9a, PCI, 2011a) was the most commonly used steel girder connection. The prestressed concrete girder with studs detail (Figure 2.9d, PCI, 2011a) was commonly implemented with prestressed concrete girders. Thus details utilizing shear studs were most commonly implemented for both steel and concrete girders. A projected reinforcement detail (Figure 2.9e, PCI, 2011a) and stirrups (Figure 2.9f, DET 3425 of ODOT, 2015) were used for the concrete girder but not as commonly as the shear stud detail. The other connection types that were included in the survey were seldom or never used.

Figure 2.9: Various panel-to-girder connection details
Figure 2.10: Number of DOT’s implementing various deck-to-girder connection details
2.3.2.3 Partial Depth Connections
Implementation ratings were surveyed for partial depth panel-to-girder and panel-to-panel connections in the same way as for the full-depth panels. Figure 2.12 shows the partial depth panel-to-panel and panel-to-girder connections. Figure 2.13 shows how many DOTs used each connection type, and Figure 2.14 shows the number of states selecting each usage rating in pie chart format.
The most commonly used partial depth panel-to-girder connection was a detail with welded steel studs extending from steel girders (Figure 2.12a, PCI, 2001) and a prestressed concrete girder with haunch reinforcement (Figure 2.12b, TXDOT, 2006). U-girders with partial depth panels (Figure 2.12c, TXDOT, 2006) were also a common configuration among different DOT’s.

Only one type of partial depth panel-to-panel connection was identified by the authors to include in the survey. As a result, DOT’s were asked an open ended question to indicate differences in their own implementation relative to the model connection (Figure 2.12e, TXDOT, 2006). Most DOT’s indicated that instead of leaving a 1 inch gap, the panels are pushed directly against each other and a concrete deck pour is used to seal the joint.

Figure 2.12: Various partial-depth connection details: a-d. panel-to-girder connection detail, and e. panel-to-panel connection detail
Figure 2.13: Number of DOT’s that use each partial depth connection
2.3.4 Usage of Overlays

DOT’s were asked to indicate the standard practice for application of overlays to bridges with both partial-depth and full-depth deck panels. DOT’s preferred a variety of options for full depth deck panels including asphalt, concrete, a 3/8” multilayer, and no overlay. Asphalt overlays and no overlay were selected most frequently among the options. No overlay was the most commonly chosen option for partial depth deck panels. Figures 2.15 and 2.16 show the number of DOT’s that used each overlay option for full-depth and partial depth deck panels.
2.4 Deck Panel Evaluations

2.4.1 Deck Panel Performance Problems

Common performance problems with the prefabricated deck panels have been identified. To evaluate the performance of the deck panels in the field, DOT’s were presented with a list of potential performance problems and asked to evaluate whether the problem: a) was observed frequently, b) had been observed in the past but it was not common, or c) had never been observed. Figure 2.17 shows the number of responses indicating a frequently observed problem.
for full-depth panels and Figure 2.19 shows the number of responses indicating a frequently observed problem for partial-depth panels. Figures 2.18 and 2.20 show the percentage of states selecting each rating for each performance issue for full-depth and partial depth deck panels, respectively. The performance problems observed most frequently for full-depth deck panels were closure pour cracking and joint leakage. The respondents that reported these as common problems indicated that both of these problems could be corrected by applying UHPC for the joints and closure pour. DOT’s indicated that reflective cracking, excessive surface wear, concrete spalling, and differential panel movement were not commonly observed. Reflective cracking was the most commonly observed performance problem for partial depth deck panels while differential panel movement, closure pour cracking, concrete spalling, excessive surface wear, and joint leakage were not indicated as frequently occurring problems for DOT’s. Reflective cracking was reported as a major issue for several states and resulted in multiple DOT’s prohibiting the use of partial depth panels. Other partial depth deck panel performance problems such as joint leakage and closure pour cracking could be corrected with UHPC.

![Full-Depth Frequently Observed Problems](image)

Figure 2.17: Number of DOT’s indicating a frequently observed problem for full-depth deck panels
Figure 2.18: Evaluation of performance problems for full depth panels, shown as percentage of states selecting each rating of the problem.
Figure 2.19: Number of DOT’s indicating a frequently observed problem for partial depth deck panels
2.4.2 Ratings of Deck Panel Performance

At the conclusion of the survey, DOT’s were asked to evaluate the overall performance of full-depth and partial depth deck panel systems. Respondents were asked to rate the panels on a scale with four options: poor, fair, good and excellent. A rating of “poor” indicated that the prefabricated deck panel system had numerous problems and did not perform as expected, while a rating of “excellent” meant that the system had no issues and only required standard
maintenance. The performance of full depth deck panels was rated highly by the DOT’s with 17 out of 20 selecting a rating of good or excellent. The partial depth panels received mixed reviews with 11 out of 20 DOT’s selecting a rating of fair or poor. The poor rating of partial depth deck panels was largely due to performance issues and is related to the reflective cracking discussed above. Even though many DOT’s indicated an unsatisfactory rating for partial depth deck panels, Texas, which uses the most partial depth panels, indicated an excellent rating for partial depth deck panel systems. Figures 2.21 and 2.22 show the number of DOT’s that indicated each performance rating for full and partial depth deck panels.

![Full-Depth Performance](image)

**Figure 2.21: Full-depth deck panel performance rating**

![Partial Depth Performance](image)

**Figure 2.22: Partial-depth deck panel performance rating**
2.4.3 Preference Trends for Full-Depth vs Partial-Depth Panels
An open ended question was asked to determine whether state DOT’s preferred full-depth or partial-depth panel systems. Figure 2.23 shows the percentage of states preferring partial depth or full-depth deck panels and percentage indicating no preference. The majority of respondents either prefer full-depth panels or currently do not have a preference. Multiple DOT’s reported being in the experimental phase for full-depth panels and therefore, currently do not have an opinion. Colorado reported preferring partial-depth to full-depth deck panels because using partial-depth panels is standard practice in the state. Iowa, Minnesota, and Louisiana reported preferring full-depth deck panels because severe performance issues had been observed with partial depth panels.

![Preferred Deck-Panel Choice](image)

Figure 2.23: Preferred deck panel system

2.5 Deck Panel Limitations
Consensus regarding limitations on the use of prefabricated deck panel systems appears to be lacking in the literature. Questions regarding skew, curvature and superelevation were included in the survey to determine if DOT’s had made decisions to limit deck panel implementation under certain circumstances. Figure 2.24 shows the percentage of states fully restricting, partially restricting, or not restricting the use of full-depth or partial-depth panels for bridges with skew, curvature and superelevation. Figure 2.24 shows that use of full-depth deck panels was less restricted than partial-depth panels. Most DOT’s did not place a restriction on the use of full-depth deck panels regardless of skew, curvature or superelevation. Among all parameters, curvature prompted restrictions for the greatest number of states; for instance 28% of states disallowed the use of full-depth deck panels for bridges with curvature. In general, DOT’s did not indicate specific limits, but four states indicated maximum skew angle of 30 degrees, and two states indicated a maximum superelevation (2% and 4%). Some DOT’s indicated that
limitations had not been placed on the implementation of deck panels was because the panels had not been used enough to develop guidelines for the use of deck panels.

More restrictions were placed on the implementation of partial-depth deck panels than full-depth deck panels. Partial depth deck panels were not permitted to be used for almost half of the DOT’s when the bridge had skew, curvature or superelevation. However, 44% of DOT’s indicated no limit had been placed on the implementation of partial depth deck panels when the project included skew, curvature or superelevation. Specific limits that were indicated were a maximum skew angle of 40 degrees, and maximum superelevation in the range of 3-5%.

Figure 2.24: Percentage of states limiting the use of prefabricated deck panels
Another limitation on prefabricated deck panels is the maximum panel size as governed by transportation considerations. Several states specified a minimum or maximum length, width, or depth applied to deck panels based on transportation limits. Table 2.1 shows the ranges for the maximum length and width that states had imposed for the transportation of panels, as well as a minimum depth.

Table 2.1: Transportation limits placed on full-depth deck panels by states

<table>
<thead>
<tr>
<th>Transportation Limits Range Among States</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length (maximum allowed)</td>
</tr>
<tr>
<td>Width (maximum allowed)</td>
</tr>
<tr>
<td>Depth (minimum allowed)</td>
</tr>
</tbody>
</table>

2.6 Implementation of Site Casting

Nevada has an added challenge in implementing ABC due to the lack of a certified precast concrete plant in the state. Because of this, it was important to determine the feasibility of site casting different components of a bridge. The survey asked whether site casting had been used for girders, columns, pier caps, footings, abutments, full-depth deck panels and partial-depth deck panels. For each component, respondents were asked whether site casting: has been used regularly, has been used sometimes depending on the project details, has been attempted once or twice, or has never been attempted. Figure 2.25 shows the percentage of DOT’s that indicated the frequency of site-casting for each type of component. The responses from the DOT’s indicated that site casting has not been attempted for any bridge components in a majority of states. Pier caps and abutments were reported as being site-cast the most often compared to other components. Full-depth and partial-depth deck panels have been site cast in a few states, but site casting of these components is very infrequent. If NDOT desires to use site casting for deck panels, the authors recommend contacting Texas and Utah for specifics on usage of site casting. The numbers reported in Figure 2.25 should be interpreted with caution. Based on the authors’ knowledge of site casting in various states, it appears that a small number of respondents may have misinterpreted site casting as CIP construction.

As discussed in Section 1.5.3, when Utah began implementing ABC construction, contractors were selected that were not certified for precast construction. Rather than opt for site casting, Utah responded to this situation by requiring the contractor to obtain certification prior to completing the project, which was addressed financially by adding the cost to the project.
Figure 2.25: Percentage of states using site casting of various bridge components
2.7 Recommendations for Nevada Implementation

The main goal of the survey was to develop recommendations on panel and connection type for the Nevada DOT by determining what has been successfully applied in other states. To develop an overall perception of performance for each connection detail, survey responses on implementation of specific connection details (Figures 2.6, 2.9, 2.12) were superimposed with performance evaluation responses for corresponding full-depth and partial depth panels (Figures 2.21, 2.22). Figures 2.26, 2.27, and 2.28 present the implicit performance ratings for each connection detail according to this method. Specifically, if a DOT indicated frequent use of a specific connection, the DOT’s associated general rating for the panels was applied toward that connection. For example, one DOT indicated frequent use of the female-to-female shear key connection and rated their experience with full-depth deck panels as “good”; thus the female-to-female shear key was assigned one “good” rating. Increasing number of “good” and “excellent” ratings in Figures 2.26 – 2.28 indicates more widespread favorable impression of the connection.

Recall that the full-depth panel-to-panel connections reported as frequently used by the most states were the PCI female-to-female connection (Figure 2.6a) for the transverse connection and the Oregon DOT cast-in-place joint (Figure 2.6g) for the longitudinal connection. Each of these connections were used by DOT’s that rated full-depth panel performance as good, and the connections did not have any poor ratings associated with their use. The other full-depth panel-to-panel connections such as the diamond shear key were not commonly used or were associated with DOT’s that did not rate performance of full-depth deck panels as favorable as the DOT’s that used the PCI or ODOT connections. Two alternative approaches for panel-to-panel connections have been found to help prevent leakage: post-tensioning and UHPC. The use of UHPC will lead to shorter connections because of the decreased anchorage length requirements and also will simplify the fabrication and installation of the panels. However, little long term performance data is available for full-depth decks with UHPC joints. Post-tensioning has been used by many DOTs to prevent leakage. Because of the tradeoffs between the two methods, the authors believe either approach could be effective. The design team should use the approach they are more comfortable with.

DOT’s indicated four options for full-depth panel-to-girder connections that were commonly used. The steel girder with shear studs and grouted haunch connection (Figure 2.9a), which is the only steel girder connection in current use by DOT’s, was associated with DOT’s that rated full-depth deck panel performance as generally favorable. Specifically, the connection received 7 good ratings and 1 fair rating, and thus appears to be an acceptable connection for steel girder panel-to-girder connections. All three commonly used options for concrete girders: steel studs, hooked reinforcement, and steel stirrups (Figure 2.9d, e, and f) received good reviews and were all used with the same frequency among states. Because the shear stud configuration (Figure 2.9a) and the prestressed concrete girder shear stud connection (Figure 2.9d) were used the most frequently among states with favorable performance reviews, the authors recommend both of these full-depth deck panel-to-girder connections.
The performance of partial depth panels was rated by DOT’s to be significantly lower than the performance of full-depth panels. Because the majority of DOT’s indicated having no preference between full-depth or partial-depth deck panels or preferring full-depth panels, the authors recommend that Nevada pursue application of full-depth panels before attempting to include partial-depth panels in standard construction practices. If partial-depth panel systems are selected by NDOT, the survey results suggest that any of the options for partial depth panel-to-girder connections are acceptable and no strong preference is indicated (Figure 2.28). Both steel and concrete girder panel-to-girder connection details were highly rated as well as the tub girder connection detail. The transverse panel connection detail developed by Texas DOT was the only panel-to-panel connection included in the survey, and was used frequently by states that rated the performance of partial-depth panels systems to be good.

In summary, the authors recommend that NDOT use full-depth deck panels because of the higher ratings associated with the use of full-depth deck panels compared to partial-depth deck panels. The recommended panel-to-panel connection details are the PCI female-to-female shear key (Figure 2.6a) for the transverse connection and the ODOT cast-in-place connection (Figure 2.6g) for the longitudinal connection. Either longitudinal post-tensioning or UHPC should be used to help prevent joint leakage. The recommended full-depth panel-to-girder connection is the PCI detail for both steel (Figure 2.9a) and prestressed concrete (Figure 2.9d) girders. If partial depth deck panels are implemented in Nevada, the authors recommend to adopt Texas specifications on partial-deck panel connections (Figure 2.12b, c, e) because of their positive ratings of partial-depth deck panels and extensive experience.
Figure 2.26: Connection ratings for full-depth panel-to-panel connections
Figure 2.27: Connection ratings for full-depth panel-to-girder connections
Figure 2.28: Connection ratings for partial-depth panel-to-girder connections and panel-to-panel connections
References


24. Texas Department of Transportation (TXDOT), (2006), Prestressed Concrete Panels Deck Details.


Appendix A: NCHRP Report 584 DOT Survey

Q1: Has your organization used any full-depth precast concrete deck panel systems in highway bridges during the last 10 years?

Yes _____
No _____ (please, give reasons):
Incremental cost
Lack of specifications or guidelines
Unsatisfactory performance in the past
Other (specify)

Q2: Approximately, how many bridges, utilizing full-depth precast concrete panels, have you constructed during the last 10 years? ______

Q3: Approximately, how many square feet of full-depth precast concrete panels have you constructed in the past 10 years? _____ sq. ft

Q4: Of the bridges listed in answer to Questions 3 & 4, please, indicate the type of transverse (normal to traffic direction) reinforcement.

Pretensioned in the precast yard %
Post-tensioned in the field %
Conventionally reinforced %
Partially pretensioned and partially conventionally reinforced %
Other (specify) %

Q5: How were the panels connected in the longitudinal direction (parallel to the traffic direction)?

Using longitudinal post-tensioning %
Splicing reinforcing bars using commercial mechanical couplers %
Using special mechanical devices %
Other (please specify)%

Q6: What is the percentage of the systems built compositely with the supporting girders? %

Q7: Did you use an overlay?

Yes _____ (if Yes, please, provide the overlay type and percent of decks)
Asphalt % Thickness
Concrete % Thickness
Other (specify) % Thickness
No _____ (If No, did you provide special treatment to the top surface of the precast panels to provide for ride-ability?)
Yes No
If yes, what type? Roughening in the precast plant during production
Grooving in the precast plant during production
Grinding in the field after construction
Sand blasting in the field after construction
Other (specify)

Q8: What is your overall evaluation of the performance of full-depth precast concrete deck panels?
Excellent
Good
Fair
Poor
Please comment and indicate whether or not you will use full depth precast deck panel systems again in future projects:

Q9: Have you developed guidelines or specifications for design, fabrication or construction of full depth precast concrete panel systems?
Yes (please, attach a copy of the specifications)
No

Q10: Successful grouting of the panel-to-panel and the deck-girder joints is considered one of the key elements of having a durable and high performance deck. Have you developed specifications for the grout properties and the grouting process?
Yes (please, attach a copy of the specifications)
No

Q11: In order to simplify the connection between the concrete deck and the steel girders and to facilitate deck removal in the future, the state of Nebraska has used 1¼ in. diameter steel studs successfully. One 1¼ in. steel stud is equivalent to two 7/8 in. studs. Do you see any problems with use of individual or clustered 1¼ in. steel studs with full depth precast deck panels.
Yes
No (please, give reasons)

Q12: AASHTO Specifications stipulate a maximum spacing of the shear connectors between the girder and the deck of 24 inches. Relaxing this limit could simplify deck placement and removal. Do you see a need for research on the performance of shear connectors at 4, 6 or even 8 feet?
Yes
No
Please comment:

Q13: Please, provide the name, phone number and e-mail address of one person on your staff who can help in answering questions on issues related to design and construction with precast concrete deck panels.
Name:
Title:
Phone:
E-mail:

Q14: Are you interested in receiving a copy of the findings of this survey?
Yes
No
Appendix B: NDOT Prefabricated Deck Panel Survey

Please tell us who is filling out this survey:

Name:

E-mail address:

Phone number:

Title:

State department of transportation or organization represented:

1. Has your state constructed bridges or performed deck replacement projects with prefabricated deck panels in the past 10 years?
   ___ Yes
   ___ No

2. In the past 10 years, how many new bridges have been constructed using prefabricated deck panels?
   List number: _____________

3. In the past 10 years, how many bridge deck replacement projects have been completed using prefabricated deck panels?
   List number: _____________

4. Please indicate approximately how many of each of the following types of projects have been completed in the past 10 years.
   New bridges with full depth precast deck panels and steel girders
   New bridges with full depth precast deck panels and prestressed concrete girders
   New bridges with partial depth precast deck panels and steel girders
   New bridges with partial depth precast deck panels and prestressed concrete girders
   Bridge deck replacement with full depth precast deck panels and steel girders
   Bridge deck replacement with full depth precast deck panels and prestressed concrete girders
Bridge deck replacement with partial depth precast deck panels and prestressed concrete girders

5. Considering all projects in the past 10 years (new bridges and bridge deck replacements) that used full-depth precast deck panels, please indicate approximately how many applied each of the following LONGITUDINAL connection details (provide continuity across transverse joints):
   - Longitudinal post-tensioning with UHPC
   - Longitudinal post-tensioning with standard grout
   - Splicing reinforcing bars with UHPC
   - Splicing reinforcing bars with standard grout
   - Other type of longitudinal connection (please specify both connection type and number of applications)
   - Not applicable

6. Considering all projects in the past 10 years (new bridges and bridge deck replacements) that used full-depth precast deck panels, please indicate approximately how many applied each of the following TRANSVERSE connection details (provide continuity across longitudinal joints):
   - Transverse post-tensioning with UHPC
   - Transverse post-tensioning with standard grout
   - Splicing reinforcing bars with UHPC
   - Splicing reinforcing bars with standard grout
   - Other type of transverse connection (please specify both connection type and number of applications)
   - Not applicable

7. Considering all projects in the past 10 years (new bridges and bridge deck replacements) that used partial-depth precast deck panels, please indicate approximately how many applied each of the following reinforcement types:
Conventional longitudinal and transverse reinforcement

Conventional longitudinal and transverse reinforcement with transverse prestressing

Conventional longitudinal reinforcement with transverse prestressing

8. In general, are deck overlays used for full depth prefabricated bridge deck construction?
   ___ Yes
   ___ No

9. Which of the following overlay options most closely represents standard practice for full depth panels?
   o Asphalt
   o 3/8" Multilayer
   o 3/4" Polymer concrete
   o Other: ____________

10. In general, are deck overlays used for partial depth prefabricated bridge deck construction?
    ___ Yes
    ___ No

11. Which of the following overlay options most closely represents standard practice for partial depth panels?
    o Asphalt
    o 3/8" Multilayer
    o 3/4" Polymer concrete
    o Other: ____________

12. Does your department prefer using partial depth or full depth precast deck panels over the alternative and if so, why?
    ______________________________________________________________________________________
    ______________________________________________________________________________________

13. We have compiled a menu of details for full-depth panel-to-panel joint details based on information found in publicly available documents. For each of the details shown, please indicate which statement best represents your state’s position.
   a. We use a detail similar to this regularly
b. We have applied a similar detail but it is not considered regular practice

c. We have applied a similar detail but would not use it again

d. We have NOT applied a similar detail

i. Female-to-female shear key

ii. Female to female shear key with welded steel plate
iii. Transverse Shear Key with Shear Plate

iv. Female to Female Shear Key with HSS

v. Female to female diamond shear key
vi. Female to Female Shear Key with Bent Reinforcement

vii. Longitudinal cast-in-place joint
viii. Longitudinal cast-in-place joint over girder ________

ix. Longitudinal Joint with spliced reinforcement ________
14. We have compiled a menu of details for full-depth panels deck-to-girder details based on information found in publicly available documents. For each of the details shown, please indicate which statement best represents your state's position.

a. We use a detail similar to this regularly
b. We have applied a similar detail but it is not considered regular practice
c. We have applied a similar detail but would not use it again
d. We have NOT applied a similar detail

i. Steel girder with shear studs and grouted haunch _______
ii. Steel girder with welded C-channel _______

iii. Tub Girder to Deck Connection _______
iv. Prestressed concrete girder with shear studs

v. Prestressed concrete girder with projected reinforcement
vi. Prestressed concrete girder with stirrups _______
15. We have compiled a menu of details for partial-depth panels deck-to-girder details based on information found in publicly available documents. For each of the details shown, please indicate which statement best represents your state's position.

a. We use a detail similar to this regularly
b. We have applied a similar detail but it is not considered regular practice
c. We have applied a similar detail but would not use it again
d. We have NOT applied a similar detail

i. Steel girder with welded steel studs ________

ii. Prestressed concrete girder with partial depth panels ________
iii. Prestressed concrete tub girder with partial depth panels _______

iv. Prestressed concrete tub girder with partial depth panels and leveling screws _______
16. The following represents a standard detail for partial-depth panels panel-to-panel connection based on information found in publicly available documents. Please indicate which statement best represents your state's position regarding this particular detail.
   a. We use a detail similar to this regularly
   b. We have applied a similar detail but it is not considered regular practice
   c. We have applied a similar detail but would not use it again
   d. We have NOT applied a similar detail

**Partial Depth Transverse Panel Connection**

If your state’s detailing for the partial depth panel-to-panel connection varies from the sample detail shown, please explain:

_______________________________________________________________________________
_______________________________________________________________________________

17. The following questions contain full-depth deck panel problems that have been reported by DOT’s and research studies. Please indicate for each problem which statement best describes the frequency of occurrence for your state
   a. We frequently observe this problem
   b. We have observed this problem in the past but it is not common
   c. We have never observed this problem

Joint leakage

Excessive surface wear

Concrete spalling

Closure pour cracking

Reflective cracking

Differential panel movement
18. The following questions contain partial-depth deck panel problems that have been reported by DOT’s and research studies. Please indicate for each problem which statement best describes the frequency of occurrence for your state.
   
a. We frequently observe this problem
   b. We have observed this problem in the past but it is not common
   c. We have never observed this problem

Joint leakage

Excessive surface wear

Concrete spalling

Closure pour cracking

Reflective cracking

Differential panel movement

Other full-depth deck panel issue (please specify the issue)

19. Please indicate limitations that have been imposed by the state on the use of full-depth prefabricated deck panels for bridges with skew.
   
o. Not permitted
   o. Permitted up to a maximum skew angle
   o. No limit

If applicable, what is the maximum skew angle (in degrees) permitted for the use of full-depth prefabricated deck panels?
20. Please indicate limitations that have been imposed by the state on the use of full-depth prefabricated deck panels for bridges with curvature.
   - Not permitted
   - Permitted up to a maximum curvature
   - No limit
If applicable, what is the minimum radius (ft) permitted for the use of full-depth prefabricated deck panels?

21. Please indicate limitations that have been imposed by the state on the use of full-depth prefabricated deck panels for bridges with superelevation.
   - Not permitted
   - Permitted up to a maximum superelevation
   - No limit
If applicable, what is the maximum superelevation (%) permitted for the use of full-depth prefabricated deck panels?

22. Please indicate limitations that have been imposed by the state on the use of partial-depth prefabricated deck panels for bridges with skew.
   - Not permitted
   - Permitted up to a maximum skew angle
   - No limit
If applicable, what is the maximum skew angle (in degrees) permitted for the use of partial-depth prefabricated deck panels?

23. Please indicate limitations that have been imposed by the state on the use of partial-depth prefabricated deck panels for bridges with curvature.
   - Not permitted
   - Permitted up to a maximum curvature
   - No limit
If applicable, what is the minimum radius (ft) permitted for the use of partial-depth prefabricated deck panels?

24. Please indicate limitations that have been imposed by the state on the use of partial-depth prefabricated deck panels for bridges with superelevation.
   - Not permitted
   - Permitted up to a maximum superelevation
   - No limit
If applicable, what is the maximum superelevation (%) permitted for the use of partial-depth prefabricated deck panels?

25. Please indicate limitations on maximum panel size that have been imposed by the state for transporting the panels
   Panel length
     o  No limit
     o  Specified limit
   If applicable, specify a length limit (in ft)

   Panel width
     o  No limit
     o  Specified limit
   If applicable, specify a width limit (in ft)

   Panel depth
     o  No limit
     o  Specified limit
   If applicable, specify a depth limit (in ft)

26. Which statement best represents the department’s use of site casting for the following pre-fabricated bridge components?
   a. Site casting is used regularly
   b. Site casting is used sometimes depending on the project details
   c. Site casting has been attempted only once or twice
   d. Site casting has NEVER been attempted

   Girders
   Columns
   Pier caps
   Footings
   Abutments
   Deck Panels (Full)
   Deck Panels (Partial)
27. If your department uses site casting of prefabricated deck panels, please answer each of the following:
Please comment on the ease of construction and overall performance in comparison to panels manufactured in a certified precast contractor facility.
__________________________________________________________________________________
__________________________________________________________________________________
__________________________________________________________________________________
Are site cast specifications different than factory cast specifications? If so, how?
__________________________________________________________________________________
__________________________________________________________________________________
__________________________________________________________________________________
If your department uses site casting of prefabricated deck panels, please answer each of the following:
What are the prequalification or certification requirements for the contractors in order to do site casting?
__________________________________________________________________________________
__________________________________________________________________________________
__________________________________________________________________________________
__________________________________________________________________________________
28. Is there a bridge project that represents the general practices of your department? If so, would you indicate below so that we may follow-up to get more information?
__________________________________________________________________________________
__________________________________________________________________________________
__________________________________________________________________________________
29. Is there a bridge project that presented a unique challenge to your department? If so, would you indicate below so that we may follow-up to get more information?
__________________________________________________________________________________
__________________________________________________________________________________
__________________________________________________________________________________
30. Does your state have standards and specifications for the design, fabrication and construction of prefabricated deck panel systems? If so, would you indicate below so that we may follow-up to get more information?
__________________________________________________________________________________
__________________________________________________________________________________
__________________________________________________________________________________
31. Which statement best represents your department perception of the overall performance of the full depth prefabricated deck panel systems that have been used in your projects?
   ___ Excellent (Panel installation had no problems, only standard upkeep has been needed...)
   ___ Good (Panel installation only had minor issues, and/or minor maintenance needed...)
   ___ Fair (Panel installation had multiple problems, significant repairs needed...)
   ___ Poor (Panel installation had major issues, major renovations required during bridge life...)

32. Which statement best represents your department perception of the overall performance of the partial depth prefabricated deck panel systems that have been used in your projects?
   ___ Excellent (Panel installation had no problems, only standard upkeep has been needed...)
   ___ Good (Panel installation only had minor issues, and/or minor maintenance needed...)
   ___ Fair (Panel installation had multiple problems, significant repairs needed...)
   ___ Poor (Panel installation had major issues, major renovations required during bridge life...)