A Prefabricated Precast Concrete Bridge System for the State of Alabama

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Traditionally, cast-in-place concrete construction has been the primary method for building bridges in the State of Alabama. This method requires large amounts of time and labor at the project site. These factors can lead to traffic impacts in the forms of road closures, delays, congestion, detours, and often the need to work over a traveled roadway. With greater demand being placed on U.S. roadways and a large number of the nation’s bridges in need of repair or replacement, bridge construction and rehabilitation are becoming national issues for state departments of transportation. For the reasons aforementioned the Federal Highway Administration has, for the last few years, been actively promoting the use of prefabricated precast concrete bridge systems to state departments of transportation because of their ability to rapidly decrease construction times for bridge projects.

The objective of this study was to examine the existing practices in prefabricated bridge construction using precast concrete components, and to recommend a totally prefabricated concrete bridge system for use on the Alabama highway system. Current trends and practices in prefabricated bridge construction were reviewed. The specific elements examined were deck panels, girders, bent caps, columns, piers, and abutments. Superstructure and substructure systems were also examined and summarized. A number of systems were ranked according to their suitability for use in Alabama. Guidelines for selection, design, and construction of these systems were produced. Finally, a system was proposed to the Alabama Department of Transportation. Based on this evaluation and consultation with the Alabama Department of Transportation, a totally prefabricated precast concrete system, referred to as the University of Alabama at Birmingham (UAB) precast bridge system was developed. The system, consisting of bulb-tee girders, rectangular voided bent caps, rectangular voided columns, and a precast abutment cap, was selected for construction of grade separation structures in Alabama.
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Executive Summary

Traditionally, cast-in-place concrete construction has been the primary method for building bridges in Alabama. This method requires large amounts of time and labor at the project site. These factors can lead to traffic impacts in the forms of road closures, delays, congestion, detours, and often the need to work over a traveled roadway. With greater demand being placed on U.S. roadways and a large number of the nation’s bridges in need of repair or replacement, bridge construction and rehabilitation are becoming national issues for state departments of transportation.

For the reasons aforementioned the Federal Highway Administration has, for the last few years, been actively promoting the use of prefabricated precast concrete bridge systems to state departments of transportation because of their ability to rapidly decrease construction times for bridge projects.

The objective of this study was to examine the existing practices in prefabricated bridge construction using precast concrete components and to recommend a totally prefabricated concrete bridge system for use on the Alabama highway system. An extensive literature review was conducted to assess current trends and practices in prefabricated bridge construction. The specific elements examined were deck panels, girders, bent caps, columns, piers, and abutments. Superstructure and substructure systems were also examined and summarized. To accomplish the study’s objective, several tasks were performed. Concepts and ideas were developed. A number of systems were ranked according to their suitability for use in Alabama. Guidelines for selection, design, and construction of these systems were produced. Finally, a system was proposed to the Alabama Department of Transportation.

Based on this evaluation and close communications with the Alabama Department of Transportation, a totally prefabricated precast concrete system, referred to as the University of Alabama at Birmingham (UAB) precast bridge system was developed. The system, consisting of bulb-tee girders, rectangular voided bent caps, rectangular voided columns, and a precast abutment cap, was selected for construction of grade separation structures in Alabama.
Section 1
Introduction

Background

With the combination of an aging infrastructure and a higher level of traffic than anticipated on many of America’s roadways, a growing number of the nation’s bridges are in need of repair. In 2002 the Secretary of Transportation’s biennial report to Congress stated that a number of the nation’s bridges were considered structurally deficient (Shahawy 2003). Of the 594,000 publicly owned bridges in the United States, approximately 25 percent fall into the category of structurally deficient or functionally obsolete. Bridges that are considered functionally obsolete can no longer convey traffic in a safe and effective manner (Public Works 2003). Additionally, for the number of bridges that are rehabilitated each year, approximately 3,000 more enter into the categories of either structurally deficient or functionally obsolete (Ralls, et al. 2004). The American Society of Civil Engineers (ASCE) 2005 Report Card for America’s Infrastructure states that bridges ranked second in Alabama’s top three infrastructure concerns (ASCE 2005). A grade of “C” was assigned to bridge structures for the whole nation. This report also states that 30 percent of Alabama’s bridges are either structurally deficient or functionally obsolete.

Most traditional highway concrete bridge construction projects to date are sequential in nature. These bridges vary in span from 20 to 125 feet (Sprinkel 1985). After constructing the foundations and requiring that the cast-in-place concrete in the foundations reaches adequate compressive strength, formwork is then erected for columns and abutments. Cast-in-place concrete is then poured to complete the columns, piers, or abutments, depending on the bridge design. Forms are then removed from the columns and re-erected to construct the bent caps. Cast-in-place concrete is poured into the bent cap forms to complete the bent cap construction portion of the project. Once bent cap formwork has been removed and the concrete used for the bent caps reaches an adequate strength, precast girders are usually erected across the spans. Formwork must then be erected for the deck of the bridge. Cast-in-place concrete is then placed to create the deck of the bridge. Once the concrete used for the deck has reached a sufficient strength, the formwork is removed; in the case of stay-in-place formwork this step is unnecessary, and the deck is allowed to cure until the specified strength is reached before traffic is allowed to move across the bridge. Barrier rails are normally constructed with the slip form method of placement after the deck reaches a sufficient strength and before the bridge is officially opened.

Overall, bridge construction is a labor-intensive, lengthy process primarily due to curing times for concrete and formwork construction. Due to the sequential nature of the construction all of these activities tend to impact the critical path of the project (FHWA 2005). Total construction time typically varies from a few months for simple short span bridges to more than a year for
longer span bridges. In cases of bridge repair projects it can often mean lengthy road closures and traffic detours.

The use of precast prefabricated concrete bridge components in bridge construction projects can aid in accelerating construction by removing curing times from the critical path of the project and eliminating the need for temporary formwork (Ralls and Tang 2003). The use of completely prefabricated precast concrete bridge systems could further accelerate the construction of bridges in the state by combining the advantages of precast prefabricated elements together with the advantages of a predesigned system.

**Research Significance**

The American Association of State Highway and Transportation Officials Technology Implementation Group (AASHTO-TIG) has selected prefabricated bridge elements and systems as a market-ready technology and has made numerous efforts to promote its use among individual state departments of transportation (Ralls, et al. 2004; Ralls and Tang 2003). The Federal Highway Administration (FHWA) states that prefabricated concrete bridge elements and systems offer many attractive incentives, such as making the work zone safer, lowering disruptions to traffic, improving quality of the construction, lowering environmental impacts, and lowering life cycle costs (FHWA 2005). Environmental impacts are lessened by such means as reducing the amount of equipment required on the project site, eliminating the need to place piers in stream crossings, and reducing the amount of emissions produced by delayed traffic. The FHWA in conjunction with AASHTO has been working to further the utilization of prefabricated concrete bridge systems on bridge projects across the country (Ralls, et al. 2004). This study is an attempt to select a totally prefabricated precast concrete bridge system that is suitable for use on Alabama highways.

**Objectives of Study**

This study was aimed at assessing the current state-of-the-art practices in prefabricated bridge design and construction in the United States and Europe. The goals of the project were to first analyze the existing practices and systems and then develop a completely prefabricated precast concrete system or systems suitable for use by the Alabama Department of Transportation (ALDOT). The specific objectives of the study were as follows:

1. Collect, examine, and summarize the available literature on prefabricated concrete bridge elements and systems;
2. Develop concepts and ideas for a prefabricated precast concrete bridge construction system(s) that are suitable for use in Alabama and rank them according to their suitability for use;
3. Propose a prefabricated precast concrete bridge system for use in the state;
4. Provide guidelines for the selection, design, and construction of this system(s);
5. Prepare a typical design example that illustrates the components and details of the proposed system.
Scope of Study

This study examined the design and construction procedures for various precast concrete bridge components, including abutments, columns, piers, piles, bent caps, girders, and decks. Only precast concrete components were taken into consideration in the scope of this study. Once the literature review was performed, design concepts were developed and a bridge design using prefabricated precast concrete bridge components was produced. Connections for bridge elements were also taken into consideration in the scope of the study. Foundations were not included in the scope of the study. Traditional methods for foundations were recommended. ALDOT is currently using traditional methods for foundation construction.

Section one is an introduction to the project. The background of the project is addressed along with the research significance and the study’s objectives and scope. An overview of the literature reviewed is provided. Section two provides a detailed review of the current literature pertinent to prefabricated precast concrete bridge design and construction methods. Section three proposes a prefabricated precast concrete bridge system for use in Alabama. Section four provides a design example of the proposed system. The example illustrates the design approach, the bridge structural layout, and the design details of the system.

Literature Review

A comprehensive literature review was performed to gather pertinent information on prefabricated precast concrete bridge systems and to survey the current practices in prefabricated concrete bridge design and construction. Information was gathered from various foreign and domestic articles, technical papers, journals, and reports. The literature review was instrumental in the selection of the proposed University of Alabama at Birmingham (UAB) precast bridge system. Details of the literature review are provided in the next section.
Section 2
Review Of Prefabricated Bridge Elements And Systems

Background

Merriam-Webster’s dictionary defines the term prefabricate as “to fabricate the parts of at a factory so that construction consists mainly of assembling and uniting standardized parts.” For the scope of this project a prefabricated precast concrete element is any structural element of the bridge that is made away from the bridge site (Sprinkel 1985). A totally prefabricated precast bridge system may consist of different combinations and variations of the following individual precast concrete elements: precast deck panels, precast girders, precast bent caps, precast piers, precast columns, precast piles, and precast abutments. Individual precast concrete elements may also be used in combination with traditional cast-in-place construction to shorten bridge construction project duration times.

Superstructure systems combine decks and girders into one unit through various methods or eliminate the need for individual deck panels or girders. Substructure systems consist of varying combinations of precast abutments with either precast piers or precast columns in combination with precast bent caps (Ralls, et al. 2004; Aktan, et al. 2000). These different individual elements and systems are examined further in the following sections.

Individual Prefabricated Elements

Deck Panels

The traditional and most popular method for constructing a bridge deck in the United States is the cast-in-place method of concrete placement. A smooth deck surface is insured when using this method due to the ability to vary the profile of the concrete and the fact that the cast-in-place concrete creates one continuous riding surface. This method requires that either temporary formwork or stay-in-place steel formwork be erected to contain the concrete used for the deck. Once formwork is erected, reinforcement must be placed and then cast-in-place concrete must be placed into the forms. Cast-in-place concrete must also be allowed to cure, usually 28 days, until a specified strength is attained before any heavy traffic loads may be allowed on the deck (Aktan, et al. 2000; Jakovich and Alvarez 2002). Overall this process can take a long time and requires a large amount of on-site labor (Culmo 2000). These constraints may be avoided by utilizing precast deck panels when completing the deck portion of a bridge project.

The two main types of precast concrete deck panels are full-depth deck panels, shown in Figure 2-1, and partial-depth deck panels, shown in Figure 2-2. Full-depth and partial-depth deck panels may be poured away from the site at a precasting plant and shipped to the site once they
have cured and are ready for placement (Jakovich and Alvarez 2002). This feature helps to minimize disruptions to traffic, improve construction quality, and lower overall construction time. Once at the site the deck panels may be placed directly on precast girders and connected through different methods that will be discussed later in this section (Ralls, et al. 2004; Aktan, et al. 2000; Shahawy 2003).
The first main type of deck panels, full-depth deck panels, aid in accelerating construction of a bridge by completely removing the casting and curing times for the deck from the critical path of the project, which is accomplished by eliminating the need for any cast-in-place concrete. They also eliminate the need to erect temporary formwork. Full-depth deck panels may vary in size and may use both transverse and longitudinal joints between panels. They are usually fabricated using only transverse joints such that the panel length is the full width of the roadway for standard width bridges (Aktan, et al. 2000; Issa, et al. 1995a, b). Full-depth deck panels may be prestressed at the casting yard and may also be posttensioned after erection. Longitudinal posttensioning helps stabilize the panels and create continuity in the deck. Posttensioning also ensures that deck panels will be securely tightened together (Issa and Islam 2002). The panels may be posttensioned both transversely and longitudinally. Full-depth deck panels not only accelerate construction but also reduce the effects of shrinkage (Ralls, et al. 2004; Aktan, et al. 2000; Shahawy 2003).

Different types of connection methods between adjacent deck panels and connections between panels and supporting elements were identified during the course of the literature review. The main joints identified were grouted shear key types of joints. The shear key joints identified were female-female, male-female, or match cast types of joints that could be posttensioned, passively reinforced, welded, or bolted. Posttensioned joints are the most complicated of the possible types but provide the best performance. The induced compression on the joints helps close cracks that form under service conditions. Passively reinforced joints, shown in Figure 2-3, use reinforcement that extends from the prefabricated pieces into the joint, which is later filled with concrete. The reinforcement in these types of joints is usually lapped by various methods. The drawbacks to passively reinforced joints are twofold because of the need to erect formwork in order to contain the concrete at the joint and the time required for the concrete to cure. Welded plate connections use steel plates that are cast into the deck panels at specified locations. The metal plates line up once the panels are erected in the field. The plates are welded together and the keyway is grouted to complete the construction. Bolted connections are used primarily with proprietary systems (Ralls, et al. 2004; Shahawy 2003; Issa, et al. 2004). Another type of connection identified for decks during the literature review was use of a connecting panel. The predominant method identified for connecting panels to supporting members utilizes studs that are fastened to the supporting members and extend upward into a void in the precast panel commonly referred to as a block-out or shear pocket. The shear pockets are cast into the precast panels at specified intervals according, to the design of the bridge. Once erected, shear studs are fastened to the supporting members and the pockets are filled with a non-shrink gout to complete the connection (Issa, et al. 1995a, b, 2004; Shahawy 2003; Culmo 2000).

A sub-type of full-depth deck panels is lightweight deck panels, shown in Figure 2-4. These types of panels may use lightweight concrete with steel grids or fiber-reinforced polymer reinforcement. These panels may be used in special cases where it is desirable to reduce loads from the superstructure, or where smaller erection equipment is mandated to place deck panels because of job constraints (Shahawy 2003).
The second main type of deck panels, partial-depth deck panels, act as stay-in-place forms when used to construct bridge decks. A topping slab is used in conjunction with these types of deck panels. In this method of construction partial-depth deck panels are erected and may be posttensioned together if the design calls for it. Once the panels are erected, a layer of cast-in-place concrete is poured over the tops of the panels to complete the deck. The use of partial-
depth deck panels accelerates construction of a bridge by removing erection of formwork for the
deck from the activities list. One drawback of using partial-depth deck panels is the possibility
of reflective cracks in the surface concrete at the locations of the precast deck panel joints

A system for leveling precast deck panels was identified while performing the literature review.
In this system deck panels have a leveling bolt located at each corner of the panel. A threaded
socket is precast into the panel and houses the leveling bolt. The bolts are adjusted up or down
with a wrench until the desired elevation is attained at the surface of the deck panel. The bolts
bear directly on the flange of the girder until grout is placed to bear the weight of the deck panel
(Shahawy 2003; Issa, et al. 2004, 1995a, b). Leveling shims may also be used to achieve proper
deck elevations before grouting the spaces between deck panels and girders.

Overlays may be used in conjunction with full-depth and partial-depth deck panels. Overlays
help to ensure a smooth riding surface but also add additional weight to the deck. Overlays are
achieved by pouring a layer of concrete or placing a layer of asphalt over the tops of the deck
panels. One alternative to overlays is grinding the surface of the full-depth deck panels to ensure

Another type of deck system identified in this literature review was hybrid steel and concrete
deck units (FHWA 2004). The FHWA scan team observed this technology currently employed
in Japan. The units consist of a steel bottom plate that rests on the bottom flange of the
transverse beam and reinforcing bars that rest on the top flange of the transverse beam. The
transverse beams span across the main girders of the bridge. Once the units are erected concrete
is placed into the units to finish the construction (FHWA 2004).

**Beams and Girders**

Concrete precast prestressed girders are probably the most widely made and accepted of all
precast bridge elements. Figure 2-5 shows long span girders in use in Alabama. Precast
prestressed girders were first used in the United States on the construction of the Walnut Lane
standardized type I through IV girder sections, which were developed by AASHTO and the
Precast/Prestressed Concrete Institute (PCI), have been in existence since the late 1950s. A
study conducted in the late 1970s to evaluate the efficiency of girder sections concluded that
bulb-tees provided the highest savings in cost and weight when compared to other standard
sections (Ralls, et al. 2004). The results of that study prompted PCI to develop the PCI bulb-tee
standard girder (Aktan 2000). The precast concrete I-girder is the most widely used shape in the
United States and has a low cost compared to alternatives. Also, forms are readily available to
cast this shape. Other shapes often used for girder sections in the United States are U, single-tee,
double-tee, rectangular box, solid slab, and voided slab girders (Aktan, et al. 2000; Chakraborty,
et al. 1995; Sprinkel 1985). Other commonly used sections such as bulb-tee girders, single-tee
girders, double-tee girders, channel girders, slab spans, and box girders will be discussed further
in the section on prefabricated superstructure systems.
Bent Caps

Bent caps have been the most widely prefabricated of all substructure components to date (Ralls, et al. 2004). Prefabricated bent caps facilitate accelerated construction in much the same way as deck panels and girders, by removing forming and curing times from the critical path of the project (Ralls, et al. 2002; Shahawy 2003). Their use also decreases the impact of construction and rehabilitation on the traveling public. Texas first used precast bent caps on the Redfish Bay and Morris Cummings Cut Bridge in 1994. The use of precast bent caps reduced the project duration by six months (Medlock, et al. 2002). It is also easier to construct bent caps in the precasting plant versus the field due to factors such as elevated construction of the bents in the field, environmental constraints of the field, and complicated formwork construction (Ralls and Tang 2003). Prefabrication of the bent caps also increases safety by decreasing the amount of time that workers will have to perform their tasks at dangerous heights. The two types of bent caps identified in the literature review were rectangular bent caps and inverted-tee bent caps. Precast bent caps may be used in conjunction with either cast-in-place columns or piers. They may also be used with precast columns, piers, or piles. Bent caps were connected to columns through various methods that will be discussed later in this section.
Rectangular precast concrete bent caps were the first and most-widely used bent cap shape identified in this study. Figure 2-6 shows a rectangular precast bent cap being set on concrete piles. Rectangular bent caps were placed on top of either precast or cast-in-place columns, piers, or piles in the various projects in which they were utilized. The bent caps are rectangular in shape and may be either solid or voided to reduce the weight of the cap (Shahawy 2003).

![Figure 2-6. Precast rectangular bent cap (FHWA 2006j)](image)

Inverted-tee bent caps, as the name implies, are in the shape of an inverted-tee. Figure 2-7 shows a picture of an inverted-tee bent cap. Beams or girders rest on the flanges of the cap, instead of the top of the cap as with the traditional rectangular bent (Shahawy 2003; Billington, et al. 1999).

The four main connection methods for connecting bent caps to columns or piers identified during the course of the literature review were grouted pockets, grouted vertical ducts, bolted connections, and grouted sleeve couplers (Matsumoto, et al. 2001). Figure 2-8 shows an illustration of a cap-to-column connection using grouted couplers. In the connection methods identified, connecting tendons normally extended from the column or pier vertically upward into voids or ducts that were precast into the cap. Caps were set on shims to ensure proper elevation of the caps. Once the caps were set in place, grout was placed into the voids or ducts to complete the connection between the caps and columns (Matsumoto, et al. 2001).
Columns and Piers

A substructure may use a variety of different supporting structural elements depending on the requirements of the design. For the purpose of this study, the term pier is used to indicate an intermediate bridge support that is either a wide rectangular gravity type of structure or a multiple column pier. Multiple column piers consist of two or more columns that support a pier cap. Columns are slender supporting elements that may be circular in shape and support a bridge in combination with other columns to make a pier or as a single column structure. As with the
other previously discussed precast elements, using precasting with substructure components facilitates rapid construction. Construction time is reduced by removing the time required due to erection of extensive formwork, placement of reinforcement, and curing times for the concrete from the critical path of the project (Ralls, et al. 2004; Billington, et al. 2001). The following precast elements were identified during the course of this literature review: segmental columns, whole columns, segmental piers, whole piers, piles, and Sumitomo’s precast form method for resisting earthquake and rapid construction (SPER). These different structural elements may also incorporate additional technologies, such as prestressing, match casting, posttensioning, and voided construction. These different elements will be looked at more closely in the following paragraphs.

A segmental column consists of column segments of varying length, depending upon the design, that are stacked vertically until the desired total column height is reached. Figure 2-9 shows a picture of a segmental column. Once the column segments are erected they may be vertically posttensioned together and to the foundation for stability. Segmental columns provide easier handling and erection than whole columns of equal height. Segmental column systems may incorporate many technologies, such as match casting, epoxy coating of joints, shear keys, and voided sections to reduce element weight (Billington, et al. 1999, 2001; Shahawy 2003). The Dallas/Fort Worth International Airport People Mover, shown in Figure 2-9, is an example of a project that used precast segmental columns. These types of columns will be discussed further in the section on prefabricated substructure systems.

Whole columns are cast and erected as one complete unit according to the height required by the design. Figure 2-10 shows a precast whole column used for the Edison Bridge. Whole columns may be placed directly onto cast-in-place footings and vertically posttensioned to anchors in the footing (Billington, et al. 2001). Connecting tendons may be used to connect the columns to the footings. Ducts may be cast into the base and top of the column to house the connecting tendons that extend vertically from the footing. The Edison Bridge project in Fort Meyers, Florida, is an example of whole columns being used to speed construction. The bridge used I-shaped whole columns with precast bent caps. The I-shape reduced the weight of the columns. Whole columns may also be rectangular and voided in order to reduce weight (Shahawy 2003).

Precast box piers, like columns, may be segmental or whole. Segmental piers are cast in multiple units of varying heights, depending upon the design requirements. Figure 2-11 shows a completed segmental pier. The units are stacked vertically much like segmental columns until the desired pier height is reached. The segments are posttensioned to one another and also to the cast-in-place footing with connecting tendons that extend vertically from the cast-in-place footing into base of the piers. Segmental piers may also use match casting to ensure proper alignment in the field like the segmental columns described earlier. Segments may also be voided or hollow to reduce the dead load on the foundations and make handling of the segments easier. The void may also be utilized to run drains down the interior of the pier. Segmental piers may be vertically posttensioned once erection is completed. The Cross Westchester Expressway Viaducts in Westchester county, New York is a good example of match cast segmental piers (FHWA 2006a; Shahawy 2003; Billington, et al. 2001).
Figure 2-9. Segmental column (FHWA 2006k)

Figure 2-10. Precast whole column (FHWA 2006m)
Precast whole piers are similar to precast whole columns in concept. The piers are precast to the desired height according to the design of the bridge. Figure 2-12 shows a precast whole pier. Precast piers are placed directly onto cast-in-place footings and connected to anchors or connecting tendons that are cast into the footing and extend upward vertically into the base of the piers. Ducts are cast into the base of the piers or the entire length of the pier depending upon the design requirements. Once the piers are set in place they are vertically posttensioned to the footings. Precast whole piers may also utilize technologies such as prestressing and voided shapes. Voided whole piers reduce element weights, which makes handling and erection easier. The Baldorioty de Castro Avenue Overpasses in San Juan, Puerto Rico are good examples of precast whole columns (FHWA 2006o; Shahawy 2003).

Another type of precast column consists of precast driven or drilled shaft piles. The piles extend up to the required elevation of the bent caps and act as columns. Figure 2-13 shows a detail where precast piles were used. Traditional cast-in-place or precast bent caps may be used with precast piles. Connecting tendons may extend up from the top of the pile into the precast bent cap. Once the pile cap is set, the cap is posttensioned to the pile and the ducts are grouted to complete the connection between the caps and the piles. State Highway 361 over Redfish Bay and Morris-Cummings Cut Bridge in Arkansas county, Texas is a good example of precast piles that act as columns (Billington, et al. 2001, 1999; FHWA 2006b).
Sumitomo’s precast form method for resisting earthquake and rapid construction, otherwise, known as the SPER system, utilizes concrete panels as stay-in-place forms during construction. Figure 2-14 shows the SPER system in use. Once the concrete panels have been erected, cast-in-place concrete is placed to create a composite pier. Panels can be used as outer formwork for solid piers or both interior and exterior formwork for hollow piers. It is estimated that use of the SPER system reduces normal construction time of piers by 60 to 70 percent (FHWA 2006c; Shahawy 2003). Overall, accelerated construction time and improved constructability are afforded by the system.
Abutments

A precast posttensioned abutment system used by the Pennsylvania Department of Transportation was identified during the course of the literature review. The system consisted of a precast abutment wall and wing wall sitting on a cast-in-place footing. Segments for the abutment wall were 16 feet in length and 8 feet high. The abutment wall was connected to the cast-in-place footing through the use of anchors placed in the footing during casting. The anchors extended upward from the cast-in-place footing to couplers that were cast into the base of the abutment segment. The connection ducts were grouted once the abutment segments were set into place. Vertical and horizontal joints between abutment segments were grouted after segments were set into their proper position. Posttensioning was performed after the cast-in-place footings and grouted joints had attained the specified strength. Flowable fill was used to backfill against the abutment wall and eliminate the need for compaction equipment. This system was used on State Road 3026 over Miller’s Run Creek in Pennsylvania (Scanlon, et al. 2002).

Prefabricated Superstructure Systems

General

Superstructure systems serve to combine decks and girders into one unit through various methods or to eliminate the need for individual deck panels or girders (Ralls, et al. 2004; Aktan, et al. 2000). The use of complete superstructure units involves setting complete units between supports and eliminating the need to set girders and then set decks. Using prefabricated superstructure units greatly cuts down the time required to construct or replace a bridge’s
superstructure. This system is faster than traditional methods using cast-in-place concrete or prefabricated techniques where girders and slabs are placed separately.

**Precast Box Beams Placed Adjacent To One Another**

A system composed of precast box beams placed adjacent to one another was used on the Baldorioty de Castro Avenue Overpasses in San Juan Puerto Rico (Shahawy 2003). In this system precast concrete box girders are placed side by side between supports and may be posttensioned together for stability. Precast box beams may also be prestressed. Element size, length, and width may vary according to the project requirements. Sprinkel (1985) suggests that box beams are most suitable for spans of 50 to 100 feet. Grouted keyways may be utilized to connect adjacent box beams in this superstructure system much the same way they are used to connect adjacent precast deck panels. This superstructure system eliminates the need for deck panels or a cast-in-place deck. An overlay of cast-in-place concrete or asphalt may be applied to improve the ride quality. Figure 2-15 shows an illustration of precast box beams placed side by side.

![Figure 2-15. Precast box beams side by side (FHWA 2006o)](image)

**Poutre Dalle System**

The Poutre Dalle System of superstructure construction uses inverted-tee beams placed side by side between the bridge piers or abutments. Once the beams are erected cast-in-place concrete is poured between the webs of the beams and over the tops of the beams to make a composite deck. This system eliminates the need for any temporary formwork. Interlocking steel extends from the inverted-tee beams into the joints between the beams to lock adjacent beams together once concrete has been placed. This system is applicable to spans of 20 to 105 feet (FHWA 2006c). This system of superstructure construction was observed by the FHWA scan team in France. An inverted-tee beam superstructure system based on the Poutre Dalle System has been used by the Minnesota Department of Transportation on T.H. 8 over Center Lake Channel and T.H. 72 over the Tamarac River (Hagen 2005). Figure 2-16 shows a Poutre Dalle beam.
Precast Double-Tee Beams and Channel Beams Placed Adjacent to One Another

Precast double-tee beams were used on the Pelican Creek Bridge in Alaska in 1992 to rehabilitate the bridge in five weeks (AASHTO TIG 2004). Double-tee beams are placed side by side between supports eliminating the need for formwork or deck panels. The diaphragms are posttensioned together transversely after the beams are set in place. Grouted keyways or welded plates like those used for deck panels may be utilized to connect adjacent double-tee or channel beams. Generally an overlay of concrete or asphalt is added to improve ride quality although an overlay is not required. These systems are most practical for spans ranging from 20 to 65 feet in length (Shahawy 2003; Sprinkel 1985). The same methods are used for channel beams. Figure 2-17 shows an illustration of double-tee beams and channel beams placed adjacent to one another.

Bulb-Tee Girders

Bulb-tee girders were used on the Rock Cut Bridge over the Kettle River in Washington and in 1997 by the Idaho Transportation Department on U.S. Route 95. In this system, as in previously identified systems, bulb-tee girders are placed adjacent to each other. The bulb-tee girders have a wide top, and when placed adjacent to each other, flanges of adjacent beams touch. The flanges of the girders act as the riding surface of the bridge. Flanges of adjacent girders may be connected by the same methods previously stated for deck panels, box beams, double-tees, and channel beams. The use of bulb-tee girders eliminates the need for deck panels or cast-in-place concrete to form a riding surface thus accelerating construction of a project (Nicholls and Prussack 1997; PCI 2006). As previously stated in the section on beams and girders, a FHWA-sponsored study performed in the late 1970s (Aktan, et al. 2000) showed that bulb-tee girders...
were the most efficient shape of all girder shapes available at the time. These girder types offered reductions in weight and savings in cost when compared to other girder shapes (Aktan, et al. 2000). As with the previously mentioned superstructure systems, bulb-tee girders may be covered with a topping slab to ensure a smooth ride for motorists. Figure 2-18 shows an illustration of a bulb-tee girder.

Figure 2-17. Precast double-tee beams and channel beams placed adjacent to one another (Sprinkel 1985)

Figure 2-18. Precast bulb-tee girder (Sprinkel 1985)
Slab Girders Placed Adjacent To One Another

This system composed of slab girders placed adjacent to one another was used on the Mitchell Gulch Bridge in Colorado (AASHTO 2004; Ralls and Tang 2003). Slab girders are placed side by side between supports and are transversely posttensioned together once erected. Figure 2-19 shows a slab girder being erected at Mitchell Gulch. Grouted keyways or welded plates may be utilized in this system to connect adjacent girders in a manner similar to the methods observed with deck panels. This system seems more suited to short span bridges. Slab sections may be voided or solid according to the needs and constraints of the project. Sprinkel states that solid slabs may be utilized for spans less than 30 feet. For lengths over 30 feet, posttensioned spans with voided cross-sections should be used (Sprinkel 1985).

Composite Deck and Girder Systems

A composite deck and girder superstructure system consists of precast concrete beams that are made composite, into a single liftable unit, with precast concrete deck panels. This system was used on the I-10 Bridge over Lake Pontchartrain in New Orleans, Louisiana (AASHTO 2004). In this system concrete slabs are cast onto precast concrete girders and made composite. Once the superstructure units are complete they can be lifted into place as a deck and girder unit (Shahawy 2003). This system was also used on the Richmond San Rafael Bridge in California. These units may generally be heavy and require large equipment to lift the units into place. Figure 2-20 shows a composite deck and girder unit being erected.
Prefabricated Substructure Systems

General

National Cooperative Highway Research Program (NCHRP) Synthesis Of Highway Practice No. 324, titled “Prefabricated Bridge Elements and Systems to Limit Traffic Disruption During Construction” (Shahawy 2003), states that construction of the substructure may expend 60 to 70 percent of the total construction time required for a project. For this reason the use of prefabricated substructure systems can greatly reduce construction time dedicated to the substructure and accelerate the overall construction of a bridge. A complete substructure system combines some form of precast column or pier system with the use of precast bent caps. Members can be prestressed, concrete-filled, hollow, boxed whole units, boxed segmental units, segmental posttensioned units, precast posttensioned whole units, or precast segmental match cast units (FHWA 2006d). One of the column or pier systems described earlier may be utilized in conjunction with one of the types of bent caps described earlier to create a complete substructure system. It was observed that systems were usually posttensioned during erection or once erection was completed.

Segmental Columns with Precast Bent Caps

The use of segmental columns with precast bent caps is described in detail in NCHRP Synthesis 324 (Shahawy 2003; Billington, et al. 1999). As previously stated, a segmental column consists of shorter column segments that are stacked vertically to attain a specified total height. In this system, column segments are posttensioned together and to a cast-in-place footing. A template section is placed as the last column segment. The template section ensures a proper fit and alignment for the precast bent cap. A precast bent cap is then placed on the template segment to complete the erection sequence. The entire structure is vertically posttensioned once erected (Billington, et al. 1999, 2001; Shahawy 2003). Figure 2-21 shows a segmental column with a precast bent cap.
It is suggested that column segments be match cast to ensure proper fit and quick erection in the field but it is not a mandatory requirement of a segmental system (Billington, et al. 1999, 2001). These segments may also have shear keys to ensure proper alignment of the units during installation. Epoxy may be placed on the surfaces of segments being fitted together and is recommended (Billington, et al. 1999, 2001). If precast bent caps are used to make a substructure system, then the addition of a template section is suggested. The template section allows for a match cast fit with the bent and correct alignment of the bent. Hollow segments are lighter, which eases handling of the segments. Wall sections of the columns must be thick enough for posttensioning ducts or connecting tendons. Internal drain lines can also be installed inside of hollow columns as an added benefit. This style of substructure may be applied to single column structures typically known as hammerhead bents and multicolumn structures.

**Whole Columns With Precast Bent Caps**

Whole columns, with precast bend caps, indicates that the column is not segmental. This system was used with the Edison Bridge in Fort Myers, Florida. In this system whole precast columns
are set on cast-in-place footings and a precast bent cap is then placed atop the column or columns. Figure 2-22 shows a precast column and precast bent cap from the Edison Bridge. The columns are connected to the footings using connecting tendons that extend from the footing into the base of the columns. Connecting tendons are used to connect the precast bent caps to the columns as well. Tendons extend upward from the tops of the columns into the base of the bent caps through some form of duct, pocket, or sleeve. Whole columns may have varying shapes, such as the I-shape of the columns on the Edison Bridge, to reduce column weight. Whole columns may also be voided to reduce weight. It is recommended that the ends of voided columns be solid for a specified length for connection and shear transfer purposes. Walls of voided columns must be thick enough to provide sufficient room for reinforcing steel and any connecting or post-tensioning ducts (Billington, et al. 1999, 2001; Shahawy 2003).

Precast Box Piers With Or Without Bent Caps

Precast box piers were used at the Baldorioty de Castro Avenue Overpass in conjunction with precast pier caps and precast box girders. Construction was finished in 36 hours for the first overpass and 21 hours for the second overpass (Shahawy 2003). The piers and caps were vertically post-tensioned to cast-in-place footings. It appears that the Cross Westchester Expressway Viaduct utilized piers without pier caps (FHWA 2006e). These projects show that bent caps may or may not be utilized in conjunction with piers according to project constraints and requirements. Figure 2-23 shows a precast box pier in conjunction with a precast bent cap.
Figure 2-23. Precast box pier with precast bent cap (FHWA 2006a)
Section 3
The Proposed UAB Precast Bridge System

Selection of the Proposed System

A precast concrete bridge system was developed for use in Alabama. This system was based on concepts identified in the literature review along with input from a local precast concrete manufacturer. This system is described in further detail in the following section. The following criteria were considered in the development of the proposed UAB precast bridge system for Alabama: availability of materials, ability to manufacture elements at the plant and deliver them to the site, ability of elements to accelerate construction, ease of handling of elements, size limitations of elements due to weights, previous experience of the precast concrete company, and enhancement of safety.

The following precast bridge elements are the main components of the UAB precast bridge system: superstructure deck and bulb-tee girder system, bent caps, columns, and abutments caps. These components can be manufactured at the precast concrete facility and shipped to the project site for erection of the bridge. It was decided to use the traditional cast-in-place method of construction for bridge foundations for this system due to concerns that arose regarding the feasibility and practicality of using a precast foundation in this system. Extensive time and preparation of the foundation excavations prior to placement of the precast foundations would be required to ensure proper surface elevations and levelness of the precast foundations once placed. This requirement would increase the difficulty of on-site construction of the foundations.

Materials and Limitations

The primary materials for this system are precast prestressed concrete and reinforced concrete. The concrete used to cast the caps, columns, and abutments was specified to have a compressive strength of 5000 pounds per square inch (psi) when removed from the forms and a compressive strength of 6000 psi at 28 days. The concrete used to cast the bulb-tee girders for the superstructure was specified to have a compressive strength of 8000 psi at 28 days and a release strength as required by the design. The concrete used for the topping was specified to have a compressive strength of 4000 psi at 28 days.

A nonshrink grout that meets AASHTO designations and specifications is required for the joints between foundations, columns, and bent caps (AASHTO 2002). The mechanical coupling system specified for the connections between foundations, columns, and caps calls for a special proprietary grout that is part of the overall proprietary mechanical coupling system.
The second most used material in this system is steel. This system utilized prestressing steel strands; mild steel reinforcement; welded steel plates to connect adjacent girders; and mechanical couplers to connect foundations, columns, and caps together. The mechanical coupling system selected to connect substructure elements together was the Nisso Master Builders (NMB) splice sleeve system, which is a grouted sleeve connection that utilizes a coupler that houses dowels that extend from the precast elements into the sleeve. Once elements are erected, the sleeve is grouted to complete the connection between elements.

The following limiting factors were considered in the development of this system: element weights, element dimensions, slenderness ratio for columns, and geometric considerations for the overall roadway. One of the main limitations considered in the design and development of this system was element weight. Element weight influenced the ability to handle elements at the precasting plant and on the jobsite as well as the ability to transport elements from the precasting plant to the jobsite. Element weight also influenced the design of the foundations due to the dead load on the foundations from the substructure and superstructure elements.

The main limitations considered in the design of the girders were weight of individual girders and span length. It was decided to limit the weight of an individual girder to 160,000 pounds and to limit the maximum simple span of a single girder to 130 feet.

The main limitations considered in the design of the bent caps and abutment caps were weight of individual caps, cap width, cap length, and cap height. It was decided to limit the weight of an individual cap to 150,000 pounds, which limited the depth to 5 feet, the width to 4 feet 6 inches, and the length to 45 feet.

The main limitations considered in the design of the columns were weight of individual columns and the slenderness ratio for an individual column. It was decided to limit individual column weights to 100,000 pounds for a single column. A maximum slenderness ratio of 100 was used for a single column.

Limitations considered for the overall bridge geometry were roadway width, maximum span, horizontal alignment, and cross slope of the bridge deck. It was decided to limit the overall clear roadway width to 40 or 44 feet. The two roadway widths were chosen based on ALDOT bridge width recommendations for two-lane roads (ALDOT 2003). This bridge system was designed for tangent horizontal alignment. Skewed bridges and horizontal curvature were not addressed in the scope of this project. Cross slopes of the deck were limited to two percent and the normal crown was six feet.

**Superstructure**

A deck bulb-tee girder superstructure system was chosen for the UAB precast bridge system. As mentioned previously, bulb-tee girders were found by the FHWA-sponsored study in the late 1970s to be the most efficient shape for precast concrete girders. This superstructure system was selected because it offers acceleration of construction, enhanced safety, and flexibility. In addition, the local precaster has had previous positive experience with casting of bulb-tee
girders. Upon reviewing the current ALDOT construction practices for bridge superstructure construction, it was identified that the most common configuration used is either AASHTO girders or PCI bulb-tee girders spaced 7 to 8 feet on center. A cast-in-place deck slab, normally 7 inches thick, is then poured on top of the precast girders to form the riding surface of the bridge. This typical arrangement is shown in section A-A of Figure 3-1. The conventional cast-in-place deck slab requires formwork between the girders and also for the slab overhangs. The proposed UAB precast bridge system superstructure scheme can be seen in Figure 3-2. It consists of deck bulb-tee girders erected adjacent to each other. The cross-sections for these girders are patterned closely after the deck bulb-tee configuration presented in the PCI Bridge Design Manual; Appendix B of the manual contains the general configuration proposed by PCI. The modifications made to the PCI sections can be seen in Figure 3-3. Primarily, the bottom of the beam is configured more closely to the PCI bulb-tee girder that is currently in use, and the web width was increased to 7 inches to facilitate concrete placement. Also, because a cast-in-place topping is to be used with the proposed system, the minimum thickness of the top flange has been reduced to 5 inches at the outside edge. The depth of these sections is set at a minimum of 36 inches and can be increased in 14-inch vertical increments to 50 and 64 inches, respectively. Significant versatility can be derived from this configuration in that these beams can conceivably be used in spans ranging from 40 feet all the way up to 130 feet. As mentioned in the previous section on materials and limitations, the deck bulb-tee girders were configured for two commonly used roadway widths. These clear roadway widths were 40 and 44 feet. The resulting beam top flange widths required to provide these two roadway widths is 85 inches for the 40-foot roadway and 93 inches for the 44-foot roadway. Six girders placed adjacent to each other will result in the required cross-section to provide the desired clear roadway width plus enough space to install standard barrier rails.

Also included in Figure 3-3 are the nominal section properties for the proposed sections. Finally, a maximum span is presented for each of the proposed sections. The maximum span was derived assuming that the beams are provided with a 5-inch minimum structural topping made composite with the top flange for live load. The maximum span listing also assumes that the 28-day concrete strength of the beams is a minimum of 8000 psi. Prestressing will be used in the deck bulb-tee girders. Pretensioning will reduce the vulnerability of the product to cracking during the manufacturing, shipping, and erection phases of a project.

As aforementioned, this superstructure system was selected because it offers acceleration of construction, enhanced safety, and flexibility. The primary way that it accelerates construction is by eliminating the need to pour a conventional cast-in-place bridge deck. It includes elimination of separate precast or metal deck panels between girders and the conventional slab overhang. In addition, the elimination of these field forming activities enhances safety. With respect to flexibility in construction, this system can be used in a number of configurations. It has been noted that the scope of this design was limited to 40-foot and 44-foot roadway widths, with simple spans up to 130 feet maximum and tangent alignment. However, this system can be used beyond these parameters. It is anticipated that this system can be used in tangent alignments with left or right ahead skew. Applications with horizontal curvature will be limited. It is also anticipated that the system can accommodate vertical grade and some small degree of vertical curvature. Finally, variations in the width of the top flange can be used to derive different
Figure 3-1. Typical bridge superstructure section and elevation
Figure 3-2. UAB precast bridge system: superstructure section and elevation
Deck Bulb Tee Girders

![Diagram of Deck Bulb Tee Girder](image)

**Dimensions and Properties**

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<th>W (in.)</th>
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<th>Inertia (in⁴)</th>
<th>Y_{bottom} (in.)</th>
<th>Weight (Kip/ft.)</th>
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* Based on simple span. HS-20 loading and f'_c = 8,000 p.s.i. Nominal Span Lengths shown. Brg to Brg. distance is (length shown - 2.0 ft.). Maximum span lengths are derived using a minimum 5" thick cast-in-place structural topping.

**Figure 3-3.** Deck bulb-tee girder and section properties
roadway widths or extension of spans for a particular depth of cross section. A method is being investigated to cast the cross slope of the roadway into the flange of the bulb-tee girder. This method will cut down the overall weight and depth of the required topping slab that will be cast over the top of the girders.

Flange-to-flange welded plate connections will be required for the deck bulb-tee girders. The function of this connection will be two fold. The first function will be to provide stability against rotation of the beams during placement of the concrete for the topping slab. The second function will be to provide live load shear transfer between adjacent girders. The edges of the flanges will be provided with female shear keys that will create a female-to-female shear key joint. The welded plate connections will provide the initial connections during erection of the girders prior to concrete placement. These keyways will be filled concurrently with the placement of the topping slab. A backer rod will be used in the bottom of the shear key to contain concrete.

The structural topping slab is used primarily for a quality riding surface. The preliminary girder designs for a 130-foot span indicated erection cambers of approximately 4.5 inches. Based on this camber, a structural topping will be needed to provide a level riding surface. The structural topping for the maximum span of 130 feet will range from 8 inches thick at the bearings to 5 inches thick at mid-span. The reinforcement required for the topping will be a single transverse mat and a single longitudinal mat. The transverse mat will function as structural reinforcement and the longitudinal mat will function as distribution reinforcement. Two inches of clear cover will be required for the transverse mat. The barrier rail reinforcement will be extended from the top flange of the fascia girder to facilitate installation of the standard barrier rail. The barrier rail will be constructed with slip form concrete. It is the opinion of the research team that since the proposed deck bulb-tee girder system requires no formwork, the proposed structural topping slab and slip-form barrier rail can be installed in a rapid manner. This configuration will also provide a higher quality bridge deck.

**Substructure**

Upon reviewing current ALDOT bridge construction practices, it was determined that the primary substructure components used in a typical bridge construction project are bents and abutments. Figure 3-4 depicts a typical bridge-bent configuration. The typical configuration consists of round or rectangular cast-in-place columns with cast-in-place concrete bent caps. As discussed previously, and shown in the figure, the bent’s foundation is typically a spread footing, drilled shaft, or a pile cap.

In determining a method to precast this particular type of substructure component, it became apparent that segments would have to be light enough and sized properly to enable transportation to the project site. A logical approach was to separate the bent into column and cap components. As discussed in the section on materials and limitations, reasonably sized sections were developed using this approach.

The first item developed was the precast column. After evaluating size and weight limitations, it was decided that the columns could be manufactured in a single piece. The proposed single
Figure 3-4. Typical cast-in-place bent and foundation types
piece column is rectangular with an internal void. This shape was chosen for its simplicity and functionality. The local precaster has had previous positive experience precasting whole rectangular concrete piles for use in bridge substructures. The internal circular void aids in reducing the weight of the individual columns for handling and erection purposes. The reduced element weight in turn reduces the overall dead load on the foundation and the bearing soil. Corners of the rectangular columns will be chamfered to keep the corners of the column from breaking and chipping during transportation and erection. Figure 3-5 shows the cross-sectional view and elevation view of a typical column for this system.

Column lengths were governed by two factors. The first factor was the slenderness ratio, set to a maximum value of 100 as illustrated by Eq. 3-1:

\[
\left( \frac{K \times L}{r} \right) \leq 100 \quad \text{Eq. 3-1}
\]

In Eq. 3-1 \(L\) is the length measured in inches. The radius of gyration, \(r\), is measured in inches. \(K\) is the effective length factor. An effective length factor of 2.0 was used for a fixed end-free end scenario. The radius of gyration for these calculations was determined using Eq. 3-2:

\[
r = (0.30 \times W) \quad \text{Eq. 3-2}
\]

In Eq. 3-2 \(r\) is the radius of gyration measured in inches. \(W\) is the width of the column measured in inches. This equation was taken from Section 8.16.5.2.2 in AASHTO Standard Specifications for Highway Bridge (AASHTO 2002). This section deals with the design of compression members. The following excerpt was taken from this section:

“The radius of gyration, \(r\), may be assumed equal to 0.30 times the overall dimensions in the direction in which stability is being considered for rectangular compression members and 0.25 times the diameter for circular compression members. For other shapes, \(r\) may be computed for the gross concrete section.”

The other main limiting factor in the column design was the column weight. The column weight for a single column was set at a value of 100,000 pounds. The maximum weight limit for a single column was chosen based on the maximum load that can be safely lifted by a single crane at the precasting plant. Figure 3-5 lists column cross-sectional dimensions, column weights, and the corresponding maximum lengths for each column size.

A mechanical coupling system will be utilized at the base of each column to create the connection at the column-foundation interface. Connecting dowels will extend from the foundation upward into the couplers that are cast into the base of the columns. Four feet of each end of a column will be solid to provide sufficient room for the mild steel reinforcement that will be utilized with the mechanical coupling system. Grouted sleeve couplers will be precast into the base of the column to create the void that will house the connecting dowels. In evaluating the requirements for the column-foundation connection, it was determined that the grouted sleeve connection offered the most flexibility with respect to ease of manufacturing and field erection. The particular product that was selected was the NMB splice-sleeve system. This product was selected because it already has an established history in this type of application. A particular example is use of the system in the construction of the Edison Bridge in Fort Myers, Florida.
Figure 3-5. UAB precast bridge system: precast column cross section, elevation, and properties
After the column is erected onto the projecting field dowels, it is secured with temporary bracing. A special grout that is part of the proprietary coupling system will be pumped into the mechanical grouted sleeve couplers to complete the connection between the foundation and the columns. After the appropriate set time for the sleeve grout, the temporary bracing may be removed.

A final feature of the precast column system is the prestressing. As mentioned previously, the tops and bottoms of the columns will be provided with mild steel at the ends for connection purposes. In addition, the columns will be pretensioned their entire length, similar to the methods used for prestressed concrete piles. Use of pretensioning will reduce vulnerability of the product to cracking during the manufacturing, shipping, and erection phases of a project. Preliminary details for the proposed columns can be found in Figures 3-6 and 3-7. It is noted that the design of the members will have to be verified before utilization in an actual project.

After development of the precast column section, consideration was given to precasting the bent caps. As discussed in the section on materials and limitations, the maximum weight for a precast cap section was limited to 150,000 pounds. This additional weight will mean that the product will require two handling vehicles at the precasting yard. However, this weight will allow the member to be transported to a jobsite with available hauling equipment. Mild steel reinforcement will be utilized in the bent caps. The shape selected for the cap section is rectangular. For purposes of this study, the depth of the cap was set at a constant 5 feet. The width of the cap may be varied in 6-inch increments from a minimum of 3 feet 6 inches to a maximum of 4 feet 6 inches. At the maximum cross-sectional dimension of 4 feet 6 inches wide by 5 feet deep, the precast bent cap may be cast up to 45 feet long. These dimensions will accommodate a 44-foot roadway bridge on a tangent section. As a weight reduction measure the cantilevers may be tapered down to a depth of 3 feet 6 inches at the tip of the cap. Figure 3-8 provides a general schematic of the precast cap configuration.

Bent caps are to be connected to the columns in the same manner that the columns are connected to the foundations. In this case, the dowels will project positively from the top of the precast columns. These dowels will extend vertically upward into grouted sleeve couplers that are precast into the bent caps. Grouting of the sleeves completes this connection. Temporary bracing of the precast cap is not required for this condition. Bearing pedestals for the bridge superstructure will be field cast on top of the precast bent. Figure 3-9 provides a schematic of the assembled precast bent.

As a point of interest, precast caps may be utilized when steel or concrete piles are left projecting above the ground line and used as columns. This method is common practice for short span bridges. Typically, a cast-in-place cap is poured around the projecting piles to provide support for the superstructure, as shown in Figure 3-10. A precast cap with recessed grout pockets could be used in lieu of the cast-in-place cap. Connection of the cap to the projecting piles is accomplished by filling the grout pockets with an epoxy grout. This proposed configuration can be seen in Figure 3-11.
Figure 3-6. UAB precast bridge system: precast column details

<table>
<thead>
<tr>
<th>Column Size</th>
<th>Void Diameter &quot;d&quot;</th>
<th>Weight per Lin. Ft. (kips/ft.)</th>
<th>Voided Moment of Inertia (in.²)</th>
<th>Voids per Cross Section (X Spaces)</th>
<th>Total Number of Strands</th>
<th>Initial Pretension (ksi)</th>
<th>Maximum Casting Length &quot;L&quot;</th>
<th>Mechanical Coupler Layout</th>
<th>Maximum # of Couplers</th>
<th>Mechanical Coupler Size</th>
<th>Dowel Size</th>
</tr>
</thead>
<tbody>
<tr>
<td>36&quot;</td>
<td>18&quot;</td>
<td>1.085</td>
<td>1,041.53</td>
<td>134.915</td>
<td>5</td>
<td>20</td>
<td>536</td>
<td>46'-0&quot;</td>
<td>3</td>
<td>#10</td>
<td>#9</td>
</tr>
<tr>
<td>42&quot;</td>
<td>24&quot;</td>
<td>1.366</td>
<td>1,311.61</td>
<td>243.022</td>
<td>6</td>
<td>24</td>
<td>511</td>
<td>60'-0&quot;</td>
<td>3</td>
<td>#14</td>
<td>#11</td>
</tr>
<tr>
<td>48&quot;</td>
<td>30&quot;</td>
<td>1.664</td>
<td>1,597.14</td>
<td>492.607</td>
<td>7</td>
<td>38</td>
<td>486</td>
<td>56'-0&quot;</td>
<td>4</td>
<td>#14</td>
<td>#11</td>
</tr>
<tr>
<td>54&quot;</td>
<td>36&quot;</td>
<td>1.977</td>
<td>1,889.12</td>
<td>620.140</td>
<td>8</td>
<td>32</td>
<td>470</td>
<td>46'-0&quot;</td>
<td>5</td>
<td>#14</td>
<td>#11</td>
</tr>
</tbody>
</table>

* Mechanical coupler is oversized to provide additional erection tolerance.

36" – 54" Column Details
**General Notes**

1. The concrete in the prestressed columns shall have a minimum of 5,000 psi compressive strength prior to receiving prestressing force and a minimum 28 day compressive strength of 6,000 psi.

2. Prestressing strands shall be ASTM A416 1/2" diameter 770 ksi. Low-relaxation strands. All strands shall have an initial tension of 30,000 lbs per strand.

3. All steel reinforcement shall be AASHTO M31 Grade 60.

4. Mechanical Couplers shall be NMB Splice-Sleeve system or approved equal.

36" - 54" Column End Details

*Figure 3-7. UAB precast bridge system: precast column end details*
Figure 3-8. UAB precast bridge system: precast bent cap
Figure 3-9. UAB precast bridge system: precast bent assembly and foundation types
Figure 3-10. Cast-in-place cap on driven piles
Figure 3-11. UAB precast bridge system: precast cap on driven piles
As mentioned earlier in the section, the other common substructure element is the abutment. Typically, bridge abutments are founded on driven piles, drilled shafts, or spread footings. A typical bridge abutment and foundation system can be seen in Figure 3-12. The abutment caps developed for this system will be a standard rectangular shape much like the bent caps. The abutments will be solid throughout excluding the mechanical couplers or grout pockets that will be cast into the base of the abutments for connection purposes. Abutment caps will be connected to the foundations in the same manner that bent caps are connected to columns unless driven piles are utilized for support. In this case a grout pocket will be cast into the abutment caps and the tops of the pile will extend up into the grout pocket. The grout pocket will then be formed and epoxy grout will be poured into the grout pocket to complete the connection between the piles and the abutment caps. The backwall, wingwalls, and pedestals for the abutment will be cast in the field. The proposed precast scheme is shown in Figure 3-13.

**Foundations**

Upon investigation of foundation systems currently used by ALDOT, it was found that there are several methods in use. Bridge abutments are typically founded on driven piles, spread footings, or drilled shafts. Cast-in-place columns are typically founded on pile caps, spread footings, or drilled shafts. In addition to these methods, on short spans, driven piles are allowed to project above the ground line and function as columns for bents. Caps are cast on top of the piles to provide bearing for the superstructure elements.

Due to constructability issues, it has been determined that these methods are currently effective and practical. No attempt has been made in this project to devise new foundation construction methods. All precast elements developed for this project begin above the completed foundation installation.

Foundations for this system vary between cast-in-place concrete and driven piles, depending on whether the foundation in question is for a column or an abutment. Column foundations are specified as cast-in-place concrete for spread footings, drilled shafts, and pile caps. Abutment cap foundations are specified as cast-in-place concrete for spread footings and drilled shaft foundations. When driven pile foundations are utilized, the abutment caps will be placed directly onto the driven piles. This decision was made on the basis of constructability. The surface of foundations must usually attain a specified elevation. The top surfaces of foundations must also be level to ensure the stability of elements placed on top of and connected to the foundations. These issues might become complicated and time consuming when trying to place a precast foundation. The surface of a cast-in-place foundation may be leveled and elevations may be adjusted by adding or removing concrete as needed.

Abutment caps and columns for this system will be connected to foundations through the use of connecting dowels that will extend upward from the foundation into the base of a column or abutment cap. Grouted sleeve couplers will be precast into the base of the columns or abutment caps to house the connecting dowels. In the case of driven piles for an abutment cap, a grout pocket will be precast into the base of the abutment cap.
Figure 3-12. Typical cast-in-place bridge abutment
Figure 3-13. UAB precast bridge system: precast abutment
System Handling, Transportation, and Erection

Handling, transportation, and erection schematics for each of the precast products can be seen in Figures 3-14, 3-15, and 3-16. The precast prestressed column sections will be manufactured with steel side forms and a steel soffit form. The side forms will be removable to allow stripping of the product from the production molds. The lift points for stripping and handling on the precasting yard will be located at a distance of 0.21 $L$ from the ends of the product, where $L$ is the overall length of the section. The location of lift points will balance the positive and negative self-weight moment. Lifting devices will be required at these locations. Lifting devices are typically 270-ksi prestressing stands bent into lifting eyes that are embedded into the concrete an adequate depth. The number of strands required is a function of the total weight of the product. Guidelines for the required number of strands and embedment depth can be found in the *PCI Design Handbook*.

All products will be delivered via truck transport to the jobsite. Dunnage supports for the precast columns while on the transport vehicle will also be located at a distance of 0.21 $L$. Once at the jobsite, the columns will be erected with a one-point pickup at either a distance of 0.30 $L$ from the end or at the end of the column. A lifting device will be required at this location as well. Once vertical, the columns will require temporary bracing on two faces. One face will be in plane with the bent and one face out of plane with the bent.

After the column-foundation connection is made as previously described, the precast bent caps may be erected. The precast bent caps will be manufactured on the same type of form system as the columns with the exception that the bent caps will be mild steel reinforced only. The precast bent caps will be stripped and erected with lifting devices located at a distance 0.21 $L$ from each end. The transportation dunnage support for the precast bent caps will also be located at 0.21 $L$. Installation of the precast bent cap will be as described in the previous section. No temporary bracing is required. Upon completion of installation of the precast bent caps and field casting of the pedestals, the precast prestressed girders may be installed. The prestressed concrete girders will require a special rollback side form system. Another feature of this form will be the ability to adjust the cross slope at the top flange. Prestressed concrete girders are typically lifted by special lifting devices located near the end of the beam. Typically, the lift and storage location points for the prestressed girders will be approximately 0.10 $L$ from the end of the beam. Bolster supports for vehicle transport are typically located at or near the lifting device. Erection of the girders is accomplished using the same lifting device as was used in stripping of the product. Once the girders are in place, flange-to-flange connections will be made and precast diaphragms may be installed at the ends of the span. The structural topping reinforcement and slab placement will follow. Finally, the standard barrier rails will be cast-in-place to complete the superstructure construction.
Figure 3-14. Column handling

2 Point Pickup Stripping

2 Point Storage & Transportation

1 Point Pickup Erection

1 Point Pickup Erection – (Alternate)

NOTE: Temporary Bracing
Reqd. Two Faces of Column (In-Plane & Out-of-Plane)

Temporary Bracing
Before Grouting
Mechanical Couplers

Precast Columns (Stripping, Storage, Transportation & Erection)
Figure 3-15. Bent cap handling
Figure 3-16. Girder handling

2 Point Pickup Stripping

2 Point Storage & Transportation

2 Point Pickup Erection

Prestressed Girder (Stripping, Storage, Transportation & Erection)
Section 4  
Design Example

General

A design example is included in this report to illustrate a typical design of the proposed UAB precast bridge system intended for use in Alabama in bridge construction. The example provides some practical suggested details for the proposed system. The design specifications used in the design example is the *AASHTO Standard Specifications for Highway Bridges* (AASHTO 2002). The design live load vehicle is HS 20-44, which is the design live load vehicle commonly used by ALDOT for bridge design. Reinforced concrete members were designed using the service load method as outlined in Section 8 of the AASHTO specifications. Design of prestressed concrete members is covered by Section 9 of the specifications.

Design Approach

Commercially available software was used to design selected components in the example. The computer program CONSPAN, licensed by LEAP Software (CONSPAN 2005) of Tampa, Florida, was used to design the prestressed concrete girders. The design method used by the program is the AASHTO load factor design method (LFD) (AASHTO 2002). As required by the design code, the prestressed concrete girders were checked for both the service load and ultimate load conditions. The computer program RC-Pier (RC-Pier 2005), also licensed by LEAP Software of Tampa, Florida, was used in the design of the precast bents. The precast caps are reinforced concrete members designed using the service load method. The program was also used to evaluate the mild steel connections at the base and top of the columns. The RC-Pier Program does not have the ability to address the design of prestressed concrete members. Therefore, an additional check was required for the precast prestressed columns that utilize a combination of both mild steel reinforcement and prestressed strand. The software tool used to check the pretensioning of the column was an Excel spreadsheet developed by PCI. The spreadsheet is entitled “PCI Prestressed Concrete Pile Interaction Diagram Spreadsheet.” This program was used to plot an interaction diagram for the precast prestressed column. This program does not handle biaxial moments, so the x-axis and z-axis moments were converted into a single resultant moment. The service axial load and resultant moment were factored by 2.5 for evaluation on the interaction diagram developed by the spreadsheet.

Bridge Structural Layout and Dimensions

The structure selected for the design example is a simple span grade separation bridge structure consisting of three spans with the following lengths: 60, 130, and 60 feet. This structure is...
This type of structure was selected because it will be a common application for the proposed system. The configuration of the design example’s grade separation bridge is frequently used by ALDOT. It consists of two end approach spans and a main span that provides a standard 17 feet minimum vertical clearance over an existing roadway. A 130-foot main span was selected because it is the maximum span length evaluated for this project.

Column heights at bents two and three will be approximately 20 feet. The foundations shown for the design example are cast-in-place concrete pile caps for the bents and driven piles for the abutments. Details of the abutments are not included in the design example.

The clear roadway width selected for the design example is 44 feet, which is also the maximum roadway width considered in the project. It is also the maximum roadway width considered based on ALDOT practices for two lane roads (ALDOT 2003). Bent caps for roads that exceed two lanes or 44 feet could not be transported to the project site via truck and trailer due to excessive weight. This roadway configuration requires the use of six girders in the cross section. Figure 4-2 shows a typical section for the design example. It can be seen from the figure that the cross slope for the deck is two percent with a six-foot parabolic crown at the center of the bridge. Also, the tops of the girders are sloped at two percent to give a uniform depth for the structural topping in the transverse direction.

The selected girder size is controlled by the length of the 130-foot middle span, which is the maximum length for sections developed for this project. This span length requires the use of the 64-inch deck bulb-tee girders. In order to give the superstructure a uniform appearance, the end spans were also designed using the 64-inch sections.

**Design Details**

The design process begins with the upper most element, the topping slab, and ends with the design of the foundation. The topping slab depth varies for the design example from 8 inches thick at the supports to 5 inches thick minimum at mid-span. The reinforced top flange of the girder is designed to carry the dead load of the topping plus a construction live load of 25 pounds per square foot. In addition, the top flange reinforcement must be adequate to carry the positive live load moment. The positive live load moment is present with the assumption that the flange-to-flange connection of the beams acts as a hinge. The reinforced topping slab is designed to be composite with the top flange of the girder. The transverse steel reinforcement in the topping is designed to carry the negative live load moment. The negative live load moment is distributed and computed based on equations found in Section 3.24.5.1.1 of the AASHTO standard specifications (AASHTO 2002). Use of these equations was based on the assumption that the composite slab and girder flange behave as a cantilevered slab projecting from the face of the girder web. This approach ignores the flange-to-flange girder connection and should be conservative with respect to the magnitude of the computed negative live load moment. Longitudinal distribution reinforcement is provided for the topping slab at a quantity equal to 67 percent of the provided main transverse reinforcement per Section 3.24.10.2 of the AASHTO
Design Criteria:
Design Live Load: HS 20–44
44’-0” Roadway Width with Standard Barriers

Figure 4-1. Bridge layout for the design example
Figure 4-2. Typical cross-section for design example
standard specifications (AASHTO 2002). Details of the topping slab can be seen in Figure 4-3. A proposed detail for the slab over the exterior girder is also shown in this figure.

Figures 4-4 through 4-10 depict the design of the girders for each of the three spans. Designs for all three spans utilize 0.6-inch diameter prestressing strand. The design sketches indicate the required strand patterns and reinforcement schemes for shear and top flange reinforcement. It is noted that with the two percent cross slope cast into the top of the girder, the beam is actually 65 inches deep at the centerline. Included with the girder details is a flange-to-flange connection and a proposed detail at the open joints. These details can be seen in Figures 4-11 and 4-12, respectively. The design example does not address a mid-span diaphragm detail. However, based on the requirements outlined in Section 9.10 of the AASHTO standard specifications (AASHTO 2002), midspan diaphragms will be required for spans over 40 feet in length. Due to accessibility and construction issues raised by the adjacent top flange configuration of the deck bulb-tee girders, a structural steel bolted diaphragm might offer the most economical and practical solution for this condition.

The precast reinforced bent cap for the design example is 4 feet 6 inches wide by 5 feet deep. The member was designed based on the design loadings outlined in the AASHTO specifications. The particular load combinations used for this case were Service Groups I, II, and III. Details for the precast cap can be seen in Figures 4-13 and 4-14.

The selected column size for the design example is 48 inches by 48 inches. As mentioned above, the columns are mild steel reinforced and pretensioned. Figures 4-15 through 4-17 depict column details and connection of the column to the cast-in-place foundation. The foundations are not designed in this system because current methods used by ALDOT are effective and practical. A typical foundation for this system will consist of a cast-in-place pile cap on top of driven shaft piles. Other common foundation types, such as spread footings or drilled shafts, may be used depending on site conditions.
Figure 4-3. Topping slab

* BARS SHOWN HERE SIMILAR TO BARRIER RAIL REINFORCEMENT BARS B2 #5 & B3 #5 (DEPICTED ON ALDOT STD. DWG. I-131). BARRIER REINFORCEMENT STEEL SPACING & ARRANGEMENT WILL BE SIMILAR TO SCHEMATIC DEPICTED ON STD. I-131 (SHEET 3 OF 8)
Figure 4-4. End span girder, strand pattern
Figure 4-5. End span girder, reinforcement
Figure 4-6. End span girder, steel layout
Figure 4-7. Middle span girder, strand pattern
Figure 4-8. Middle span girder, reinforcement
Figure 4-9. Middle span girder, steel layout
Figure 4-10. Steel details
Figure 4-11. Flange-to-flange connection details

NOTE: SPACING OF ABOVE FLANGE-TO-FLANGE CONNECTION SHOULD NOT EXCEED 15'-0" CENTER TO CENTER.

64" DECK BULB TEE GIRDER
FLANGE-TO-FLANGE CONNECTION DETAIL
Figure 4-12. Open joint details
Figure 4-13. Precast cap elevation and details
Figure 4-14. Precast cap sections
12 @ #11 PROJECTING DOWELS.
SAME ARRANGEMENT AS
FOUNDATION PLAN. ERECT PRECAST
CAP ONTO THESE DOWELS.

#11 FIELD DOWELS
(12 REQ'D) CAST-IN-FOUNDATION
WITH TEMPLATE FOR REQUIRED
DOWEL SPACING SEE FOUNDATION
PLAN VIEW.

MECHANICAL COUPLER

CONVENTIONAL
CAST-IN-PLACE
FOUNDATION

48" COLUMN

Driven Piles
Figure 4-16. Precast column sections
MECHANICAL COUPLERS SHALL BE NMB SPlice-SLEEVE SYSTEM OR APPROVED EQual SHIMMING, RED GROUT (\(\frac{3}{4}''\)), AND GROUTING OF SLEEVEs SHALL BE IN ACCORDANCE WITH MANUFACTURER'S PUBLISHED RECOMMENDATIONS.

TYPICAL MECHANICAL COUPLER DETAIL

* DOWEL PROJECTION ABOVE FOOTING = 1'-0\(\frac{1}{2}''\)

Figure 4-17. Mechanical coupler detail and foundation plan
Section 5
Summary And Conclusions

Summary

Prefabricated precast concrete bridge systems are a growing technology to aid in rapid bridge construction to minimize traffic impacts. Prefabrication also improves the quality of the finished product because elements are manufactured under controlled conditions in the precasting plant and high-performance materials may be used during casting. Prefabrication also improves safety in the work zone by reducing the amount of activity that is required over traffic or at high elevations to construct the bridge.

An extensive literature review was performed to assess the current practices in prefabricated precast concrete bridge construction and to identify viable prefabricated concrete bridge systems for construction of grade separation structures in Alabama. The elements and systems identified in the literature review were categorized according to type and examined to determine the most appropriate systems that were suitable for use in Alabama. Once this task was performed the preliminary design phase of the project began.

A proposed totally precast bridge system, referred to as the UAB precast bridge system, was developed. Precast element shapes were designed based on the previous experience and current casting ability of the local precaster. Size limitations were determined according to the handling and lifting abilities of equipment at the local precasting yard. With these factors taken into consideration, a superstructure system consisting of adjacent deck bulb-tee girders with a topping slab was developed. The substructure consists of precast rectangular bent caps resting on rectangular precast prestressed voided columns. A conventional foundation system of driven piles and cast-in-place pile caps was determined to be the most suitable foundation system for this project. Connection methods were analyzed, and a proven mechanical coupling system was selected to connect columns to foundations and bent caps to columns. Welded plate connections were utilized in conjunction with a female-female grouted shear key joint to connect adjacent deck panels for the superstructure system. Once the preliminary phase of the project was completed, a design example was performed to finish the project.

For the final stage of the research project, a detailed design example illustrating the structural design of the proposed UAB precast bridge system was performed. The objective of the design example is to illustrate the feasibility of designing and constructing the proposed UAB precast bridge system. The design example covers a three-span bridge with a center span of 130 feet and end spans of 60 feet. The calculations were performed in accordance with AASHTO Standard Specifications for Highway Bridges. The design vehicle live load was a HS 20-44. The clear roadway width for the design example bridge was 44 feet. Six deck bulb-tee girders that were 64
inches deep with 7-foot-9 inch wide top flanges were placed adjacent to one another and connected with the shear key joint mentioned previously to create the superstructure for the design bridge. A topping slab of cast-in-place concrete was designed for placement over the top flanges of the girders to create the riding surface of the deck. 44-foot-7-inch long bent caps that were 5 feet tall and 4 feet 6 inches wide support the superstructure for the bridge. The bent caps are connected to the columns with the NMB mechanical coupling system described in the section on substructures. Column heights for the design bridge were set at 20 feet with a cross-section of 48 inches by 48 inches. Columns are connected to the 9-foot-6-inch by 9-foot-6-inch pile cap with the same NMB mechanical coupling system. The 20-foot columns provided for a vertical clearance of 17 feet over the existing roadway in the example.

In summary this project has assessed the current trends in prefabricated bridge construction and developed a working prefabricated bridge system for use in Alabama to accelerate bridge construction.

Conclusions

The literature review indicates that prefabricated bridge construction is a growing technology, which can be used for new projects as well as bridge rehabilitation. Prefabricated bridge construction will result in rapid construction and minimized delays to the traveling public. It was determined that project durations were highly accelerated depending on the level of prefabrication. The shortest project durations identified during the course of the study were the Baldorioty de Castro Avenue Overpasses being constructed in just 36 and 21 hours for the first and second overpass, respectively.

A feasible prefabricated precast concrete bridge system, referred to as the UAB precast bridge system and considered suitable for use in Alabama, was developed under close communication with the local precaster. With the development of this prefabricated precast concrete bridge system, this type of technology may be applied in Alabama to bridge projects that require rapid construction or rehabilitation.

Recommended Research

Construction of a prototype bridge, using the UAB precast bridge system, is recommended to study the actual performance of the proposed system in a real world application. As a first step, the proposed design will be reviewed with ALDOT engineers to obtain their comments and suggestions. These comments will be incorporated to produce a revised design of the proposed system. It is anticipated that this system will result in significant savings in construction time, and as a result, minimize economic loss due to vehicular traffic downtime. An economic analysis will be performed as part of this prototype construction project.

A second recommendation, and the next logical step, would be constructing a prototype secondary road structure using the proposed system. For example a three-span bridge with minimum span lengths of 60 feet would be the most likely scenario. Although the system is
designed for a 40 or 44-foot roadway, it can be adapted to a 28-foot roadway, which is typical for a secondary road bridge. Actual field conditions may help determine the level of efficiency and speed of construction that this system may offer. Additional research funds will be needed to complete the recommendations of this project.
Section 6
List Of References


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*RC-Pier Version 4.0.1*, LEAP Software, Inc., 11602 North 51st Street, Tampa, Florida 33617.


Section 7
Abbreviations

ALDOT Alabama Department of Transportation
AASHTO American Association of State Highway Transportation Officials
ASCE American Society of Civil Engineers
FHWA Federal Highway Administration
NCHRP National Cooperative Highway Research Program
PCI Precast/Prestressed Concrete Institute
SPER Sumitomo’s Precast Form Method for Resisting Earthquake and Rapid Construction
UAB The University of Alabama at Birmingham