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State-of-the-Art Report on Precast Concrete Systems for Rapid Construction of Bridges

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16. ABSTRACT

More extensive use of precast concrete components, which are fabricated off-site and then connected on-site, could allow bridges to be constructed more rapidly. The increased use of precast components in bridges also promises to increase work-zone safety and reduce environmental impacts for bridges that span waterways.

This report discusses precast concrete systems that have been used for rapid bridge construction outside of Washington State and evaluates whether they are suitable for use within Western Washington. The report also identifies key features that are important for successful precast concrete system applications. Information on previously used systems was gathered through an extensive review of published literature. Washington State Department of Transportation (WSDOT) design and construction engineers, precast concrete producers, and bridge contractors were also consulted to obtain their input on the positive and negative aspects of applied systems.

Most applications have been used in areas of low seismic potential. By contrast, Western Washington is subject to strong earthquakes. Because precast systems contain connections, and connections are typically vulnerable to seismic loading, a qualitative evaluation of the expected seismic performance of each system was deemed necessary.

The researchers identified four types of precast concrete superstructure systems: full-depth precast concrete panels, partial-depth precast concrete panels, prestressed concrete multibeam superstructures, and preconstructed composite units. The four systems appear to have acceptable seismic behavior, but there are concerns associated with constructability and durability.

Precast concrete substructure systems have received much less attention than have superstructure systems. The use of precast substructure components can provide significant time savings by eliminating the time needed to erect formwork, tie steel, and cure concrete in the substructure. The success of the system depends strongly on the connections, which must have good seismic resistance, have tolerances that allow easy assembly, and be suitable for rapid construction.

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The traffic delays caused by bridge construction are becoming less tolerable as traffic volumes and congestion increase in Western Washington state. Developing ways of constructing bridges more rapidly is therefore desirable. One way of achieving that goal is to make more extensive use of precast concrete components, which are fabricated off-site and then connected on-site. The increased use of precast components in bridges also promises to increase work-zone safety and reduce environmental impacts for bridges that span waterways.

This report discusses precast concrete systems that have been used for rapid bridge construction elsewhere and evaluates whether they are suitable for use in Western Washington. The report also identifies key features that are important for successful precast concrete system applications. Information on previously used systems was gathered through an extensive review of published literature. Washington State Department of Transportation (WSDOT) design and construction engineers, precast concrete producers, and bridge contractors were also consulted to obtain their input on the positive and negative aspects of applied systems.

Most applications have been used in areas of low seismic potential. By contrast, Western Washington is subject to strong earthquakes. Because precast systems contain connections, and connections are typically vulnerable to seismic loading, a qualitative evaluation of the expected seismic performance of each system was deemed necessary.

The researchers identified four types of precast concrete superstructure systems. The four systems appear to have acceptable seismic behavior, but there are concerns associated with constructability and durability. Each system is briefly discussed below.

Full-depth precast concrete panels span the width of the bridge and are placed adjacent to one another. Grouted joints are used between adjacent panels. Post-tensioning may also be used to keep these transverse joints in compression. Shear studs are placed in pockets in the panels and are connected to the supporting steel girders to create a composite deck. Full-depth panels require little formwork and eliminate the lengthy cure time needed for cast-in-place decks. However, leaking and spalling of the joints in full-depth precast concrete systems have been observed in applications, although

improved joint details have been developed and are likely to reduce these problems. Significant effort is required to level the panels and grout the joints between the panels. Although the majority of full-depth panel applications have been on steel girders, they can be used with precast, prestressed concrete girders, which are commonly used in Washington State. Full-depth precast concrete panels can also be used for deck replacement, allowing the redecking to be performed during night closures only.

Partial-depth precast concrete panels span between girders, and cast-in-place concrete is placed on top of the panels to create a composite, full-depth deck. Although partial depth panels eliminate the majority of the required formwork, the overhangs must still be constructed with conventional construction methods. In previous applications, cracks have been observed in the cast-in-place concrete over the transverse joints between panels in the negative moment regions of a bridge. These cracks have prompted WSDOT to use the panels only in the positive moment regions of a bridge, where the top of the deck is in longitudinal compression. The cast-in-place concrete portion of the deck should provide sufficient diaphragm action to deliver seismic forces to the substructure. To maximize the benefits of using partial-depth panels, their application would need to be extended to the negative moment regions.

Prestressed concrete multibeam superstructures consist of girders with the deck attached that are placed adjacent to one another to provide both the superstructure support and bridge deck. Deck bulb Ts are a common example. Grouted joints are used between the members. A wearing course is typically used to provide a smooth riding surface. Multibeam superstructures can be erected quickly because the number of necessary precast pieces is smaller than that needed for other systems, and a cast-in-place deck is not needed. However, removing differential camber between the girders can create construction difficulties. Cracking and spalling of the joints between girders have been observed. Until an improved joint detail is developed that eliminates the current joint durability problems, prestressed concrete multibeam superstructures provide limited potential for use on highways in Western Washington.

Preconstructed composite units (PCUs) consist of bridge girders and a deck slab assembled off-site and then placed as a unit at the bridge site. PCUs are typically large and heavy, and their configurations vary significantly among applications. The size of

PCUs makes them difficult to transport on highways, so PCUs are most often used at water crossings where barge transportation is possible or at sites where nearby, adequate staging areas are available.

Precast concrete substructure systems have received much less attention than have superstructure systems. Substructure systems at intermediate supports consist of precast concrete column components and cap beam components. The connection between components is critical for both constructability and seismic performance. The variety of connections that have been used can be separated into two general categories. The first are match-cast pieces that meet at epoxy-filled joints and are connected by post-tensioning, and the second are grouted joints and spliced mild steel bars. A majority of the precast substructure applications have been in areas of low seismic potential. These kinds of connections are likely to require significant changes to provide the seismic performance needed in Western Washington. The use of precast substructure components can provide significant time savings by eliminating the time needed to erect formwork, tie steel, and cure concrete in the substructure. The success of the system depends strongly on the connections, which must have good seismic resistance, have tolerances that allow easy assembly, and be suitable for rapid construction.

CHAPTER 1:
INTRODUCTION

1.1 MOTIVATION

Increasing traffic volumes and congestion in Western Washington state are fueling the need for bridges that can be constructed rapidly and with a minimum impact on traffic. The majority of highway bridges currently constructed in Washington state consist of reinforced, cast-in-place concrete abutments and piers, precast concrete girders, and a reinforced, cast-in-place concrete deck slab. Although cast-in-place abutments, piers, and deck slabs have been durable and performed well during earthquakes, they do not lend themselves easily to rapid construction. Cast-in-place concrete construction is time intensive, in part because the formwork must be installed, the concrete must be placed and allowed to cure, and then the formwork must be removed. Cast-in-place concrete also requires many on-site construction procedures that create negative impacts on traffic flow, work zone safety, and the environment. There is significant demand for the development of systems that can be constructed quickly while maintaining durability and seismic performance equal to or better than their cast-in-place concrete counterparts.

Precast concrete construction provides a promising alternative to cast-in-place concrete construction. Previous precast concrete applications have achieved significant reduction in construction time and decreased traffic impacts, as will be illustrated in this report.

In addition to rapid construction, precast concrete components have many other advantages. Precast concrete members are often more durable and more uniformly constructed than their cast-in-place concrete counterparts because of the controlled fabrication environment and stringent quality control in precast concrete production plants. A study on bridge durability found that a smaller percentage of prestressed concrete bridges were "structurally deficient" than cast-in-place bridges of similar age and span length (Dunker and Rabbat 1992). Precast components also reduce the time during which workers are exposed to high-speed traffic and other on-site construction hazards, increasing work-zone safety. Precast concrete components are advantageous for bridges being constructed over water, wetlands, and other sensitive sites where environmental concerns and regulations discourage the use of cast-in-place concrete. Precast concrete systems also require significantly less formwork than cast-in-place concrete. This provides an additional advantage for situations in which clearance is limited and sufficient room is not available for formwork extending below the bridge superstructure.

The vast majority of the previous precast applications have been in regions of low and moderate seismic activity. The development of precast concrete components for bridges located in seismic areas is complicated by increased requirements on structural continuity, increased ductility, and increased development length for the reinforcement. These requirements make the design of connections between the precast components more difficult than the connections used in low and moderate seismic regions.

1.2 OBJECTIVES OF THE STUDY AND SCOPE

The objective of this study is to develop a durable precast concrete system with sufficient seismic capacity that can be used in Washington state for rapid bridge construction. Attributes for an acceptable system include the following:

1. rapid construction
2. limited impact on surrounding traffic flow due to construction activities
3. sufficient seismic performance
4. sufficient durability to provide required service life

To achieve the above objectives, information was gathered on precast concrete components and systems that have been previously used for rapid bridge construction. This report outlines the design, construction, and performance issues associated with these precast components and systems. The information presented in subsequent chapters was compiled from published literature, AASHTO design specifications, publications produced by state departments of transportation, and additional miscellaneous sources. The authors met with Washington State Department of Transportation (WSDOT) bridge engineers, local precast concrete producers, and bridge contractors. The goal of the meetings was to evaluate the existing systems and determine the promising aspects for a system to be used in Washington state. The comments, ideas, and concerns received during these meetings are included in the following chapters.

This report is not intended to be an exhaustive review of every application of rapid bridge construction in the past but a summary of key ideas and systems related to precast concrete. Descriptions of commonly used systems with extensive notes on fabrication, construction, design, and durability are included to serve as a basis for the development of precast concrete components and systems with the most promising features for use in Washington state.

1.3 REPORT ORGANIZATION

The first part of this report, chapters 2 through 5, covers superstructure systems composed of precast concrete components. Chapter 6 covers precast concrete substructure systems. In the past, precast concrete components have been used more often in bridge superstructures than in the substructures. Accordingly, a larger quantity of information is available relating to superstructure systems and so superstructure systems constitute a correspondingly larger proportion in this report. The superstructure systems can be divided into the following four broad categories, each of which is addressed in a separate chapter.

- Chapter 2: Full-Depth Precast Concrete Deck Panels
- Chapter 3: Partial-Depth Precast Concrete Deck Panels
- Chapter 4: Multi-Beam Precast Concrete Bridges
- Chapter 5: Pre-Constructed Composite Systems

Because substructure systems are still developing, well-defined systems with unique characteristics have yet to emerge. Therefore, only Chapter 6 is devoted to substructure systems. A more in-depth seismic discussion is provided for substructure systems than for superstructure systems because substructure components are more vulnerable during earthquakes and have a greater effect on structural performance. Precast abutment systems have seen limited use, therefore they are not discussed in-depth in this report. Precast abutment systems have been developed for short single-span, low traffic bridges. These systems have been used throughout the United States, including eastern Washington State, Idaho, and Montana. They systems have been found to be easy to fabricate and construct. (Central Pre-Mix Prestress Co., personal communication, August 17, 2004). Further information on precast abutments can be found in the following references: Scanlon et al. 2002, Randall and Wallace 1987, Pritchard 1978, and Szautner 1984.

Each chapter includes the following sections:

- **Description of System** - Provides a description of the system, the components used in it, and the method used to connect the components.
- **Fabrication Details** - Contains methods for fabrication of the precast components utilized in the system.
- **Construction Procedure -** Outlines the procedure for constructing the system.
- **Summary of Use –** Provides a summary of where and how often the system has been used previously.
- **Performance Evaluation –** Discusses performance and describes problems that have arisen in previous applications.
- **Key Issues -** Describes key issues associated with the fabrication, construction, and design of the system. These include items that are critical to the performance of the system, as well as issues not traditionally considered in design.
- **Evaluation of System -** Provides a summary of positive and negative aspects associated with the system.

Chapter 7 contains a brief discussion of the attributes required for acceptable precast concrete systems and the extent to which the systems presented meet these requirements, for both the superstructures and substructures.

CHAPTER 2:
FULL-DEPTH PRECAST CONCRETE DECK PANELS

2.1 DESCRIPTION OF SYSTEM

Full-depth precast concrete deck panels are used for both new bridge construction and replacement of deteriorated, cast-in-place concrete decks on existing bridges. A typical system is shown in Figure 2.1. This system typically consists of precast concrete panels, approximately 8-in. thick, placed adjacent to one another on bridge girders. Panels typically span the full width of the bridge deck and are approximately 10 ft long.

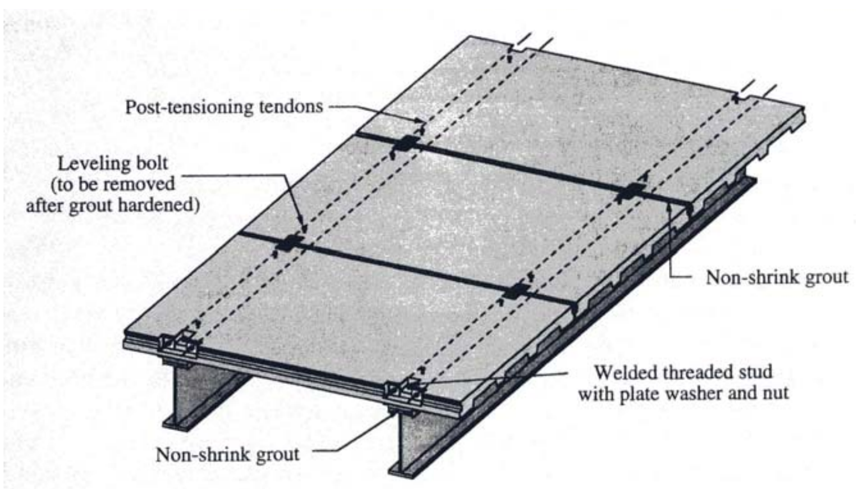


Figure 2.1: Typical full-depth precast concrete deck panel application on steel girders (Yamane et al. 1998)

Grouted shear keys are used in the transverse joints between adjacent panels. The panels are typically connected to the girders using shear pocket connectors, which consist of a mechanical connector (such as shear studs or reinforcing bars) encapsulated in grouted pockets. These connections cause the panels to develop composite action with the girders.

Panel reinforcement configuration varies among applications. Some applications use pretensioning in the transverse direction, while others use mild steel reinforcement. Most recent applications contain longitudinal post-tensioning. The post-tensioning places the transverse joints between panels into compression, improving durability and promoting monolithic behavior.

Full-depth panels do not necessarily require a wearing course. Nonetheless, in many applications, an asphalt, latex modified concrete, or micro-silica modified concrete wearing course is applied to create a smooth riding surface.

2.2 FABRICATION DETAILS

Panel design and the producer's facilities can significantly affect the fabrication of full-depth panels. Panels that do not include pretensioning can be fabricated at a precast plant or temporary yard near the construction site (Osegueda and Noel 1988). A typical fabrication procedure is as follows:

1. The bottom layer of reinforcing steel is placed in the precasting bed.
2. Formwork is placed for the sides of the panels, the shear keys, and the shear pockets.
3. The top layer of reinforcing steel is placed.
4. The concrete is cast and consolidated into the forms.

Panels including pretensioning are typically only fabricated in precast concrete plants. The fabrication procedure is as above with the added step of placing prestressing strands and stressing them after placing the bottom layer of reinforcing steel. If all the panels have the same length dimension (parallel to the bridge girders axes), they are typically cast end-to-end in a long-line prestressing bed. A better quality transverse joint could be achieved by casting the panels side-to-side because of higher quality formwork, however this would require increased labor and plant space, resulting in increased costs (Concrete Technology Corporation (CTC), personal communication, February 26, 2004). Short-line beds with adjustable formwork are used when panels have different dimensions, such as those used for curved or skewed bridges. Congestion problems can occur as a result of mild steel, prestressing steel, block-out formwork, and post-tensioning ducts. This problem is exacerbated if oversized post-tensioning ducts are used to improve on-site constructability (CTC, personal communication, February 26, 2004).

2.3 CONSTRUCTION PROCEDURE

Typical steps for the installation of full-depth precast concrete deck panels on steel girder bridges are as follows:

1. Girders are cleaned and variations in elevation are corrected with shims.

2. Panels are lifted and placed onto the girders.
3. Panels are leveled using leveling screws or shims.
4. Transverse joints between the panels are filled with grout and allowed to reach the required compressive strength.
5. When longitudinal post-tensioning is included in the design, tendons are fed through ducts in the panels and stressed.
6. Shear connectors, such as shear studs, are welded to the girders inside the shear pocket openings in the panels.
7. The shear pockets, the haunch between the girders and panels, and post-tensioning ducts are filled with grout and allowed to cure.
8. An overlay or wearing course is applied to provide a smooth riding surface.

It is important to apply the longitudinal post-tensioning after the transverse joints have been grouted but before the panels are connected to the girders to prevent the introduction of undesirable stress into the girders (PCI NER 2002).

A similar construction procedure can be used for placing full-depth panels on prestressed concrete girders. Placing the panels on prestressed concrete girders is slightly more involved than placing them on steel girders because the shear pocket voids in the panels must be properly aligned so that the stirrups protruding from the girders fit into the voids. Another option when full-depth panels are to be used with prestressed concrete girders is to use dowels to connect the panels to the girders instead of the web reinforcement. When the panels and girders are connected this way, the construction procedure is the same as the steel girder procedure, but now, instead of placing shear studs, holes are drilled into the top of the concrete girder, through the voids in the full-depth panels. Then dowels are placed in the holes and epoxied in place.

The construction procedure can be adapted to meet the specific needs of a project. This is common when full-depth panels are used to replace deteriorated cast-in-place concrete decks and full closure of the bridge is not possible. Two methods are commonly used in this situation. In the first method, one side of the bridge is closed and the deteriorated deck is replaced with full-depth panels while traffic is maintained on the other side. Once the first side is finished, traffic can be switched to the newly redecked side, so that the redecking can be completed on the other side. When redecking is complete, transverse post-tensioning is used to connect the two halves of the completed bridge deck. The second method requires multiple night-time and weekend closures of the entire bridge. During each of these closures a portion of the deteriorated deck is removed and replaced with full-depth panels. With this method the bridge work can be performed during nights or weekends, and the bridge can remain open during high traffic hours.

2.4 SUMMARY OF USE

Full-depth panels have been used since the early 1960s (Biswas et al. 1984). They were originally used for non-composite construction. The first application of full-depth panels for full composite construction was in 1973 (Biswas 1986). Use of full-depth panels has been documented in over 18 states, Japan, Great Britain, Canada, and Mexico (Issa et al. 1995b and Matsui 1994). Full-depth panels have been used primarily for the replacement of deteriorated, cast-in-place concrete decks; however, they have also been used for new bridges.

The majority of the previous applications have been on steel girder bridges. One possible explanation for this is because most prestressed concrete girder bridges are not old enough to require a deck replacement, which is the most common use for panels (Slavis 1982). Another possible explanation is that connections between panels and steel girders are simpler than between panels and prestressed concrete girders. Some research contends that full-depth panels perform better on prestressed concrete girders because of their larger stiffness relative to steel girders (Issa et al. 1995b). The increased stiffness of precast girders may reduce cracking and fatigue problems associated with steel girders. At least one bridge has been constructed in Washington state using full-depth panels (CTC, personal communication, February 26, 2004). On this project, a few difficulties arose, but they were attributed to the geometric complexity of the project and the inexperience of the contractor with full-depth deck panels (CTC, personal communication, February 26, 2004).

2.5 DURABILITY EVALUATION

The durability of bridges using full-depth panels has varied widely (Issa et al. 1995b). The primary damage observed in full-depth panel decks is cracking of the transverse joints between panels. This cracking allows water to leak through the joints. In many cases, the water contains chlorides, which cause deterioration of the concrete and reinforcement in the panels. Leaks in the transverse joints can also accelerate deterioration of the bridge girders. Cracking between adjacent shear pockets has also been documented (Issa et al. 1995b). Spalling at discontinuities in the deck, such as joints between panels and shear pocket connectors, has been recorded and found to accelerate deck deterioration (Kropp et al. 1975). The daily traffic carried by a bridge has been identified as a major factor affecting deck performance. Decks with small amounts of daily traffic have, in general, performed much better than decks with high traffic volumes (Issa et al. 1995b, WSDOT Bridge and Structures Office (BSO), personal communication, January 29, 2004). Large numbers of trucks and particularly overloaded trucks are also believed to accelerate the deterioration of the decks, especially the joints between the panels (WSDOT BSO, personal communication, January 29, 2004).

The long-term performance of full-depth panels is still uncertain. The majority of the bridges constructed with full-depth panels have been constructed in the past 20 years (Issa et al. 1995b). Accordingly, the majority of the decks on which observations have been documented are relatively young. Supporters of full-depth deck panels also believe that the durability problems have been caused by inadequate details that were used in early applications (Issa et al. 1995b). Bridges incorporating newer, improved, details are too young to display deterioration problems, and the claim that the details improve durability can be neither supported nor refuted. Because of the limited use, no information on the durability of this system in Washington state applications was found.

2.6 KEY ISSUES

2.6.1 Key Issue #1: Bearing of Panels on Girders

Differential camber among bridge girders and other fabrication variations can cause the bearing of the full-depth deck panels on the girders to be uneven. The full-depth panels should be leveled and bear evenly on the girders to ensure good performance. If the panels are not leveled, extensive spalling of the transverse joints and a poor riding surface may result (Kropp et al. 1975). Placing the panels directly on the girders leaves voids that can cause the panel and joint to crack and spall (Issa et al. 1995a). To alleviate this problem, the void between the girders and panels must be filled completely with grout to provide a solid, uniform bearing surface. The gap between the panels and girders can be very small (~1/8 inch) making it difficult to achieve a quality bearing surface (Kropp et al. 1975). This problem can be avoided in many applications by temporarily supporting the panels slightly above the girders and then forming a haunch region that can be filled with grout. Leveling bolts or shims are commonly used to level the panels and temporarily support them above the girders. Leveling bolts and shimming are further discussed in the following sections.

Bridge girders with a large amount of camber require that the full-depth panels be leveled to produce the proper bridge profile, as illustrated in Figure 2.2. There are two common ways of achieving this. The first involves leveling the full-depth panels on the girders so that panels align with the bridge profile (Figure 2.2a). This allows for a thin wearing course to be used, promoting rapid construction. When this method is used, the clear distance between the bottom of the panels and top of the girders can become very large near the ends of the bridge, making it difficult to level the panels and form the haunch region (CTC, personal communication, February 26, 2004). The second method involves leveling the full-depth panels with the girder profile a short depth above the girder (Figure 2.2b). This minimizes the distance between the bottom of the panels and the top of the girders, making the leveling easier, but the difference between the girder profile and bridge profile must be made with a varying depth wearing course. This can result in a wearing course that is over 3 inches thick (WSDOT BSO, personal communication, January 29, 2004), which requires longer to cure because of its increased thickness. The weight of the deck is also increased, which increases the seismic forces on the bridge.

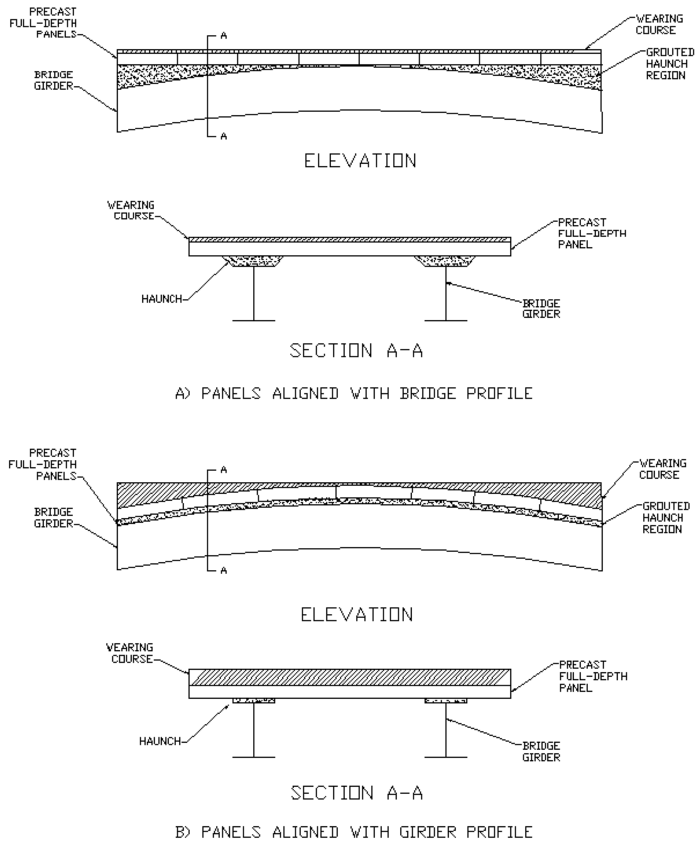


Figure 2.2: Methods of` leveling full-depth panels to account for bridge profile

Leveling Bolts

Leveling bolts are standard bolts that extend through the full-depth panel and bear on the bridge girders. After the panels have been lifted into place, the leveling bolts are adjusted to level the panels. The haunch region is then formed and grouted. Figure 2.3 shows a typical leveling bolt detail. Leveling bolts improve the speed and ease of construction (Culmo 2002) and are not believed to negatively affect deck durability (Issa et al. 1995a). There is debate over whether the leveling bolts need to be removed after the haunch region has been grouted. Some feel that the leveling bolts should be completely removed so that there is less chance of a stress concentration forming at the leveling bolts (PCI-NER 2002). It is agreed that the leveling bolts should be "backed out" after the grout has hardened, so that at least the leveling bolts do not continue to bear on the girder flanges. No durability problems have been attributed to leaving leveling bolts in place as long as they have been "backed out." Leaving them in allows construction to proceed more rapidly. Bridge contractors employed by WSDOT prefer the use of leveling bolts to shims (WSDOT BSO, personal communication, January 29, 2004).

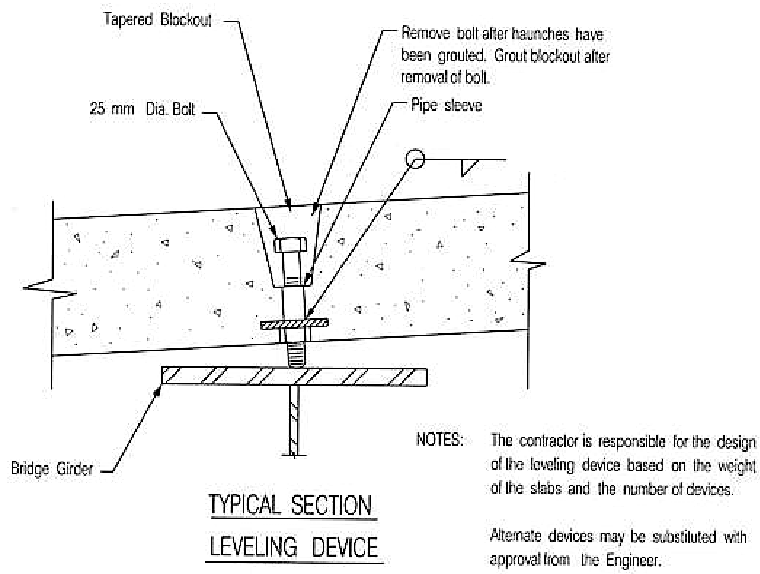


Figure 2.3: Detail of leveling bolts used for positioning the precast deck slabs (PCI-NER 2002)

Shims

Semi-flexible shims made of elastomer or epoxy mortar are attached to the edges of the girders before the panels are lifted into place. The shims can deform slightly when the panels are placed to eliminate any irregularities in the panel and girder surfaces. The void between the bottom of the panels and top of the girders is then filled with grout to form the haunch. Using shims eliminates the need for side forms when grouting the haunches. Each panel should rest on its own set of shims because the required shim height for every panel may be different (Osegueda and Noel 1988). In previous applications, contractors working for WSDOT have found shimming to be tedious and time consuming and, therefore, prefer leveling bolts over shims (WSDOT BSO, personal communication, January 29, 2004).

2.6.2 Key Issue #2: Connection of Panels to Bridge Girders

Full-depth panels must be adequately connected to the girders so that full composite action can be achieved. The panels must be connected to the girders to prevent them from lifting off the girders, which can create fatigue and vibration problems (Tajima et. al 1966). Insufficient connection of the full-depth panels to the girders can cause cracking in the panels and at the joints (Issa et al. 1995b). Several types of connection have been used in previous applications. Each connection consists of a mechanical connector and a grouted region. In each case, high quality grout in the shear pockets and between the slab and girders is critical for developing full composite action (WSDOT BSO, personal communication, January 29, 2004). The most common connection types are presented in the following subsections.

Shear Pocket Connections

Shear pocket connections, shown in Figure 2.4, are the most commonly used connection between the full-depth panels and the girders. The connection emulates a cast-in-place slab-to-girder connection. Full composite action can be developed without the need for an excessive number of connectors. Shear pockets are generally trapezoidal in shape; they are wider at the top to prevent the panels from lifting off the girder. Headed shear studs are commonly used for connections to steel girders. Channel sections have also been used (Tajima et al. 1966). Figure 2.4 shows a typical shear pocket connection to a steel girder. Connections to precast concrete girders can be more difficult. Typically, the panels must be fabricated so that the transverse reinforcement of the girder extends into the shear pockets in the panels. As an alternative to extending the web reinforcement out the top of the girder, reinforcing bars can be grouted into the girders through the shear pockets in the panels after they have been placed (PCI-NER 2002).

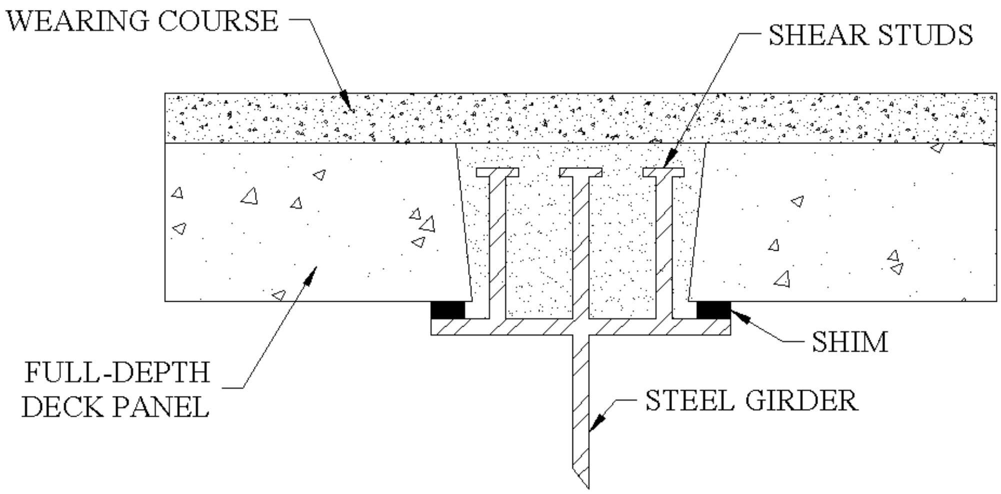


Figure 2.4: Detail of shear pocket connector with shear studs (Issa et al. 1995b)

A problem with panel shear pocket connections is that they are located in the area of greatest negative moment, making them prime locations for the initiation of cracks (Moore 1994 and Issa et al. 1995b). To help prevent crack initiation, the corners of the shear pockets can be rounded to reduce stress concentrations (Issa et al. 2000). There have also been instances when the grout has split into multiple pieces and popped out of the shear pockets (Issa et al. 1995b).

Bolted Connections

Bolted connections use high-strength bolts to connect full-depth panels to steel girders, as shown in Figure 2.5. Ducts are cast into the panels, which align with the bolt holes in the flanges of the steel girders. After the gap between the girder flange and panel has been grouted, bolts are fed through ducts and corresponding holes in the flange and tightened to secure the connection. The connection is capable of developing full composite action, but the composite action can be lost as a result of deterioration of the connections.

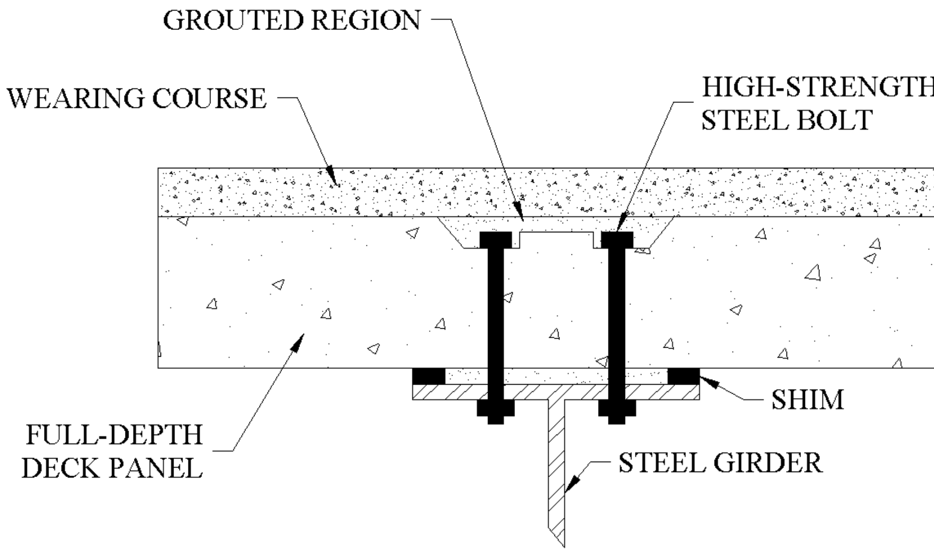


Figure 2.5: Detail of bolted connection (Issa et al. 1995b)

In the initial applications, the gap between the panels and the girders was not grouted. This was problematic because the required bolt tension could not be achieved without cracking the panels (Biswas 1986 and Issa et al. 1995b). When the void between the panels and girders is grouted, the compressive stress in the panels due to the bolts may cause extensive creep in the concrete immediately surrounding the connection. This can cause the panels to crack and the bolts to lose tension (Tajima et al. 1966). In the field, this detail has performed poorly (Biswas 1986, Issa et al. 1995b, and Yamane et al. 1998). Because of the poor field performance and the problems stated above, the use of this connection is not recommended.

Tie-Down Connections

Tie-down connections consist of mechanical connectors used to clamp the panels against steel girders. Figure 2.6 shows a typical tie-down connection. A block-out in the panel is not required, reducing the likelihood of cracking in the panel. However, the connection must be completed from below the bridge deck, making construction more difficult. Issa et al. (1995b) asserted that these connections do not provide full composite action. Bridges using this type of panel-to-girder connection are designed as non-composite. Although this connection does not produce full composite action, field tests of a bridge using it found that a significant amount of composite action was developed from friction between the panels and girders (Kropp et al. 1975).

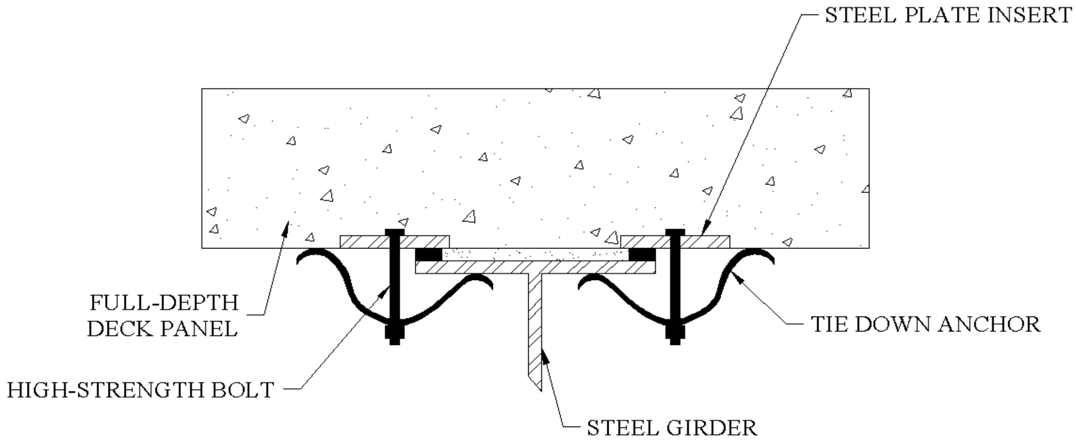


Figure 2.6: Detail of tie-down connection (Issa et al. 1995b)

Field investigations have found that tie-down connections can become loose because of the vibrations caused by traffic (Issa et al. 1995b) and have fractured as a result of fatigue (Kropp et al. 1975). The performance of these connections during an earthquake is likely to be poor because of the reverse lateral loading (Issa et al. 1995b). Because of the inability of this connection to develop full composite action and poor performance in previous applications, it is not recommended (Issa et al. 1995b; Kropp et al. 1975).

Combination Connections

Connections using a combination of the details described above have also been used. For example, Yamane et al. (1998) proposed using shear pockets with headless shear studs and tie downs, as shown in Figure 2.7. This connection detail makes it easier to remove the slab should a future replacement be required.

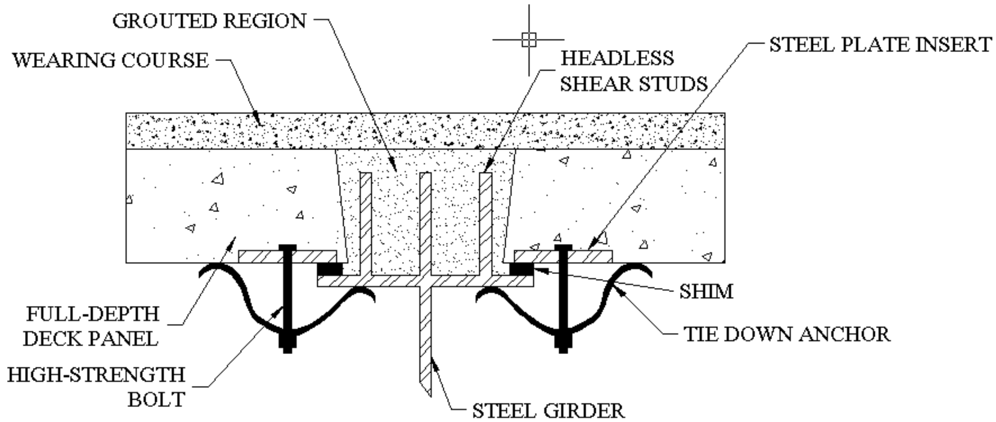


Figure 2.7: Panel connection proposed by Yamane et al. (1998)

2.6.3 Key Issue #3: Joints between Panels

Transverse joints between panels must be able to transfer wheel load shear and axial forces, as well as prevent leakage through the deck. Longitudinal joints also may be needed when full-width panels are not practical. This may occur where a crown is required in the deck, construction is staged, the width of the bridge exceeds the maximum panel length, or there is a unique situation, such as an exit or spur on the bridge (Issa et al. 1995a). Most of the joints used in previous applications have been female-to-female shear key type connections (Issa et al. 1995b). Male-to-female shear key connections have been used and were found to perform poorly because of panel irregularities preventing a perfect fit (Kropp et al. 1975). Many similar connection details with slightly different dimensions have been used in previous applications. A sketch of a typical joint is shown in Figure 2.8.

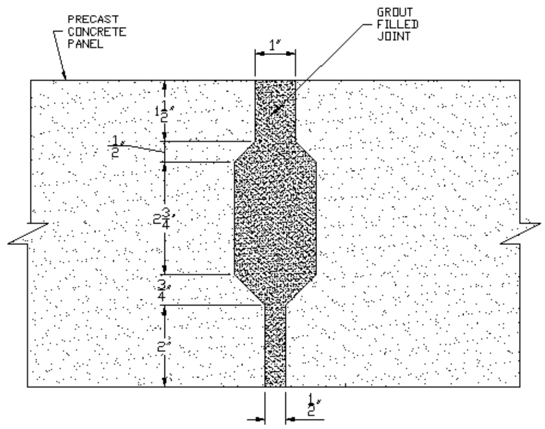


Figure 2.8: Detail of a typical female-to-female type shear key joint (Issa et al. 1995b)

Many joints have performed poorly in previous applications because of low quality joint material, poor joint details, inconsistent/inadequate construction procedures, and lack of routine maintenance (Issa et al. 1998 and Nottingham 1996). In order to improve joint performance, the following requirements for joint detailing and construction have been proposed. Joints should be designed to allow easy flow of grout (Issa et al. 1995b). Research has found that larger joints with easy flow paths and numerous access points from above perform better than joints with less room for grout to flow (Nottingham 1996). Female-to-female shear key joints should have a full-depth gap to allow for dimensional irregularities in the panels (Issa et al. 1995b). Failure to provide this gap can cause undesirable stresses in the panels if they bear against one another. The full depth of the female-to-female shear keys should be grouted. Grouting only the top of the joint may cause poor performance in the negative moment regions because of the reduced bearing area (Issa et al., 1995b). Grout should be high quality and have a high-early strength, high-bond capability, and low shrinkage (Nottingham 1996). In many projects, including some in Washington state, achieving acceptable grout quality in the field has proven to be difficult (WSDOT BSO, personal communication, January 29, 2004). Grouts with high-early strengths allow for post-tensioning shortly after grouting so that construction can proceed rapidly (Issa et al. 1995a). Joints should not incorporate foam-packing rods. Precast panel tolerances can cause the rods to be placed incorrectly, resulting in poor performance (Nottingham 1996). Possible joint construction problems are illustrated in Figure 2.9. If vertical faces of the panels are smooth and slick from being cast against steel forms, the vertical faces should be sandblasted and pressure washed before grouting. The bonding capacity of the grout to the deck panel is greatly reduced if this procedure is not performed (Gulyas 1996; Nottingham 1996; Issa et al. 2003). The preparation of a transverse joint is shown in Figure 2.10. The quality and proper installation of grout has a large effect on joint performance. Research has shown that, when high quality grout is properly installed, failure of the joint will occur in the panel adjacent to the grouted region rather than in the grout itself (Issa et al. 2003). For an improved joint detail, see research by Stanton and Mattock (1986).

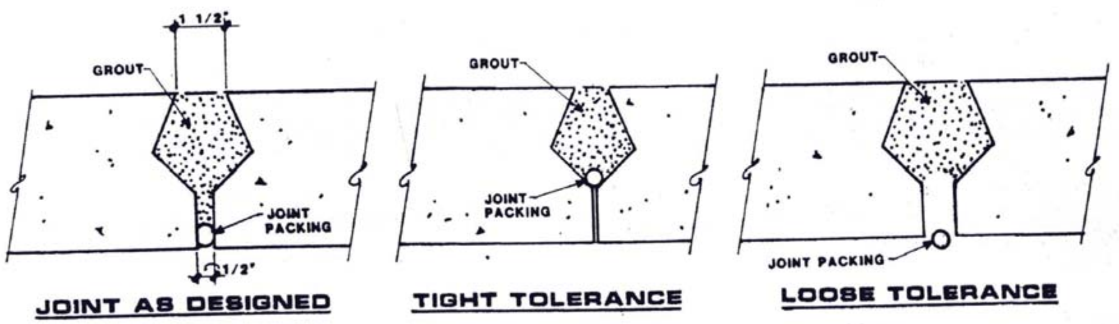


Figure 2.9: Detail of foam packing rods misaligned as a result of panel misalignment (Nottingham 1996)

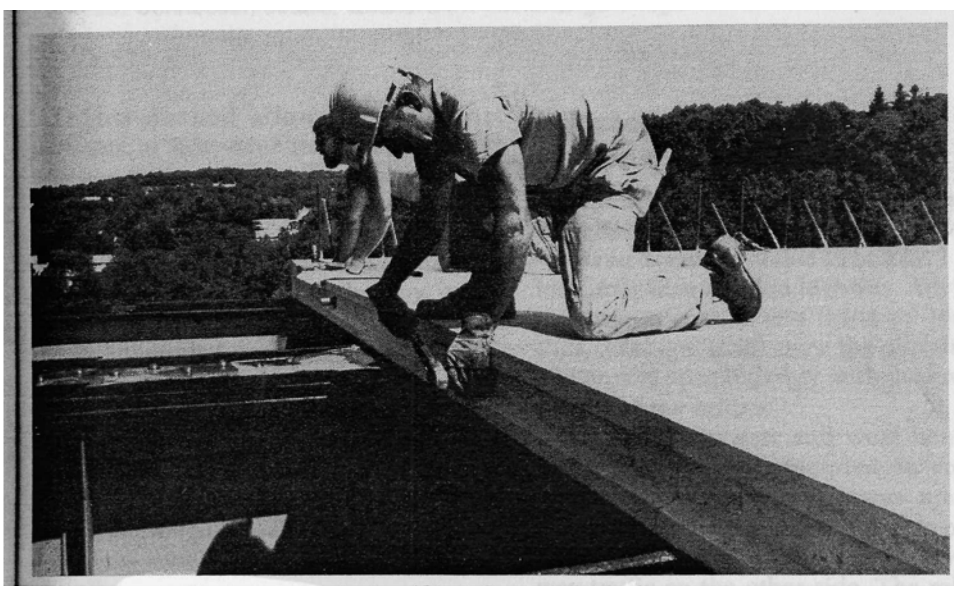


Figure 2.10: Preparation of one side of a female-to-female shear key (Biswas 1986)

In recent applications, longitudinal post-tensioning has been incorporated to achieve a snug fit between adjacent panels and to place the transverse joint into compression. Placing the joint in compression helps to prevent cracking due to applied loads or shrinkage of the concrete, and helps prevent subsequent leakage through the joint (Issa et al. 1995b). Providing sufficient longitudinal post-tensioning to keep the panels in compression under service loads also increases the shear force and bending moment capacities of the joint, especially in the negative moment region of the deck (Issa et al. 1998).

For simple-span bridges a minimum prestress level in the range of 150 to 200 psi is needed to keep the joints in compression under service loads. For continuous-span bridges a minimum prestress level in the range of 300 to 450 psi is required to keep the joints in compression under service loads (Issa et al. 1998; Issa et al. 1995b). AASHTO requires a minimum prestress of 250 psi throughout the joint (AASHTO 1998). Aligning the post-tensioning ducts in the panels during construction can be difficult (WSDOT BSO, personal communication, January 29, 2004). The use of oversized ducts can alleviate this problem but may lead to congestion problems during fabrication of the panels (CTC, personal communication, February 26, 2004).

2.6.4 Key Issue #4: Transverse Pretensioning of Panels

Most full-depth panels used in previous applications have been fabricated from precast, reinforced concrete. Precast, prestressed concrete panels have been growing in popularity in recent years (Yamane et al. 1998). These panels are pretensioned in the transverse direction of the bridge. This procedure allows for thinner panels, provides better crack control, and helps reduce damage to the panels during transportation and erection (Yamane et al. 1998). Prestressed panels are still avoided by some designers because it is difficult to develop the pretensioning steel at the ends of panels, where they overhang the exterior girder (Culmo 1991). Adding additional mild steel in these regions can help to alleviate this problem (Yamane et al. 1998).

2.6.5 Key Issue #5: Wearing Surface

Full-depth panel deck slabs have a rough surface because of the grouted joints and shear pockets. A typical full-depth panel deck is shown in Figure 2.11. Such a rough surface is not acceptable on many bridges, especially those with large volumes of high-speed traffic, so a wearing surface is added. This is required for safety, rider satisfaction, and improved durability (WSDOT BSO, personal communication, January 29, 2004). Materials commonly used for the wearing surface include asphalt, polyester concrete, micro-silica modified concrete, and latex-modified concrete.

Figure 2.11: Typical surface of a full-depth deck panel bridge without a wearing surface (Issa et al. 1995b)

2.7 EVALUATION OF SYSTEM

Full-depth precast/prestressed deck panels appear to be a viable option for the rapid construction of highway bridges. Full-depth panel deck slabs may be more expensive than similar cast-in-place concrete decks, but any additional cost may be offset by the reduction in construction time (CTC, personal communication, February 26, 2004; Culmo 1991). Costs could also be reduced by developing a standardized system that could be used in a wide range of applications. Adoption of a standardized system would also be beneficial because it would allow fabricators to invent high quality steel forms, resulting in more accurate panel dimensions and fewer fit-up problems on-site (CTC, personal communication, February 26, 2004).

Full-depth precast slabs provide the potential for significant time-savings over cast-in-place concrete slabs. Using full-depth panels eliminates the curing period required for a cast-in-place slab, which can be as long as 14 days (WSDOT BSO, personal communication, January 29, 2004). The need for placing and removing formwork is eliminated, which saves a significant amount of on-site labor time and eliminates the impact on traffic that is typically caused by formwork operations (WSDOT BSO, personal communication, January 29, 2004). In order to obtain the full time savings of using full-depth panels, an experienced contractor should be hired (CTC, personal communication, February 26, 2004; WSDOT BSO, personal communication, January 29, 2004).

Full-depth panels are versatile. They can be used on a variety of bridge geometries and bridge lengths. Panel dimensions can be altered to accommodate skewed and curved bridges. Panels can be used for both new construction and the replacement of deteriorated cast-in-place decks. Deteriorated decks can be replaced in portions with full-depth panels, allowing a bridge to be redecked with only night-time and weekend closures. Partial width redecking (one or two lanes of traffic at a time) is also possible, allowing bridges to remain open with a reduced traffic volume.

The panels can also be used with either prestressed concrete or steel girders. Installation on steel girders appears to be easier than on prestressed concrete girders because more options exist for connecting the panels to the girders. It may be difficult to adapt this system to the standard prestressed concrete girders used in Washington state because of the locations of transverse reinforcement extending from the girder. Girder reinforcement details would need to be altered to accommodate placement of full-depth panels. Strict fabrication and construction tolerances would be required to ensure that the transverse reinforcement extending from the girder fit correctly with the pockets fabricated in the panels (WSDOT BSO, personal communication, January 29, 2004).

Despite extensive efforts to improve the durability and constructability of full-depth panel systems, some problems still remain. Even with improvements in joint details and the inclusion of post-tensioning in design, the long-term serviceability of this system is still questionable. Deterioration problems, particularly associated with the transverse joints, have occurred in previous WSDOT applications (WSDOT BSO, personal communication, January 29, 2004). Many current full-depth panel designs have openings in the top of the slab caused by shear pocket connectors in locations of maximum negative moment. This reduces the flexural cracking capacity of the panels in the area of maximum demand. The pockets cause cracks to initiate, resulting in decreased deck durability. Alterations in design to eliminate these pockets should be explored further. The system also requires a large quantity of grouting, which can be time consuming in the field. WSDOT experience has shown that it is difficult to consistently produce a high quality grout and to completely fill the gaps between the slab and girders. WSDOT has encountered many serviceability problems when the grout is low quality or there are gaps or voids in the grout (WSDOT BSO, personal communication, January 29, 2004).

The seismic performance of a bridge with full-depth panels must be investigated. No mention of seismic performance was found in the literature. It would appear that, with enough post-tensioning, the deck could be capable of developing the required diaphragm action. The expected damage to transverse joints caused by a seismic event should also be considered. Cracking or spalling of the transverse joints during an earthquake may leave the joint prone to leakage and other deterioration. The ability of the full-depth panels to withstand vertical accelerations should also be considered. The combination of headed shear studs and trapezoidal shaped shear pocket connections should prove sufficient to withstand the required forces.

Research has been performed at Virginia Tech University to investigate the horizontal shear connection of the haunch area of full-depth panels (Roberts-Wollmann 2004). The Ontario Ministry of Transportation has also recently performed tests on full-depth panel decks and plans to construct its first bridge with full-depth panels in 2004 (Au 2004).

CHAPTER 3:
PARTIAL-DEPTH PRECAST CONCRETE DECK PANELS

3.1 DESCRIPTION OF SYSTEM

Partial-depth precast concrete deck panels, shown in figures 3.1 and 3.2, are thin prestressed concrete panels that span between girders and serve as stay-in-place forms for the cast-in-place concrete deck. Panels are typically 3.5-in. thick, 8-ft long in the longitudinal direction of the bridge, and sufficiently wide to span transversely between the girders. The panels are pretensioned with strands located at mid-depth running in the bridge's transverse direction. The panels are placed adjacent to each other along the length of the bridge. The panels are not connected to one another at the transverse joints. The prestressing strands in the panels serve as the bottom layer of reinforcing steel in the bridge deck. After the panels are in place, the top layer of reinforcing steel is placed, and the cast-in-place concrete portion of the deck (typically 4.5-in. thick) is poured on top of the panels. The cast-in-place concrete and panels act as a composite deck slab.

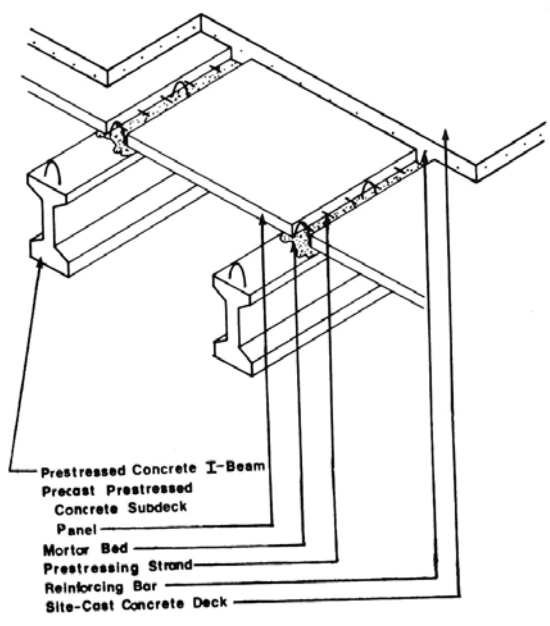


Figure 3.1: Typical precast/prestressed partial-depth deck panel (Sprinkel 1985)

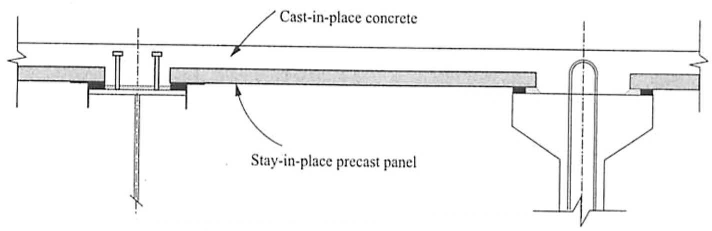


Figure 3.2: Typical partial-depth precast panel system (Tadros 1998)

3.2 FABRICATION DETAILS

Partial-depth panels are relatively simple to fabricate and should provide little problem for most precast concrete producers. PCI-NER (2001) provides recommended tolerances for the fabrication of panels, along with design and detailing requirements. Panels are typically fabricated in a long-line bed so that many panels can be fabricated at one time.

Two methods are used to produce panels of the proper width. The first method places end forms or spacers in the formwork at the desired intervals. After the panels have cured, the strands between panels are cut, resulting in panels of the desired width with small extensions of prestressing strand extending beyond the faces of the panels. The second method involves casting one long, continuous panel. After the panel has set, it is saw-cut into shorter panels of the desired width. In this case, the ends of the prestressing strands are flush with the faces of the panels (Fagundo 1985). Fabricators for WSDOT projects have typically utilized the first method (CTC, personal communications, February 26, 2004).

The panel's top surface is typically roughened during fabrication to increase the bond between the panel and cast-in-place concrete (Goldberg 1987). The Prestressed Concrete Institute–New England Region (PCI-NER 2001) recommends that the top surface be broom roughened to an amplitude of approximately 0.06 inch.

Because the panels are typically thin, they can be fragile. The potential for damaging a panel should be reduced by limiting the number of times the panels must be moved during precasting operations (Sprinkel 1985). It is important to select the location for lifting hardware so that handling stresses are minimized (Sprinkel 1985). Guidelines for the location of the lifting hardware are provided in the WSDOT Bridge Design

Manual (WSBDM). It is important to analyze the panels for each condition to which they will be exposed throughout their life, including release of prestressing, handling and shipping, placement of topping, and service loads.

3.3 CONSTRUCTION PROCEDURE

The following procedure is commonly used to construct bridge decks incorporating partial-depth deck panels. Detailed construction issues are addressed in the *Key Issues* section.

1. Panels are delivered to the bridge site in the order in which they will be placed. Figure 3.3 shows panels being unloaded at the construction site.

Figure 3.3: Removing panels from a flatbed trailer (Courtesy of WSDOT)

2. Grout dams are placed on the top flange of the bridge girders.
3. Temporary supports are provided to position the panels until they are grouted into place.
4. The panels are placed on the girders, as shown in Figure 3.4.

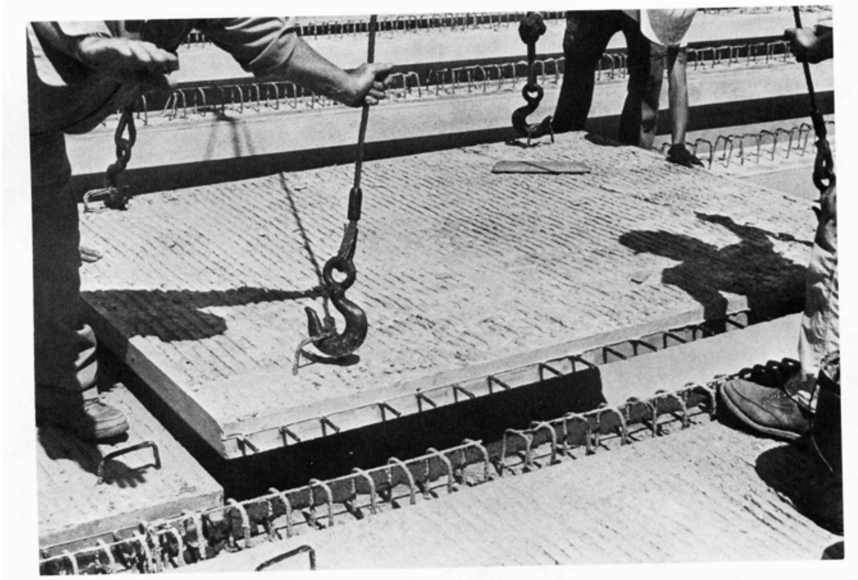


Figure 3.4: Typical partial-depth precast panel installation (Sprinkel 1985)

5. A grout layer is poured between the bottom of the panels and the top flange of the girder.
6. Formwork for the deck overhangs is constructed concurrently with steps 1-5 or after they have been completed.
7. The top layer of reinforcement for the composite deck is placed on top of the partial-depth panels as shown in Figure 3.5.
8. The cast-in-place concrete portion of the deck is placed and allowed to cure.
9. The temporary formwork for the overhangs is removed after the cast-in-place concrete reaches the specified strength. The temporary formwork could possibly be eliminated by using temporary props to support partial-depth panels for the overhangs. However, WSDOT is concerned that using partial-depth panels for the overhang portion of the deck would make future widening of the bridge difficult.

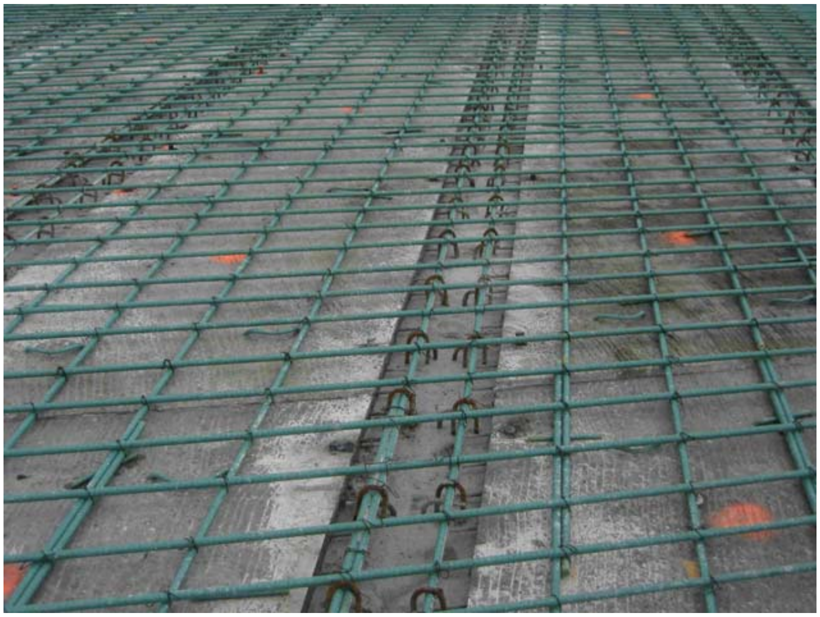


Figure 3.5: Top layer of reinforcing steel in place on top of partial-depth panels (Courtesy of WSDOT)

3.4 SUMMARY OF USE

Partial-depth deck panels were first used in the 1950s for bridges on the Illinois Tollway project. In the late 1960s and early 1970s, many other states began to incorporate them into their bridges (Goldberg 1987). Such panels have been used in at least 28 U.S. states and Canadian provinces (Goldberg 1987). They are currently used in Washington state for spanning between adjacent girders and over the void in tub girders. WSBDM (1998) standard drawings show typical details for partial-depth deck panels. If the superstructure is not post-tensional, WSDOT limits the use of partial-depth panels to the dead load positive moment regions of the deck. A typical full-depth, cast-in-place deck is used in negative moment regions (WSDOT BSO, personal communication, January 29, 2004). Previous applications have had slightly higher construction costs but resulted in overall savings because of reductions in the amount of traffic control required (WSDOT BSO, personal communication, January 29, 2004). Using the system more frequently would decrease the costs as contractors and fabricators became more accustomed to it (CTC, personal communications, February 26, 2004).

Partial-depth panels have been used in a variety of applications. They have been used for both deck replacement and new construction. They have also been used on both steel girders and precast prestressed concrete girders. Details have been presented for both cases in many references, including Ross Bryan Associates, Inc. 1988 and PCI-NER 2001. The panels have been used on skewed bridges. For lightly skewed (15° or less) bridges, the panels are saw cut to match the skew (Tadros 1998). On larger skews, traditional forming is used at the ends of the bridge.

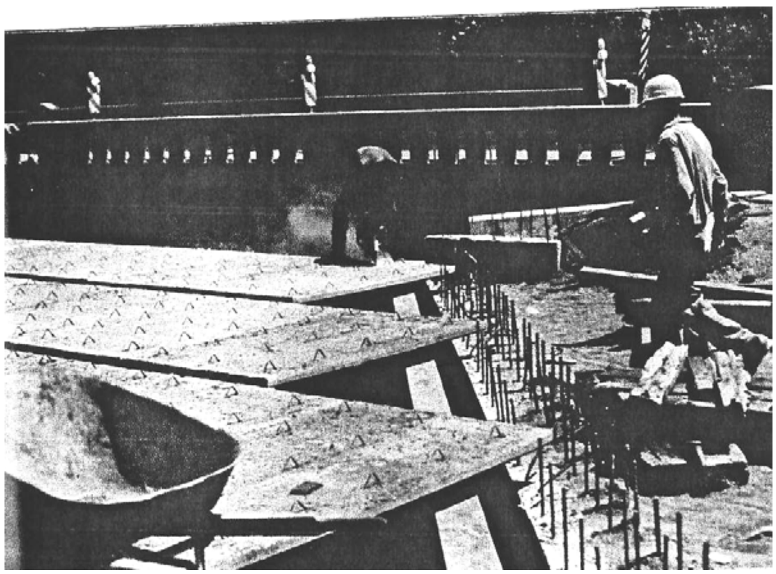


Figure 3.6: Rectangular panels to be saw cut in the field on a skewed bridge (Goldberg 1987)

3.5 DURABILITY EVALUATION

The most common problem with the use of partial-depth panel decks is cracking in the cast-in-place concrete portion of the deck at both the transverse joints between panels and at the locations where the panels bear on the girders (Ross Bryan Associates, Inc. 1988). Although cracks have been observed in the deck surface directly above the transverse joints between panels, dissection of old decks has revealed that the cracks usually extend only part-way through the cast-in-place portion of the deck slab. The cracks are not believed to affect the structural performance of the deck slab significantly (Goldberg 1987 and Sprinkel 1985). However, the cracks do present a deterioration concern because they permit the ingress of moisture and corrosion agents to the reinforcement in the deck. This is a serious concern in highly corrosive environment such as coastal areas or regions where deicing salts are used. This type of cracking has prompted WSDOT to use partial-depth slabs only in dead load positive moment regions of the bridge deck. In the negative moment region, tension in the top of the slabs can promote the development of the transverse cracks. Such cracking is especially likely because the shrinkage of the cast-in-place concrete will be concentrated at the joints between the precast panels. In the positive moment region there is compression in the top of the slab, which helps to inhibit the development of the transverse cracks (WSDOT BSO, personal communication, January 29, 2004). With proper design, detailing, and construction, the resulting composite deck slab can have good durability (Goldberg 1987).

3.6 KEY ISSUES

3.6.1 Key Issue #1: Placement of Panels on Girders

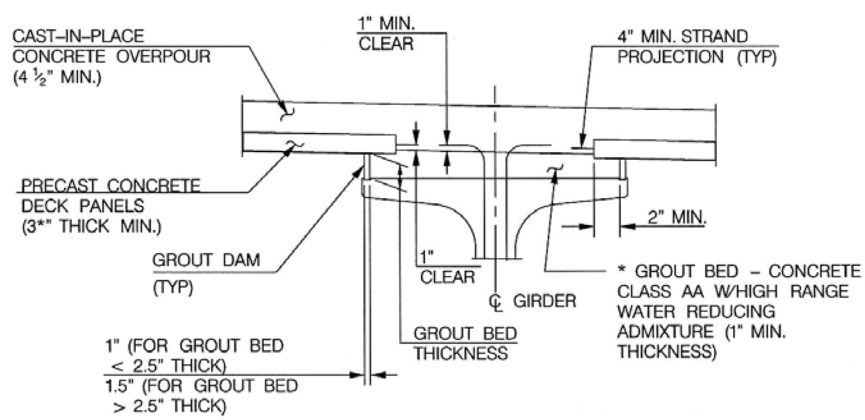


Figure 3.7: Typical detail for panels bearing on a concrete girder (PCI NER 2001)

The bearing of the partial-depth panels on the supporting girders is a critical issue that must be addressed properly to produce acceptable performance. A solid, uniform bearing between the partial-depth panels and the girders must be provided. In the first generation of designs, fiberboard was used as the bearing material between the partial-depth panels and the supporting girders. The fiberboard deformed in the rain and restricted the flow of cast-in-place concrete between the panels and girders (Medlock

2002). Because the panels lacked a solid, uniform bearing region, cracks in the cast-in-place concrete formed in the bearing area. Using soft bearing materials, such as fiber board, has caused other problems: the bridge deck may behave like simple spans between girders instead of a continuous span over girders; the ends of the panels may delaminate from the cast-in-place concrete near the joints, forcing the cast-in-place concrete to carry all of the live load shear; and cracking may occur over the joints. Current designs require the panels to be firmly bedded in grout or concrete on the supporting girders (Tadros 1998 and Fagundo 1985). The grout must completely fill the void between the panels and the girder flanges and be allowed to reach the required strength before the placement of the cast-in-place concrete (PCI NER 2001). Sufficient overlap between the panel and girder flange is also required to assure proper bearing. WSBDM (1998) standard drawings specify a minimum 4-in. overlap.

To construct the grout bed, the panels must be supported on the girders, and the side-forms for the haunch region must be installed. The haunches are usually formed with grout dams, which are typically high-density expanded polystyrene foam strips that are continuous over the length of the girder. The WSBDM (1998) requires a “closed cell foam” to be used for the grout dam. The grout dam is attached to the top of the girder with an adhesive. An adhesive is also placed on top of the grout dam, so that when the panels are placed, the grout dam will also be fixed to the bottom of the panel. The panels are supported on the girders with either the grout dams or leveling bolts. Using the grout dams as shims for the panels eliminates the need for any extra hardware; however, the dams must be cut to the proper height in the field, which is tedious and time consuming (PCI NER 2001 and Medlock 2002). Leveling bolts work in the same way in partial-depth panels as they do in full-depth panels. Panels with leveling bolts awaiting installation are shown in Figure 3.8. WSDOT prefers the use of leveling bolts over shims because leveling bolts facilitate quicker and more accurate leveling of the panels in the field (WSDOT BSO, personal communication, January 29, 2004). The gap between the girder and the panel is formed with a grout dam.

Any temporary material located in the bearing area should be compressible. If materials such as steel or hard plastic are left in place and the grout shrinks, the unyielding material will become the primary support likely resulting in undesirable cracking in the bridge deck over the points of rigid bearing (Ross Bryan Associates, Inc. 1988). PCI-NER (2001) recommends that temporary supports consist of continuous, high-density, expanded polystyrene grout dams with a minimum compressive strength of 55 psi and with an approved adhesive used to adhere the grout dam to the girder and deck panel. WSDOT's bridge standard drawings show a detail with closed-cell foam and leveling bolts, as shown in Figure 3.9. To ensure that the leveling bolts do not become the primary bearing point, they must be loosened by two turns after the grout has reached the required strength.

When the bearing has been properly detailed and constructed with grout or concrete, development of cracks has been reduced significantly (Goldberg 1987).

Figure 3.8: Panels stacked on site with leveling bolts and prestressing extensions visible (Courtesy of WSDOT)

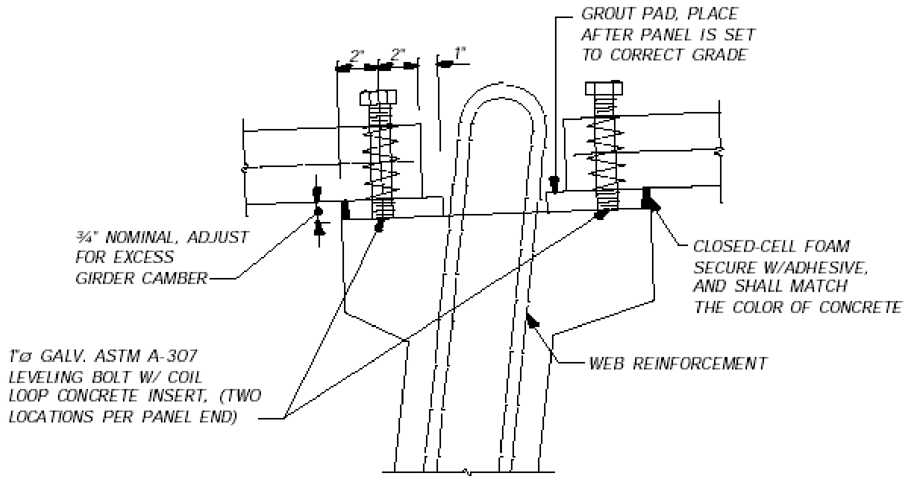


Figure 3.9: Typical detail for panels bearing on a concrete tub girder (from WSDOT standard bridge drawings)

3.6.2 Key Issue #2: Prestressing Extensions

As mentioned in the *Fabrication Details* section above, two methods are commonly used to fabricate panels to the desired width. The first method involves cutting one long, continuous panel into shorter panels so that the ends of the prestressing strands are flush with the faces of the panels (Fagundo 1985). The second method uses end forms between the panels, resulting in small lengths of prestressing strand extending beyond the face of the panel, as shown in Figure 3.8.

The effectiveness of prestressing extensions in increasing the performance of partial-depth panel decks is currently under debate. Several research projects examining the effects of prestressing extensions have found that the strand extensions did not significantly affect the behavior of the deck system (Fagundo 1985, Goldberg 1987, Klingner and Bieschke 1988, and Ross Bryan Associates, Inc. 1988). Research performed at the University of Texas at Austin showed that there is no difference in the size of cracks in decks constructed from panels with or without strand extensions (Tadros 1998). AASHTO states that prestressing strands and/or reinforcing bars in the panel need not extend into the cast-in-place concrete above the beams (AASHTO LRFD 9.7.4.3.2 1998).

In contrast, research performed in Florida showed that a 6-in. projection of prestressing strands from the ends of the panels is required to anchor the panels to the cast-in-place deck (Tadros 1998). Although the AASHTO LRFD Specification does not require extensions, it does include in the commentary that the absence of extended reinforcement may affect transverse load distribution because of a lack of positive moment continuity over the beams. This could result in reflective cracking where the panels rest on the girders. It is important to protect reinforcing steel in the deck slab in case transverse cracks do develop, especially in areas where deicing salts are used (AASHTO LRFD C9.7.4.3.2 1998). If extensions are to be used, PCI-NER (2001) recommends using a minimum extension of 4 inches. It is also important to verify that there is no conflict between the prestressing extensions and the girder's web reinforcement (Sprinkel 1985).

3.6.3 Key Issue #3: Development of Prestressing within Panels

The span of partial-depth panels is limited to the distances between bridge girders. This leaves only a short distance for the transverse prestressing strands to develop. Research performed at the University of Florida found that in most cases the development length provided in partial-depth panels was less than that required by AASHTO specifications (Tadros 1998). Because of the limited development length, it is important to avoid additional conditions that could decrease the bond of the strand to the concrete, such as dirty strands (Ross Bryan Associates, Inc. 1988). The de-tensioning procedure used by the fabricator can also affect the development length. Sudden release of the prestressing force, caused by flame-cutting the strands, has been shown to increase the needed development length. Therefore, the prestressing force should be reduced gradually for partial-depth panels (Ross Bryan Associates, Inc. 1988). Other potential solutions include applying a grit to the outside of the strand or using an indented strand. It is not desirable to decrease the development length too much because small development lengths can increase the potential for splitting the panels at their ends (CTC, personal communication, February 26, 2004). WSDOT typically uses a 3/8- or 7/16-inch, seven-wire strand to decrease the required development length (WSDOT BSO, personal communication, January 29, 2004 and WSBDM 1998).

3.6.4 Key Issue #4: Composite Action between CIP Concrete and Precast Panel

Partial-depth panels must be capable of developing sufficient composite action with the cast-in-place concrete to be an effective bridge deck system. As a composite deck system, the cast-in-place concrete and the partial-depth panels create the total thickness of the slab, with the panel's reinforcing steel serving as the positive moment steel in the transverse direction. One way to increase the composite action between the cast-in-place concrete and the partial-depth panels is to intentionally roughen the top surface of the panel (AASTHO LRFD 9.7.4.3.3 1998, Fagundo 1985, and Goldberg 1987). Ross Bryan Associates, Inc. (1988) and the WSBDM (1998) recommend that panels be raked in the direction parallel to the strands in order to minimize the reduction in the section modulus. To ensure full bond between the cast-in-place concrete and panels, it is important that laitance or other contaminates be removed from the panels before placement of the cast-in-place concrete (PCI NER 2001). Typically, bond agents and mechanical connectors are not needed to develop full composite action (AASHTO LRFD C9.7.4.3.3 1998). Research has shown that mechanical connections, such as U bars, extending from the top of the panels do not significantly affect the structural performance of the bridge deck (Klingner and 1988). There have been cases in which partial-depth panel bridges have been demolished and the bond between the panels and the cast-in-place concrete was found to be still intact (CTC, personal communication, February 26, 2004).

Limitations are also placed on the percentage of the total deck thickness that can comprise partial depth panels. AASHTO LRFD (1998) Section 9.7.4.3.1 and PCI-NER (2001) recommend that the panels neither exceed 55 percent of the finished deck depth nor be less than 3.5 inches. This requirement helps to minimize the development of cracks in the cast-in-place concrete over the panel joints and to ensure that composite action will develop. WSBDM standard drawings specify a panel depth of 3.5 inches. (WSBDM 1998). PCI-NER (2001) requires that the thickness of the cast-in-place portion be at least 4.5 inches.

3.7 EVALUATION OF SYSTEM

Partial-depth concrete panels appear to be a promising system for rapid deck construction in some applications. It has been asserted that because partial-depth panel systems require a significant amount of cast-in-place concrete they may not be as well suited for rapid construction as some other systems (Sprinkel 1985). However, the durability of partial-depth panel systems is good and the benefits of partial-depth panels for rapid construction are substantial when the amount of grouting required for other systems is considered.

A significant reduction in on-site labor results from using partial-depth panels. Less concrete and reinforcing steel must be placed on-site in comparison to that required in a cast-in-place bridge deck. One of the largest benefits of using partial-depth panel deck slabs is the decrease in expensive and time-consuming forming operations because there is no need to install and remove formwork, except for the deck overhangs (Ross Bryan Associates, Inc. 1988). The need for overhang formwork is not a significant drawback in Washington state. Most superstructure systems would require overhang formwork to facilitate the casting of traffic barriers (WSDOT BSO, personal communication, January 29, 2004).

Some problems still exist with the use of partial-depth panels. At this time, WSDOT only allows the use of partial-depth panels in the positive moment region of the deck. Therefore, over interior piers, a full-depth, cast-in-place slab must be used. This mixed use of partial-depth panels increases the cost and construction time for the deck. Ideally, one type of bridge deck should be used for the entire deck (Atkinson Construction Company (ACC), personal communication, March 24, 2004 and CTC, personal communication, February 26, 2004). Another problem that has arisen in some uses of partial-depth panel deck slabs is the height in the haunch area because of camber, cross-slopes, and tolerances. If the haunch gets too high, problems can arise in forming the bearing area for the panels on the girders (CTC, personal communication, February 26, 2004 and WSDOT BSO, personal communication, January 29, 2004). The repair of partial-depth panel bridge deck slabs can also be more difficult than that of full-depth cast-in-place deck slabs. If an exterior girder is damaged and requires replacement, traditional cast-in-place decks can be cut between girders and only half needs to be removed (WSDOT BSO, personal communication, January 29, 2004). On the other hand, when the deck includes partial-depth panels, the deck slab has to be removed back to the first interior girder. Despite these problems, partial-depth panels are still very versatile. They can be used on both steel and concrete girders, short and long bridges, and skewed bridges (Sprinkel 1985 and Tadros 1998).

Most precast concrete producers should have little problem fabricating partial-depth panels because they are relatively simple. Most producers can fabricate the partial-depth panels with multi-purpose forms, which can also be used for other types of structures (Sprinkel 1985). Because more than one fabricator can produce the panels, price competition should keep costs down (WSDOT BSO, personal communication, January 29, 2004).

The NUDECK system is a hybrid system utilizing aspects from both partial-depth deck systems and full-depth deck systems. The NUDECK system was developed with the goal of eliminating some of the drawbacks found in a conventional partial-depth deck system. These include the need to utilize traditional forming techniques for the overhangs and the cracking, which has been known to develop over both the transverse joints and above the girders in traditional partial-depth panels systems. The NUDECK system utilizes precast partial-depth panels (4.5-in. thick) that extend over the entire width of the bridge connected in the transverse direction by both prestressing and reinforcement. These units are connected in the longitudinal direction by reinforcement, allowing the NUDECK system to be continuous in both the transverse and longitudinal directions. This helps to reduce cracking (Badie et al. 1998). The system also eliminates the need for traditional forming of the overhangs. The Skyline Bridge located in Omaha, Nebraska, is the first bridge constructed with the NUDECK system (Sun 2004).

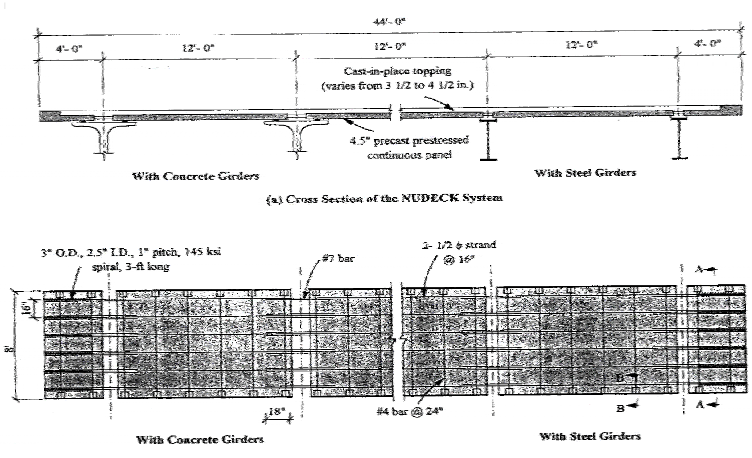


Figure 3.10: Diagram of the NUDECK System (Badie et al. 1998)

CHAPTER 4:
PRESTRESSED CONCRETE MULTIBEAM SUPERSTRUCTURES

4.1 DESCRIPTION OF SYSTEM

Prestressed concrete multibeam superstructures consist of precast/prestressed concrete girders, such as double tees, box beams, deck bulb-tees, and channels placed adjacent to one another spanning the longitudinal direction of the bridge. Figure 4.1 shows an example of a multibeam superstructure. The girders serve as both the deck and the support system for the superstructure. The girders are connected to each other by longitudinal, grout-filled shear keys, mechanical fasteners, and/or transverse post-tensioning. A structurally composite, cast-in-place concrete slab is not required. In many applications, a non-composite wearing course is added to provide a smooth riding surface.

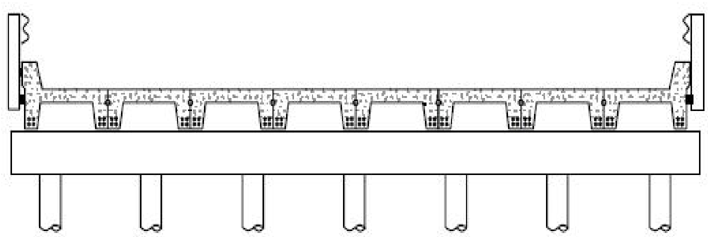


Figure 4.1: Elevation view of bridge superstructure composed of precast/prestressed concrete channel beams (Ingersoll et al. 2003)

4.2 FABRICATION DETAILS

Girders for prestressed multibeam concrete superstructures can be either pretensioned or post-tensioned. Pretensioning is commonly used because it results in reduced costs and helps girders withstand transportation and erection loads. The presence of pretensioning effectively limits the production of the girders to PCI certified production plants (ACC, personal communication, March 24, 2004). Most precast concrete producers are experienced at fabricating the types of girders typically used. In many cases, the producers have reusable formwork for the most common shapes, such as deck bulb-tees and box beams (CTC, personal communications, February 26, 2004).

Typically, girders are cast in long-line pretensioning beds. The cost and complexity of fabricating prestressed girders vary, depending on the beam cross-section. Double tees and ribbed beams can be easier to fabricate because the formwork does not have to be movable (Badie et al. 1999). The members are cast and then lifted out of the unmoving formwork (Csagoly and Nickas 1987). Bulb-tees are typically fabricated with removable, reusable side forms. Placing and removing side forms is a common procedure for most producers but can result in increased costs because of the more complex forms and the additional labor required. Box girders and voided slabs require either permanent polystyrene or removable steel void forms, making fabrication more difficult and expensive. These sections can also be produced by using a mandrel in combination with a low-slump concrete mixture. This procedure is cheaper but has a large initial cost (CTC, personal communications, February 26, 2004).

4.3 CONSTRUCTION PROCEDURE

The following construction procedure is typically used for prestressed concrete multibeam superstructures. It is independent of the shape of the girder. Variations in shape would only affect the number of girders that need to be lifted into place.

1. Place beams adjacent to one another on the pier caps.
2. Remove the differential camber from the beams by either jacking them against each other or positioning large loads on individual girders.
3. Weld/bolt the mechanical connectors in the longitudinal joints.
4. If required by design, grout the longitudinal shear key joints between the beams and allow the grout to reach a specified compressive strength.
5. If required by design, apply transverse post-tensioning. WSDOT requires either a longitudinal grout key or transverse post-tensioning, but not both.
6. Place a wearing course, if required by design. WSDOT always requires a wearing surface (overlay) and membrane.

4.4 SUMMARY OF USE

Prestressed multibeam concrete superstructures without an additional cast-in-place concrete slab have been used extensively for bridges throughout the United States (El-Remaily et al. 1996). The system is most common in remote areas where fresh concrete is difficult to obtain and traffic volumes are low (WSDOT BSO, personal communication, January 29, 2004 and CTC, personal communications, February 26, 2004). Figure 4.2, based on data in the 1990 National Bridge Inventory (NBI), shows the percentage of prestressed bridges built by type from 1950-1989. The figure shows that prestressed concrete multibeam superstructures are commonly used and are growing in popularity (Dunker and Rabbat 1992).

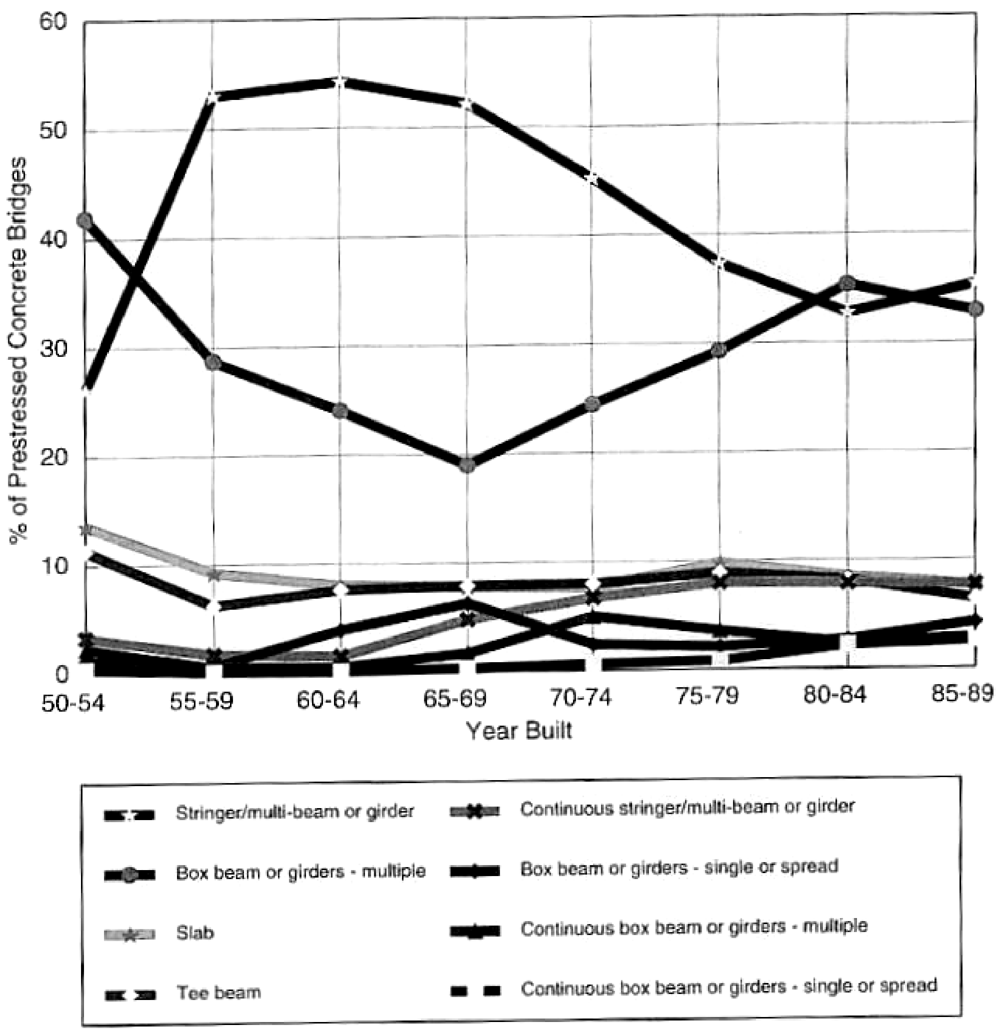


Figure 4.2: Percentage of prestressed concrete bridges constructed with each section type (Dunker and Rabbat 1992)

Prestressed multibeam superstructures have been used in Washington state for a significant period of time. Deck bulb-tees, voided slabs, and ribbed beams are standard girder cross-sections in Washington state for multibeam concrete bridges (WSBDM 1998) and are used to construct highway bridges without additional cast-in-place concrete slabs (WSDOT BSO, personal communication, January 29, 2004). According to the NBI compiled in December 2002, at least 750 prestressed multibeam concrete bridges have been constructed in Washington state without cast-in-place concrete slabs. WSDOT has not constructed bridges of this type in approximately four years because of construction and serviceability problems (WSDOT BSO, personal communication, January 29, 2004). The reason for the limited use is the poor performance of the longitudinal joints between the girders under heavy traffic (WSDOT BSO, personal communication, January 29, 2004). Prestressed multibeam concrete bridges are still being constructed in Washington state by local agencies and the U.S. Forest Service (WSDOT BSO, personal communication, January 29, 2004). Section 6.2.4 of the Washington State Bridge Design Manual (1998) states that prestressed multibeam concrete bridges should only be used for bridges with low annual daily traffic (ADT). This is consistent with the data from the NBI. Of the approximately 750 prestressed multibeam concrete bridges in Washington state, only four have an ADT of over 10,000.

4.5 DURABILITY EVALUATION

The longitudinal joints between girders have caused the vast majority of the durability problems associated with this system. In bridges without wearing courses, many instances of cracking and spalling of the longitudinal joints have been recorded (WSDOT BSO, personal communication, January 29, 2004). Cracking has been identified in the overlay on many prestressed multibeam concrete bridges. The cracks appear directly over the location of the longitudinal joints between girders and run the length of the bridge (Badie et al. 1999 and El-Remaily et al. 1996). These conditions leave the joint prone to leakage, which can cause deterioration of both the prestressed girders and substructure components. Leakage can also become a driving hazard for bridges with highway underpasses (Badie et al. 1999 and El-Remaily et al. 1996). WSDOT uses asphalt wearing courses to help prevent the cracking that occurs in concrete overlays (WSDOT BSO, personal communication, January 29, 2004).

Deterioration of prestressed multibeam concrete bridges is most prevalent in bridges without thick wearing courses and inadequate transverse post-tensioning (El-Remaily et al. 1996). Because of problems associated with joint leakage, the AASHTO LRFD Bridge Design Specifications (1998) do not recommend the use of prestressed multibeam concrete bridges without cast-in-place concrete slabs in regions where deicing salts are commonly used (AASHTO 1998).

4.6 KEY ISSUES

4.6.1 Key Issue #1: Design of Individual Girders

Individual girders are typically designed according to the requirements of the AASHTO LRFD Bridge Design Specifications (1998) or the AASHTO Standard Specification for Highway Bridges (2002). Design includes consideration of the ultimate limit states of flexure and shear, as well as the service limit states of stresses in the beam at transfer and service. The requirements for transfer length, development length, and anchorage must also be considered. Stresses induced in the girders during construction should also be considered in girder design. These stresses can be large for prestressed multibeam concrete bridges because differential camber between girders must be eliminated inducing large stresses in the girders (Stanton and Mattock 1986).

To make design and fabrication more efficient, standard cross-sections have been developed by AASHTO, the Prestressed Concrete Institute (PCI), and WSDOT (Nawy 2003 and WSBDM 1998). Using standard cross-sections, and thus reusable formwork, improves the economy of fabrication (CTC, personal communication, February 26, 2004). The Washington State Bridge Design Manual, Section 6.2.4, includes requirements for standard precast/prestressed beams used in short span bridges (WSBDM 1998). Three standard cross-sections are used in Washington state for prestressed concrete multibeam bridges. They are voided slabs, ribbed beams, and deck bulb-tees, as shown in Figure 4.3. Plans for the standard shapes are provided in appendices 6.6-8 (WSBDM 1998). Standard box girder cross-sections have been developed by AASHTO. A combined AASHTO/PCI initiative developed standard deck bulb-tee cross-sections (Nawy 2003). Other standard girder cross-sections have been proposed in the literature. They include

- Csagoly and Nickas (1987)—Developed a double T section for the Florida Department of Transportation.
- Badie et al. (1999) —Developed a modified box beam by widening the top flange. This change was intended to reduce the torsional stiffness at the longitudinal joints to prevent cracking. The largest section is 11 ft. wide and capable of supporting a single lane of traffic. The proposed cross-section is shown in Figure 4.4.

Each standard section is designed for several girder depths and material properties. Typical L/d ratios, maximum span lengths, and weight for each shape are compiled in Table 4.1.

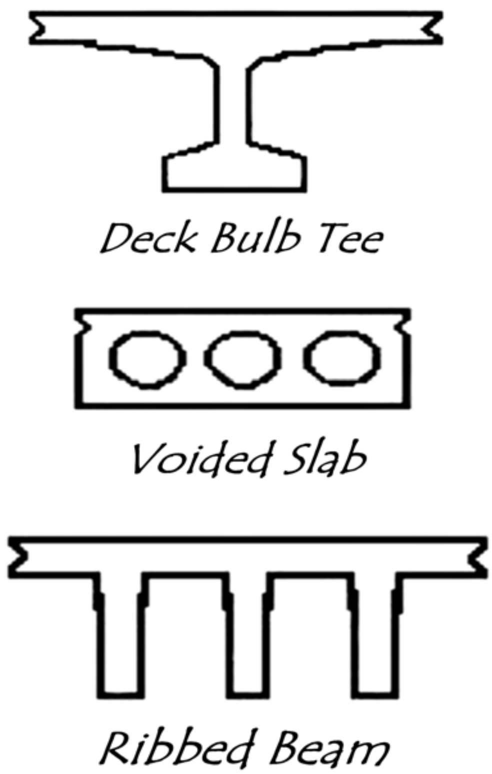


Figure 4.3: Standard girder cross-sections used by WSDOT (WSBDM 1998)

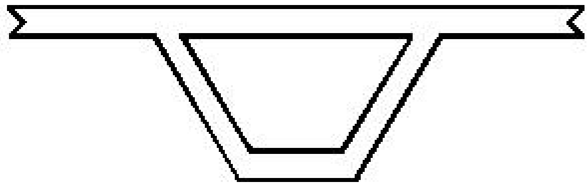


Figure 4.4: Trapezoidal beam cross-section proposed by Badie et al (1999)

Table 4.1: Typical L/d values and span lengths for standard girder cross-sections (WSBDM 1998, Badie et al. 1999, and Nawy 2003)

Shape	Type	Flange Width (in.)	L/d	Max Span (ft)	Weight/ft (kips)	Max Span Weight (kips)
WSDOT Deck Bulb-tee	W65DG*	48	26.8	145	1.11	161
		60	25.8	140	1.03	144
		72	23.1	125	0.95	119
WSDOT Precast Slab	12-in. Solid	varies	40.0	40	0.92	37
	18-in. Voided	varies	38.0	57	1.00	57
	26-in. Voided	varies	34.6	75	1.38	104
AASHTO/PCI Deck Bulb-tee	BT-35	48	34.3	100	0.75	75
		72	26.7	78	0.91	71
		96	22.3	65	1.07	70
AASHTO Box Beams	BI-36	36	40.9	92	0.58	53
	BI-48	48	40.9	92	0.72	66
Badie et al. Trapezoidal Beam	800	87	33.9	89	1.06	94
		142	26.7	70	1.40	98

*Shallower girders (W35DG, W41DG, and W53DG) are also available.

4.6.2 Key Issue #2: Longitudinal Joint Design

The girders are connected to one another with longitudinal joints. The joints are composed of a continuous grouted shear key and welded mechanical connectors spaced at 4- to 8-foot intervals. Transverse post-tensioning can be used to place the joint in compression. The longitudinal joints should be designed for out of plane shear caused by wheel loads and in-plane tension cause by shrinkage of the slab (Stanton and Mattock 1986). The fatigue performance of the joint and susceptibility to leaking should also be considered in design. Both the grouted and mechanical components of the joint should be considered in design (Csagoly and Nickas 1987). Section 5.14.1.2.8 of the AASHTO LRFD Bridge Design Specifications (1998) contains several provisions for the longitudinal connection between girders. The Washington State Bridge Design Manual (1998) contains standard plans for the shear keys and mechanical connectors between girders in appendices 6.6-8. The shear key alone is not sufficient to withstand the forces between adjacent girders. It is important to also have adequate mechanical connectors between the girders to ensure that there is adequate compression on the joint (Central Pre-Mix Prestress Co., personal communication, August 17, 2004).

The continuous grouted joint is commonly a female-to-female shear key. The shear key is commonly designed to carry the entire live load shear force on the joint (Stanton and Mattock 1986). Two standard shear key configurations are currently used for the majority of connections between bulb-tees and ribbed beams. They are shown in Figure 4.5. Girders without flanges, such as box girders and voided slabs, have traditionally been connected with narrow, shallow shear keys. El-Remaily et al. (1996) proposed using wider, deeper shear keys to prevent cracking of the joint. The traditional and proposed joint configurations are shown in Figure 4.6. AASHTO (1998) recommends using a V-joint shear key for the longitudinal joints as shown in Figure 4.7. This joint configuration was tested by Csagoly and Nickas (1987), who found it to perform adequately. This conflicts with other reports that found the joint to be undesirable because of direct contact of the precast members (Issa et al. 1995b). AASHTO (1998), Section 5.14.1.2.8, requires that shear keys be at least 6.5 inches deep and filled with non-shrink grout that reaches a compressive strength of 5000 psi in 24 hours.

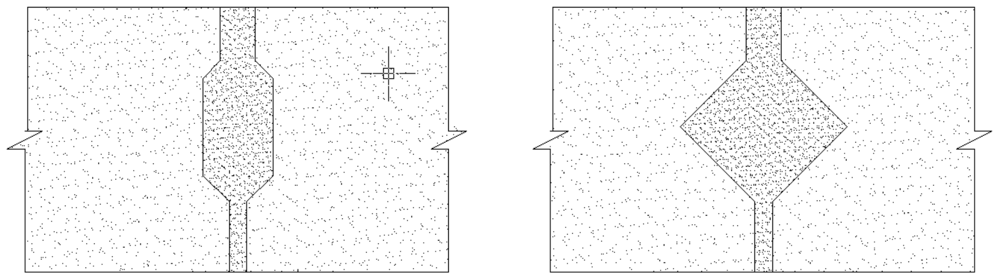


Figure 4.5: Common shear key designs used for connecting girders with flanges

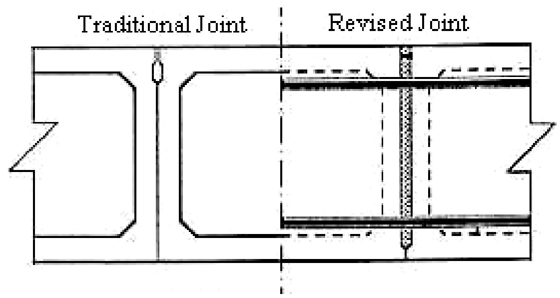


Figure 4.6: Shear key designs for connection girders without flanges
(Adapted from El-Remaily et al. 1996)

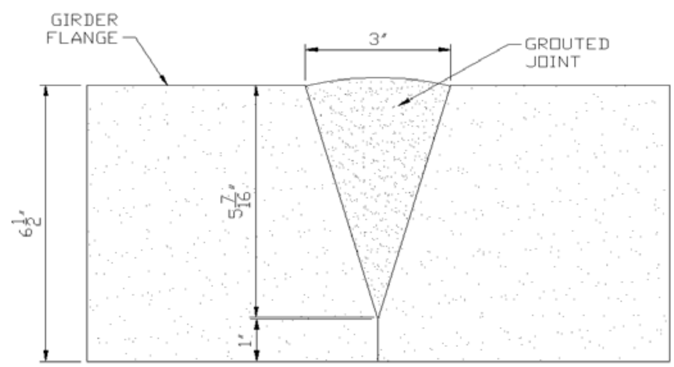


Figure 4.7: V-Joint longitudinal connection detail
(Adapted from Csagoly and Nickas 1987)

Many mechanical connector configurations have been designed and implemented. The mechanical connectors are required for carrying tensile loads between the girders because of shrinkage and torsional effects (Stanton and Mattock 1986) and because of shear due to differential camber between girders (WSDOT BSO, personal communication, January 29, 2004; Stanton and Mattock 1986). The standard mechanical connection configuration used by WSDOT is shown in Figure 4.8 (WSBDM 1998). During the past two decades, transverse post-tensioning has been incorporated into the design of multibeam bridges. The post-tensioning places the joints into compression, reducing cracking attributable to service loads (Hill 1988). Transverse post-tensioning also increases the joints' capacity to carry shear (Csagoly and Nickas 1987) and a limited amount of moment (Stanton and Mattock 1986). If post-tensioning is used, AASHTO LRFD Bridge Design Specifications (1998), Section 5.14.1.2.8, requires a minimum compressive stress of 250 psi induced across the joint. Previous applications have shown this amount of post-tensioning to be acceptable (Hill 1988). However, aligning post-tensioning ducts during construction can be very difficult (WSDOT BSO, personal communication, January 29, 2004). Consequently, WSDOT does not typically use transverse post-tensioning.

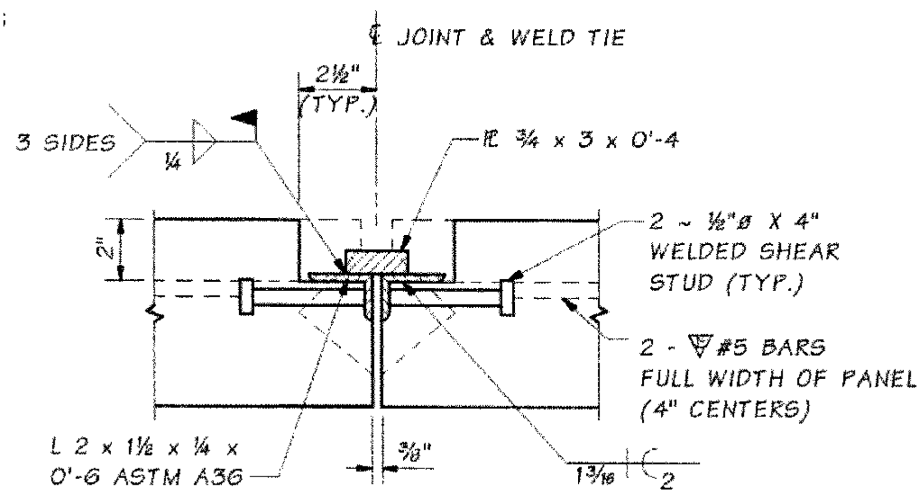


Figure 4.8: Standard mechanical connection detail (WSBDM 1998)

4.6.3 Key Issue #3: Load Distribution among Beams

The amount of wheel load transferred from a loaded beam to an adjacent unloaded beam must be considered to determine appropriate design loads for the girders. The amount of load that can be transferred depends on the ability of the joints between the girders to transfer forces as well as the sectional properties (e.g., torsional stiffness) of individual girders and the system as a whole (Stanton and Mattock 1986). Load distribution factors for multi-beam precast/prestressed concrete bridges without cast-in-place decks are provided in Section 4.6.2.2 of the AASHTO LRFD Bridge Design Specifications (AASHTO 1998).

4.6.4 Key Issue #4: Limits on Length and Weight of Beams

The length and weight of individual girders must be small enough that the girders can be transported from the precasting yard to the construction site and safely erected with standard equipment. The following limits are provided in the Washington State Bridge Design Manual (WSBDM 1998). Girder weight should be limited to 200 kips to accommodate equipment used in precasting plants in Washington state. Girder weight must be under 180 kips because of transportation constraints. Reducing the beam weight to under 155 kips allows for faster transportation and requires no special permits. Girder length should be less than 130 feet to accommodate transportation. Contractors commonly have cranes capable of lifting up to 360 tons vertically. The weight that can be lifted is reduced if the crane boom is inclined at an angle. It is also common for two cranes to be used to lift heavy girders. In most situations, a girder under the allowable transportation weight limits should be erectable at the construction site. Site location and geometry have an impact on the ability of the cranes to locate themselves where they can safely lift the weight (WSDOT BSO, personal communication, January 29, 2004).

4.7 EVALUATION OF SYSTEM

Multi-beam precast/prestressed bridges appear to be a viable option for rapid construction. One key advantage of the system is that it has been used previously in Washington State, providing vital experience for fabricators, contractors, and engineers. The system does have a disadvantage in that it is recommended only for applications with low ADTs (under 10,000) because of the poor durability of longitudinal joints under heavy truck traffic. Continuity and restrained moments are also concerns.

The fabrication of girders for prestressed multibeam concrete bridges should be relatively economical and fast. Many precast concrete producers already manufacture the girders commonly used in multibeam bridges. Accordingly, the producers already have reusable formwork, eliminating initial costs. Multiple girders can also be cast at the same time on a long-line bed, resulting in rapid fabrication.

Transportation constraints can limit the use of prestressed multibeam concrete bridges. Long span girders for multibeam bridges often approach the transportation limits for both length and weight. This is a concern because length and weight limitations for transportation can determine the maximum span length on multibeam bridges unless girders are spliced, which can hinder rapid construction.

The versatility of prestressed concrete multibeam bridges in accommodating a variety of bridge geometries and lengths is limited. The system does provide some flexibility in using different cross sectional shapes to accommodate different bridge lengths and widths. The different cross-sections allow greater architectural freedom. But the system also limits the maximum span of a prestressed concrete multibeam bridge to the longest girder capable of carrying the required load that can be transported to the bridge site and erected. This limits the span length of these bridges to approximately 180 feet (Stanton and Mattock 1986); use of shorter spans (~80 to120 feet) is more common. This system is also limited to straight, non-skewed bridges because the differential camber between girders is exacerbated when girders are placed on a skew. Pretensioning curved girders is also extremely difficult. In most curved bridges, straight girders would be placed as chords rather than using curved girders.

Construction of prestressed concrete multibeam bridges has several distinct advantages and disadvantages. The main construction advantage of multibeam bridges is that girders serve as both the superstructure support system and deck. This requires a significantly smaller number of components that need to be lifted into place, resulting in more rapid construction, especially when construction is limited to night and weekend closures. However, girders can become heavy, requiring large lifting equipment and limiting the boom angle at which the girders can be placed. Another advantage is that multibeam bridges require no deck formwork, eliminating the work time required for both placing and removing formwork, and thus resulting in more rapid construction. Any impact on traffic from formwork operations is also eliminated. Another advantage is that all superstructure construction can be performed from on top of the bridge. This eliminates the need for a "sub deck," saving both time and money. Safety conditions for construction workers and traffic under the bridge are also improved. No cast-in-place concrete is required for multi-beam bridges, eliminating the curing period and speeding construction.

One of the greatest construction difficulties is eliminating the differential camber between the girders (WSDOT BSO, personal communication, January 29, 2004). It is important to develop an adequate means of removing the differential camber between the girders on site instead of attempting to shuffle girders in the fabrication yard in an attempt to place girders adjacent to other girders with similar camber (Central Pre-Mix Prestress Co., personal communication, August 17, 2004). Transverse post-tensioning is required to improve the durability of multibeam bridges. Installing the post-tensioning requires a specialty crew and can increase the construction time. Cast-in-place concrete diaphragms have also been required by WSDOT in previous applications to provide acceptable performance (WSDOT BSO, personal communication, January 29, 2004). This can possibly reduce the rapid construction advantage of this system over a cast-in-place slab superstructure. A wearing course is required to provide a smooth riding surface and aesthetically acceptable appearance for high speeds and large traffic volumes. A waterproofing membrane may also be required to prevent leakage through the longitudinal joints (WSDOT BSO, personal communication, January 29, 2004).

Accommodations must be made for the camber of prestressed concrete girders. There are several possible ways to do this. One possible solution is to align the girder profile with the bridge profile. This can be accomplished by specifying a curved bridge profile that matches the girder profile, or by casting the girders with downward sag to offset the camber caused by the prestressing, resulting in a zero camber girder for use on straight bridge profiles. The difference in profiles can also be made up by using a cast-in-place topping on the prestressed girders. Although this allows the greatest construction and casting tolerances, it reduces the rapid construction advantage of the prestressed multibeam concrete bridge system and leads to an increase in the seismic weight of the superstructure.

A hybrid of this system, referred to as the partial-depth prestressed concrete multibeam bridge, appears to have promising attributes for rapid construction. The system consists of thin, flanged (~3-in.) deck bulb-tee girders topped with approximately 5 inches of cast-in-place reinforced concrete to form the composite bridge superstructure.

Thin-flanged deck bulb-tees are produced by a local precast concrete producer and are available in widths of up to 8 feet (CTC, personal communication, February 26, 2004). The aim of this system is to combine the rapid construction benefits of the prestressed concrete multibeam bridge system, such as elimination of formwork, with the durability of a partial-depth deck panel system. One disadvantage of the system is that large amounts of cast-in-place concrete is required to produce a flat deck because of the camber of the girders. This is expensive and increases the seismic weight of the bridge. It should be avoided by matching the roadway profile to the girder profile or vise versa (CTC, personal communication, February 26, 2004). Currently, this system is being used primarily by counties and local governments in Washington state. It has been avoided by WSDOT in most cases because of concerns about excessive cracking of the bridge deck and durability (CTC, personal communication, February 26, 2004).

No mention of the seismic performance of these bridges has been found in the literature. The ability of the beams to produce sufficient diaphragm action must be considered. Such diaphragm action should be achievable by providing an adequate amount of transverse post-tensioning. The AASHTO LRFD Bridge Design Specifications, Section 5.14.1.2.8 (1998), support this by stating that the superstructure can be assumed to be monolithic in design if sufficient transverse post-tensioning is provided. Resistance to vertical accelerations must also be considered, but this should be similar to standard designs.

CHAPTER 5:
PRE-CONSTRUCTED COMPOSITE UNITS

5.1 DESCRIPTION OF SYSTEM

Pre-constructed composite units (PCUs) are steel or concrete girders prefabricated with a composite concrete bridge deck. PCUs are typically fabricated off site and brought to the bridge site by barge, truck, or rail. Truck or rail transportation may limit the PCUs' size and weight. After arriving on site, the PCUs are lifted into place as a unit, which greatly reduces on-site construction time (Federal Highway Administration (FHWA 2004). After several units are in place, the deck joints between units are grouted. The units are typically post-tensioned both longitudinally and transversely to provide compression across the joints. The units can be prefabricated complete with non-structural elements, such as barrier walls, light posts, wearing surfaces, electrical conduits, deck drains, and striping thereby further reducing construction time. Examples of PCUs used in previous applications are shown in figures 5.1 and 5.2. The use of PCUs has been relatively limited; therefore, less documentation can be found on their application and performance. They tend to be used on large projects, each of which is unique, so common details have yet to emerge.

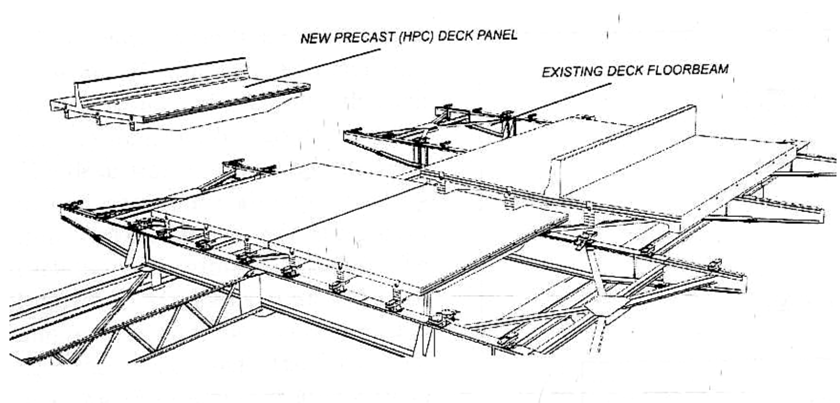


Figure 5.1: Pre-constructed composite units used for the Jacques Cartier Bridge (Zaki 2003)

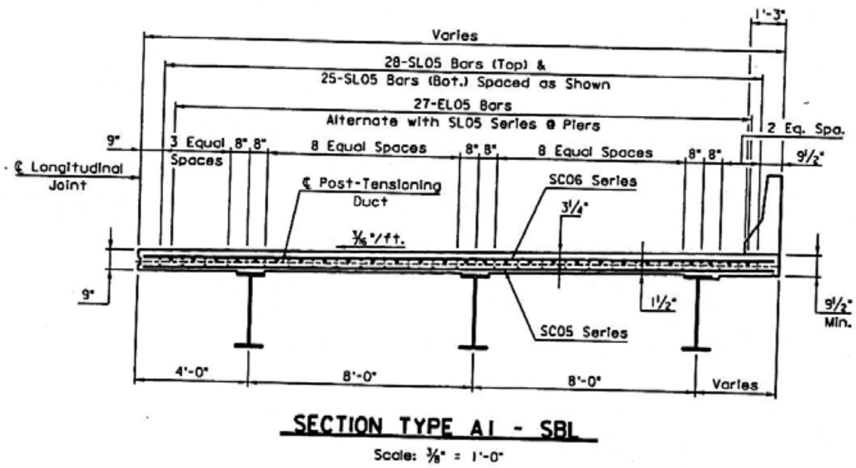


Figure 5.2: PCU from preliminary plans for superstructure replacement for Route I-95 over Overbrook Road in Richmond, Virginia (URS 2003)

5.2 FABRICATION DETAILS

Because of the infrequent use of PCUs, standard types, details, or methods for fabrication have not been developed at this time. Unique types, details, and design have been developed and used for each project. Consequently, the fabrication procedure varies for each of the PCUs utilized on each project. In general, a reinforced concrete deck slab is cast with either steel or concrete girders. The shapes, dimensioning, and fabrication procedures differ for either steel or concrete girders. For concrete girders, multiple stems are cast monolithically with the reinforced concrete deck slab. For steel girders, one process that has been proposed is the INVERSET system, which is described briefly below.

To fabricate the PCUs on the Tappan Zee Bridge project, the INVERSET system was used. This process involves inverting steel beams that are supported at their ends, hanging deck forms from the beams, and placing concrete in the forms. Under the dead load, the unit deflects to a desired point, at which time the steel beams reach a desired stress. After the concrete has reached a specified strength, the unit is inverted to the upright position. Two benefits of the INVERSET fabrication system are that the most-dense concrete surface is located at the top of the deck, and it also places the concrete deck in a permanent crack-resistant compressive stress state (White 2000), similar to that achieved by prestressing.

5.3 CONSTRUCTION PROCEDURE

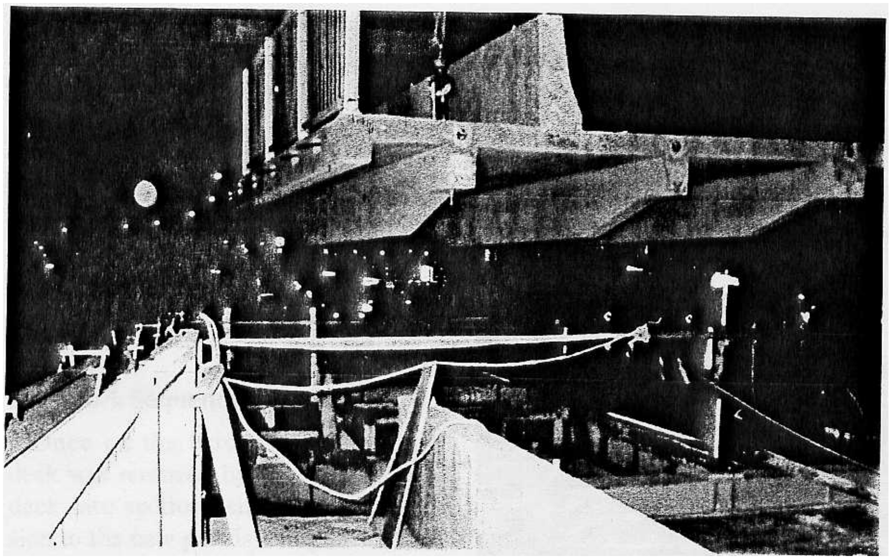


Figure 5.3: PCU being placed on the Jacques Cartier Bridge (Zaki 2003)

The construction procedures vary for each bridge project. Typically, the PCU is fabricated away from the bridge site. After the PCU arrives at the site, it is lifted by crane and placed. Once a number of PCUs are in place, grout is placed in the joints between the units. The units are then post-tensioned longitudinally and transversely to place the joints between units in compression. The joints between units were then waterproofed. Typically, a waterproofing membrane with an asphalt overlay is used.

5.4 SUMMARY OF USE

The application of PCUs appears to have begun in the 1990s, when they were first used for large-scale bridge superstructure replacement projects (FHWA 2004). Since that time, units have begun to be used in some states for smaller scale projects, such as typical overpass structures (URS Rte. I-95 over Overbrook Road Preliminary Plans). Previous projects (George P. Coleman Bridge located in Yorktown, Virginia, and Norfolk Southern Railroad Bridge over I-76 located in Pennsylvania) have involved preconstructing entire truss spans and barging them to the bridge site. With this method, six old spans were removed and replaced in nine days (FHWA 2004). The PCUs utilized for the I-95 Bridge over the James River in Richmond, Virginia, were composed of a reinforced concrete deck slab composite with steel girders. The use of the units allowed construction crews to completely remove and replace span sections during night hours only, keeping all travel lanes open during daytime hours (FHWA 2004). The Tappan Zee Bridge over the Hudson River near New York City utilized PCUs made of a concrete deck slab composite with galvanized 24-in.-deep, wide-flange beams (White 2000). The deck reconstruction of the Jacques Cartier Bridge, a long truss structure in Montreal, Canada, utilized PCUs composed of a multi-stem integral deck slab and girder system. After the units were placed, they were post-tensioned in the transverse and longitudinal directions (Zaki 2003). For the Lions' Gate Suspension Bridge in Vancouver, B.C., PCUs were utilized to replace both the deck and truss elements simultaneously. The PCUs were brought to the bridge site by barge and then lifted into place (FHWA 2004).

There are also plans to utilize PCUs on smaller projects in some states, including Virginia. One such project is the proposed widening and superstructure replacement of Rte. I-95 over Overbrook Road in Richmond, Virginia. The three-span (30-ft, 56-ft, 30-ft) structure is approximately 116 feet long . This is the first example that the University of Washington research team was able to find in which PCUs would be utilized on a typical overpass structure.

Figure 5.4: Truss PCU being barged in for the George P. Coleman Bridge (FHWA 2004)

Figure 5.5: PCU being raised into place on the Lions' Gate Suspension Bridge, Vancouver, B.C. (FHWA 2004)

5.5 PERFORMANCE EVALUATION

Use of pre-constructed composite units is relatively new. Accordingly, the performance of these systems is not well documented. Two years after the Tappan Zee Bridge project was completed, an inspection found that the PCUs were in excellent condition, with no evidence of cracking. It also found that the joints between the units were performing well, with no noticeable leakage (White 2000).

5.6 EVALUATION OF SYSTEM

There is less experience with and less literature discussing PCUs than exist for other systems. Therefore, any problems that may arise from the use of PCUs are not fully known at this time. PCUs have been used primarily for larger scale bridge projects, although their use for smaller projects, such as overpass structures, appears to be starting. The largest benefit of utilizing PCUs is the major reduction in construction time needed onsite. This is achieved by prefabricating as many components as possible off-site.

Although no literature was found on problems specifically associated with PCUs, it is assumed that they could experience problems similar to those suffered by full-depth deck panels and prestressed concrete multibeam superstructures. One key issue could be the longitudinal and transverse joints between the units and the waterproofing of these joints. Also, different top slab elevations of adjacent units could be another key issue. Another important issue is the weight and size of the PCUs and, therefore, the ability to transport and place them. One major drawback of preconstructed composite units is that designs are commonly unique to a particular project and are difficult to apply as standardized plans (WSDOT BSO, personal communication, January 29, 2004).

CHAPTER 6:
PRECAST CONCRETE PIER SYSTEMS

6.1 DESCRIPTION OF SYSTEM

Precast concrete pier systems utilize the combination of precast concrete columns and precast cap-beam components to create a pier. Both single-column piers and multiple-column piers can be constructed with precast concrete. Precast pier systems are compatible with a variety of foundation and superstructure types. Individual components in a precast pier system are connected to one another with mild reinforcing steel splices and/or post-tensioning. The design and construction of precast pier systems can vary significantly, depending on the application. Figure 6.1 shows an example of a single-column precast concrete pier composed of precast components.

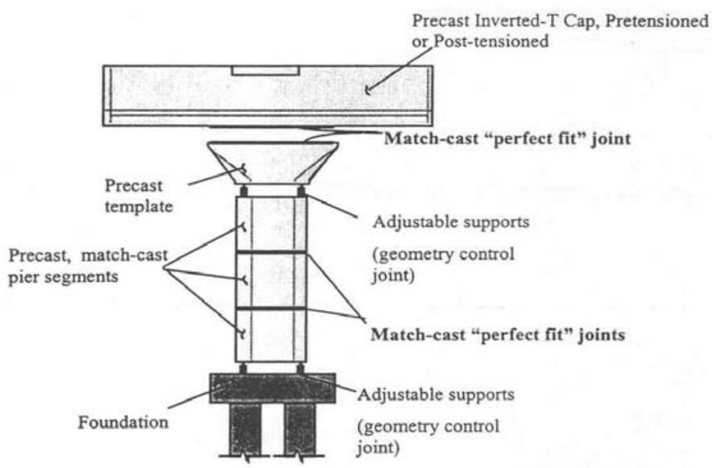


Figure 6.1: Single-column pier composed of precast concrete components (Billington et al. 1999)

6.2 FABRICATION DETAILS

The fabrication procedures for column and cap beam components vary significantly depending on the characteristics of the pier and the type of joints between the components. The two types of joints most commonly used between precast components are grouted joints and match-cast joints. Match-cast joints have been used in many applications and can significantly alter the fabrication process. They are fabricated by using the joint face of a previously completed component as part of the formwork for the next component. This procedure results in a "perfect" fit joint between the two components. Fabrication of match-cast joints is typically more labor and time intensive than grouted joints, but match-cast joints allow for easier erection, resulting in more rapid on-site construction. Because the procedures for fabrication of column and cap-beam components vary significantly from one another, each is discussed separately in the subsections below. The components that incorporate match-cast and grouted joints are both considered along with the fabrication advantages and disadvantages of each type of joint.

6.2.1 Column Fabrication

It is preferable to fabricate columns in full-height segments. This eliminates the need for joints between multiple column segments that are time consuming and costly both to fabricate and construct. Component weight must be limited to allow full-height fabrication of long columns. Hollow sections fabricated with either sono-tubes or collapsible formwork have been used to reduce the weight. For large-scale production, a system that uses a mandrel and low-slump concrete could be employed (CTC, personal communication, February 26, 2004). If multiple column segments are required, they are connected with match-cast or grouted joints. Column components can be fabricated horizontally with procedures similar to those used for precast beams and piles. In most applications, column segments have not included pretensioning because the segments have not been long enough to provide sufficient transfer and development length for the prestressing steel. Longer components have utilized pretensioning (Cruz Lesbros et al. 2003), usually to inhibit cracking during handling and transportation. Circular cross-sections, which are popular for bridge columns, can be difficult to cast horizontally, resulting in higher costs. Casting the segments horizontally can also result in a rough finish in some areas and a smooth finish in other areas, which may be aesthetically unpleasing (CTC, personal communication, February 26, 2004). Octagonal cross-sections may be easier to fabricate and should be considered as an alternative to circular cross-sections (CTC, personal communication, February 26, 2004).

Grouted joints between segments require no changes to the fabrication procedure apart from the need to include connection hardware in the ends of the segments. Multiple segments can be cast at one time on a long line bed. The number of segments that can be produced at one time is limited only by the length of the bed and the amount of available formwork. By contrast, match-cast joints require significant alterations to the fabrication system. In match casting, the new segment is cast using, as formwork, the face of a previously cast segment. This procedure requires a number of special handling maneuvers.

One proposed procedure for producing match-cast column segments is presented in Figure 6.2 (Billington et al. 1999). This vertical casting of column segments has several drawbacks. First, the process is labor intensive because it requires repetitive moving of the segments. In addition, segment heights are limited because the vertical depth of the formwork required for long column segments would become impractical.

Local precasters have suggested that better economy and greater flexibility can be achieved by fabricating the segments horizontally (CTC, personal communication, February 26, 2004), as was done for the WSDOT Bellevue Access project. This approach allows for longer segments to be cast, and if a sufficiently long bed is available, it eliminates the repetitive moving of the segments. The most economical system is likely to be one in which the segments are made as large as possible within the constraints of handling and transportation. This approach would reduce fabrication time because each casting line or machine could only produce one segment per working day (Anon 1984 and Pate 1995), and it would reduce construction time because fewer pieces would need to be placed.

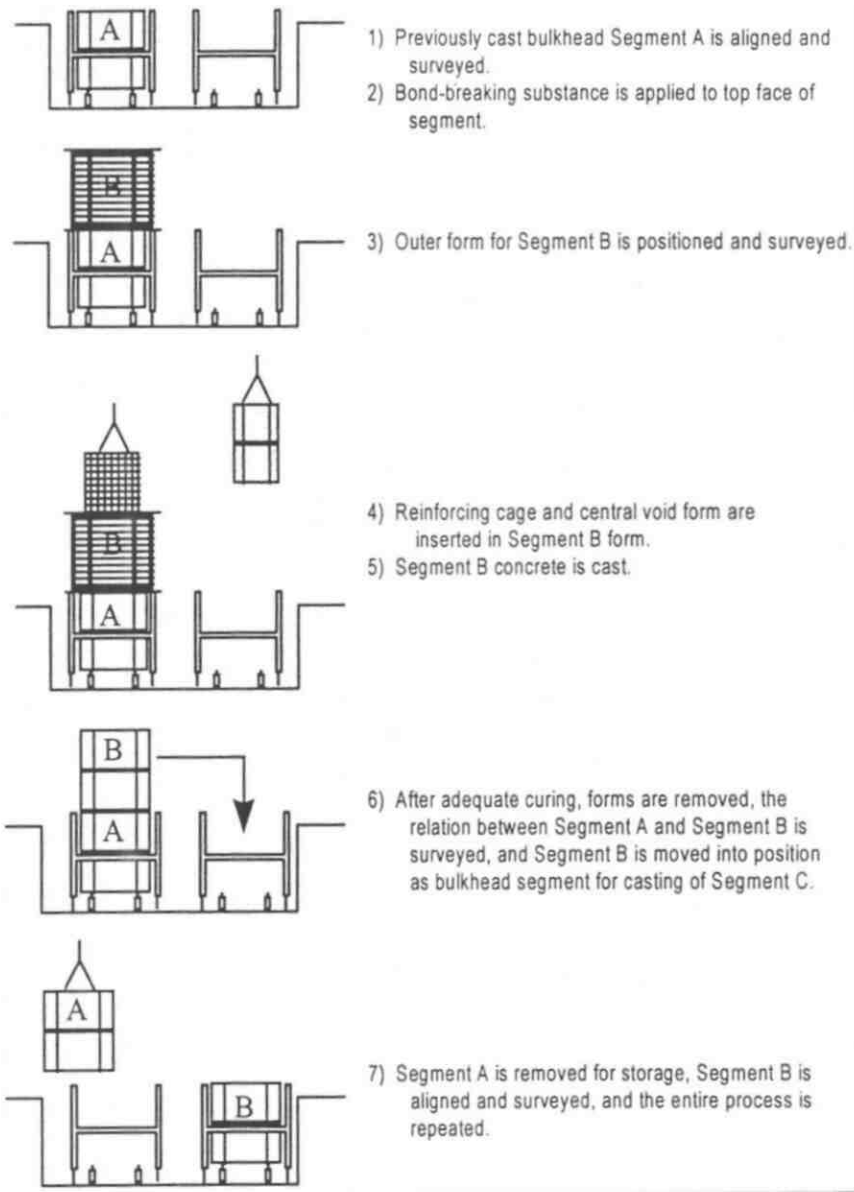


Figure 6.2: Proposed procedure for fabrication of match-cast column segments (Billington et al. 1999)

6.2.2 Pier Caps

The fabrication of pier cap-beams is similar to the fabrication of standard precast girders. Common shapes for the cap-beam include solid and hollow rectangular beams, solid and hollow box T-beams, and U-shaped beams (Lubuono et al. 1996). Cap-beams are typically pretensioned to increase their strength and improve their handling characteristics. Cap-beams are usually heavy, so measures are often required to reduce their weight. Cap-beams that contain partial voids along their length have been proposed to reduce the weight of the cap (Billington et al 1999). To fabricate the partial voided cap-beams, disposable void forms could be used, but the extra labor to create the voids would increase the cost of fabrication. Accordingly, partial voided cap-beams should only be used if they result in significant savings in other portions of the project (CTC, personal communication, February 26, 2004). It is also possible to fabricate cap-beams in multiple pieces if weight or length becomes excessive. The cap-beam segments can then be connected in the field using grouted or cast-in-place concrete joints and post-tensioning.

In some applications, the cap-beam has been fabricated with a match-cast joint between the bottom of the cap-beam and the top of column. This has been done to facilitate the alignment of the heavy pier cap during on-site construction (Billington et al. 1999 and Lester and Tadros 1995). A few methods have been proposed to fabricate the joint between the pier cap and the top column segment. One method is shown in Figure 6.3. For this method, the ledge is match-cast against the top segment of the column. After the ledge has cured to a desired strength, it is placed with similar segments in a line bed, where the stems of the cap-beams are then cast. Local precast concrete producers have suggested avoiding this type of method because it is extremely labor intensive (CTC, personal communication, February 26, 2004).

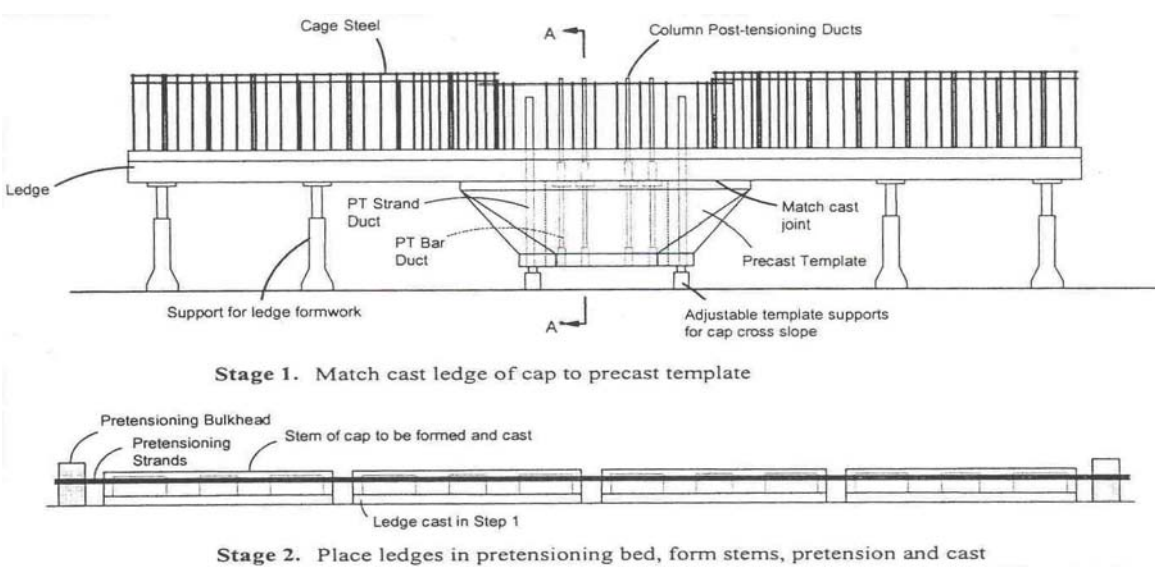


Figure 6.3: Method for creating match-cast joint between column and pier cap (Billington et al. 1999)

6.3 CONSTRUCTION

This section presents general construction procedures for precast pier systems. The specific construction procedures for an individual precast concrete pier system depend on the footing-to-column, column-to-column, and column-to-cap connection used. Specific procedures pertinent to particular connection designs are included in the *Key Issues* section.

6.3.1 Footing-to-Column Connections

Footings are constructed with the same methods as those used for cast-in-place construction. In some cases, slight variations, such as embedding ducts or additional reinforcing bars in the footing, may be required. The *Key Issue #1* section describes these variations in further detail. After the footing has been constructed and has cured, the first column segment is connected to the footing using one of the methods described in the *Key Issue #1*section.

6.3.2 Column-to-Column Connections

Columns are typically made of either a full-height segment or multiple segments that have either match-cast joints or grouted joints between them. It is recommended that precast columns be erected in one single piece when possible. This reduces the number of segments that need to be erected and eliminates unnecessary joints that increase construction time (Associated General Contractors of Washington (AGC), personal communication, April 9, 2004). Although full-height segments are preferable, transportation, erection, or other limitations may necessitate the use of multiple column segments that are connected with either match-cast or grouted connections. Designs have been proposed that use a mixture of both match-cast joints and grouted joints (Billington et al. 1999).

The following steps have been used for column segments that have match-cast joints (Billington et al. 1999).

1. The second column segment is lowered onto spacer blocks several inches above the first column segment.
2. Post-tensioning bars in the first and second segments are spliced together.
3. Epoxy is applied to the face of both column segments at the joint surface.

4. The spacer blocks are removed and the second column segment is lowered into contact with the first segment. The match-cast joint aligns the two segments properly.
5. The post-tensioning bars are stressed to produce uniform compression across the joint.

Figure 6.4 shows a column with match-cast joints under construction. Local bridge contractors prefer match-cast joints over the grouted joints (AGC, personal communication, April 9, 2004) largely because of the need to construct a leak-proof grout dam in the latter. There are concerns about whether epoxy has environmental limitations such as required moisture and temperature limits, which must be further investigated (AGC, personal communication, April 9, 2004).

Figure 6.4: Construction of a precast segmental column using match-cast joints (Pate 1995)

The following steps have been used for column segments that do not have match-cast joints.

1. The second column segment is placed onto the first and is leveled with either shims or leveling bolts that create a small gap between the two segments.
2. If post-tensioning is present, the bars between the two column segments are

spliced. This is typically done while the upper segment is hanging above the lower one.

3. The gap between the two segments is filled with grout or cast-in-place concrete.
4. After the joint material has cured to the required strength, the post-tensioning bars are stressed.
5. Some columns also use mild reinforcing steel to connect one segment to the other. In this case, mild reinforcing steel extends from one column segment and fits into splice sleeves located in the other column segment. The sleeves are filled with grout when the joint is grouted (Cruz Lesbros et al. 2003).
6. Epoxy may also be used to seal the joint (Muller and Barker 1985).

Local bridge contractors have expressed an unfavorable attitude towards grouted joints (AGC, personal communication, April 9, 2004). Each joint would require formwork before the grout could be placed, which would increase labor as well as the time to place each segment. The length of time required for the grout to cure in the field would also increase the time between placement of segments (AGC, personal communication, April 9, 2004). These two items would significantly reduce the rapid construction advantage of precast columns over cast-in-place columns (AGC, personal communication, April 9, 2004). One suggestion to improve the grouted joint is to include "feet" on the bottom of each column segment so that the required number of leveling bolts and/or shims could be reduced (AGC, personal communication, April 9, 2004).

6.3.3 Column to Cap-Beam Connection

After the final column segment has been placed, the cap-beam can be attached to the column. Connections used in previous applications have generally fallen into one of the following categories: grouted duct or post-tensioned**.** The column to cap-beam connection is similar to the column-to-column connections described above. For a single column pier, a one-segment pier cap is typically used, provided the cap does not exceed weight or length limits for transportation and erection.

For multi-column piers, either single or multiple segment pier caps may be used depending on erection and alignment concerns. If multiple segments are used, cast-in-place concrete joints with spliced reinforcing bars are typically used to connect the sections. When match-cast joints or doweled reinforcing bars are used between the cap-beam and the column, alignment problems may occur, especially for multi-column piers. The columns must be properly positioned and aligned with respect to one another to ensure that the cap beam will fit correctly (CTC, personal communication, February 26, 2004 and AGC, personal communication, April 9, 2004). Figure 6.5 shows a proposed erection procedure for constructing a multi-column pier. Figure 6.6 shows the placement of a precast multi-column cap-beam onto cast-in-place concrete columns.

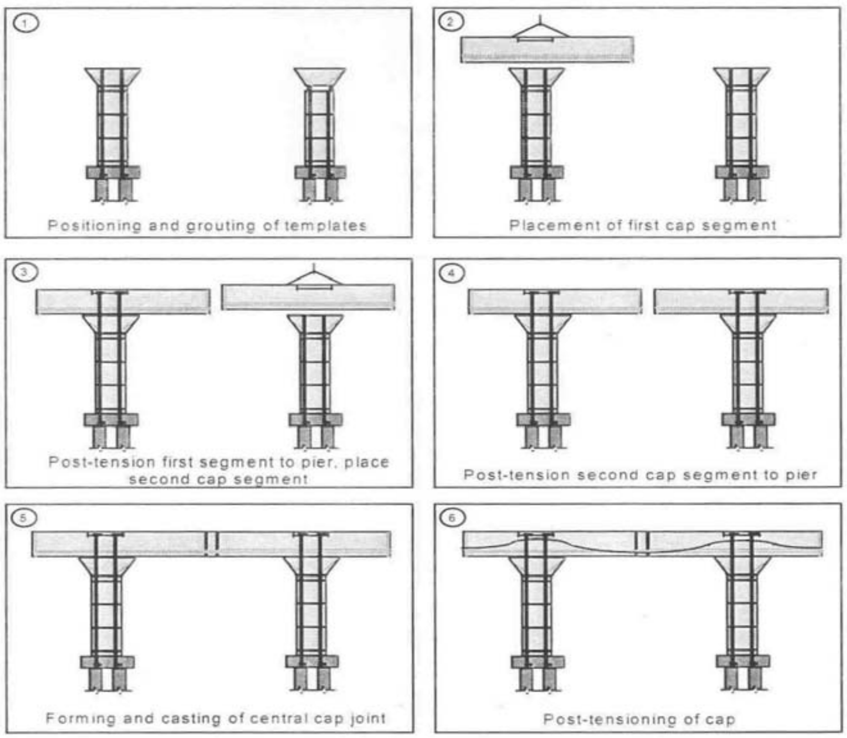


Figure 6.5: Erection sequence for a multi-column pier (Billington et al. 1999)

Figure 6.6: Erection of a precast multi-column pier cap-beam (FHWA 2004)

6.4 SUMMARY OF USE

Precast concrete piers have been used less frequently than precast superstructure systems. In the majority of applications, only one of the components, either the columns or cap beam, was precast and the other was cast-in-place with conventional methods. Precast piers have been used primarily for large-scale bridge projects, often spanning large waterways. The vast majority of previous applications and research have been for non-seismic regions. Billington et al. (1999) developed a standardized precast pier system for non-seismic areas to be used by the Texas Department of Transportation. The American Segmental Bridge Institute is considering standards for precast substructure elements (Billington el al. 2001).

The following is a partial list of large-scale bridge projects utilizing precast pier components.

- Chesapeake and Delaware Canal Bridge; St. Georges, Delaware (Pate 1995)
- Dauphin Island Bridge; Mobile, Alabama (Anon 1984)
- Linn Cove Viaduct; Grandfather Mountain, North Carolina (Muller and Barker 1985)

- Bahrain Causeway; Connection Bahrain and Saudi Arabia (Ingerslev 1989)
- Ayuntamiento 2000 Bridge; Cuernavaca, Mexico (Cruz Lesbros et al. 2003)
- Redfish Bay in Texas; Port Aransas, Texas (Medlock et al. 2002)
- Vail Pass; Colorado
- Northumberland Strait Crossing, Connecting Prince Edward Island and New Brunswick, Canada (Lester and Tadros 1995)
- Seabreeze Bridge; Daytona Beach, Florida (Billington et al. 2001)
- Edison Bridge over Caloosahatchee River; Fort Meyers, Florida (Billington et al. 2001)

Use of precast piers for short- and moderate-span bridges has been limited but is increasing in popularity (Billington et al. 2001). The following is a partial list of short- and moderate-span bridge projects that have used precast pier components.

- U.S. Hwy 183 ; Austin, Texas (Billington et al. 1999)
- U.S. Hwy 249 over Louetta Road; Houston, Texas (Billington et al. 1999)
- Pierce Elevated section of Interstate Hwy. 45; Houston, Texas (Jones and Vogel 2001)
- Lake Ray Hubbard Project; Lake Ray Hubbard, Texas (Medlock et al. 2002)
- Bellevue Direct Access- N.E. 4th St.; Bellevue, Washington (ACC, personal communication, March 24, 2004)

Precast concrete segmental substructures have also been used in Japan since the 1960s (Higuchi et al. 1968 and Takano et al. 1968).

6.5 DURABILITY EVALUATION

No mention of the performance of precast substructures has been found in the published literature. Most of the bridges constructed with precast substructures are relatively new, and long-term durability issues have yet to emerge.

6.6 KEY ISSUES

6.6.1 Key Issue #1: Connection Between Footing and Column Segments

The footing-to-column connection presents both challenges and opportunities because it involves the connection of a precast and a cast-in-place component. Several footing-to-column connections used in previous applications are presented below.

One type of footing-to-column connection consists of temporarily supporting the column in place before the footing is constructed and then pouring the footing concrete around the reinforcing steel extending out of the bottom of the column. To construct this connection, the column is supported by temporary support legs extending from the bottom of the column and bears on a leveling pad. Reinforcing bars extend downwards from the bottom of the column segment into the footing. The reinforcing steel for the footing is then placed and the footing concrete is poured. In some cases, rather than using leveling pads, the footing is poured in two layers, with the base of the column incorporated into the second pour (Muller and Barker 1985, Anon 1984, and Cruz Lesbros et al. 2003). Figure 6.7 shows this type of connection before the second layer of the cast-in-place footing has been placed. While it eliminates the need to cast a leveling pad, it introduces the need to lap-splice the column bars. The splice length may prove to be a critical design element.

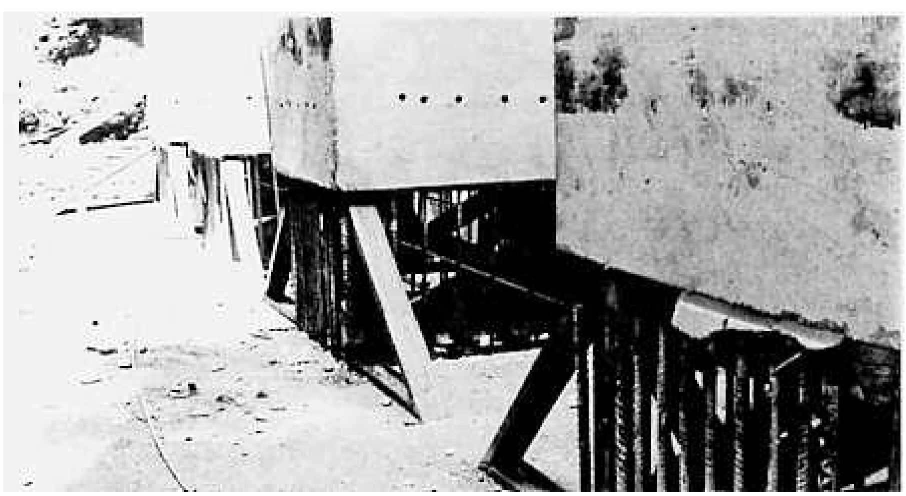
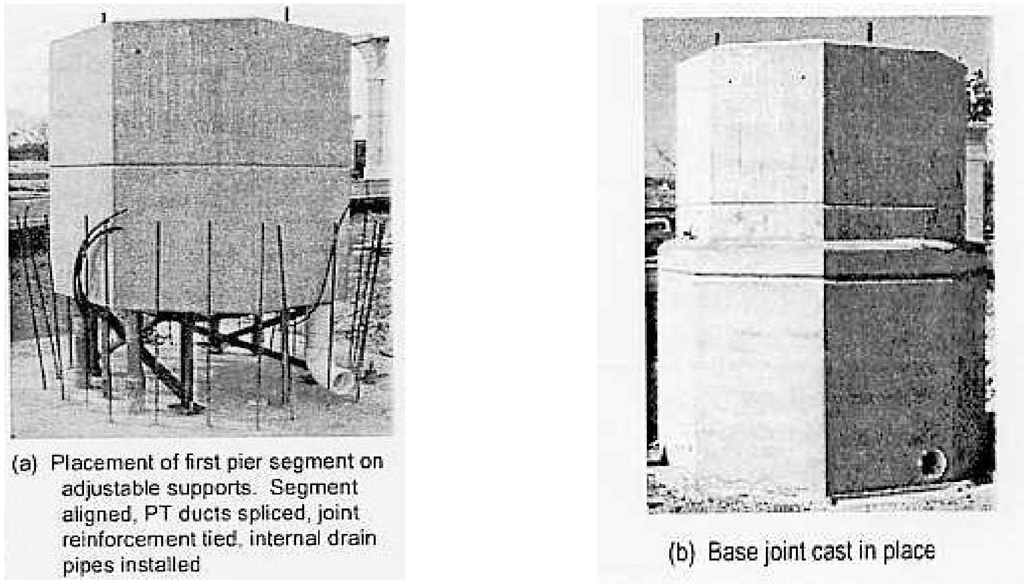


Figure 6.7: Cast-in-place connection of column component to footing (Cruz Lesbros et al. 2003)

This type of connection has been used successfully in Western Washington and resulted in significant construction time savings (ACC, personal communication, March 24, 2004). The structural design of this connection is the same as the design for a traditional cast-in-place footing and column. Congestion of reinforcing bars in the footing is typically not a problem. Although temporary bracing is required to support and align the columns before the footing is poured, similar bracing is required for the formwork of cast-in-place columns, resulting in no loss of the rapid construction benefits that precast columns have over cast-in-place columns (AGC, personal communication, April 9, 2004). Surveying is also required on both precast columns and the formwork for cast-in-place columns to assure that they are properly aligned (AGC, personal communication, April 9, 2004).

A potential issue that may arise when this connection is used is that a large column resting on a leveling pad supported by soft soils could cause excessive settlement of the leveling pad resulting in incorrect vertical alignment of the columns (AGC, personal communication, April 9, 2004). However, casting the footing in two layers, as described above, might provide a solution to this problem.

This connection requires that the columns be fabricated, brought to the site, and temporarily supported before the footing concrete can be placed. In most cases this allows the schedule to be compressed because the columns can be fabricated in a plant while the site is being prepared. However, if the situation requires simultaneous fabrication of the columns and the placement of footings, a cast-in-place concrete collar type connection, in which reinforcing bars sticking out of the footing are lap spliced with bars extending downwards from the column, should be used. This collar connection is shown in Figure 6.8.



(a) Placement of first pier segment on adjustable supports. Segment aligned, PT ducts spliced, joint reinforcement tied, internal drain pipes installed

(b) Base joint cast in place

Figure 6.8: Collar type connection of precast column and CIP footing (Billington et al. 1999)

A second type of footing-to-column connection consists of a small grouted joint between the bottom column segment and footing, secured with vertical post-tensioning. To construct this joint, the column segment is placed on the completed footing and aligned with shims or leveling bolts. Post-tensioning bars are inserted through the column segment into anchors in the cast-in-place concrete footing. The joint is filled with grout and allowed to cure. The post-tensioning bars are stressed and the ducts are grouted (Billington et al. 1999). Figure 6.9 shows a sketch of this connection. The primary difficulty with constructing this connection is properly aligning the post-tensioning ducts in the footing and column. Accomplishing this requires careful detailing and the use of identical templates for constructing the cast-in-place footing and fabricating the precast column (AGC, personal communication, April 9, 2004; ACC, personal communication, March 24, 2004; CTC, personal communication, February 26, 2004 and Josten et al. 1995).

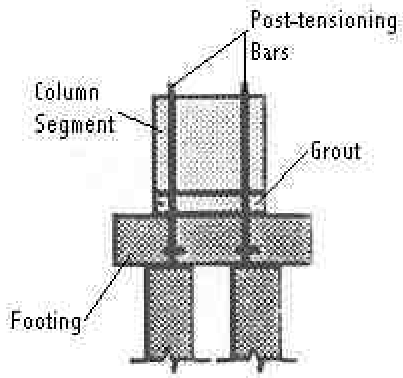


Figure 6.9: Grouted column-to-footing connection (Billington et al. 1999)

6.6.2 Key Issue #2: Connection between Column Segments

Match-cast joints have been used in most of the previous applications (Pate 1995, Anon 1984, and Muller and Barker 1985). Grouted joints may lack a uniform bearing surface and are more likely to experience edge crushing. Loose-fit joints must also be carefully aligned and held in place while the grout is placed. Poor grout placement or quality can lead to partially filled joints, stress concentrations, cracking, and corrosion of reinforcing steel. These drawbacks support the use of match-cast joints (Billington et al. 1999).

The literature provides little guidance on the design of these connections. One important consideration is the shear capacity of the joint region. Although shear friction alone should be adequate to carry the shear demand, shear keys can be included to provide further shear capacity. The epoxy joint sealer, used between column segments, may also add to the shear strength of the connection; however, it appears that this source of strength has not been taken into account in previous designs (Muller and Barker 1985). Figure 6.10 shows a typical connection surface. Another potential problem that may develop between adjacent column segments is the distance between transverse reinforcement. If clear cover is provided for the transverse reinforcement at the top and bottom of each segment, there will be a distance in the column where transverse reinforcement is spaced at a larger distance than elsewhere in the column, which could create a weakness in the column's shear capacity. This distance will be even larger if grouted joints are used (AGC, personal communication, April 9, 2004). When multiple column segments are used, the post-tensioning carries flexural demands across the joint. Another typical design requirement is that sufficient prestressing be applied so that joints do not open under service loads (Billington et al. 1999). This creates a potential problem in seismic applications. If the prestressing required to prevent the joint from opening under service loads becomes large, it could significantly reduce the ductility of the column because of the high axial stresses it induces. Large initial tendon stresses also limit the tendons reserve strain capacity, which could result in premature yielding. This problem can be solved by using a larger area of strand at lower stress if space is available in the column and connections. This will be discussed further in the section on *Seismic Considerations*.

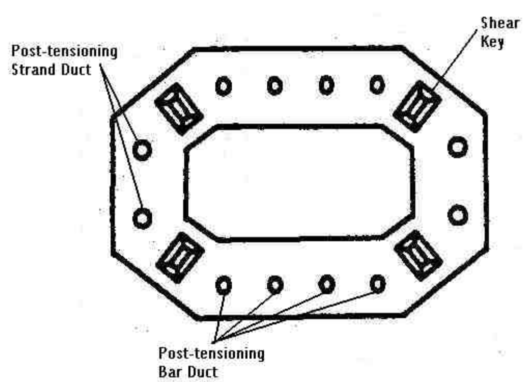


Figure 6.10: Typical segmental column connection surface (Billington et al. 1999)

<u>*6.6.3 Key Issue #3: Connection of Column and Cap-Beam*</u>

Two types of connection have been used to connect a precast cap beam to precast or cast-in-place columns (LoBuono et al. 1996). The first uses post-tensioning bars or strands (Billington et al. 1999). This connection is very similar to the connection between two column segments. In brief, the match-cast joint between the top column segment and the pier cap is coated with epoxy, post-tensioning bars are spliced, and the pier cap is lowered into place. A schematic of this connection is shown in Figure 6.11. In non-seismic applications, this connection may be adequate, but the lack of mild steel allows for little energy dissipation during a seismic event (Kwan and Billington 2003a).

Billington et al. (1999) suggested that the top column segment should be flared to reduce the design moments for the cap beam. A flare section would allow for a smaller and lighter cap to be used, but there are two concerns when doing this. The first is that by reducing the size of the cap, serviceability problems such as excessive cracking may arise (Young et al. 2002). Horizontal prestressing in the cap beam should help to reduce this possibility. The second potential problem with using the flared section is that it reduces the effective length of the column. In a seismic event, this will increase the shear demand on the column (Yashinsky and Karshenas 2003).

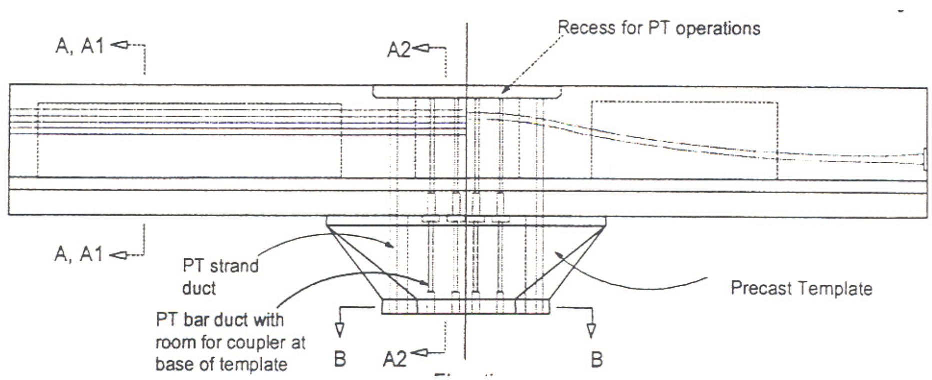


Figure 6.11: Post-tensioned connection between template segment and cap-beam (Billington et al. 1999a)

The second type of column-to-cap connection uses mild steel reinforcing bars in a grouted duct connection (Matsumoto et al. 2002, Wolf and Friedman 1994, and Mandawe et al. 2002). The connection is constructed as follows. The cap-beam is placed on the top column segment with shims or leveling bolts used to produce a gap and align the cap-beam. The cap-beam is fabricated with several full-depth ducts that receive reinforcing bars extending from the top column segment. The joint between the column and cap and the ducts in the pier cap are then filled with grout (Matsumoto et al. 2002). A schematic of this type of connection is shown in Figure 6.12.

In seismic regions the column-to-cap connection region for cast-in-place construction is extremely congested. This may limit the space to locate ducts in the cap beam. Local precast concrete producers have stated that it would be very difficult to emulate a cast-in-place connection using these grouted ducts because the ducts take up approximately twice as much room as reinforcing bars (CTC, personal communication, February 26, 2004). Construction tolerances would need to be tight to ensure that the bars fit into the corresponding ducts. Local bridge contractors agree that, although the tight tolerances are undesirable, it would be possible to make the connection work with the use of templates (AGC, personal communication, April 9, 2004).

During the construction of the Getty Center Tram Guideway in Southern California, precast cap-beams were placed onto cast-in-place columns by aligning 16 1.41-inch diameter bars into corresponding 1.5-inch diameter ducts, thereby demonstrating the capability to connect segments when templates are used (Josten et al., 1995). For multi-column, single cap-beam piers, accurate alignment is even more critical. Not only must the reinforcing bars for each column line up with the holes in the corresponding location in the cap beam, but the columns must also be positioned exactly relative to each other (AGC, personal communication, April 9, 2004). The large depth of the pier cap should provide sufficient development length for the bars. If not, headed bars can be used.

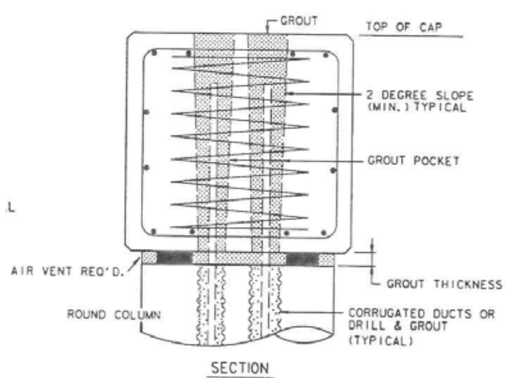


Figure 6.12: Mild steel connection between column and pier cap (Matsumoto et al. 2002)

6.6.4 Key Issue #4: Connection between Cap-Beam Segments

Long multi-column pier cap-beams often require that the cap-beam be fabricated and transported in multiple segments. The cap-beam segments may be connected to each other with grouted, cast-in-place concrete or match-cast connections. If match-cast connections are to be used, in order for the segments to align properly, it is important to correctly account for creep, shrinkage, and elastic deflection because of the cap-beam's own weight (CTC, personal communication, February 26, 2004). Alignment problems may also arise when match-cast joints are used or when the segments are connected before they are lifted into place. Weight may also become a problem when segments are connected before they are lifted into place. Although these options introduce the need for tight tolerances, they do eliminate the need for falsework.

Cap-beams may be lifted in several pieces and connected to each other with grouted or cast-in-place concrete joints. The connections are typically located at mid-span. This location is not ideal because under gravity loading, the maximum positive moment occurs near mid-span. This increases the number of reinforcing bars that need to be spliced and the required splice length (Billington et al. 1999). Ideally, the location of the connections would be at the points of contraflexure where gravity induced moments would be lowest. This can result in significant construction loads on a column before the pier cap segments are connected together. Using grouted or cast-in-place concrete joints allows for larger tolerances but requires the use of formwork and temporary supports. There are also architectural concerns when grout or cast-in-place concrete is used in the joints. Grouted joint leaking may also stain the precast concrete segments, and the cast-in-place concrete used in joints may be a different color than the precast concrete segments. This can be remedied by using pigmented sealer over the concrete surface.

6.6.5 Key Issue #5: Weight and Size Limitations

Column segments for single column piers and long cap-beams may become excessively heavy or long and so can present transportation and erection challenges. WSDOT weight limits do not apply to precast columns, but guidelines on component weight limits have been suggested. The suggested limits vary from 120,000 lbs to 180,000 lbs (Billington et al. 1999 and WSDOT BSO, personal communication, January 29, 2004). On the basis of these weights, Table 6.1 provides the maximum component length for various cross sections. Hollow section weights are calculated assuming a 1-ft. wall thickness.

Table 6.1: Maximum column length limitations

Cross Section	Weight Limit		Cross Section	Weight Limit	
	120 k	180 k		120 k	180 k
Hollow Round			**Solid Round**		
3 ft diameter	127 ft	190 ft	3 ft diameter	113 ft	170 ft
4 ft diameter	84 ft	127 ft	4 ft diameter	63 ft	95 ft
5 ft diameter	63 ft	95 ft	5 ft diameter	40 ft	61 ft
6 ft diameter	50 ft	76 ft	6 ft diameter	28 ft	42 ft
Hollow Square			**Solid Square**		
3 ft x 3 ft	100 ft	150 ft	3 ft x 3 ft	88 ft	133 ft
4 ft x 4 ft	66 ft	100 ft	4 ft x 4 ft	50 ft	75 ft
5 ft x 5 ft	50 ft	75 ft	5 ft x 5 ft	32 ft	48 ft
6 ft x 6 ft	40 ft	60 ft	6 ft x 6 ft	22 ft	33 ft

6.7 SEISMIC EVALUATION

Although the majority of the previous precast pier applications have been in non-seismic regions, some of the concepts used in non-seismic areas can likely be employed in seismic areas with modifications made to the connection details. The main concern with precast piers in seismic areas is a lack of continuity between components. Precast pier systems in seismic areas will likely require more reinforcement between components and greater development lengths for this reinforcement than similar systems in non-seismic areas. Additional reinforcement and increased development lengths may prove problematic because of geometric constraints and congestion of pier reinforcement.

Currently some research is being performed to examine the feasibility of using precast pier systems in seismic regions (Kwan and Billington 2003a, Kwan and Billington 2003b, Mandawe et al. 2002, Sritharan et al. 1999, and Yoon 2002). This research has focused primarily on the design aspects of precast pier systems in seismic areas, with little discussion of changes in construction or fabrication procedures required for seismic areas. One area of research currently being studied is the use of prestressing to connect components in precast pier systems. Including prestressing in precast pier systems induces compressive stresses in the components, potentially limiting the amount of damage experienced during an earthquake. Sritharan et al. (1999) showed longitudinal post-tensioning of the cap-beam to be very effective in reducing the amount of damage on cap-to-column joints under seismic loadings. Vertical post-tensioning of cast-in-place columns has also been shown to be effective in reducing damage and improving performance (Ikeda 1998). Prestressing increases the cracked stiffness of a column, which reduces deflections.

Another advantage of including prestressing in columns is that it provides a restoring force that reduces residual deformations after an earthquake. For this to occur, the prestressing tendons must remain elastic during the earthquake. Unbonded post-tensioning is commonly used to prevent yielding because deformation in the tendons can be distributed over the entire tendon length. Columns reinforced with unbonded post-tensioning tendons should provide elastic restoring forces and have small residual deformations. However, such columns display little energy dissipation under cyclic loading because of the lack of yielding of mild steel reinforcement (Kwan and Billington 2003a and 2003b). The effectiveness of post-tensioning may be limited by problems with concrete crushing. Concrete crushing causes shortening of the column, resulting in loss of prestressing and increasing the residual displacements of the columns. Higher compressive stresses in the pier columns caused by the post-tensioning can cause early failure by crushing the concrete and, therefore, justify the use of higher strength concrete (Kwan and Billington 2003). This problem can be reduced by providing good confinement detailing at the critical locations in the column.

Reinforcing columns with a combination of unbonded prestressing tendons and mild steel appears to be a promising way to reduce residual deformations while still achieving some energy dissipation (Kwan and Billington 2003a and 2003b). It is possible to adjust the proportions of the prestressed and non-prestressed reinforcement in the column to obtain the desired seismic performance (Stanton et al. 1997). Kwan and Billington (2003a) presented finite element analysis models showing the impact that different combinations of post-tensioning and mild reinforcement have on column performance. Including prestressing in design also allows the amount of transverse reinforcement in the cap-to-column connection to be reduced (Sritharan et al. 1999). This reduces reinforcement congestion, improving the constructability of precast pier systems.

Segmental columns also allow different materials to be used for different segments of the column. Yoon et al. (2002) examined the possibility of using segments of engineered cementitous composites (ECC) in the locations of the column where plastic hinges are likely to form. The ECC distributed cracking in the plastic hinge region resulting in many fine cracks rather than one big crack. It was determined that the columns incorporating ECC had better energy dissipation at low drift levels and significantly less damage than standard reinforced concrete columns. The improved energy dissipation compared to conventional columns diminished at large displacements. The columns incorporating ECC had larger residual displacements because misalignment prevented the cracks from closing completely (Yoon 2002). It appears that many other alterations can be made to the system to improve its seismic performance. Applying connection concepts from the PRESSS initiative and other building research may be very beneficial (Priestley 1991).

6.8 SYSTEM EVALUATION AND RECOMMENDATIONS

Fabricating pier components off-site produces several advantages. Because of high levels of congestion, placing reinforcing steel for piers can be time consuming. Using precast components allows this work to be done off site, resulting in rapid on-site construction. Standard sections must be developed to allow precast components to become economical. Standard sections allow precast concrete producers to invest in reusable formwork that lead to an economical, high quality product. The complexity of fabrication will depend on the type of component. Some components, such as square columns and rectangular cap-beams, would be easier to fabricate. Circular columns and T-shaped connection regions would be more difficult and expensive. A significant percentage of structurally deficient bridges in the United States are so categorized because of the deterioration of their piers and other substructure components. This makes the improved durability of precast concrete over cast-in-place concrete a significant advantage for use in bridge piers

Transportation and erection limits may cause problems for precast pier components, especially longer and heavier column segments used in single-column piers.

Precast pier components are typically shorter than prestressed concrete girders, decreasing their chance of exceeding transportation length restrictions. However, weight may pose a transportation problem because pier components are typically stockier than prestressed girders, preventing distribution of load over a greater number of axles during transportation. Along with transportation problems, it is important to consider erection weight limits as well. For many single column piers, columns may need to be divided into multiple pieces in order to meet the weight constraints.

If properly developed, a precast pier system could be extremely versatile. A precast pier system must be capable of being adapted to many different configurations, including single column piers and multiple column piers, as well as several different types of foundations (spread footings, pile footings, and drilled shaft footings).

Precast piers should be designed with as few components as possible. Limiting the number of components reduces fabrication time. Piers with fewer components would also require fewer connections, reducing on-site construction time. Ideally, each column and cap beam would be fabricated as one piece. In situations where it is not possible to cast columns in one piece because of transportation or erection constraints, multiple column segments could be used and connected with match-cast joints. local bridge contractors felt that if grouted joints were used to connect segments, little time savings over cast-in-place concrete columns could be achieved (AGC, personal communication, April 9, 2004). Match-cast connections would allow much more rapid construction in the field because of the fast setting epoxy and self-aligning properties of the match-cast joint. Although match-casting would slow fabrication time because only one segment per column could be produced per day, the number of segments for a given column would be small, which would limit the number of days required to fabricate the columns. Match-cast joints also require that each component be placed in the field with the component with which it was match-cast. In a standardized precast substructure system, it would be preferable to have interchangeable components. This would require diligence from the fabricator and contractor and might not be possible with a match-cast system.

For the following reasons, columns with solid cross-sections are preferable unless weight limits dictate the use of hollow sections or significant cost savings can be achieved.

- Hollow sections are not recommended for locations of the column where plastic hinges can form because inward spalling can occur, regardless of confinement.
- Local wall buckling must be considered for hollow cross-sections and can present a problem (Taylor et al. 1995).
- In solid sections all of the post-tensioning is placed in the center of the column.
- Solid sections have better impact resistance.
- Inspecting the inside of hollow cross-sections is not feasible without damaging the column, preventing proper inspection of hollow columns after seismic events.

In many situations it may be most beneficial to use a pier design that includes both precast and cast-in-place concrete components. Scheduling constraints vary significantly for every project. Accordingly, the type and extent of rapid construction measures required for a given project can vary significantly. For example, some projects may require the entire construction process to be completed in the least number of days, whereas others require only that all construction be done at night so as not to disrupt nearby traffic. Using cast-in-place concrete for some components relaxes construction tolerances, making the piers easier to build. Therefore, the mix of cast-in-place and precast construction that meets the particular project objectives should be used.

A rapid construction mindset should still be used in approaching the cast-in-place components. Through the use of prefabricated reinforcing steel cages, customized formwork, and high early strength cement, the amount of time required for construction can still be significantly reduced.

Previous research suggests that the seismic performance of precast pier systems can be greatly improved by including prestressing in the design, particularly if it is unbonded. Including prestressing will likely reduce damage and prevent residual deformation. Little research has been conducted to validate this and more is required. Precautions must be taken to protect the tendons from corrosion.

CHAPTER 7:
CONCLUSIONS

7.1 SUMMARY

This report provides a summary of precast concrete construction used for rapid construction of bridges. The descriptions of four prominent superstructure systems (full-depth precast concrete panels, partial-depth precast concrete panels, prestressed multibeam concrete superstructures, and pre-constructed composite units) and substructure systems are presented. In recent years the use of precast concrete components has increased in many parts of the United State and around the world, but in other locations, precast components have seen limited use.

In many applications, the use of precast concrete components significantly decreased the construction time required for the project. The largest benefits have been seen in areas where precast concrete systems have been used repeatedly. In these cases, the contractor's familiarity with the system led to significant reductions in construction time and improvements in overall economy. The use of precast concrete components has been shown to provide rapid construction, decrease environmental impacts, increase durability, and reduce on-site labor, resulting in better work zone safety.

The use of precast concrete bridge superstructure components began in the 1960s, with a majority of the projects performed in the last 10 years. Many of the initial problems from the early applications of precast concrete components have been addressed. However, because a majority of bridges constructed with precast components are relatively young, there is a potential for additional problems to arise. Versatile systems need to be developed further to include a larger variety of bridge types, locations, and construction schedules that can be constructed with precast concrete components. A majority of applications have been in non-seismic areas. Further research is required to develop precast concrete systems that will perform adequately in seismic regions.

7.2 PRECAST CONCRETE SYSTEM ATTRIBUTES FOR USE IN WASHINGTON STATE

Given the summaries provided in this report, the following attributes and requirements are important for a precast concrete system to be used in Washington state.

Note that many of the attributes listed below are general. Therefore, additional requirements may arise on a project-by-project basis.

7.2.1 General Considerations

- Components should conform to weight and length restrictions based on transportation and erection constraints. It is important to consider highway transportation limits because many bridge sites may only be accessible by highway. Sites near waterways, which can accommodate barging, generally have fewer constraints.
- Significant reduction or complete elimination of cast-in-place concrete should be considered for a system. In cases where cast-in-place concrete is still required, the number of pours required should be reduced if possible, and high early strength concrete should be considered, provided that heat buildup is not a problem.
- A system should be designed to require significantly less formwork than needed for the cast-in-place concrete counterpart. Less formwork saves time associated with placement and removal.
- The connection between precast components is critical to the performance of a precast system. The connections must be well designed, detailed, and constructed, and they should have a documented performance history from testing or previous use.
- Precast concrete systems should use the smallest possible number of connections, and they should be protected from the environment.
- A system should be flexible and adaptable to a variety of bridge geometries, bridge configurations, different locations with varying staging and construction areas, and different construction schedules.
- Ultimately, the system should become standardized. Standardization will allow repeated use, reducing fabrication costs through reusable formwork. Contractors will also become more familiar with the system with each reuse, resulting in reduced construction costs and higher quality products. Standardization will also

encourage investment in special templates and jigs to ensure accurate alignment on site.

- A system must also be economical. Initially, a precast concrete system may cost more than a cast-in-place system, but over multiple uses a system should be economical. Costs of a system should be kept down so that the increase in cost does not outweigh the rapid construction benefits. When a system is evaluated, it is important to consider not only the financial costs but also all the benefits of a precast system, including reduced construction time, decreased environmental impacts, less traffic delays for the public, and improved work zone safety.

7.2.2 Superstructure

- A precast superstructure system should be able to accommodate a variety of bridge geometries.
- Systems should be compatible with the types of structural components commonly used in Washington state, such as prestressed concrete girders, tub girders, and steel girders.
- It is important to provide a smooth riding surface. Systems without an inherent smooth riding surface will require an overlay.
- If possible, a system will allow all construction to be performed from above the bridge deck. This eliminates the need for a sub-deck and improves work zone safety.
- Connections between components are critical, and their durability should be carefully examined.
- The system must be able to account for the difference between the girder profile and the bridge profile.
- When applicable, leveling bolts should be used for the alignment and positioning of components rather than shims.
- When the appropriate system is selected, WSDOT experience with precast superstructure systems should also be considered.

7.2.3 Substructure

- Precast substructure systems for Western Washington should have acceptable seismic performance. This may result in significant changes to systems previously developed for use in non-seismic regions.
- Systems should consist of the smallest possible number of segments. This improves system durability and results in quicker construction.
- The connections between components should have acceptable tolerances to allow for easy on-site construction.
- Unbonded post-tensioning appears to be promising for a precast substructure system and should be explored further. The research will need to consider concerns about corrosion of the unbonded post-tensioning reinforcement.
- Systems should be able to accommodate a variety of foundations, including spread footings and drilled shafts.
- Substructure systems should be compatible with a variety of superstructure systems and components, as well as the different types of connections required between substructure and superstructure components.
- Match-cast epoxied joints require less on-site construction time and so are preferred over grouted joints. Cost-effective fabrication methods for match-cast components should be developed.

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Appendix A: Summary of Previous Research

This appendix provides tables summarizing previous research for each system.

Author	Summary of Research
AASHTO 1998	AASHTO LRFD Design Specification
AASHTO TIG 2004	(www.aashtotig.org) Provides general information on prefabricated bridge elements and systems.
AGC 2004	Meeting with the AGC/WSDOT Structures Team (April 2004)
Anon 1984	Outlines the design and construction of the Dauphin Island Bridge near Cedar Point, AL. The bridge utilized precast box segments for the columns.
Atkinson 2004	Meeting with officials of Atkinson Construction and tour of the Bellevue Access project in Bellevue, WA. (March 2004)
Badie et al. 1998	Discusses the NUDECK system, which is a unique partial-depth panel system which the authors developed.
Badie et al. 1999	Developed a new beams cross section to reduce cracking in the longitudinal joints. The trapezoidal section is similar to a box girder with an extended top flange.
Billington et al. 1999	Presents research that led to the creation of an initial standardized systems for precast substructures. Includes discussion of the design, fabrication, and construction of bridge piers as well as the connections between different components.
Billington et al. 2001	Paper discusses the development of attractive and rapidly constructed substructure systems. Makes recommendations for standardization of precast substructure systems.
Biswas et al. 1984	Tested a one-third scale model of full-depth precast panels on steel stringers. Examined the effectiveness of shear pocket connectors in creating composite action as well as fatigue and ultimate behavior
Biswas 1986	Presents a summary on the use of full depth precast deck slabs including several field observations

Cruz Lesbros et al. 2002	Presents the design and construction of the Ayuntamiento 200 Bridge in Morelos, Mexico. The bridge used precast columns and cap beams. Cast-in-place concrete connections were used between the precast components.
Csagoly and Nickas 1987	Described and tested a double tee section at the Florida Department of Transportation. Tested the ultimate strength of the section as well as the ultimate and fatigue strength of V-joints between beams.
CTC 2004	Meeting with engineers with Concrete Technology Corporation, a precast concrete producer located in Tacoma, Washington. (February 2004)
Culmo 1991	Discussed previous research on full-depth panels and the application of this research towards developing a full-depth panel system for use in Connecticut.
Culmo 2002	Presents the summary of a literature search on full depth deck applications along with a description of the development and implementation of a deck system based on the information uncovered in the literature search.
Dunker and Rabbat 1992	Results of a study of the National Bridge Inventory examining the durability of different types of bridges.
El-Remaily et al. 1996	Proposed an improved detail for connection box girders consisting of deeper grouted keys and longitudinal post-tensioning at intermediate diaphragms.
Fagundo et al. 1985	Discusses tests which were performed in order to determine in what way partial-depth panels acted, either continuously or simply supported at the girders.
Goldberg 1987	Presents guidelines for the design, manufacture and erection of precast prestressed concrete partial-depth panels
Gulyas 1996	Provides comments on Nottingham's article and provides further material information.
Higuchi et al. 1968	Introduces some connection designs for precast concrete components used by the Japanese National Railways. Includes mention of segmental column systems.
Hill et al. 1988	Described the design, construction, and performance of a deck bulb tee multi-beam bridge. Special illustration of the use of transverse post-tensioning was provided.

Ikeda 1998	Examined the effect of vertical bonded prestressing on the seismic behavior of reinforced concrete bridge columns.
Ingerslev 1989	Describes the design and construction of the Bahrain Causeway in the Arabian Gulf. The bridge used precast concrete piles for columns and precast concrete pier caps.
Ingersoll et al. 2003	Study of the durability of multibeam bridges in Iowa composed of channel shaped sections.
Issa et al. 1995 a	Provides construction procedures for the replacement of bridge decks using full-depth deck panels.
Issa et al. 1995 b	Provides field observations for approximately 30 bridges in 12 states, including critical design details and how they affected the deck performance
Issa et al. 1998	Describes the finite element analysis of two existing bridges with the goal to determine the amount of post-tensioning required between adjacent panels to keep the transverse joints in compression.
Issa et al. 2000	Examined the effect of various levels of longitudinal post-tensioning on the performance of transverse joints between slabs. Tested three steel girders with full-depth precast panels for service, fatigue, and ultimate behavior.
Issa et al. 2003	Examined the performance of several grout materials for use in the shear keys connecting adjacent panels.
Jones and Vogel 2001	Reports on the used of precast concrete pier caps in retrofit and new bridge construction in Texas.
Josten et al. 1995	Presents the design and construction of the Getty Museum People Mover in Los Angeles, CA which incorporated precast cap beams in design.
Klingner 1988	Discusses tests performed to determine the effect strand extensions had on bridge deck performance.
Kropp et al. 1975	Describes the design construction and performance of two bridges constructed in Indiana using full depth precast panels.
Kwan and Billington 2003a	Outlines a finite element analysis of the monotonic and cyclic behavior of precast concrete bridge piers with varying amounts of prestressing and mild reinforcement.

Kwan and Billington 2003b	The companion paper to the above reference outlining the expected response of the precast concrete piers subjected to a variety of ground motions.
Lester and Tadros 1995	Presents the design and construction of the Northumberland Strait Crossing between New Brunswick and Prince Edward Island, Canada. Template sections with match-cast connections were used to ease alignment of massive concrete box girders.
LoBuono et al. 1996	A report to the Florida DOT outlining the most promising shapes to use for precast bridge substructures. Construction, fabrication, and design concerns were all considered. A brief mention was given to the connections of the components.
Mandawe et al. 2002	Examined the cyclic performance of reinforcing bars grouted in corrugated ducts.
Matsui et al. 1994	Tested both individual precast panels and a specimen consisting of the precast panels on steel stringers for service, fatigue, and ultimate behavior. Also discussed the installation of the panels on a highway bridge in Japan.
Matsumoto et al. 2002	Describes the results of a research project examining the requirements for grouted connections between precast concrete columns and pier caps.
Medlock et al. 2002	Briefly summarizes eight bridge projects in Texas which have utilized precast components. Most of which were substructure components.
Moore 1994	Tested a 0.4-scale model of a steel plate girder bridge with precast concrete panels for the AISI-FHWA model bridge project.
Muller and Barker 1985	Outlines the design and construction of the Linn Cove Viaduct in North Carolina. Precast concrete box segments are used for the columns of the bridge.
Nawy 2003	Standard text on prestressed concrete. Appendices include standard AASHTO shapes and properties.
NBI 2002	National Bridge Inventory from 2002
Nottingham 1996	Describes typical precast deck panel joint details, materials, and construction practices utilized in the highly aggressive Alaskan dock environment for.

Osegueda et al. 1986	Summarized the tests of Biswas et al. (1984) as well as describing the construction and load testing of a bridge constructed using the same design as experimentally tested by Biswas et al. (1984).
Pate 1995	Outlines the design and construction of the Chesapeake and Delaware Canal Bridge. Precast concrete box segments were used for the bridge columns.
PCI BPC 1988	Presents guidelines for the design, manufacture and erection of precast prestressed concrete partial-depth panels.
PCI NER 2001	Provides guidelines and details for partial-depth precast/prestressed concrete deck panels.
PCI NER 2002	Provides design guidelines for the use of full depth precast deck panels for both new construction and replacement of existing decks.
Slavis 1982	Reports on the performance of approximately five bridges constructed using full depth precast bridge decks.
Sprinkel 1985	Report which summaries the use of precast bridge elements. NCHRP 407
Sritharan et al. 1999	Paper presenting an alternate design procedure for Bridge Cap Beam to Column joints which are more constructible than the existing conventional seismic design of bridge cap beam to column joints.
Stanton and Mattock 1986	Analytically examined the load distribution characteristics of open cell precast/prestressed beams. Also reviewed previous connection details, developed a methodology for designing connections, developed an improved detail, and performed experimental testing on a prototype connection.
Szautner 1984	Presents three precast concrete proprietary systems that can be used for rapid bridge construction. All of the systems are for small span bridges (<60ft).
Tadros 1998	Provides a summary of existing rapid bridge deck replacement methods and provides recommended procedures for future bridge deck replacements. NCHRP Report 407
Tajima 1966	Studied the strength and fatigue performance of several different types of precast panels to steel girder connections.
Takano et al. 1968	Examined the connection of precast concrete columns to cast-in-place footings and pier caps. Reports on the experimental testing of several two column piers.

Taylor et al. 1995	Presents results of a study which investigated experimentally and analytically the behavior of hollow, thin-walled concrete box bridge piers.
URS (Karl Larson)	Preliminary bridge plans utilizing PCUs for the superstructure replacement for the I-95 Bridge over Overbrook Road located in the City of Richmond, VA.
White 2000	Describes the rehabilitation of the Tappan Zee Bridge which utilized PCUs. This paper also describes the INVERSET system.
Wolf and Friedman 1994	Discusses the retrofit of the Redfish bay and Morris & Cummings Cut Bridges in Texas. Precast concrete pier caps were attached to existing cast-in-place concrete columns using a grouted mild steel connection.
WSBDM 1998	Washington State Bridge Design Manual
WSDOT 2004	Meeting with bridge engineers at the Washington Department of Transportation discussing previous experience and opinions on precast concrete superstructure systems. (January 29, 2004 Olympia, WA)
Yamane et al. 1998	Tested one specimen consisting of steel girders with three precast/prestressed slabs for service, fatigue, and ultimate behavior.
Yashinsky and Karshenas 2003	Handbook on the fundamentals of seismic bridge engineering for new and retrofit structures.
Yoon 2002	Prevents research examining the benefits of using fiber reinforced concrete segments in the location of plastic hinges for columns subjected to cyclic horizontal loading.
Young et al. 2002	Research the width and growth of cracks in the overhang portions of bridge cap beams.
Zaki 2003	Describes the reconstruction of the Jacques Cartier Bridge deck system using PCUs.