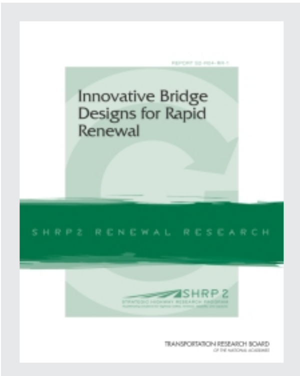

Innovative Bridge Designs for Rapid Renewal

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HNTB Corporation

The Second
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Innovative Bridge Designs for Rapid Renewal

HNTB Corporation

Genesis Structures, Inc.

Structural Engineering Associates

Iowa State University

TRANSPORTATION RESEARCH BOARD

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The research reported on herein was performed by HNTB Corp. with Kenneth Price as Principal Investigator and Bala Sivakumar as co-Principal Investigator. Also providing support were Genesis Structures (Kansas City, Missouri), Structural Engineering Associates (San Antonio, Texas), and Iowa State University (Ames, Iowa). The authors gratefully acknowledge those individuals from state departments of transportation, industry organizations, contractors, and academia who participated in the project survey and focus group meetings, and who provided important information and documentation for this project.

FOREWORD

Monica A. Starnes, *Senior Program Officer, Renewal*

As the nation's bridge inventory continues aging and the need for its renewal increases, new approaches on how to design and build bridges are paramount. This need combined with increasing traffic congestion will require the implementation of faster and less disruptive construction methods. Accelerated Bridge Construction (ABC) techniques have proved their ability to fulfill these needs in some unique bridge projects and, most importantly, in a limited number of statewide bridge programs such as Utah's.

While the key for successful implementation of ABC on a large scale requires a range of technical and programmatic solutions, one mechanism that has proved successful in implementing past bridge innovations is the idea of standard concepts and, in some cases, standard plans. This SHRP 2 project started its research with an ultimate goal of developing a set of such standard concepts.

The project first focused on identifying and evaluating the historic barriers to prevalent use of ABC. Based on the assessment, the research team led by HNTB developed a set of technical solutions to overcome those identified barriers. The solutions were directed toward modular (i.e., prefabricated) bridge substructure and superstructure systems that (1) can be installed with minimal traffic disruptions and (2) can be easily constructed by local contractors using conventional equipment. With those goals in mind, the research team set out to develop new structural concepts by incrementally improving on proven and accepted structural systems, components, and details. Structural evaluations, analyses, designs, and laboratory testing provided the tools to achieve the sought improvements.

The *ABC Toolkit* was produced with bridge practitioners in mind. It provides a series of design and construction concepts for prefabricated elements and their connections. In response to the scope of work, the toolkit also provides proposed language for AASHTO design and construction specifications. This report, a companion to the toolkit, details research into ABC approaches and offers insight into various components that can be used in accelerated bridge construction.

Since the initiation of this research in 2007, other ABC-related programs have either matured (e.g., Utah DOT's ABC program) or been established (e.g., FHWA's Every Day Counts, EDC) in parallel. While the SHRP 2 *ABC Toolkit* and R04 report provide concepts for designing and building complete bridges, these tools are not meant to be the complete collection of information on ABC or prefabricated bridge elements and systems (PBES), but rather an additional resource that complements the body of knowledge and other publications on the subject.

CONTENTS

Executive Summary

This report documents and presents the results of SHRP 2 Renewal Project R04, Innovative Bridge Designs for Rapid Renewal, to develop standardized approaches to designing and constructing complete bridge systems for rapid renewals. Bridge deterioration and the need for replacement continue to be ongoing problems in the United States. Accelerated bridge construction (ABC) techniques have the potential to minimize traffic disruptions during bridge renewals, to promote traffic and worker safety, and to improve the overall quality and durability of bridges.

Accelerated bridge construction entails prefabricating as many bridge components as feasible. Minimizing road closures and traffic disruptions is a key objective of ABC. For ABC systems to be viable and see greater acceptance, savings in construction time should be clearly demonstrated. Successful use of prefabricated elements to accelerate construction requires careful evaluation of the requirements for the bridge, site constraints, and an unbiased review of total costs and benefits.

Accelerated bridge construction applications in the United States have developed two different approaches: accelerated construction of bridges in place using prefabricated systems and the use of bridge movement technology and equipment to transfer a completed bridge from an off-alignment location to its final position. Rapid construction of bridges in place offers the promise of limited closures, possibly for days or weeks, during complete construction of a bridge. This type of construction traditionally relies on extensive prefabrication of bridge elements, including substructure and superstructure components, and the use of cranes to install these elements into their final locations. An alternative to rapid construction in place is the use of preassembled bridges, completed at off-alignment locations and then moved by various methods using techniques such as lateral sliding, rolling, and skidding; incremental launching; and movement and placement using SPMTs (self-propelled modular transporters) into the final locations.

A key objective of this project is to identify impediments and obstacles to greater use of ABC and to seek solutions to overcome them. Focus group meetings and owner surveys identified several factors that have contributed to the slow adoption of ABC in the United States. Despite life-cycle cost savings and the gradual lowering of costs, departments of transportation are hesitant to use ABC techniques because of their higher initial costs. Another great impediment to rapid construction is the slow process of custom engineering every solution. Rather than custom solutions, pre-engineered modular systems configured for conventional construction equipment could promote more widespread use of ABC through reduced costs and increased familiarity of these systems among owners, contractors, and designers.

An objective of the SHRP 2 R04 project was to develop "standardized approaches to designing and constructing complete bridge systems for rapid renewals." The aim, therefore, is to develop pre-engineered standards for modular bridge substructure and superstructure systems that can be installed with minimal traffic disruptions in renewal applications. This project takes the approach that for ABC to be successful, ABC designs should allow maximum opportunities for the general contractor to do its own prefabrication, as there is reluctance among contractors to outsource much work to precasters. In this regard, the R04 team has determined that ABC systems should be

- As light as possible,
- As simple as possible, and
- As simple to erect as possible.

To get the maximum advantage possible from the speed of on-site construction with prefabricated bridge installations, consideration should be given to using complete prefabricated bridge systems, including foundations and substructures. In many cases, foundation and substructure construction is the most costly and time-consuming part of constructing a bridge. This project provides design standards for complete prefabricated bridge systems, including superstructure and substructure systems and foundation strategies for shallow and deep foundation systems in the context of ABC projects.

A key objective of this project is to identify impediments and obstacles to greater use of ABC and seek solutions to overcome them. These challenges can be met and successfully addressed if owners, designers, and contractors innovate incrementally and collaboratively. Building on previous experience and constantly pushing the envelope will result in continued successes.

Project R04 involved three distinct phases performed over a period of 4 years. Phase I, consisting of Tasks 1 through 5, was completed in November 2009. In this phase, the team collected extensive data on ABC projects and identified current impediments and challenges to greater use of ABC by bridge owners. Phase II comprised Tasks 6 through 9 and was completed between December 1, 2009, and December 31, 2010. The findings and ABC concepts from Phase I were subjected to critical evaluations in Phase II to identify concepts that could be advanced to standard plans in Phase III. Work on Phase III commenced on January 1, 2011, and was completed in March 2012. A synopsis of project activities follows.

The literature review consisted of gathering published data and reviewing innovative design and construction concepts that involved many project examples demonstrating different approaches to ABC. Sixteen ABC design concepts developed in the Phase I investigations included new concepts or adaptations of existing concepts that are proposed as solutions to various ABC problems. ABC design concepts have been classified into five tiers on the basis of mobility impact time as follows:

Tier 1: Traffic impacts within 1 to 24 hours.
Tier 2: Traffic impacts within 3 days.
Tier 3: Traffic impacts within 2 weeks.
Tier 4: Traffic impacts within 3 months.
Tier 5: Significant reduction of overall project schedule by months or years.

The work in Phase II incrementally winnowed the collected findings and ABC concepts from Phase I through screening and further evaluation. Phase II consisted of an engineering and constructability evaluation of the concepts, as well as identification of obstacles to implementation of various Phase I concepts. These evaluations provided recommended ABC concepts and techniques that could be advanced to standard plans and field trials. Phase II proposed a short list of concepts that could be advanced to design standards and the implementation phase. From this short list, standard plans were developed for the most useful technologies that could be deployed on a large scale in bridge replacement applications. They included complete prefabricated modular systems as outlined here:

- Precast modular abutment systems
  - Integral abutments; and
  - Semi-integral abutments.

- Precast complete pier systems
  - Conventional pier bents; and
  - Straddle pier bents.
- Modular superstructure systems
  - Concrete deck bulb tees;
  - Concrete deck double tees; and
  - Decked steel stringer system.

Typical designs for superstructure and substructure modules have been grouped into the following span ranges:

- 40 ft ≤ span ≤ 70 ft;
- 70 ft ≤ span ≤ 100 ft; and
- 100 ft ≤ span ≤ 130 ft.

To assist the owners and engineers with implementation of ABC, a goal was established to develop a set of standard conceptual details demonstrating the possibilities and limits of ABC erection technologies. Guidelines were also provided for conventional erection of ABC systems by using cranes. The erection concepts presented in the drawings were intended to assist the owner, the designer, and the contractor in selecting suitable erection equipment for the handling and assembly of prefabricated modular systems. Another task entailed identifying any shortcomings in the current AASHTO LRFD Bridge Design Specifications that may limit their use for ABC designs and making recommendations for addressing such limitations. The primary deliverable was recommended specification language for ABC systems, suitable for future inclusion in the LRFD Bridge Design Specifications.

The ease and speed of construction of a prefabricated bridge system in the field is paramount to its acceptance as a viable system for rapid renewal. In this regard, the speed with which the connections between modules can be completed has a significant influence on the overall ABC construction period. Investigations of joint types and material options have identified full moment connection using ultra-high-performance concrete (UHPC) joints as the preferred connection type for modular superstructure systems to satisfy the criteria for constructability, structural behavior, and durability. The properties of UHPC make it possible to create small-width, full-depth closure pour connections between modular components. These connections may be significantly reduced in size as compared with conventional concrete construction practice and could include greatly simplified reinforcement designs. A lab testing program was carried out at Iowa State University in Phase III of this project to further evaluate the performance of UHPC in ABC applications. The tests evaluated the strength, serviceability, and constructability of both longitudinal and transverse UHPC joints.

The first ABC demonstration project under SHRP 2 Project R04 was completed in Phase III in late 2011. The project consisted of replacing the bridge located on US-6 over Keg Creek in Pottawattamie County, Iowa. The replacement structure is a three-span steel/precast modular bridge with precast bridge approaches. The principal objective of the project was to demolish and replace the existing bridge within the 14-day ABC period by using ABC standard plans developed in this project. A daylong Highways for LIFE (HfL) workshop, including a site visit, occurred during the critical accelerated bridge construction period and provided an opportunity to disseminate information to bridge owners from around the country. The workshop highlighted the innovative design and construction features advanced by this project.

CHAPTER 1

Background

Problem Statement and Research Objective

Bridge deterioration and replacement continue to be ongoing problems in the United States. Accelerated bridge construction (ABC) techniques have the potential to minimize traffic disruptions during bridge renewals, promote traffic and worker safety, and also improve the overall quality and durability of bridges. ABC has been used on emergency replacement projects as well as on planned bridge replacement projects. While most agencies are aware of ABC, very few practice it on a large scale.

Accelerated Bridge Construction (ABC) applications in the United States have developed two different approaches: accelerated construction of bridges in place using prefabricated systems and the use of bridge movement technology and equipment to move completed bridges from an off-alignment location into their final position.

Rapid construction of bridges in place offers the promise of limited closures, days or weeks at most, to allow for the complete construction of a bridge. This type of construction traditionally relies on extensive prefabrication of bridge elements, including substructure and superstructure components, and the use of cranes to install these elements in their final location, along with innovative contracting and procurement techniques. Special connections designed to integrate elements into a completed bridge are made and the bridge can be opened to traffic in a very short period of time.

An alternative to rapid construction of bridges cast in place is the use of preassembled bridges, completed at an off-alignment location and then moved via various methods into the final location using techniques such as lateral sliding, rolling, and skidding; incremental launching; and movement and placement using SPMTs (self-propelled modular transporters). This can be done, depending on complexity, in minutes, hours, or at most, several days. Very large and complex structures can be erected in this fashion; the method distinguishes itself from rapid in-place construction in the scale of the potential project that can be undertaken and the use of complete fabrication at off-site locations.

ABC has yet to gain significant traction in the United States. A key objective of this project is to identify impediments and obstacles to greater use of ABC and to seek solutions to overcome them. Focus group meetings held with representatives from more than 20 departments of transportation (DOTs) as part of Phase I of the SHRP 2 R04 project identified several factors that have contributed to the slow adoption of ABC. Despite the gradual lowering of costs and life-cycle cost savings, DOTs are hesitant about using ABC techniques because of their higher initial costs.

A great impediment to rapid construction is the slow process of custom engineering every solution. Rather than customizing every solution, pre-engineered modular systems configured for traditional construction equipment could promote more widespread use of ABC through reduced costs and increased familiarity of these systems among owners, contractors, and designers. An objective of the ongoing SHRP 2 R04 project, with HNTB as the prime contractor, is to develop "standardized approaches to designing and constructing complete bridge systems for rapid renewals." The aim therefore is to develop pre-engineered standards for modular bridge substructure and superstructure systems that can be installed with minimal traffic disruptions in renewal applications.

The research objective for the R04 project is as follows:

> To develop standardized approaches to designing, constructing, and reusing (including future widening) complete bridge systems that address rapid renewal needs and efficiently integrate modern construction equipment.

ABC entails prefabricating as many bridge components as is feasible considering site and transportation constraints. The successful use of prefabricated elements to accelerate construction requires careful evaluation of bridge requirements, site constraints, and an unbiased review of total costs and benefits. This project takes the approach that for ABC to be successful, ABC designs should allow maximum opportunities for general contractors to do their own prefabrication at staging areas adjacent to project sites or in their yards using their crews. The R04 team has determined that ABC systems should be

- As light as possible,
- As simple as possible, and
- As simple to erect as possible.

In many cases, foundation and substructure construction is the most costly and time-consuming part of constructing a bridge. To get maximum speed and advantage possible from on-site construction with prefabricated bridge installations, consideration should be given to using prefabricated components for foundations and substructures. A total substructure system may consist of modular abutments and walls and prefabricated bent caps supported by prefabricated columns. This project aims to provide design standards for complete prefabricated bridge systems, including foundation strategies for shallow and deep foundation systems in the context of ABC projects.

In past years, the potential weak link in prefabricated systems has been the connections between components. Whereas the prefabricated components are constructed in controlled environments, the closure joint construction is exposed to variability inherent in field construction. Transverse and longitudinal deck closure joints are the biggest challenge to achieving long-term durability with minimum maintenance and rideability and smoothness requirements. Recent advances in high-performance materials have introduced a new generation of connections that are more durable and also low in maintenance.

Scope of Study

As noted, a part of the mandate of the ongoing SHRP 2 Project R04, Innovative Bridge Designs for Rapid Renewal, is to develop standard plans and details for promoting more widespread use of accelerated bridge construction. A key objective of this project is to identify impediments and obstacles to greater use of ABC and to seek solutions to overcome them. These challenges can be met and successfully addressed if owners, designers, and contractors innovate incrementally and collaboratively. Building on previous experience and constantly pushing the envelope will result in continued successes.

Project R04 is composed of three distinct phases over a time period of four years. Phase I, consisting of Tasks 1 through 5, was completed in November 2009. In this phase, the team collected extensive data on ABC projects and identified current impediments to and challenges in greater use of ABC by bridge owners. Phase II comprised Tasks 6 through 9 and was completed between December 1, 2009, and December 31, 2010. The findings and ABC concepts from Phase I were subjected to critical evaluations in Phase II to identify concepts that could be advanced to standard plans in Phase III. Work on Phase III commenced in January 1, 2011, and was completed in March 2012. A synopsis of project activities follows.

The literature review consisted of gathering published data, and review of innovative design and construction concepts involved many project examples demonstrating different approaches to ABC. The following topics were included in the discussion: complete bridge systems and component-level solutions, such as innovative superstructure systems; rapid bridge deck installation and replacement concepts; substructure options; and rapid foundation construction concepts for both shallow and deep foundations. The second major focus of the literature review was the use of innovative construction methods as a solution to rapid bridge construction. These topics included bridge movement using self-propelled modular transporters, lateral sliding and skidding of completed bridges, the use of incremental launching alone or combined with other techniques, and other innovative construction techniques.

A variety of techniques were used to determine industry opinions and experience about ABC. Electronic web-based surveys, phone interviews, focus group teleconferences and meetings, in-person visits, and various e-mail communications were all used to solicit opinions from owners, engineering designers, contractors, fabricators, vendors, specialty contractors, and specialty engineers. The surveys and communications in general provided data about past experiences, successes and failures of prior projects, institutional obstacles, and responses to other questions that helped the team understand how ABC is used in various locations throughout the country.

Sixteen ABC design concepts developed by the R04 team from the Phase I investigations were described with concept sketches and photographs and then advanced to Phase I evaluations. They include new concepts or adaptations of existing concepts that are proposed as solutions to various ABC problems. The concepts cover two broad themes pertaining to rapid renewal.

- Proven concepts, and
- New and innovative concepts.

These concepts are categorized as follows. They include new concepts, or adaptations of existing concepts, that are proposed as solutions to various ABC problems.

- Modular superstructure systems;
- Segmental superstructure systems;

- Precast decks;
- Precast modular abutment systems;
- Precast complete pier systems;
- Segmental columns and piers;
- Above-deck driven carriers;
- Launched temporary truss bridge;
- SPMTs and other wheeled carriers; and
- Launching, sliding, and lateral shifting.

ABC design concepts have been classified into five tiers, based on mobility impact time, as follows:

- Tier 1: Traffic impacts within 1 to 24 hours.
- Tier 2: Traffic impacts within 3 days.
- Tier 3: Traffic impacts within 2 weeks.
- Tier 4: Traffic impacts within 3 months.
- Tier 5: Overall project schedule is significantly reduced by months to years.

Phase II incrementally winnowed the collected findings and ABC concepts from Phase I through screening and further evaluations. It consisted of an engineering and constructability evaluation of the concepts as well as an identification of obstacles to implementation of the various Phase I concepts. The purpose of these evaluations was to provide recommended ABC concepts or techniques that could be advanced to standard plans and field trials in Phase III. Any technology recommended for field trials needed to meet minimum standards of readiness for execution, provide a promise of durability, and provide value to the owner. Phase II proposed a short list of ABC concepts that could be advanced to the design and implementation phase.

The first ABC demonstration project under R04 consisted of replacing the bridge located on US-6 over Keg Creek in Pottawattamie County, Iowa. The replacement structure was a three-span steel/precast modular bridge with precast bridge approaches. The design was performed in Phase II, and the construction was completed in Phase III. The principal objective of the project was to demolish and replace the existing bridge within the 14-day ABC period. A daylong Highways for LIFE (HfL) workshop included a site visit that occurred during the critical accelerated bridge construction period and provided an opportunity to disseminate information to bridge owners around the country. The workshop highlighted the innovative design and construction features advanced by this project. This demonstration project can affect the future practices of the industry and the U.S. Department of Transportation, as technologies that are successfully implemented on this project could accelerate the adoption of ABC innovations in the United States. Acceleration could be accomplished by creating awareness and educating stakeholders about innovative features, which would increase stakeholder confidence in recommending ABC use on other projects.

Several different areas of testing needs for ultra-high-performance concrete (UHPC) joints were identified during the design of the demonstration project. UHPC joint testing was performed by Iowa State University, a member of the R04 team. The testing covered the constructability and grindability of the UHPC joints and negative bending strength of the transverse pier joints.

As noted, one goal of this research project is to develop an *ABC Toolkit* for designers to foster greater use of ABC in bridge renewal and widening projects. The *ABC Toolkit* will be composed of ABC design standards for substructure and superstructure systems, design examples, and sample specifications. In Phase III, the research team developed pre-engineered standards optimized for modular construction and ABC. Standardizing ABC systems will bring greater familiarity with ABC technologies and concepts and should foster more widespread use of ABC. Using standardized designs will serve as a training tool to increase ABC familiarity among engineers.

ABC design examples were developed to be used by future designers. They provide step-by-step guidance on overall structural design of bridge components and use the same standard bridge configurations for steel and concrete as used in the ABC design standards. To supplement the design examples (where shortcomings in the AASHTO specifications may exist), the toolkit also provides recommended load and resistance factor design (LRFD) provisions for ABC modular systems. The LRFD design specifications do not explicitly deal with the unique aspects of large-scale prefabrication, including issues such as element interconnection, system strength, and behavior of rapid deployment systems during construction.

Recommended LRFD construction specifications for prefabricated elements and modular systems were compiled by the research team with the intent that they would be used in conjunction with standard plans for steel-and-concrete modular systems. As such, these specifications for rapid replacement focus heavily on means and methods required for rapid construction with prefabricated modular systems and are part of the *ABC Toolkit.*

Task 13 focuses on the development of technical training materials suitable for a one-day course on ABC to be deployed by the National Highway Institute. Task 13 will run concurrently with Phase IV activities and is not addressed in this report.

Introduction to the Final Report

This Final Report, prepared in accordance with Task 14 requirements for this project, documents the findings of Tasks 1 through 12. It contains four chapters and eight appendices. Chapter 1 gives a review of the problem statement, the research objective, and the scope of study. Chapter 2 describes the research tasks and the findings of the literature search, survey, and focus group meetings. The 16 ABC design concepts developed by the R04 team from the Phase I investigations are described in Chapter 2 with concept sketches and photographs. Chapter 3 contains the results of the engineering and constructability evaluations of the ABC concepts detailed in Chapter 2, as well as a short list of concepts that could be advanced to the ABC design and implementation phase. Chapter 4 introduces the *ABC Toolkit* developed in the R04 Project, combined with an overview of innovative project delivery and contracting provisions for ABC.

Appendix A contains the literature search and ABC case studies compiled by the R04 team. Appendix B summarizes the surveys and focus group findings. Appendix C is the UHPC Lab Testing Report. Appendix D is the report on the construction of the first field demonstration project in Iowa. Appendix E is composed of the ABC standard plans and Appendix F provides ABC sample design calculations. Appendices G and H include the recommended LRFD design and construction specifications, respectively.

CHAPTER 2

Research Approach

Research Tasks

The research effort was organized according to the following 14 tasks:

Task 1: Conduct an international and national literature search of published and unpublished material about innovative bridge technologies pertinent to rapid highway renewal.

Task 2: Conduct multiple focus groups to identify existing obstacles in the design and construction processes that inhibit optimum use of rapid renewal technologies.

Task 3: Develop new bridge designs that are more compatible with these innovative construction techniques and technologies.

Task 4: Develop new construction techniques and technologies that are compatible with existing bridge systems and bridge systems yet to be developed.

Task 5: Develop a report for Phase I detailing the work conducted in Tasks 1 through 4.

Task 6: Evaluate new construction techniques, technologies, and bridge systems, including laboratory testing, as appropriate.

Task 7: Design a demonstration bridge project, to be constructed in Phase III, using the most promising ABC technologies; prepare criteria to evaluate constructability and performance.

Task 8: Develop a work plan for preparing standard design plans and details for ABC superstructure and substructure systems and ABC construction technologies recommended from Task 6 evaluations.

Task 9: Develop a report for Phase II detailing the work conducted in Tasks 6 through 8 and a work plan for Phase III.

Task 10

A: Conduct a workshop for field demonstration projects approved by the technical coordinating committee (TCC) to evaluate techniques and systems.

B: Develop standard plans and details for ABC superstructure and substructure systems and ABC construction technologies recommended from Task 6 evaluations.

C: Conduct lab testing for the handling and constructability of UHPC joint material.

Task 11: Working with the industry, develop AASHTO–formatted LRFD design specifications and analysis methods, details, standard plans, and detailed design examples for complete bridge systems that are designed and fabricated in less time, and then installed on site in minutes or hours, using innovative construction equipment. The products should accommodate future reuse of these systems.

Task 12: Develop an accompanying AASHTO–formatted LRFD construction specification for the new construction techniques and technologies to address procurement and contracting along with user issues. The contractor is to include simple language addressing contracting tools that bridge professionals should consider. Other SHRP 2 efforts (Projects R07 and R09) are already focusing on improved procurement tools. User and life-cycle costs shall be addressed in this project (R04).

Task 13: Develop training materials suitable for deployment by the National Highway Institute (Phase IV).

Task 14: Prepare a final report, including a discussion of future research needs.

Phase I

Project R04 is composed of three distinct phases that took place over a period of four years. Phase I focused on Tasks 1 through 4 and culminated in Task 5, the preparation of the Phase I report. A brief description of the Phase I activities and findings follows.

Task 1: Conduct an international and national literature search of published and unpublished material on innovative bridge technologies pertinent to rapid highway renewal.

The literature review consisted of gathering published data from various sources including academic journals, conference proceedings, trade publications, research reports, and similar outlets on many topics related to this project. This included publications on engineering design, research, project construction, project management, and other related aspects of accelerated bridge construction (ABC). Information was also gathered from unpublished sources in the form of interviews with owners, contractors, designers, vendors, and fabricators.

The literature review is presented as an example project (or several projects) that is used to discuss a certain facet of ABC. The literature review discusses ABC in two main branches: the use of innovative design and construction concepts and the use of innovative construction methods and equipment.

The design and construction concepts review involves project examples demonstrating different approaches to ABC. Included is a discussion of bridge systems, such as those developed by several states and trade associations, that allow for complete prefabricated erection of bridges as an ABC approach. Also discussed are a series of component-level solutions, such as various innovative superstructure systems, rapid bridge deck installation and replacement concepts, substructure options, and rapid foundation construction concepts for both shallow and deep foundations.

The second major focus of the literature review is the use of innovative construction methods as a solution to rapid bridge construction. A general discussion on various bridge movement techniques, as well as design considerations for bridge movement projects, is included. Specific projects are presented to highlight the use of promising and novel bridge construction techniques alike. These include bridge movement using self-propelled modular transporters, lateral sliding and skidding of completed bridges, use of incremental launching alone or combined with other techniques, and other innovative construction techniques.

The literature review resulted in a collection of 300 technical references. These were carefully screened for relevance, redundancy, and content. Only those references meeting the project objectives are summarized for inclusion in this report.

Those projects, both domestic and international, that are specifically applicable to the current study of ABC and for which relatively complete information was available are presented in a series of case studies. For these case studies, examples of different types of projects are presented to highlight the variety of ABC solutions available. These are grouped into projects that executed ABC through prefabrication and those that use innovative construction techniques or structure movement as the core of the solution. Many projects increasingly involve both of these components, so a strict division is not always possible. The presented case studies are subdivided into superstructure systems, deck systems, substructure systems, and bridge movement systems, and appear in Appendix A.

Task 2: Conduct multiple focus groups to identify existing obstacles in the design and construction processes that inhibit optimum use of rapid renewal technologies.

A variety of techniques were used to determine industry opinions and experience in the area of ABC. Electronic web-based surveys, phone interviews, focus group teleconferences and meetings, in-person visits, and various e-mail communications were all used to solicit opinions from owners, engineering designers, contractors, fabricators, vendors, specialty contractors, and specialty engineers. The surveys and communications were used to collect data about past experiences, successes or failures of prior projects, institutional obstacles, and responses to other questions that helped the team understand how widely ABC is implemented in various locations throughout the country.

Two distinct surveys/interviews of owners and contractors were carried out during different time periods by different R04 team members. Thirty-eight completed questionnaires were received from the first survey, and 24 agencies responded to the second survey. The second survey and focus groups were conducted to obtain further insight into obstacles to the implementation of ABC methods, the causes, and the solutions. The surveys were developed to solicit information from owners and contractors on issues related to ABC. Each survey had several mandatory questions, and based on responses to those questions, follow-up questions were posed. As a supplement to the surveys, multiple owners were contacted individually by team members to gain additional insights into various aspects of ABC.

A series of interviews was also conducted with contractors. The contractors interviewed for this work varied in size of business and experience. Some were local or regional, and others would be considered major national or international contractors. Each contractor was interviewed at length and their thoughts and experiences were captured in detailed interview notes. Several meetings were held with specialty contractors as well.

Three focus group teleconferences were organized with owner representatives for follow-up discussions about survey responses and ABC-related issues. A total of 24 owner representatives from all regions of the country participated in the focus group conference calls.

Task 3: Develop new bridge designs that are more compatible with these innovative construction techniques and technologies.

The objective of this project is to develop standardized approaches to designing and constructing complete bridge systems. The aim, therefore, is to develop pre-engineered standards for ABC construction for selected bridge substructures and superstructure systems that can be installed with minimal traffic disruptions in renewal applications.

To this end, the concepts contained in this report have been classified into five tiers based on mobility impact time as follows:

- Tier 1: Traffic impacts within 1 to 24 hours.
- Tier 2: Traffic impacts within 3 days.
- Tier 3: Traffic impacts within 2 weeks.
- Tier 4: Traffic impacts within 3 months.
- Tier 5: Overall project schedule is significantly reduced by months to years.

Sixteen ABC design concepts developed by the R04 team have been described with concept sketches and photographs. They include new concepts, or adaptations of existing concepts, that are proposed as solutions to various ABC problems. The concepts covered two broad themes pertaining to rapid renewal:

- Proven concepts, and
- New and innovative concepts.

This chapter presents new concepts, or adaptations of existing concepts, that are proposed as solutions to various ABC problems. Some concepts, such as modular bridge systems, are truly complete systems and generally focus on the use of large prefabricated components in order to expedite construction. Many of the other concepts are at the elemental level, such as superstructure systems, deck concepts, various innovative column construction ideas, and so forth, and are not complete bridge systems. Yet these elemental concepts can be used together to form complete bridge systems.

Task 4: Develop new construction techniques and technologies that are compatible with existing bridge systems and bridge systems yet to be developed.

Five ABC construction concepts developed by the R04 team have been described with concept sketches and photographs. They include adaptations of existing concepts. The intent is to develop standard concepts for erecting highway structures using adaptations of proven technology that fulfill the following requirements:

- Serve multiple functions during a bridge construction process;
- Can be easily adapted from project to project;
- Can easily be transported on both urban and rural road systems;
- Can be mobilized with minimal erection and de-erection time; and
- Are not significantly more expensive than other standard equipment (cranes).

Task 5: Develop a report for Phase I detailing the work conducted in Tasks 1 through 4.

The work of Phase II was to incrementally winnow down the collected findings from Phase I and, through screening and prioritization, provide recommended ABC concepts and techniques that could be advanced to a field demonstration in Phase III.

The R04 team distilled the promising concepts that were recommended for advancement to Phase II into three broad categories with associated subcategories as follows:

- Category 1: Precast substructure systems
  A. Abutments;
  B. Piers; and
  C. Segmental columns.
- Category 2: Precast decks and complete superstructure systems
  A. Modular beam systems;
  B. Precast decks; and
  C. Segmental systems.
- Category 3: ABC construction technologies
  A. Launching, sliding, and shifting;
  B. Jacking and mining; and
  C. Equipment
    1. Wheeled carriers and SPMT
    2. Above-deck and straddle carriers
    3. Temporary trusses.

Phase II

Phase II comprised updated Tasks 6 through 9. A brief description of Phase II activities and findings follows.

Task 6: Evaluate new construction techniques, technologies, and bridge systems, including laboratory testing, as appropriate.

The work of Task 6 was to incrementally winnow the collected findings and ABC concepts from Phase I through screening and further evaluations. The purpose of these evaluations was to recommend ABC concepts and techniques that could be advanced to standard plans and field trials in Phase III. Thus, any technology recommended for field trials needed to meet minimum standards of readiness for execution and durability and value to the owner.

So that thorough and consistent evaluations were performed on the concepts, all evaluations were required to follow a series of steps, as outlined below:

- Step 1: Compile published materials pertinent to this technology.
- Step 2: Perform an engineering evaluation of the concept.
- Step 3: Perform a constructability evaluation of the concept.
- Step 4: Discuss implementation challenges and barriers to more widespread use.
- Step 5: Develop recommendations for testing concept development and implementation.

Results of the evaluations of Phase I concepts are presented in the Task 6 Concept Evaluation Report (March 2010), which is organized into the following three parts:

- Part I: ABC Superstructure Systems;
- Part II: ABC Substructure Systems; and
- Part III: ABC Construction Technologies.

An evaluation matrix for each option was prepared and included in the Task 6 report. Each option was evaluated based on criteria such as initial cost, durability, system simplicity, market readiness for rapid construction, ease of evaluation for overload permits, and other factors. Each criterion was scored on a scale of 1 to 5, with 1 being the lowest (poor), 3 being average, and 5 being the highest (very good). The design concepts that achieved the highest rankings, on the basis of results of the Task 6 evaluations, were then identified. With the ranking of these ABC concepts as a criterion, a short list of design concepts recommended for standardizing in Phase III was prepared. These concepts are those most suited for field trials and are considered market ready for ABC use.

Task 7: Design a demonstration bridge project, to be constructed in Phase III, using the most promising ABC technologies; prepare criteria to evaluate constructability and performance.

Under this research project, a total of $250,000 has been allocated to assist an owner in constructing a bridge project that demonstrates advances in accelerated bridge construction methods. A demonstration bridge project suitable for this research was identified in Pottawattamie County, Iowa, approximately 6 miles east of Council Bluffs. The bridge carries U.S. Hwy 6 over Keg Creek and replaces an existing bridge that was constructed in 1953.

A replacement bridge was being designed by the Iowa DOT Office of Bridges and Structures for construction letting in February 2011, using conventional construction. The R04 team redesigned the replacement bridge for ABC construction in Task 7. The replacement structure would be a three-span (67 ft, 3 in.; 70 ft, 0 in.; 67 ft, 3 in.) 210-ft, 2-in. by 47-ft, 2-in. steel/precast modular bridge with decked steel modules, precast substructures, and precast bridge approaches. This application provided a unique opportunity to effectively promote ABC for rapid renewal of the bridge infrastructure and to demonstrate various ABC technologies being advanced in the R04 project.

Task 8: Develop a work plan for preparing standard design plans and details for ABC superstructure and substructure systems and ABC construction technologies recommended from Task 6 evaluations.

As noted, one goal of this project is to produce pre-engineered ABC design standards for substructure and superstructure systems. By standardizing designs, their availability through local or regional fabricators will greatly increase, reducing lead times and costs, and increasing use. The first step is to develop a work plan for the viable design concepts and construction technologies recommended from the Task 6 reviews. A detailed work plan for this effort was developed under Task 8. Producing the design standards will be done in Phase III, under Task 10B. Doing the ABC design standards in Phase III allowed the research team to benefit from lessons learned from designing the demonstration project.

Task 9: Develop a report for Phase II detailing the work conducted in Tasks 6 through 8 and a work plan for Phase III.

In Task 9, the team created a Phase II report describing the activities completed in Tasks 6 through 8, including a work plan for Phase III. The demonstration project design was also prepared.

Phase III

Phase III was composed of Tasks 10 through 14. A brief description of the Phase III activities and findings follows.

Task 10A: Conduct a workshop for field demonstration projects approved by the TCC to evaluate techniques and systems.

Task 10A required constructing a demonstration bridge that used the most-promising bridge details identified earlier in the research and incorporated the ABC standards. The US-6 bridge, which crosses Keg Creek near Council Bluffs, Iowa, is representative in size and length of a large majority of bridges across the United States, was identified and replaced as a demonstration bridge using prefabricated elements and modular systems. The demonstration bridge incorporated proven ABC bridge construction details with the use of ultra-high-performance concrete (UHPC). The normal bridge replacement period of 6 months was shortened to only 2 weeks of traffic disruption. The complete bridge system was designed and constructed using superstructure and substructure systems composed of prefabricated elements. The field assembly of the three-span steel/composite bridge was completed in October 2011, and the bridge was reopened to traffic after a 14-day closure.

A daylong workshop was held on October 28, 2011, to disseminate information to bridge owners around the country. The showcase was attended by nearly 80 people from 14 states. These participants represented state DOTs, FHWA, designers, and contractors that shared an interest in accelerated bridge construction.

Task 10B: Develop standard plans and details for ABC superstructure and substructure systems and ABC construction technologies recommended from Task 6 evaluations.

Standardizing ABC systems will bring about greater familiarity with ABC technologies and concepts, which will foster greater regional cooperation and allow the industry to accommodate region-specific practices and industry needs. Pre-engineered standards developed in this project emulated cast-in-place construction but were optimized for modular construction and ABC. These standards can be inserted into project plans with minimal additional design effort to adapt to project needs.

Task 6 identified technologies that met minimum standards of readiness for execution, were suitable for ABC, and promised durability, economy, and value to the owner. In Task 10B, standard plans were developed for the most-useful technologies that could be deployed on a large scale in bridge replacement applications. They include complete prefabricated modular systems, as outlined here:

- Precast modular abutment systems
  - Integral abutments; and
  - Semi-Integral abutments.
- Precast complete pier systems
  - Conventional pier bents; and
  - Straddle pier bents.
- Modular superstructure systems
  - Concrete deck bulb tees;
  - Concrete deck double tees; and
  - Decked steel stringer system.

ABC details for superstructure and substructure systems that are suitable for a range of spans have been developed. The details presented in the plans included with this report are intended to serve as general guidance to practitioners in the development of site-specific designs suitable for accelerated bridge construction. Full moment connections between modular superstructure and substructure components were used to emulate cast-in-place construction. The closure pours were constructed using UHPC for the superstructure modules and self-consolidating concrete for the substructures.

A set of standard conceptual details was developed to assist the owners and engineers with their ABC implementation. These details demonstrate the possibilities and limits of conventional crane-based erection and erection using ABC construction technologies. Because the ABC design standards developed are for modular superstructure and substructure systems, the conceptual details for ABC construction technologies focus on bridge erection systems that are specifically intended for delivery and assembly as modular systems. Standard conceptual details for construction technologies for rapid bridge renewal projects using modular systems have been prepared and categorized into one of the following project types:

- ABC bridge designs built with conventional construction.
- ABC bridge designs built with ABC construction technologies.

Task 10C: Conduct lab testing for the handling and constructability of the UHPC joint material.

Three suites of laboratory tests were conducted by Iowa State University to evaluate the UHPC deck joints used in the demonstration bridge. The lab tests conducted for this study included abrasion testing of the UHPC closure joint material, constructability testing of the intersecting deck joints, and strength and serviceability testing of the transverse deck joint at the pier. The strength and serviceability tests focused on a UHPC joint application previously untested, namely in a transverse joint located in the primary negative bending region over the piers of a continuous bridge. The tests covered areas of interest that were considered critical to the use of UHPC in ABC applications and in the Keg Creek demonstration project.

Task 11: Develop AASHTO–formatted LRFD design specifications and analysis methods, details, standard plans, and detailed design examples.

In Task 11, the research team worked with the industry and developed AASHTO–formatted LRFD design specifications and analysis methods, details, standard plans, and detailed design examples for complete bridge systems that could be designed and fabricated in less time, and then installed on site in minutes or hours using innovative construction equipment. The products should accommodate the future reuse of these systems.

The work in this task entailed identifying any shortcomings in the current LRFD Bridge Design Specifications that may be limiting their use in ABC designs and making recommendations for addressing these limitations. The primary deliverable was to develop recommended specification language for ABC systems suitable for future inclusion in the AASHTO LRFD Bridge Design Specifications. ABC design incorporates components from several sections of the code. As such, the recommended specification is written as if it were to be added as a new LRFD subsection (5.14.6) under Section 5, Concrete Structures, in *LRFD Bridge Design Specifications*.

The design examples developed in this task will serve as training tools to increase familiarity about ABC among engineers. Three design examples are provided to illustrate the ABC design process for the following prefabricated modular systems:

- Decked steel girder;
- Decked precast prestressed girder; and
- Precast pier.

The design examples pertain to the same standard bridge configurations for steel and concrete used in the ABC standards. The intent was to design examples that could be used in conjunction with the ABC design standards developed in Task 10 so that practitioners will get a comprehensive view of how ABC designs are performed and translated into design drawings and details.

Task 12: Develop an accompanying AASHTO–formatted LRFD construction specification for the new construction techniques and technologies to address procurement and contracting along with user issues.

For this task, the contractor was to include simple language addressing contracting tools that bridge professionals should consider. Other SHRP 2 efforts (Projects R07 and R09) are already focusing on improved procurement tools. User and life-cycle costs shall be addressed in this project (R04).

ABC construction specifications pertain specifically to prefabricated elements and modular systems and are intended to be used in conjunction with the standard plans for steel and concrete modular systems developed in Task 10. As such, these specifications for rapid replacement focus heavily on means and methods requirements for rapid construction using prefabricated modular systems. The specifications have been prepared as if they were to be added as a stand-alone section in the LRFD Bridge Construction Specifications. Task 12 also compiled fundamental information on construction contracting and delivery methods that may be used to enhance the implementation and delivery of ABC construction projects.

Task 13: Develop training materials suitable for deployment by the National Highway Institute.

Technical content suitable for inclusion in a one-day short course on the Rapid Renewal of Bridges Using Modular Systems was developed. A portion of the course would be an introduction to the various ABC methods, while the rest of the course would be focused on the outcomes of this project, including introducing the new standard bridge concepts, systems, details, design and construction requirements, the design and construction of the demonstration project, and a guided walk-though of some of the sample designs. This work was completed in 2012, concurrently with Phase IV.

Task 14: Prepare a final report, including a discussion of future research needs.

The project deliverables include a final report that documents the entire research effort. A draft final report was submitted first for panel review. Upon receiving panel comments the final version of the report was prepared. The final report incorporates the following:

- Selected portions of prior Phase I and Phase II reports, including key information such as the literature review, development and evaluation of ABC concepts, and so forth;
- Recommendations from the project team about promising ABC design and construction technologies;
- Details from lab testing report of UHPC;
- Design standards for selected modular ABC systems;
- ABC design examples;
- Recommended LRFD design and construction specifications;
- Results from the field demonstration project;
- NHI training materials; and
- Recommendations for further study.

Owner–Contractor Survey and Focus Groups

Various focus group and outreach efforts will be described and the results from the efforts presented. Detailed results from various questionnaires are also provided in Appendix B. The data are presented in two major groupings—results from a series of electronic surveys and solicitations and more detailed individual outreach efforts to owners and contractors, including focus group teleconferences. The objectives of the outreach efforts were to gather information on current

ABC practices and to identify obstacles currently inhibiting ABC use for a greater number of bridge replacements. No meaningful response from the precaster surveys was obtained; thus, only the owner and contractor surveys are included.

Two distinct surveys/interviews of owners and contractors were carried out. The survey results are presented separately, as first and second owner surveys, as they were performed during different time periods by different R04 team members. The second survey and focus groups were conducted to obtain further insight into obstacles to the implementation of ABC methods, the causes and the solutions. Thirty-eight completed questionnaires were received from the first survey, and 24 agencies responded to the second survey. The surveys were developed to solicit information from owners and contractors on issues related to ABC. Each survey had several mandatory questions, and based on responses to those questions, additional follow-up questions were posed. Three focus-group teleconferences were organized with owner representatives for follow-up discussions on survey responses and ABC-related issues. A total of 24 owner representatives from all regions of the country participated in the focus-group conference calls. A summary of findings from these surveys and outreach efforts follows.

Summary of Findings from Surveys, Interviews, and Focus Groups

The summary of findings from the surveys and outreach efforts begins with some key findings distilled from the results of the outreach efforts. The findings highlighted in italicized text are concerns which solutions to be developed during the course of this project could either eliminate or significantly mitigate. A central theme of this research is to develop standardized approaches to designing, constructing, and reusing (including future widening) complete bridge systems that address rapid renewal needs and efficiently integrate modern construction equipment. This project will provide the needed national research support to advance ABC as a less risky, less costly, and more standard way of doing business for bridge renewals.

Key Findings from Surveys, Interviews, and Focus Groups

Based on the team's research, agencies are generally pleased with the results of ABC projects, yet ABC has yet to gain significant traction. The largest impediment to increased use of ABC appears to be the *higher initial costs.* Sufficient repetition would make the precast construction more economical. *It has been shown that costs have come down with repetition* (noted by a representative of the Utah DOT). Despite this cost savings, concerns remain about the ability to balance the increase in construction costs for ABC projects against the user costs savings. The owner has no way of collecting any savings of these costs.

Besides cost, a main deterrent of implementing ABC is that it is perceived as a method that *raises the level of risk associated with a project.* Some of this perceived risk is associated with how contractors see the process as too complex. Lack of familiarity with ABC methods adds to this perceived risk. States are looking for design manuals and other aids to help them design and implement ABC. Training could be beneficial. Design considerations that suggest how structures should be moved, acceptable deformation limits during movement, and better specifications all are needed.

Standardizing components is good practice, but it also offers challenges in getting the industry and states to come together in a regional approach to ABC. Contractors would be more willing to make equipment purchases if bridge construction became more standardized or industrialized, and was based on methods of erection that would speed assembly. This would increase the prospects for reusing the same equipment, creating a more efficient operation. ABC designs should be adaptable to a number of placement options to be cost competitive. A majority of the contractors are not receptive to owners requiring that a specific method of construction be used in ABC contracts.

In terms of performance, there are concerns about quality of construction, durability of joints, connections in precast elements, and seismic performance of precast elements and connections in seismic regions. Data are needed to show that ABC projects reduce accidents and increase worker and traffic safety.

Location is a factor that plays a role in the acceptance of ABC projects. *Lack of access for equipment and the need for large staging areas unavailable in urban locations hinder large-scale prefabrication. The use of precast elements for substructures has been impeded by the weight of components and hauling.* The use of smaller elements for superstructure and substructure that can be assembled on site will overcome mobility issues. Contractors have concerns about the diminished profitability of projects using large precast elements due to the greater outsourcing of work to precasters and specialty subcontractors. There is a culture of using cast-in-place construction among bridge contractors. Lack of a precasting industry in some states has also impeded ABC. Proper planning of the entire project is essential for ABC. It is not sufficient to accelerate bridge construction if the bridge is not on a critical path.

First Owner Survey and Interviews

The first survey results show that 73% of the agencies had experience with some aspect of ABC delivery. Eighty-two percent of the agency respondents indicated that an impetus to implement ABC within their agency currently exists, suggesting that many agencies are looking for ways to enhance the speed with which bridge projects are delivered. When asked why ABC techniques were used, the criterion most likely to be considered "very important" or "important" was traffic disruption mitigation, at 98%.

Though there is an overall high regard for ABC, lack of information has been identified as a hindrance for many agencies. Roughly three-quarters of respondents said that they do not have an identification system to select projects that are well suited for ABC implementation. While ABC has distinct benefits that are definitely attractive to many agencies, despite the gradual lowering of costs and life-cycle cost savings, DOTs are hesitant about using ABC techniques because of their higher initial costs.

Some states feel that a regional consensus is required for ABC to move forward, since contractors and fabricators in their part of the country work in multiple states. They believe that the local fabricators would embrace new shapes and technologies as long as a commitment to a large number of projects was made. Missouri, for example, has used alternative technical concepts (ATCs) on several projects to provide an incentive for contractors to develop confidential cost and time-saving innovations that will give them a competitive advantage. The Texas DOT districts are limited to using only 5% of the project cost for incentives. Also, no more than 25% of the road user delay costs may be used for incentives. In some cases, the roadway construction, but not the bridge construction, is on the critical path. These are impediments to ABC. The Utah DOT is unique in the level of support from the agency for using user costs as a strong consideration in weighing whether to use conventional or accelerated delivery methods. The Utah DOT is delivering its ABC program through a combination of design–build contracts and a method known as CM/GC (construction manager/general contractor).

There is resistance by local contractors to use extensive prefabrication because of the large project share that is subcontracted out to specialists. Also, there has to be sufficient repetition to make the precast components more economical and their construction more efficient than cast-in-place construction. ABC contains more factors, which contributes to risk, and agencies are generally risk averse. ABC is perceived as raising the level of risk associated with a project. The level of risk needs to be shown to be manageable in order for ABC to gain traction with some agencies. To properly analyze risk in ABC, it would be useful if quality differences could be demonstrated between field-constructed projects and projects built using prefabricated elements. Additionally, the rate of accidents in work sites could provide justification for using ABC.

Second Owner Survey and Interviews

The foremost concern with ABC is funding. Owners believe it costs more to do ABC, which takes money away from other critical projects. Contractors are concerned that ABC will lead to greater subcontracting thereby reducing profits gained by keeping their labor force employed and productive. Difficulty in considering user costs in project costs has impeded the wider use of ABC. User costs are difficult to quantify, and a funding mechanism to capture these costs for specific projects has not yet been developed. Accurate methods for determining life-cycle costs and developing user costs could help advance ABC. States with low traffic volume have found it more difficult to justify ABC costs. An exception may be the need for very long detours in some rural states.

States consider ABC a worthwhile method for certain projects but not a standard method. There is a cast-in-place (CIP) culture among owners and contractors that needs to be changed if more ABC projects are going to be considered. Owners are generally supportive of attempts to standardize elements and systems suitable for ABC but remain doubtful that it will result in reduced ABC costs. They agree that developing the elements and systems could promote greater use. Some owners feel there is a lack of familiarity with ABC, which may be offset by standardized elements and details. Also, a lack of available workspace at bridge sites is a great impediment to using SPMTs and heavy lift equipment.

Owner Focus Group Conference Calls

In the opinion of many of the focus group participants, ABC costs more than conventional construction. It uses money that cannot be recovered and takes funding away from other bridge projects. States do not usually consider user costs in project costs. There is no accepted national approach to estimating user costs. More projects can be justified if states can recover user costs through federal funding. States have used A+B bidding. The main concern is with estimating the realistic user-cost portion of the "B" part and the ability to recover user costs from federal funding.

Local communities seem to prefer a short closure with rapid construction compared with a long staged construction. Owners again discussed that there is a prevalent CIP construction culture among contractors. Contractors have proposed CIP construction within the same duration (same cost) when precast ABC was specified. Some of this CIP construction culture comes from contractors that like to keep as much work to themselves as possible to keep crews employed and maximize internal profits. A precast option may require work to be subcontracted out and reduces the control of the prime contractor.

Rapid construction requires the designer to spend a lot more time on site and to be instantly available in the design office to work with contractor and DOT construction personnel to provide speedy responses to questions. This is a key to success for ABC projects. States are often restricted in what they can do to engage the contractor during the design phase, which makes ABC more difficult. Some states have used CM/GC contracting to improve this early communication without going to design-build.

States don't see any particular impediments with current designs as far as ABC is concerned. States are not using special designs for ABC, but simply modifying CIP details to fit ABC. Standardized designs and design examples for ABC could be helpful. FHWA's recently published *Connection Details for Prefabricated Bridge Elements and Systems* was welcomed by the states.

Relaxing concrete curing standards by using a strength-based (maturity method) approach works better for ABC projects. The use of self-consolidating concrete has made a big difference in ABC projects. The use of UHPC for joints and closure pours is also being tried on ABC projects. Contractors can perform precasting of conventionally reinforced bridge sections either on or off site. Lack of a nearby ready mix plant can be a reason to use precast. Only prestressed components will require a precast manufacturer. Many states are reluctant to rely on posttensioned concrete systems for ABC due to difficulty in future inspection and maintenance. It's been positively noted that states have had success with precast abutments and pier caps on pilings. However, precast substructure elements have in some cases been impeded by the weight of components and hauling. In general, some of these precast substructure elements do not provide much tolerance for field installation. Greater care must be exercised when using precast elements for fit up in the field versus cast-in-place construction. Trucking and lifting can be issues with the larger precast elements.

Some comments and concerns brought up in the conference calls include weather issues, which can have detrimental effects on the speed of construction. It's been positively noted that mobility and environmental concerns are pushing greater use of ABC. Current precast deck systems have so many steps (posttensioning, grouting joints and ducks) that the time savings is minimal at best. A simpler system would speed the process. Partial-depth precast deck panels have a proven history and do save some construction time. Also, all ABC technologies have not been validated for high seismic regions. Seismic connections are the biggest concern among all surveys and are the subject of ongoing research.

Contractor Surveys

Contractors indicate that ABC was employed because the owner required it (i.e., it was not the contractor's decision). They noted that most of these projects consisted of using special equipment to move prefabricated elements. In response, a majority of contractors said they would invest in new equipment (lease or buy) specifically to be able to complete a project using rapid replacement techniques if the owner commits to many ABC projects (not just one). When further questioned if bridge construction became more standardized and based on certain methods of erection to speed the assembly, would they positively consider such a purchase, 90% of contractors responded "yes."

A majority of the contractors are not receptive to owners requiring that a specific method of construction be used in ABC contracts. If agencies mandate a specific method of construction, the freedom of the contractor to develop the best solution is diminished, and this has the potential to limit the contractor's ability to bid on certain projects. Of all responses, 67% of contractors indicated that they did not think appropriate incentives existed to accelerate the work in their contracts. They indicated that many forms of contracts have been used on their accelerated projects, which supports the notion that there are various procurement methods suited for ABC projects, not just one method.

It is believed by some contractors that the availability of standardized bridge elements would help lower construction costs. Others believe a site-specific design that acknowledges constraints and crane needs is a preferred approach. Limitations on the cranes that can be used at a site are a major concern with prefabricated elements. One hundred tons is a reasonable upper limit for such elements.

Tiered Approach to ABC

Minimizing road closures and traffic disruptions is a key objective of ABC. For ABC systems to be viable and gain greater acceptance, the savings in construction time should be clearly demonstrated. To facilitate the investigations and discussions in this project, ABC design concepts have been classified into five tiers based on mobility impact time as follows:

- Tier 1: Traffic Impacts within 1 to 24 hours.
- Tier 2: Traffic Impacts within 3 days.
- Tier 3: Traffic Impacts within 2 weeks.
- Tier 4: Traffic Impacts within 3 months.
- Tier 5: Overall project schedule is significantly reduced by months to years.

Modular systems allow a more versatile option to ABC that is not limited by available space at the bridge site. Modular bridge systems are particularly suited for Tier 2 concepts for weekend bridge replacements or as Tier 3 concepts, where the entire bridge may be scheduled to be replaced within one to two weeks using a detour to maintain traffic. Tier 1 concepts include preassembled superstructures, completed at an off-alignment location and then moved via various methods into the final location using techniques such as lateral sliding, rolling and skidding, incremental launching, and movement and

Figure 2.1. Tier 1 or Tier 2 ABC: Bridge move by using SPMT.

placement using self-propelled modular transporters (SPMTs) (Figure 2.1). Tier 5 concepts involve accelerating a statewide bridge renewal program by months or years by applying ABC technologies included in the other tiers.

Development of ABC Design and Construction Concepts

Introduction

This section addresses the development of bridge design concepts and construction techniques and technologies that can be used for future ABC projects. This section also presents suggestions and recommendations for development and implementation in the subsequent phases of this project. Though structured as separate tasks, it was concluded that the issues of design, details, and construction techniques were best presented together since they are so strongly interrelated. ABC projects of the future will likely use combinations of innovative designs, coupled with advanced construction methods and other planning and design tools to bring projects to fruition.

Feasible bridge design and construction concepts that will allow for rapid renewal under the guidelines of the following tasks are discussed in this section:

- Task 3: Develop new bridge designs that are more compatible with these innovative construction techniques and technologies.
- Task 4: Develop new construction techniques and technologies that are compatible with existing bridge systems and bridge systems yet to be developed.

Various efforts were undertaken to meet the challenges of these tasks. In order to conceptualize ABC projects of the future, the R04 team focused heavily on gathering, understanding, and following up on past ABC projects. Chapter 3 presents many of those findings. In addition to collecting and summarizing the literature in Chapter 3, there was also a significant outreach effort, described in Chapter 4, to gather additional information on ABC from various perspectives. The teams' activities related to new concept development included the following:

- Outreach to various DOT, contractor, and design professionals throughout Phase I activities. Specific attention was paid to owner and contractor concerns with regard to ABC design and implementation.
- Focus group teleconferences to identify and discuss perceived obstacles to ABC implementation, as well as potential solutions.
- Multiple internal meetings and working sessions of the R04 team, including two two-day sessions in HNTB Corporation's New York office with multiple HNTB representatives and subconsultants to discuss and brainstorm ideas. These sessions included several HNTB senior bridge engineers from across the United States that have expertise in various aspects of bridge design and construction.
- Brainstorming workshop in Kansas City, Missouri, attended by owners, contractors, and railroad representatives to review and critique initial design and construction concepts to ensure that the concepts were feasible. The concepts were then ranked, and a teleconference was held to discuss the highest-ranking concepts with DOT representatives who could not attend the Kansas City meeting.

Target Outcomes for Design

Historically, the formulation of infrastructure costs has given little or no consideration to the costs borne by those who will use the bridges in question. Since many bridges on Interstate highways are now in high-traffic urban centers, the cost to users should be considered from both an economic and political standpoint. Inadequate infrastructure must be replaced with the least possible effect on the traveling public in terms of lost time, lost productivity, and wasted energy. To achieve these objectives, bridge construction should focus on objectives that will save time and money and lead to better overall results. The following strategies are suggested to achieve the accompanying results:

- As light as possible
  - Improves the load rating of existing foundations and piers; and
  - Simplifies the transportation and erection of bridge components.
- As simple as possible
  - Fewer girders;
  - Fewer field splices;
  - Fewer bracing systems; and
  - No temporary bracing to be removed.

- As simple to erect as possible
  - Fewer workers on site;
  - Fewer fresh-concrete operations;
  - No falsework structures required;
  - Simpler geometry; and
  - Bearings or seismic I/D systems instead of pier continuity.

ABC should demonstrate flexibility and scalability in all three aspects of renewal, which are

1. Accelerated *retrofitting* of existing bridges
   - Micropiles, lateral drilled shafts, jet grouting;
   - Steel jackets on pier columns;
   - Use of high-performance steel (HPS), twin I-girder systems, and UHPC deck-slab panels;
   - Non-invasive strengthening of foundations and piers;
   - Lighter deck for increased load capacity; and
   - Replacing a precast concrete (PC) span with a steel/composite span may improve load rating of existing foundations and piers.
2. Accelerated *replacement* of existing bridges
   - Accelerated bridge removal and off-site demolition; and
   - Accelerated bridge construction under existing constraints.
3. Accelerated *construction* of new bridges
   - Easier transportation and equipment availability in remote sites;
   - More local sources for CIP concrete supply;
   - Greater ability to overcome site constraints;
   - Greater access to the area under the bridge;
   - More workable height of piers;
   - Simpler bridge geometry;
   - More workable bridge length and amortization of specialty investments; and
   - Adaptable to climates ranging from tropical to arctic.

Target Outcomes for Construction

The concepts illustrated in this chapter are derived from time-tested bridge construction methods.

- Access from above
  - Launching gantry erection of girders or precast segments;
  - Launching gantry erection of macrosegments; and
  - Wheeled transportation and placement of full-length precast spans.
- Access from underneath
  - Crane lifting of precast spans;
  - Wheeled transportation and placement of precast spans; and
  - Strand jacking of full-length spans or macrosegments.
- Access from the abutment
  - Monolithic launching of single spans with a temporary pier;
  - Incremental launching of continuous spans; and
  - Launch followed by structural changes.
- Lateral access
  - Launching of continuous spans combined with lateral shifting; and
  - Span removal by reverse launching.

Proposed Design and Construction Concepts

While most agencies are aware of ABC, very few practice it on a large scale. According to the survey, many ABC techniques are ready for implementation, yet DOTs are hesitant about using ABC techniques because of their higher initial costs. When certain techniques have been used, agencies were generally pleased with the results, but very few have committed to using these same techniques as part of an overall ABC program.

Advancing the state of the art to overcome obstacles to ABC implementation and achieve more widespread use of ABC is a goal of this research. The knowledge and understanding of ABC issues gained from the initial tasks provide a sound basis for formulating a direction for future research, which will lead to the desired outcomes for this project. Findings from the outreach efforts of owner and contractor concerns and impediments to ABC implementation as described in Chapter 4 and summarized here served as a starting point for the R04 team to explore ABC solutions, specifically design and construction concepts that could potentially be further developed for implementation in the next phase of this project.

- The largest impediment to increased use of ABC appears to be the higher initial costs. Reducing cost was a priority with most owners.
- Concerns exist about durability of joints and connections in precast elements.
- Concerns exist about seismic performance of precast elements and connections in seismic regions.
- ABC is perceived as raising the level of risk associated with a project. It is also perceived as being too complex by some contractors. Proven superstructure and substructure systems that reduce overall risks would be quite attractive to owners and contractors.
- Lack of familiarity with ABC methods is a concern. States are looking for design manuals and other aids that could help them design and implement ABC. Training could be beneficial.

- When designing, consideration should be given for structures to be moved, for acceptable deformation limits during movement. Overall, there is a need for better specifications.
- ABC designs should be adaptable to a number of placement options to be cost competitive. A majority of contractors are not receptive to owners requiring that a specific method of construction be used in ABC contracts.
- Lack of access for equipment and the need for large staging areas unavailable in urban locations hinder large-scale prefabrication. The use of precast elements for substructure has been impeded by the weight of components and hauling. Using smaller elements for superstructure and substructure that can be assembled on site will overcome mobility issues. The modular concept of building bridges could overcome this concern.
- Standardizing components is a good idea but also offers challenges in getting the industry and states to come together in a regional approach to ABC. Developing ABC standards that could be adopted regionally is one goal.
- Contractors will be more willing to make equipment purchases if bridge construction becomes more standardized or industrialized and is based on certain methods of erection that speed assembly. These steps would increase the prospects for reusing equipment.

The findings were used as a guide by the R04 team to pursue solutions during Tasks 3 and 4 that could either eliminate or significantly mitigate these concerns. The concepts developed and described in this chapter are aimed at meeting the objective of the R04 project, which is to develop standardized approaches to designing, constructing, and reusing (including future widening) complete bridge systems that address rapid renewal needs and efficiently integrate modern construction equipment.

Twenty-one design and construction concepts developed by the R04 team during Tasks 3 and 4 are listed in Tables 2.1 and 2.2.

Table 2.1. ABC Design Concepts

Concept	Description	Tier
Design Concepts		
D-1	Precast abutments and piers	2
D-2	Hybrid drilled shafts	2
D-3	Segmental piers	3
D-4	GRS abutments	3
D-5	UHPRC substructure systems	3
D-6	UHPRC superstructure systems	3
D-7	Concrete-filled steel or FRP shell columns	3
D-8	Complete composite steel superstructure systems	1
D-9	Complete precast concrete superstructure systems	1
D-10	Modular superstructure systems	1
D-11	Pre-topped U-beams	2
D-12	Space frame superstructures	1
D-13	Precast deck systems	3
D-14	Concrete-filled steel tube design	3
Next-Generation Design Concepts		
ND-1	Next-generation design concepts	3
ND-2	Next-generation material concepts	3

Note: FRP = fiber-reinforced polymer; GRS = geosynthetic reinforced soil; UHPRC = ultra-high-performance reinforced concrete.

Description of Design and Construction Concepts

Design Concept D-1: Precast Abutment and Pier Details

Concept Description

DOTs around the country have tried using precast abutment and pier details with varying results. Precasting as much of the substructure as possible allows for faster construction of the bridge and reduced interference with normal system operation (see Figures 2.2 and 2.3). One goal of this research project is to produce pre-engineered plans that can be readily implemented by state DOTs with minimal additional effort.

Advantages

The main advantage of precast abutment and pier details is a shorter period of construction-related disruption for transportation facilities. Other advantages include the following:

- Factory-produced precast concrete product.
- Better control of precast element tolerances.
- Less CIP concrete required in the field.
- Reduced exposure of construction workers to traffic conditions.

Table 2.2. ABC Construction Concepts

Construction Concept	Description	Tier
C-1	Above-deck driven carrier systems	1, 2, 3
C-2	Launched temporary truss bridge	1, 2, 3
C-3	Wheeled carriers or SPMTs	1
C-4	Launching and lateral sliding	1
C-5	Jacking and mining	3

- Reduced exposure of traveling public to construction activities.
- Less time curing concrete in the field.
- Possible reduced cost of elements via standardization.
- Building the new precast abutment behind the existing abutment would increase the hydraulic opening and limit the need for a hydraulic analysis.
- Use of wall piers would possibly increase the use and production of identical pieces (abutment sections), lowering the total precast option cost.

Disadvantages

Some disadvantages of using precast elements follow:

- They are heavier, requiring larger cranes to place.
- They may be more difficult to deliver to remote locations.
- With the contracting community still learning the system, the cost of initial installations tends to be higher.

Connections

Reference is made here to the FHWA publication *Connection Details for Prefabricated Bridge Elements and Systems* (Culmo, 2009). This publication contains examples of connections of various precast elements that have been used on previous projects throughout the United States. This publication should be used when developing connection details for substructures.

A Proven Precast Abutment System

Precast abutment details have been tried in several states over the past 10 years. Some details have proven more successful than others. In reviewing details from around the country, the research team found the current details employed by the Utah DOT to be proven and complete. These details have been included here as a reference on how to approach detailing precast abutments. Individual states may want to modify the details presented to fit their local needs and market conditions.

The Utah system provides some unique details. The use of multiple large vertical cavities within the wall elements that are later filled with high early strength concrete allows for larger precast elements and leads to lighter shipping and lifting weights. The use of fewer elements speeds the field construction process. The details also show the use of precast footings using leveling bolts and a bed of grout to seat the footings. This minimizes the amount of CIP concrete and keeps the project moving. By having the vast majority of the abutment concrete delivered to the job site as precast elements, the time needed to form, cast, and cure the abutment is greatly reduced. This reduces the level of inconvenience to the traveling public. Pictures of a typical precast abutment installation are given in Figures 2.2 and 2.3.

The research team would like to thank the Utah DOT for providing these details. (For a complete set of precast abutment and pier details and notes, please see the Utah DOT website.)

Figure 2.2. Precast abutment installation.

Figure 2.3. Filling of voids in precast abutments.

Design Concept D-2: Hybrid Drilled Shaft/Micropile Foundation System

Concept Description

The intent is to develop a high-capacity drilled shaft foundation system that eliminates the need for large lifting equipment and complex rigging, handling, and fabrication. Micropile foundation systems have a number of key advantages for ABC. One is the use of low-cost, small-footprint, all-terrain drilling rigs for installation and segmented 5- to 12-in. nominal diameter high-strength steel casings that allow for rapid installation in low head-room conditions (Figure 2.4). Another key advantage of the use of micropiles is reduced construction risk, since a failed micropile can simply be abandoned (and replaced with a closely adjacent micropile). Also, micropile installation methods pose the least potential impact on closely adjacent structures, even fragile structures. Large-diameter drilled shaft construction requires large equipment, ample head room, and complex logistics for fabrication, installation, concreting, and integrity testing operations, in a riskier construction environment. If a drilled shaft does not meet integrity tests, expensive and time-consuming repair operations must be undertaken.

Another advantage of a hybrid system is the lack of need for a circular-shaped foundation system, as illustrated in Figure 2.4. Any shape will do the job. Wall-type piers or those of any other cruciform shape may readily be integrated into a hybrid foundation system.

A number of distinct construction advantages result from a hybrid of the two systems, whereby the upper portion (10 to 20 ft) of the deep foundation is conventional drilled shaft construction and the lower portion of the shaft is composed of micropiles. Above grade, the drilled shaft is extended to serve as a circular pier column, eliminating pile cap foundation construction. Below grade, the drilled shaft portion of the hybrid foundation need extend only to the extent required by design, with due consideration of flexural demands and extreme events relating to scour and seismic design.

Figure 2.4. Typical drilled shaft/micropile system for circular column.

A key benefit of this approach is that a significant length of the drilled shaft in soil is replaced by micropiles. By using a much shorter drilled shaft length, significant time and logistics savings accrue in excavation, reinforcement placement, and concreting. In addition, this proposal is applicable where rock is too deep to serve as a viable founding stratum.

Design Considerations

The following design considerations are critical to the development and implementation of this concept:

- Shear load transfer mechanism between micropile and drilled shaft. Shear load transfer along the length of the micropile will be an important consideration in minimizing the length of the upper drilled shaft portion of the hybrid system, particularly in circumstances where it is important to mobilize casing strength.
- Behavior of micropile clusters in a scour environment. Upper shaft length can be minimized in circumstances where micropiles have adequate buckling stability under the design scour event.
- Lateral behavior of the hybrid foundation system from a soil structure interaction perspective. Given the significant difference in lateral flexibility of the two systems, design behavior of the hybrid system under lateral loads will be critical. In addition, the hybrid design strategy allows for different-shaped drilled shaft/column extensions (e.g., rectangular, oval) and for much larger shafts than have been previously contemplated. For large hybrid shafts and hybrid shafts of arbitrary cross section, proper design under lateral loads becomes more complex.
- Influence of battered micropiles on overall system performance. The potential for battered hybrid drilled shafts, as well as vertical hybrid shafts that employ battered micropiles, introduces further design issues and opportunities.
- Group effects for hybrid piles, including the difference between group effects for the shaft portion versus micropile portion, figure into design consideration.

Research and Testing Needs

Given that the proposal combines two well-researched deep foundation techniques, the team did not see the need for a significant research program. Research and testing should be focused on both lateral load behavior and shear transfer between the shaft/micropile interface, but the team did not see this as a major effort. There are adequate design and analysis capabilities in the current state of practice to allow full implementation of this proposal without research and testing.

Design Concept D-3: Precast Segmental Columns with Precast Caps

Concept Description

Construction of the substructure may expend 60% to 70% of the total construction time required for a project. However, the employment of precast abutment and pier details has been used sporadically by DOTs around the country with varying results. Typically, highway bridges are constructed of CIP reinforced concrete abutments and piers. Although these practices generally produce durable bridges, they also contribute significantly to traffic delays because of the sequential nature of the construction. Foundations must be formed, poured, and cured before columns and pier caps can be placed. Columns and pier caps must be formed, poured, and cured before the girders and deck are placed.

Precast concrete bridge piers offer a promising alternative to their cast-in-place concrete counterparts. Enormous benefits could arise from their use because precast concrete bridge pier components are typically fabricated off site and then brought to the project site and quickly erected. Precast pier components also provide an opportunity to complete tasks in parallel. For example, the foundations can be cast on site while the precast pier components are fabricated off site. The use of precast components has the potential to minimize traffic disruptions, improve work zone safety, reduce environmental effects, increase quality, speed up construction time, and lower life-cycle costs. Some projects are in rural areas where traffic is minimal but the shipping distance for wet concrete is expensive. The use of precast concrete bridge elements can provide dramatic benefits for bridge owners, designers, contractors, and the traveling public.

A segmental column consists of column segments of varying length that are stacked vertically until the desired total column height is reached, as shown in Figure 2.5 and Figure 2.6. Once the column segments are erected they may be vertically posttensioned together and to the foundation for stability. Segmental columns are easier to erect 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.

Segmental piers may use match casting to ensure proper alignment in the field. Segments also may be voided or hollow to reduce the dead load on the foundations and make it easier to handle the segments. Segmental piers may be vertically posttensioned once erection is completed.

Segmental pier columns may also be used without pier caps. The Route 36 Highlands Bridge in Highlands, New Jersey, used segmental piers without pier caps (Figure 2.5). Whole columns, with precast bent caps, may also be used with smaller columns.

Source: New Jersey DOT.

Figure 2.5. Segmental piers on Route 36 Highlands Bridge.

Figure 2.7. Bridge with GRS abutments over waterway.

One of the goals of this research project is to produce pre-engineered plans for precast piers that can be readily implemented by state DOTs with minimal additional effort. The intent is to provide standardized details that are applicable to both seismic and non-seismic regions.

Design Concept D-4: Geosynthetic Reinforced Soil Abutments

Concept Description

There are at least two types of geosynthetic reinforced soil (GRS) structure applications for support of bridge abutments. Traditionally, mechanically stabilized earth (MSE) abutment designs provide support for a bridge by deep foundations such as piles or drilled shafts. Less common, but growing in consideration, are MSE wall bridge abutments supported on spread footings within the reinforced mass, as shown in Figures 2.7 and 2.8. These newer systems will be discussed in the following paragraphs.

The FHWA and AASHTO have proven standards for design of MSE wall bridge abutments. In Section 11.10 of the AASHTO *LRFD Bridge Design Specifications,* 4th edition, the code specifically addresses the design of MSE bridge abutments. Modular systems, such as those used for GRS abutments, are addressed in Section 11.11.

The foundation soil conditions, abutment loadings, and tolerance to settlement are among the most important considerations when evaluating whether a spread footing can be used. MSE wall bridge abutments are generally used to

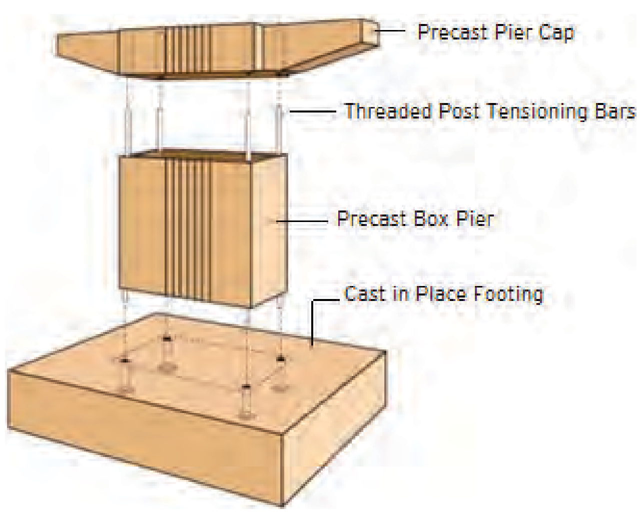


Figure 2.6. Typical components of a segmental pier.

Figure 2.8. GRS abutment under construction.

provide cost savings by shortening the bridge length. But they can also provide benefits because they can accelerate the construction schedule, require smaller equipment to construct, can tolerate significant settlements, and allow for the mitigation of settlement before placement of the bridge.

The GRS bridge abutment technology to date has been applied to 70- to 90-ft single-span low-volume road bridges. The basis of the design is the combining of the superstructure and substructure into an integral abutment system. The abutment face consists of modular concrete block wall units. The abutment mass consists of layers of compacted fill with layers of geosynthetics (geogrid or geotextile) alternating to the height of the wall. Design can generally allow for the use of either native soil backfill or a more select material.

A lower-quality backfill may require more geosynthetic reinforcement. No abutment footing is needed; the precast concrete box beam superstructure is placed directly on the GRS abutment. No approach slab is needed because the bridge beams and approach roadway fill are backfilled with GRS materials. This provides a more uniform support condition that allows for a gradual transition from the bridge to the roadway, reducing the potential for a "bump" at the end of the bridge. GRS bridge abutments can be used for stream crossings with proper design for scour conditions. A geosynthetic-reinforced soil foundation supports the abutment, which is protected by a riprap slope toe protection designed for the appropriate stream velocity.

Advantages

The accelerated construction benefits of using a spread footing abutment supported on the reinforced mass versus deep foundation support are significant. Constructing a spread footing on the reinforced mass considerably reduces the time it takes for the staging and installation of deep foundation elements at the abutments as well as the time for associated tasks such as preboring and the placement of pile sleeves. The use of the spread footing option may have some limitations related to bridge type and span length as the standard bearing pressure is on the order of 2,000 psf. Geotechnical investigations may require more detailed information in order to evaluate the spread footing option.

GRS bridge abutments are simpler and therefore much quicker to construct than conventional deep foundation supported bridges due to their integrated design technology. Time savings of 50% over normal construction time have been reported for projects completed using the GRS system. Construction can be performed, for the most part, quickly and easily by hand labor and small equipment and is less dependent on weather conditions and associated delays. Materials required are generally readily available and easily transported, and the need for creating access for heavy equipment is greatly reduced.

Design and construction guidelines for GRS bridge abutments are available in NCHRP Design Report 556 (Wu et al., 2006).

Design Concept D-5: Ultra-High-Performance Concrete (UHPC) Substructure Systems with Reinforcing Capacity

Concept Description

The use of precast substructure components with conventional concrete was presented earlier in Design Concept D-1: Precast Abutments and Pier Details. The benefits and opportunities of using normal strength concrete are evident.

As the next step in precast pier components, ultra-high-performance concrete (UHPC) can be used to create extremely strong, durable forms for substructure elements. As noted in Concept D-6, compressive strengths of 18,000 psi to 30,000 psi can be achieved with this material, depending on the mixing and curing process. In addition, tensile capacities on the order of 2 ksi can be used to supplement the main reinforcing structure. The high strength and durability of this material make it a valid candidate for standardized accelerated bridge construction.

A precast concrete shell pier system, the SPER system, was developed by the Sumitomo Construction Company of Japan. The SPER acronym represents the Sumitomo Precast form method for resisting Earthquakes and Rapid construction. This system uses concrete panels as stay-in-place forms during construction. Once the concrete panels have been erected, cast-in-place (CIP) concrete is placed in the forms to create a composite pier. Panels can be used as outer formwork for solid piers or both interior and exterior formwork for hollow piers.

The primary advantage to this system is the precast shells forms serve not only as stay-in-place forms but, more importantly, as structural members.

The erection sequence is listed as follows:

- Erect the interior precast form on the footing or other foundation element.
- Erect the exterior precast form on the footing.
- Fill with cast-in-place concrete.

Conventional concrete piers in the 30- to 40-ft range require 4-in. precast shells. The forms are simply stacked one atop the other surrounding the primary reinforcing and filled with CIP concrete. Note that the lateral confinement reinforcing is placed as part of the precast shell elements, which considerably reduces the on-site labor and saves additional construction time.

Figure 2.9. Taller columns using UHPC forms.

For taller piers (see Figure 2.9), which have been constructed up to 200 ft in height, the SPER piers are constructed using an inner and outer form that provides a large hollow space and provides considerable material and weight savings. The larger hollow forms are precast in channel-shaped sections that permit handling with somewhat lighter cranes and transport without the need for specialized trucking permits. It is estimated that use of the SPER system reduces pier construction time by 60% to 70%.

The use of UHPC materials for a precast shell pier system, as shown in Figure 2.10, has been investigated by researchers in Switzerland. The FEHRL (Forum of European National Highway Research Laboratories) has investigated the use of 2-in.-thick stay-in-place shell forms for bridges over a highly traveled highway system. These shell elements are connected using epoxy resin and are used to create the forms and main reinforcing elements.

Advantages

UHPC offers a number of advantages for accelerated pier construction:

- Extremely high strength-to-weight ratio and smaller, more easily transported pier sections.
- Nearly zero permeability that is extreme durability, especially when used in roadway splash zones.
- Higher modulus of elasticity than conventional concrete.
- Very ductile behavior that permits gradual deformations under high loads.

Figure 2.10. Use of UHPC columns for highway overpasses.

- Thin sections permit lightweight pieces that can be erected by smaller cranes.

Disadvantages

- This material currently is projected to cost three to five times as much as conventional concrete.
- A longer cycle of casting and heat curing is required to achieve extremely high compressive strength.
- Limited number of casting locations in the United States.

Design Concept D-6: Ultra-High-Performance Concrete (UHPC) Superstructures

Concept Description

Ultra-high-performance concrete (UHPC) was developed in France in the 1990s and has seen limited use for bridge structures in the United States. Only one state bridge owner, the Iowa DOT, has constructed bridges with members using this innovative material, and the New York State DOT has used UHPC for joints. Nonetheless, the extremely high strength and durability of UHPC make it a valid candidate for consideration in standardized accelerated bridge construction (ABC) components.

UHPC consists of fine sand, cement, and silica fume in a dense, low water-to-cement ratio (0.15) mix. Compressive strengths of 18,000 psi to 30,000 psi can be achieved, depending on the mixing and curing process. The material has an extremely low permeability and is highly durable due to the almost non-existing intrusion of chloride-laden water. To improve ductility, steel or polyvinyl alcohol (PVA) fibers (approximately 2% by volume) are added, replacing the use of mild reinforcing steel. For this project, the patented mix Ductal developed by Lafarge North America was used with the steel fibers.

The Iowa DOT has constructed three generations of these bridges, beginning in 2006 with a conventional 110-ft-long prestressed concrete bulb tee beam bridge crossing a small creek in Wapello County. Funding for this project was provided through the FHWA Innovative Bridge Research and Construction Program (IBRC). One advantage of this system for ABC applications is the reduced number of pieces to be erected in short closure periods.

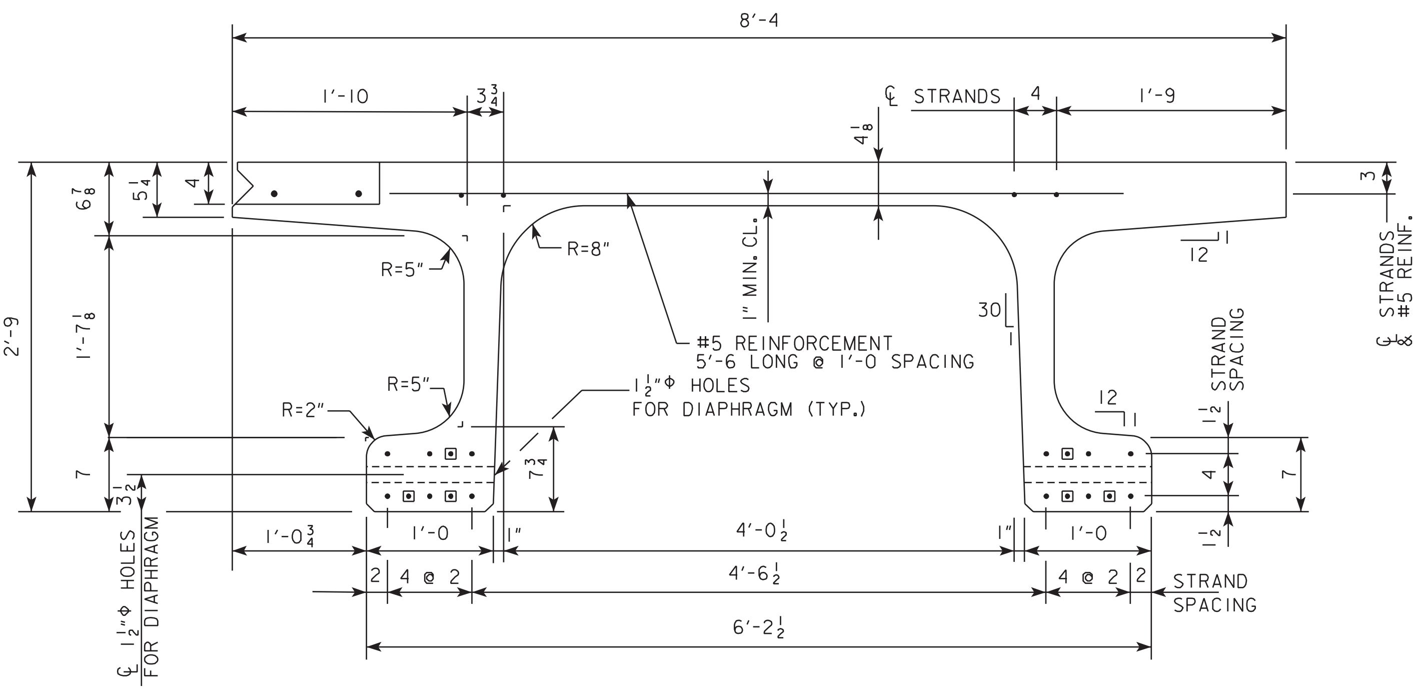
The second-generation UHPC superstructure project, designed to take greater advantage of the strength of the component materials, was constructed in Buchanan County in 2008. The subject bridge consisted of a 50-ft pi-girder span encased in a CIP concrete diaphragm. These pi-girder sections, so titled because they look much like the Greek letter pi, as shown in Figures 2.11 and 2.12, are essentially a modified double tee section with very thin deck and web components. Extensive laboratory and analytical testing at FHWA and Iowa State University determined that a more robust section would better distribute live loads to the girder webs.

Source: Iowa Department of Transportation.

Figure 2.11. UHPC pi-section superstructure construction in Iowa.

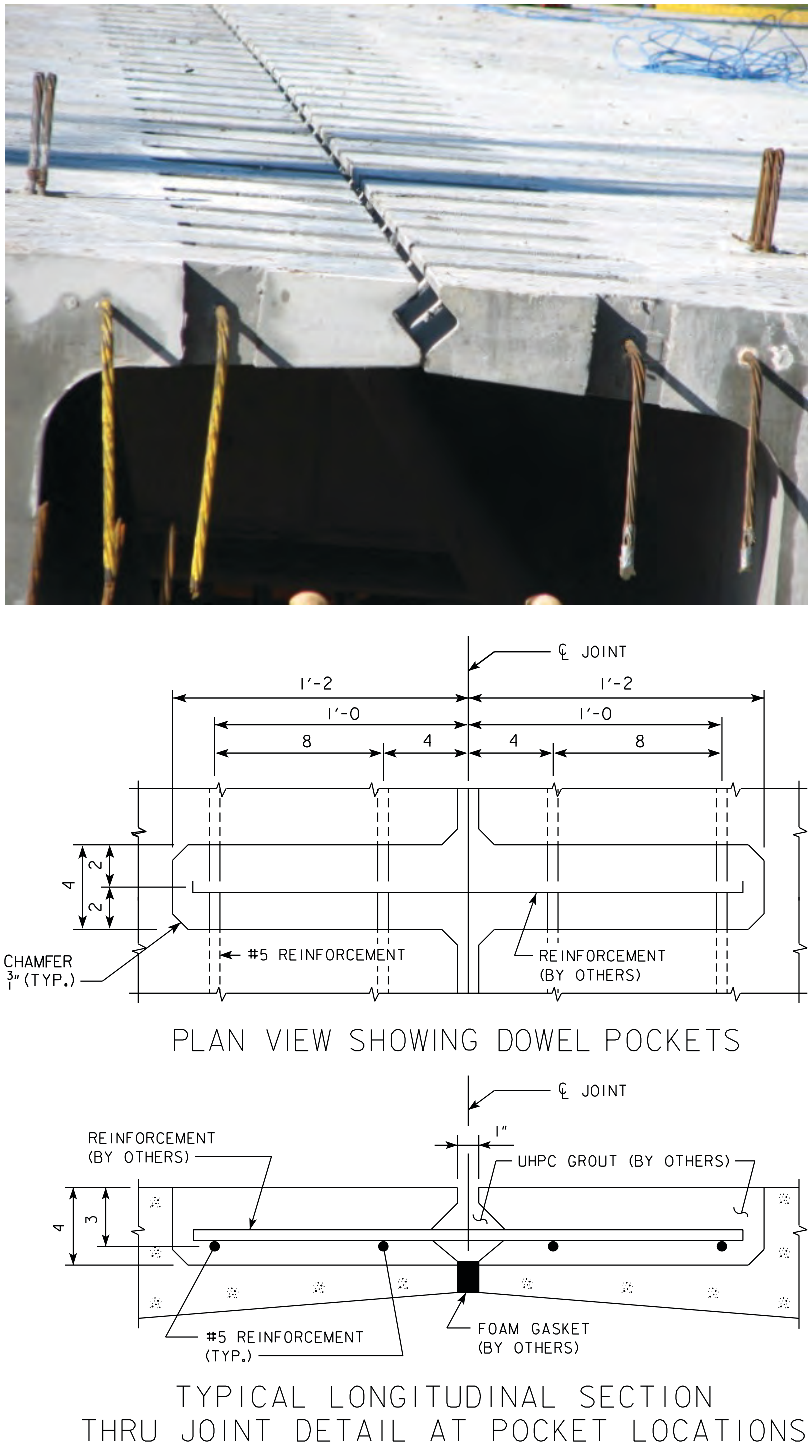
The pi-girder sections are connected using a dowel pocket connection, as opposed to transverse posttensioning, as shown in Figure 2.13. Although the UHPC material was designed to function without conventional reinforcing, previous owners have chosen to provide nominal reinforcing as a redundant system.

A third generation of UHPC superstructures is currently being developed by the Iowa DOT. This system, which consists



Source: Keierleber et al. 2008.

Figure 2.12. Typical UHPC pi-girder section.



Source: Keierleber et al. 2008.

Figure 2.13. Typical UHPC pi-girder longitudinal joint.

of a UHPC waffle slab, could be used as either a short-span bridge or, in a slightly modified configuration, as a fast-track deck replacement system for a girder bridge.

Advantages

UHPC offers a number of advantages for accelerated construction:

- Extremely high strength-to-weight ratio.
- Nearly zero permeability, which provides a very durable material.
- Higher modulus of elasticity than conventional concrete.
- Very ductile behavior that permits gradual deformations under high loads.
- Thin sections permit lightweight pieces that can be erected by smaller cranes.

Disadvantages

- Currently, this material is projected to cost three to five times as much as conventional concrete.
- A longer cycle of casting and heat curing is required to obtain extremely high compressive strength.
- Limited number of casting locations in the United States.

Design Concept D-7: Concrete-Filled Steel or FRP Shell Columns

Concept Description

The use of steel or FRP jacket systems for retrofitting and strengthening of existing concrete piers has achieved moderate success for many years. These form systems are installed as a jacket and then filled with grout containing corrosion-resistant admixtures. Jacketing has been used to extend the life of bridge columns that may suffer from significant spalling due to corrosion of reinforcing steel, or for columns that must be upgraded for seismic considerations. External jacketing is used to provide the desired level of confinement without the need for expensive, time-consuming replacement.

A significant concern with this type of system is the inability of future bridge inspectors to truly understand and document the condition of the enclosed concrete. Advances in nondestructive evaluation (NDE) and testing have evaluated a variety of solutions for monitoring jacketed piers, but the industry appears to remain unconvinced of the value of these inspection methods.

The concept of column jacketing can be used not only as a retrofit for providing additional capacity, but could provide a means for accelerated construction as a "lost form" system, as illustrated in Figure 2.14. The primary goal of such a system would be to maintain typical and accepted detailing convention while quickly increasing the strength of the columns for rapid construction.

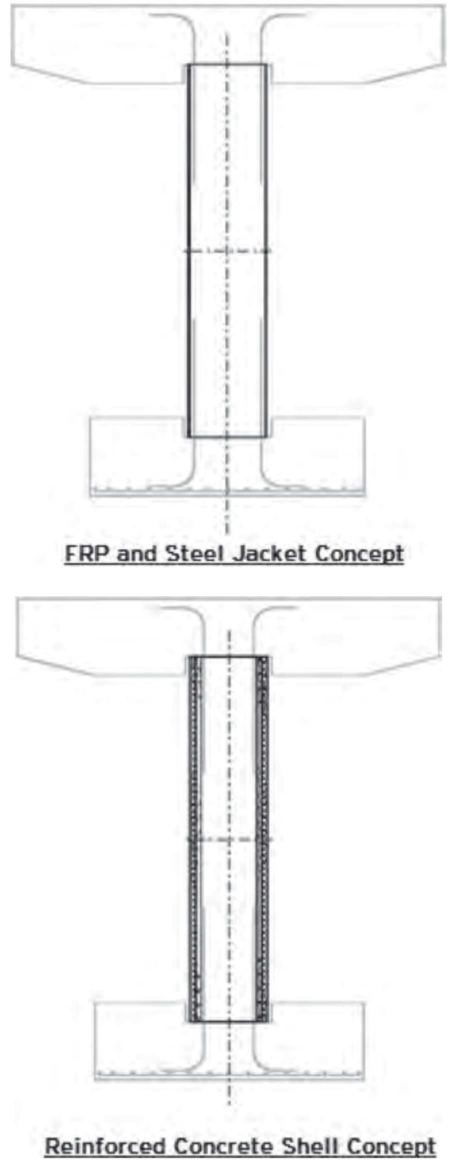


Figure 2.14. Lost form system of column construction using steel or concrete forms.

Fully leveraging the forming system as a primary load carrying member requires connection details that transmit full loads and function as a continuous element from top to bottom. The shells would be erected on site and could be used to support precast or prefabricated cap-beam elements during the construction sequence. The interior space of the shells would then be filled with CIP concrete.

Advantages

- Factory-produced shell components can be easily standardized in a variety of commonly used shapes and sizes.

- Ductile connections and similar details can be developed for seismic applications.
- Easy transportation and erection on site.
- No on-site formwork to be constructed and stripped.
- Suitable for use with all foundation types, including footings and drilled shafts.
- Precast concrete shell system is generally more stable than steel or FRP, but this may be offset by considerably higher weight.

Disadvantages

- May require additional shoring to generate sufficient load capacity during all construction stages.
- Future inspection limitations.
- Heavier weight and larger cranes required for concrete shell system.

Design Concept D-8: Complete Composite Steel Superstructure Systems

Concept Description

The intent with this concept is to develop large construction systems that can be built in the shop in large scale, transported to the site, and then erected by assembling the pieces together with a minimal need of formwork. Steel/composite girder structures lend themselves to this type of ABC because when they are built in the shop the individual pieces are strong and stiff enough to be transported and erected with minimal need for additional stiffening or shoring.

These systems can be made out of steel plate girder systems or steel tub girders, as shown in Figure 2.15, with the decks cast in place, precast at a manufacturing facility, or even on site if there is enough room adjacent to the project site. In addition, these systems can be fabricated as longitudinal sections that can be erected piece wise and assembled together using in-place posttensioning, or they can be fabricated as full width deck systems that can be erected in a single piece. These full deck systems can be made out of composite regular tub girders or as edge girder systems (see alternate concept below).

Figure 2.15. Steel tub girder composite superstructure system.

Trapezoidal steel box girders are very suitable for this type of large building block approach construction. They offer light, cost-effective solutions while providing structural efficiency during transportation, erection, and service life. In addition, they are suitable for curved geometry in situations where bridges are carrying ramp traffic of various curvatures. Furthermore, they offer an aesthetically pleasing solution for bridges being constructed or replaced.

Trapezoidal box girders building blocks can be designed with a single box, two boxes, or as many as needed to carry the width of the deck. However, the most standard use to date involves twin tub girders. These bridges can be designed and constructed to function as simple spans or continuous structures. Several connection details are available and can be used to provide continuity for dead and live loads, either as standard splice construction procedures or specific details applicable to the particular situation at the site.

As mentioned above, steel tubs can be constructed in building blocks that include one longitudinal box, which can extend from splice to splice, as long as a span or more to block lengths that fit the concept visualized by the designer in agreement with the erector.

Steel tub girders are usually designed with the bottom flange parallel to the top of the deck. This lends itself to standard bridge geometry, with the deck of the tub exactly fitting the designed crown of the roadway. Also, the internal geometry of the steel tub is constant, thus simplifying the detailing of the cross frames while these building blocks are being fabricated in the shop.

Depending on the site, composite decks could be designed and erected as cast in place on top of the erected tub girders, which is current standard practice. However, for accelerated construction the decks could be cast in place in the shop (pretopped) for each building block, and then assembled in the field during erection using posttensioning techniques. Construction of the composite deck can also be accomplished using precast, prestressed deck panels placed in the field piecewise and made composite through the secondary pours at the recessed sections of the panels, which are designed to coincide with blocks of shear studs attached to the steel tub girder top flanges, as illustrated in Figure 2.16.

Erection is usually accomplished using shored construction. In certain instances, when spans are short enough, shoring may even be avoided altogether with the building blocks extending from abutment to pier or pier to pier as the weight of the building blocks and the capacity of the cranes permit, as shown in Figure 2.17. However, in most instances, erection could be accomplished using only vertical shoring at certain locations, as necessary from design, but with minimal additional formwork. In either case, shoring can eliminate interference with traffic below, with the steel tub girders providing a working platform to perform deck erection and assembling activities.

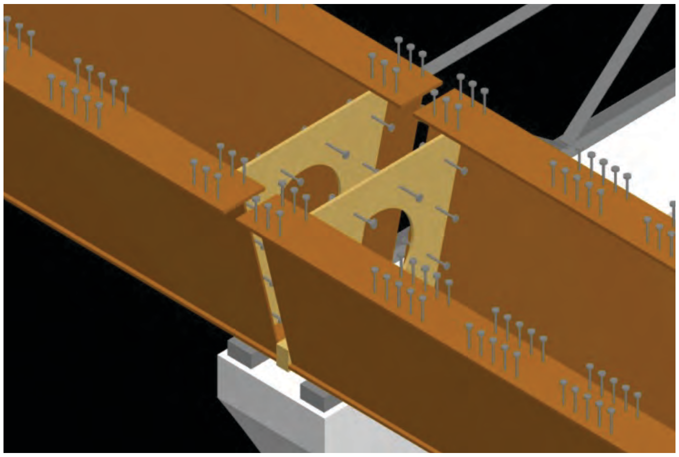


Figure 2.16. Steel tub girder showing shear connectors.

Advantages

Steel/composite superstructures have great potential for economy and for ABC as described below:

- They are lighter primary structural members than precast, prestressed girder structures. This allows for longer spans and larger building blocks to be prefabricated in the shop.
- For simple supported structures they offer structurally efficient solutions with the steel section in tension and durable compression slab in compression. In fact, longitudinal slab compression is achievable in two-span continuous bridges by lowering at the central pier or jacking at the abutments.
- When assembled as structural steel framework-only building blocks, they are very suitable for the rapid application of full-depth precast slab panels. Such panels that are cast and cured in the shop provide for very good dimensional stability due to their long curing times. In addition, they are built to very accurate dimensions and to high quality from industrialized casting processes. Deck panels can even be match cast to fit better in the field during assembly. Using these techniques, field operations involving fresh concrete can be minimized. In addition, these deck panels can be made continuous using longitudinal posttensioning in the field during assembly.
- Because most of the superstructure is built in the workshop it provides for improved quality, planning, and risk management.
- Building blocks prepared in the shop provide long durability from the application of different and renewable protective treatments in the shop combined with touch ups in the field.
- These structures are modifiable to new service conditions.
- The building block approach provides for simpler dismantling at the end of service life.
- Few clearly recognizable structural elements for enhanced aesthetics.
- This type of construction is readily suitable for combination with rapid foundation systems, rapid pier erection, and rapid steel/composite deck assembly. There could be additional weight savings through the use of higher strengths of high-performance steel (HPS) such as grade 70 and grade 100.
- Steel tub girders offer superior stiffness in bending and torsion, thus providing better deflection and vibration control.
- Additional weight savings could be achieved by the use of precast UHPC or lightweight concrete deck panels.
- In most cases, an efficient two-girder system can be implemented.
- Weight saving is achieved with full-length span prefabrication, especially in the case of simple span structure with the composite action applied for all load components: self-weight, super imposed, and live load.
- The system can easily be applied to wider roadway bridges just by adding more pre-decked box girders.
- The same system could be applied with two-plate girders per module connected with cross frames.

Figure 2.17. Erection of steel tub girders.

Disadvantages

Concrete bridges are made of a single material; although that material is not homogeneous or isotropic, for the sake of design we do presume it to be one material. That has worked well so far for designers of concrete bridges of any kind.

Steel bridges are not usually made exclusively from steel. Most of the time, steel bridges are a composite of concrete and steel, which attempts to efficiently provide the functional needs of the bridge of having a rideable surface deck while trying to take advantage of the low cost, versatility, and compressive strength that concrete as a building material has to offer.

To achieve the same quality as conventional construction, a superior knowledge of the behavior of these materials, separately as well as when combined to form an efficient composite material, is usually required. This is even truer in the case of building-block-approach construction. In this case, the behavior of the materials is complicated further by the piecewise application of the assembly, the application of prestressing or posttensioning, the use of closure pours made out of fresh concrete with behavior differing from that of the precast pieces due to time-dependent effects, the different structural layout of shoring or structural continuity to be achieved, and so forth.

These types of structures are often overlooked by funding agencies due to their higher initial cost. Steel tub girders tend to use more steel than traditional steel plate girders for a given deck length and width. However, the savings obtained due to minimizing shoring and formwork and the expedited construction often overcome the initial expenditure for the extra steel.

In addition, many owners are concerned that twin tub girder structures are in fact two girder systems and therefore qualify as fracture critical and hence need to be inspected and maintained as such. The life-cycle costs of required fracture critical inspections discourage owners from choosing these structures. However, the current availability and application of HPS with its superior toughness, combined with recent research performed in Wisconsin for the Marquette Interchange Reconstruction and at the University of Texas at Austin, have shown that these types of structures are much more robust as currently accounted for in design and they exhibit as much redundancy as other conventional structure types.

On the other hand, marrying steel and concrete in the shop requires the steel shop fabricator and concrete precaster to work at a different level and combine their activities in the shop. This could present logistical as well as preferential challenges.

Transportation of these large building blocks also presents logistical challenges concerning available transportation routes, permitting, sizes of the pieces as compared to available routes, available transportation and erection equipment, and so forth.

Contractors may not be familiar with this type of construction and may initially shy away from selecting it as a viable cost-effective option.

Other disadvantages include the following:

- The need for heavy structural shoring designed for the weight of most of the span;
- In some instances, the need for additional right-of-way;
- Incompatibility with irregular or inaccessible sites; and
- The need for casting facilities and storage areas in multi-span bridges.

Alternative Concepts

In cases of quick bridge replacement, while maintaining traffic in the existing bridge, steel edge box girders can be effectively used for full-width, full-span applications subject to weight and size limitations. These systems can be erected outside the edge of the existing bridge and can be designed as simple spans, or multiple continuous-span systems with appropriate details for continuity. The sequence of erection for this alternative concept is as follows:

- The new foundation shafts and piers are erected on either side of the existing bridge.
- Two parallel edge box girders are built on either side of the existing bridge. Incremental launching could be used for short to medium spans, while balanced-cantilever construction of varying-depth girders can be used for longer spans.
- Gantries running along the edge girders lift the existing spans, or full-length strips of the span, and transport them to the abutment for demolition.
- The same gantries are used to place transversely ribbed precast UHPC deck slab panels between the two edge tub girders for the final deck.
- Installing all the precast deck panels before connecting them to the steel structure eliminates permanent tensile stresses in the deck.

These systems with edge girders and crossbeams are conventionally used in cable-stayed bridges and other two-girder systems.

Design Concept D-9: Complete Precast Concrete Superstructure Systems

Concept Description

This category contains three options: precast segmental decks, voided slabs, and the channel bridge section.

There is an unfortunate perception in many parts of the country that voided slabs have been plagued by performance and durability problems. This is not the case, considering that the United States has not used voided slabs as defined elsewhere in the world. True voided slab systems have been successfully designed and constructed at low initial cost in Canada and Europe, where they are known for superlative strength and durability.

Voided slabs, as defined in U.S. practice, generally mean box girders that have been used for short and intermediate spans with inadequate detailing, poor workmanship, and a

Source: New York State DOT.

Figure 2.18. Typical section during erection, showing simple lifting frame.

lack of proper posttensioning. For spans up to approximately 100 ft, the system can be simplified considerably by using solid slab sections that are match cast and posttensioned for extremely high durability and low cost. ABC concepts provide the most economical option for falsework erection where traffic conditions allow.

There are several examples of this technology already in use in the United States, including SR-54 in Delaware. The so-called channel bridge section was originally endorsed by the New York State DOT, and two prototype structures were built, as shown in Figures 2.18 and 2.19.

This type of system originally was hindered by two limitations:

- Private interests were hoping to develop this as a proprietary product; and
- Width restriction occurred, based on design limitations.

Source: New York State DOT.

Figure 2.19. Channel bridge, SR-17M in New York, showing skew and under-slung erection frames.

Both of these limitations can be readily overcome, and some applicable design and construction features are exhibited in these two demonstration projects, including the following:

- Small-scale application of precast segmental technology in the United States;
- Incorporation of the barrier into the structure cross section for efficiency;
- Demonstrated redundancy with one barrier completely destroyed by accident; and
- Small-scale application of proven under-slung erection gantry technology for typical grade separation bridges.

The best example of full span precast systems to date in the United States is the Robert Moses Parkway in New York State.

Complete spans can be prefabricated on site or at an established precasting facility, depending on proximity and local site conditions. Complete spans can be modularized per Concept D-10 for decks that become prohibitively wide or heavy for shipping and erection.

Developing Standardized Designs

The intent for this concept is to develop pre-engineered standards for complete spans for bridges up to 140 ft that can be transported and erected in one piece. For short spans, these segments can be purchased and erected by owner crews using conventional equipment in a few days.

Design Considerations for Standardized Deck Segments

- Pre-engineered standards for modular construction. Designs can be used for most sites with minimal bridge specific adjustments.
- Optimized designs for ABC and use of high-performance materials. Simplicity and efficiency of design, availability of sections, and short lead times are key considerations.
- Length ≤140 ft, weight ≤100 tons, width ≤8 ft.
- Skew can be readily dealt with for all three options.
- Deck systems should all be match cast.
- Prestressing or posttensioning can be used in the longitudinal direction.
- Posttensioning criteria can be provided to eliminate tension and creep-shrinkage cracking, with significant improvements in durability and reduced maintenance costs.
- Posttensioning also will result in increased life expectancy of superstructure systems in all cases.
- Prestressing will require the use of relatively expensive casting beds and initial investment.
- This will require large scale application for economy.
- Transverse posttensioning in the field for match cast joints between modularized versions of the channel bridge option.

- Segments designed for transportation and erection stresses. This is an issue only for full spans in the channel bridge option.
- Segments should be match cast in all cases.
- Geometry control has historically been a "black box" issue for most owners, but it is rather straightforward and means that a smooth ride is easily achievable without an overlay.
- All three options can be adapted for simple spans and for continuous spans (simple for dead load and continuous for live load), with details to eliminate deck joints and provide for live load continuity at piers.
- All three options can be developed with sidewalks, curbs, and barriers manufactured integrally with the cross section.
- The integral behavior of curbs, barriers, and sidewalks will typically increase the total moment of inertia of the cross section by up to 25%, or possibly higher. This is a significant advantage that can be integrated into the design of the cross section for higher efficiency, lower weight, and reduced cost.
- Standard details for durable connections between deck segments.

Design Concept D-10: Modular Superstructure Systems

Concept Description

The intent with this concept is to develop pre-engineered standards for modular deck segments for concrete and steel bridge superstructures with spans of up to 140 ft that can be transported and erected in one piece. Longer spans can be transported in sections, spliced on site, and erected using special techniques such as girder launching. Standardizing designs to no more than five sections (for each of the three deck segments) that will cover the span range from 40 ft to 140 ft will increase their availability through local or regional fabricators, reduce lead times, lower costs, and increase familiarity among local contractors. Segments for short spans can be purchased and erected by county crews using conventional equipment in a few days. The deck segment concepts incorporate proven elements or details used by the New York State DOT and the Washington State DOT. Similar details have been used by the Utah DOT, the Idaho DOT and other state DOTs. Refinement of these concepts in Phase II will entail the development of standardized deck sections optimized for ABC, as well as modular construction that addresses specific ABC design considerations, as discussed below.

Four Options for Standardized Modular Deck Segments

- Deck bulb tees with integral deck.
- Double tees with integral deck.
- Decked stringer system (two beam steel sections with slab).
- Decked trapezoidal boxed girders.

Design Considerations for Standardized Deck Segments

- Pre-engineered standards for modular construction. Designs that can be used for most sites with minimal bridge-specific adjustments.
- Optimize designs for ABC and use of high-performance materials. Simplicity and efficiency of design, availability of sections, and short lead times are key considerations.
- Usually length ≤140 ft, weight ≤100 tons, width ≤8 ft for transportation and erection using conventional construction equipment.
- Able to accommodate moderate skews. For rapid renewal, it would be more beneficial to eliminate skews altogether by making the bridge spans slightly longer and square.
- Segments designed for transportation and erection stresses, including lifting inserts. Sweep of longer beams should not be an issue for erection as there is an opening between the beams.
- Segments that can be installed without the need for cross frames or diaphragms between adjacent segments. This improves speed of construction and reduces costs. Use of diaphragms is optional based on owner preference.
- Segments that can be used in simple spans and in continuous spans (simple for dead load and continuous for live load). Details to eliminate deck joints at piers. Details for live load continuity at piers to be included for use as required.
- Use of high-performance materials: HPC/UHPC concrete, HPS, or A588 weathering steel. Consider lightweight concrete for longer spans to reduce weights of deck segments.
- Deck tees and double tees with minimum 8-in. flange to function as decks with integral wearing surface so that an overlay is not required. Use of overlay is optional (see below).
- Cambering of steel sections for longer spans. Control fabrication of concrete sections, time to erection, and curing procedures so that camber differences between adjacent deck sections are minimized. Leveling procedure to be specified to equalize cambers in the field during erection.
- Deck segments when connected in the field should provide acceptable ride quality without the need for an overlay. Deck segments to have ¼-in. concrete overfill that can be diamond ground in the field to obtain desired surface profile.
- Limit the number of standardized designs for each deck type to five, which should cover span ranges from 40 ft to 140 ft. Consider steel rolling cycles and sections widely available.
- Segments designed to be used with either full moment connection between flanges or with shear-only connections. Each flange edge needs to be designed as a cantilever deck overhang.
- Design for sections that can be transported and erected in one piece, as shown in Figure 2.20. Lengths up to 140 ft may be feasible in certain cases. Provide one method of erection. (Spans longer than 140 ft may be erected by

Figure 2.20. NY-31 Bridge—installation of deck sections.

shipping the segments in pieces, splicing on site, and using a temporary launching truss for erection, as discussed in the section on Construction Concept C-2.)
- Design for sections that can be transported in pieces and spliced on site before erection to extend spans to 200 ft and beyond. Develop two alternate erection techniques when conventional lifting with cranes may not be feasible due to weight or site constraints.
- Edge sections of deck with curb piece ready to allow bolting of precast barriers.
- Provide standard details for durable connections between deck segments.

Achieving Ride Quality with Precast Deck Segments

- While the application of an overlay helps overcome finite geometric tolerances, it also requires another significant critical path activity prior to opening a structure to traffic.
- Today's availability of low-permeability concretes and corrosion-resistant reinforcing steel allows owners to forgo the use of overlays on bridge decks.
- With prefabricated superstructure construction, the challenge is to develop methods that achieve a final ride surface without the use of overlays. Control of cambers during fabrication and equalizing cambers or leveling in the field is intended to achieve the required ride quality.
- An attractive option is diamond grinding decks with sacrificial cover to obtain the desired surface profile. Such a method can be faster and more cost-effective.
- For continuous or multiple simple spans, beam cambers may affect ride quality to a point where an asphalt overlay system may be recommended (see the following discussion).

Camber Control During Fabrication and Equalizing Cambers in the Field

- Differential camber of the beams can lead to dimensional problems with the connections.
- Schedule fabrication so that camber differences between adjacent deck sections are minimized. Measure camber on each deck section immediately after the transfer of prestress forces. (The Washington State DOT requires that at transfer of prestress, the difference in camber between adjacent deck sections of the same design must not exceed ¼ in. per 10-ft span length, or a maximum difference of ¾ in., whichever is less.)
- Equip all deck sections with leveling inserts for field adjustment or equalizing of differential camber. The inserts with threaded ferrules are cast in the deck, centered over the beam's web. The Washington State DOT specifies a minimum tension capacity of 5,500 lb for the inserts. After all adjustments are complete and the deck sections are in their final position, fill all leveling insert holes with a nonshrink epoxy grout.
- The welded joint details can accommodate minor differential camber. If the differential camber is excessive, the contractors in some states will apply dead load to the high beam to bring it within the connection tolerance. A leveling beam also can be used to equalize camber.
- Have a leveling beam and suitable jacking assemblies available for attachment to the leveling inserts of adjacent beams, as shown in Figure 2.21. Adjust the deck sections to the tolerances required. More than one leveling beam may be necessary.
- If the prescribed adjustment tolerance between deck sections cannot be attained by use of the approved leveling

Figure 2.21. NY-31 Bridge leveling procedure for adjacent beams.

system, shimming the bearings of the deck sections may be necessary.

Preservation Strategy and Use of Asphalt Overlay

- The combination of high-performance concrete and high-quality construction will provide a long service life for these systems. Some owners may, however, have concerns about the long-term durability of bare decks. Use of an asphalt overlay with a membrane could be a desirable option in such situations to provide enhanced durability. In most cases, the overlay can be installed in a day prior to opening the bridge to traffic, or the overlay can be done during night lane closures at a later point. If the bridge is constructed and opened during the cold-weather months, the asphalt overlay can be installed when warm weather returns and the asphalt plants open.
- Asphalt overlay will provide improved ride quality.
- The use of asphalt overlay may be required in bridge widening and for multiple simple spans, to even out the roadway profiles.
- European practice is to always use an asphalt overlay with a membrane as a protective system for bridge decks. Their experience indicates that keeping water away from bridge decks significantly improves service life.
- The preservation strategy for bridges with an asphalt overlay would be to replace the overlay on an as-needed basis. For bridges without an overlay, a new bonded concrete overlay or topping slab may be added to compensate for any deck deterioration.
- The team recommends investigating the substitution of FRP bars in place of steel rears in the deck slab/top flange to achieve a longer deck life. The FRP bars may cost two or three times more than steel, but the overall cost impact would not be much. Several FRP-reinforced bridges are in service and have performed well.

Design Considerations for Connections Between Deck Segments

ABC considerations for joint details include the following:

- They can achieve durability at least equal to that of the deck.
- Joint designs should consider truck traffic severity to achieve durability.
- Joint details suitable for heavy/moderate/light truck traffic sites.
- They can achieve acceptable ride quality (similar to CIP decks).
- Do not require overlays (overlay use is optional).
- Do not require posttensioning.
- Details can accommodate slight differential camber.
- Can be opened to traffic in a matter of hours or days.
- Avoiding the need for placement and removal of formwork is preferable, requiring access from below.

Potential Joint Types

- Match cast and posttensioned joints are acceptable alternatives for which designers can find information on from other sources.
- Passively reinforced joints (full moment connections suitable for heavy truck traffic sites).
- Welded or bolted joints (shear-only connections suitable for moderate to light truck traffic sites).

Heavy Truck Traffic Sites: Full Moment Connection Using Ultra-High-Performance Concrete (UHPC) Joints

- Passively reinforced 6-in.-long joint, no posttensioning.
- Full moment connection suitable for heavy-truck traffic sites, but can be used under less severe traffic situations.
- The placement and curing of UHPC can be performed using procedures similar to those already established for use with some high-performance concretes (HPCs). The fluid mix is virtually self-placing and requires no internal vibration.
- UHPC can provide significant durability improvements to bridge decks due to its high strength, extremely low permeability, high resistance to freeze thaw, and the improved connection details inherent in the system. Research demonstrates that UHPC exhibited almost no permeability and was not susceptible to chloride ingress.
- In the New York State DOT detail, the shorter development length of reinforcing bar in UHPC allowed a narrower joint that reduced the total shrinkage. Tests done by the New York State DOT show that a 5-in. development length was sufficient for #6 rebar. This allowed a full moment connection to be made using a 6-in. closure pour and straight rebar. The New York State DOT successfully completed a project in 2008 using a UHPC joint.
- In tests done at Michigan Tech Transportation Institute, the UHPC showed compression strength of 28,000 psi, compared with 4,000 psi for normal concrete. Tensile cracking strength was above 1,000 psi, compared with 400 psi for normal concrete. In testing for resistance to road salts and chlorides, UHPC withstood these chemicals at a rate 100 times greater than normal concrete.
- The compressive strength gain behavior of UHPC is an important characteristic of the concrete. UHPC does not have any compressive strength for nearly 1 day after casting. Then, once initial set occurs, UHPC rapidly gains

strength over the course of the next few days until over 10 ksi of strength is achieved in about 3 days. No special curing is needed for the joint material (though steam curing is beneficial when applied). Regardless of the curing treatment applied, UHPC exhibits significantly enhanced properties compared with standard normal strength and HPCs.

- Level any differential camber between adjacent beams before placing the joint. Slight differences in camber (<¼ in.) can be tolerated.
- Installation time is about 3 days, including erecting, placing, closure pours, and curing.

FHWA is conducting additional testing on UHPC joints. The results will provide improved guidance for design.

Heavy Truck Traffic Sites: Full Moment Connection Using High-Performance Concrete (HPC) Joints

- Alternate passively reinforced joint using HPC; no posttensioning.
- Full moment connection suitable for heavy truck traffic sites, but can be used under less severe traffic situations.
- The lapping of steel may be achieved with overlapping looped bars or short straight bars whose development is improved by the geometry of the joints or by external means, such as confining spirals.
- The greater widths (up to 3 ft) that are typical for these joints, relative to UHPC or welded or bolted joints, may increase the likelihood of shrinkage cracking and may require erecting and removing formwork from below.
- The interface between the precast deck and the cast-in-place closure is of particular concern, since cracks can develop due to shrinkage. A penetrating sealant should be applied to the top surface of grouted joints after curing to enhance durability.
- Level any differential camber between adjacent beams before placing the joint. Slight differences in camber (<¼ in.) can be tolerated.
- Installation time is about 3 days, including erecting, placing closure pours, and curing.
- Researchers are investigating the use of a small closure pour with headed reinforcing bars.

Moderate to Light Truck Traffic Sites: Shear-Only Connection by Welding and Grouting

- Use a welded tie connection combined with a grouted key. Steel ties, 6 in. long and ½ in. thick, are normally spaced 5 ft on center along the edge of the beam. They are welded to angles embedded in the beams and anchored with studs.
- This connection is primarily designed as a shear-only connection. There is no intent to make this connection a deck moment connection. Each flange edge needs to be designed as a cantilever deck overhang.
- The Texas DOT has researched transverse-welded connections for adjacent precast members and found that when combined with a grouted shear key, the connection is sound and durable. The Washington State DOT and the Idaho DOT have also used a welded joint detail for precast members.
- Any differential camber should be leveled before welding. Connection can be made even if there is slight camber differential between the beams.
- Installation time is about 2 days, including erecting, welding, and grouting. Multiple spans can be built in the same time frame with larger construction crews.
- The use of fiber-reinforced grouts can enhance joint performance. Some welded joints have not worked well under certain applications. The Utah DOT has had some issues with leakage. More study of welded joints is recommended.

Design Concept Sketches (see Chapter 3)

- Deck bulb tee system with diaphragms;
- Deck bulb tee system without diaphragms;
- Joint details;
- Deck slab at pier;
- Deck segment details;
- Double tee system;
- Decked stringer system; and
- Decked trapezoidal box system.

Research and Testing Needs

- Experimental investigation of UHPC joints being done by FHWA should continue to be monitored.
- Investigations of other joint-reinforcing details being done by various researchers should be completed.

Design Concept D-11: Pre-Topped Trapezoidal Concrete Tub Beams

Concept Description

The intent of this concept is to develop superstructure systems using the Texas DOT U beams in a pre-topped condition for spans up to 115 ft that can be transported and erected in one piece, as shown in Figures 2.22 and 2.23. Longer spans can be accomplished by splicing the sections on sight and posttensioning. These longer sections could be launched into place, erected with overhead gantries, or erected with two large cranes. Standards for this system would be developed to cover span ranges from 60 to 175 ft, with no more than five standard cross sections used. Standardization, coupled with current

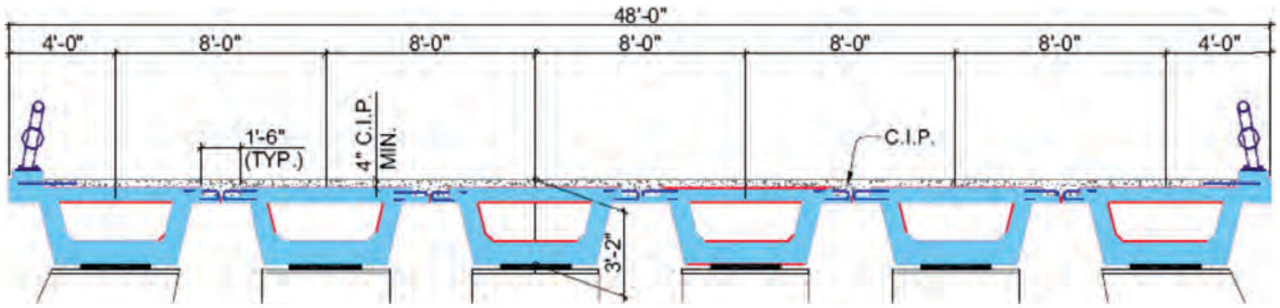


Figure 2.22. Typical bridge cross section.

widespread use in Texas, will increase the U beams' availability with local and regional fabricators and drive down the cost associated with designing, detailing, and fabricating the units. A system that can span 115 ft has already been successfully completed in Texas, where precast shell columns and complete superstructures were erected in as little as 4 days. Designs from the bridges done in Texas, developed by the Texas DOT and Structural Engineering Associates in San Antonio (part of the R04 team), would give this concept a significant head start.

Two Options for Pre-Topped U Beams

- Spans 60 to 115 ft, transported and erected in one piece.
- Spans 60 to 175 ft, transported in 10-ft lengths, and post-tensioned on site.

Design Considerations for Pre-Topped U Beams

- Standard sections already in use would be used to minimize fabrication costs.
- Designs would be optimized to use high-performance materials to reduce weight.
- Lengths less than 115 ft produce sections under 150 tons for shipment in one piece. For longer spans, or where overhead gantries or launching is the preferred method of erection, 10-ft segments would be cast and posttensioned on site.
- Units would be designed to handle transportation and erection stresses.
- An overlay can be provided with this system and still allow the bridge to be opened within 4 days of the beginning of superstructure erection.
- Limit the number of standardized sections to five, which will cover span ranges from 60 to 175 ft.
- Provide two or three suggested methods of erection, such as cranes, launching, and overhead gantries.
- Edge sections of deck with curb pieces to allow bolting of prefabricated barriers.
- Provide standard details for connections between sections.

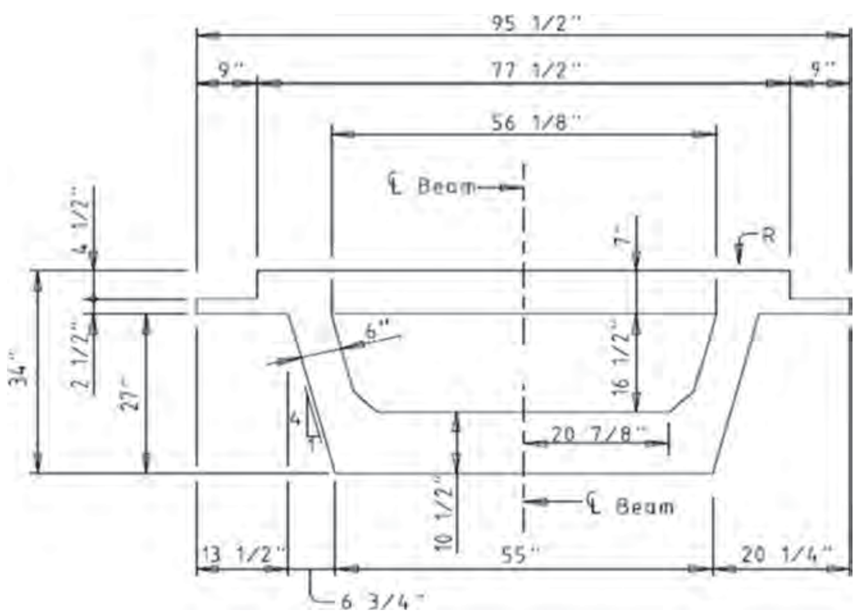


Figure 2.23. Dimensions of 32-in.-deep section used to span 115 ft.

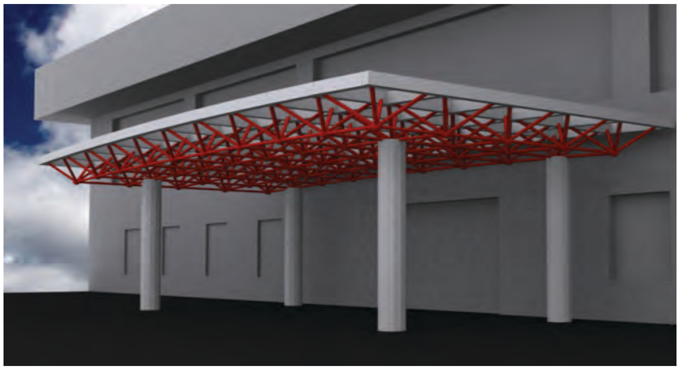


Figure 2.24. Space frame roof structures example.

Design Concept D-12: Space Frame Bridge Superstructures

Concept Description

The steel tubular space frame is a lightweight structure that has seen significant growth, primarily in the building industry, as shown Figure 2.24. Today there are numerous manufacturers of steel space frames. These manufacturers have automated the fabrication process and advanced the nodal connection details, making these structures cost competitive and potentially ready for the bridge industry.

Advantages

- Lightweight, efficient structure.
- Can span up to 150 feet.
- Large load distribution performance.
- Highly redundant; requires simultaneous failure of numerous members to precipitate a collapse.
- Two-way slab action; composite deck slab spans in orthogonal directions results in minimum slab thickness.
- Easily standardized.

Disadvantages

- Lack of performance information for bridges.
- Fatigue performance must be verified.
- Not applicable for rapid construction.

Due to their light weight, steel space frames lend themselves to rapid construction. They can be prefabricated with composite deck slab in sections, trucked to a bridge site for crane erection, and then connected together with closure pours. In another possible erection scheme, the steel space frames are trucked to the site in sections and erected vertically on both sides of the abutment to allow traffic to continue. The concrete deck slab is then poured and cured while the space frames are secured in the vertical position. After curing, the sections could be rotated down and connected together while the existing bridge is rapidly removed. The completed superstructure would then be rolled out into position using a previously erected launching beam.

Design Concept D-13: Precast Concrete Deck Systems

Concept Description

The use of full-depth precast concrete deck panels, as shown in Figure 2.25, is not a new concept. In fact, a number of state DOTs have used a variety of these systems with varying success. Traditional cast-in-place (CIP) bridge deck slabs often take up a significant portion of construction schedules while the contractor forms, places, and ties the reinforcing steel and places and cures the CIP concrete.

Eliminating the overlay allows the bridge to be reopened to traffic faster, as CIP concrete is needed only at the joints between the prefabricated panels. The preferred alternative to cast-in-place joints, which takes additional time and effort, is match cast joints.

Match cast joints have been used successfully on at least one project in the United States. In conjunction with long-line casting that requires virtually no complex geometry control, a finished, rideable surface is achievable in the precast plant.

If for some reason match cast joints are not possible, rapid-set concrete mixes that do not require skilled concrete placement and finishing workers can be used for these joints.

Whether or not match cast joints are used, field posttensioning should always be used for full-depth precast deck panels. The reasons include durability and serviceability. The amount of conventional reinforcing can be reduced significantly with an overall cost saving. This results in a virtually crack-free deck and guarantees a significant life expectancy for the deck.

The recommended technology is PVC duct, four strand flat tendons in conjunction with mono-stressing. This

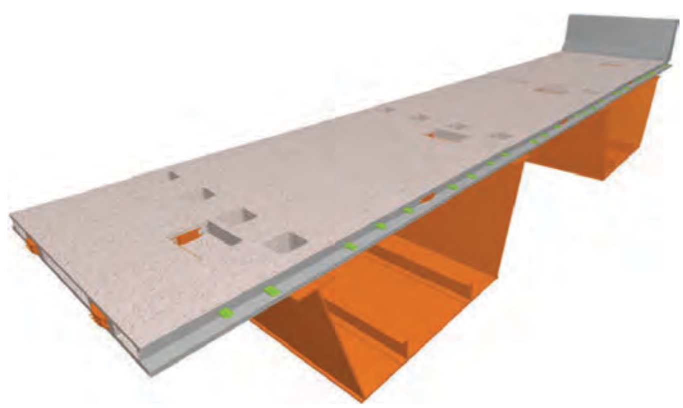


Figure 2.25. Typical full-depth precast concrete deck panel.

posttensioning system can be installed by the contractor without specialty equipment or labor quickly and easily.

Eliminating the CIP joints accelerates the schedule considerably. The addition of posttensioning does not increase the time of construction because the posttensioning is required to extrude the epoxy on match cast joints and occurs simultaneously.

When used to their full advantage, full-depth precast concrete deck panels can reduce construction time by several weeks or months when compared to cast-in-place systems, depending on the size of the bridge. A wide variety of these systems have been developed and tested by owners across the United States. NCHRP Project 12-65 and NCHRP Report 584 (Badie and Tadros, 2008) document a system that provides the optimum benefits of a full-depth system and partially addresses the opportunities for innovation listed below.

One of the challenges of a precast deck panel system is the need to provide a fully composite connection between the concrete deck panels and the steel or prestressed concrete girders. There has been significant research into the use of both larger-diameter, higher-capacity shear studs, which reduce the number of studs required to make the connection, and the current AASHTO *LRFD Bridge Design Specifications*, 4th ed., limitations of 24-in. maximum spacing between stud clusters. Research results indicate that 48-in. spacing may be acceptable, and this specification is under consideration for inclusion in future codes.

Conventional studs are not the only answer. The University of Nebraska at Omaha has developed a process in which the concrete is omitted continuously along the top flange, leaving the reinforcing exposed at the top and bottom until after erection. This results in a longitudinal closure strip, which is grouted at the same time as the haunches.

This may not be the most desirable detail, but several other composite action details are under development, as shown in Figure 2.26.

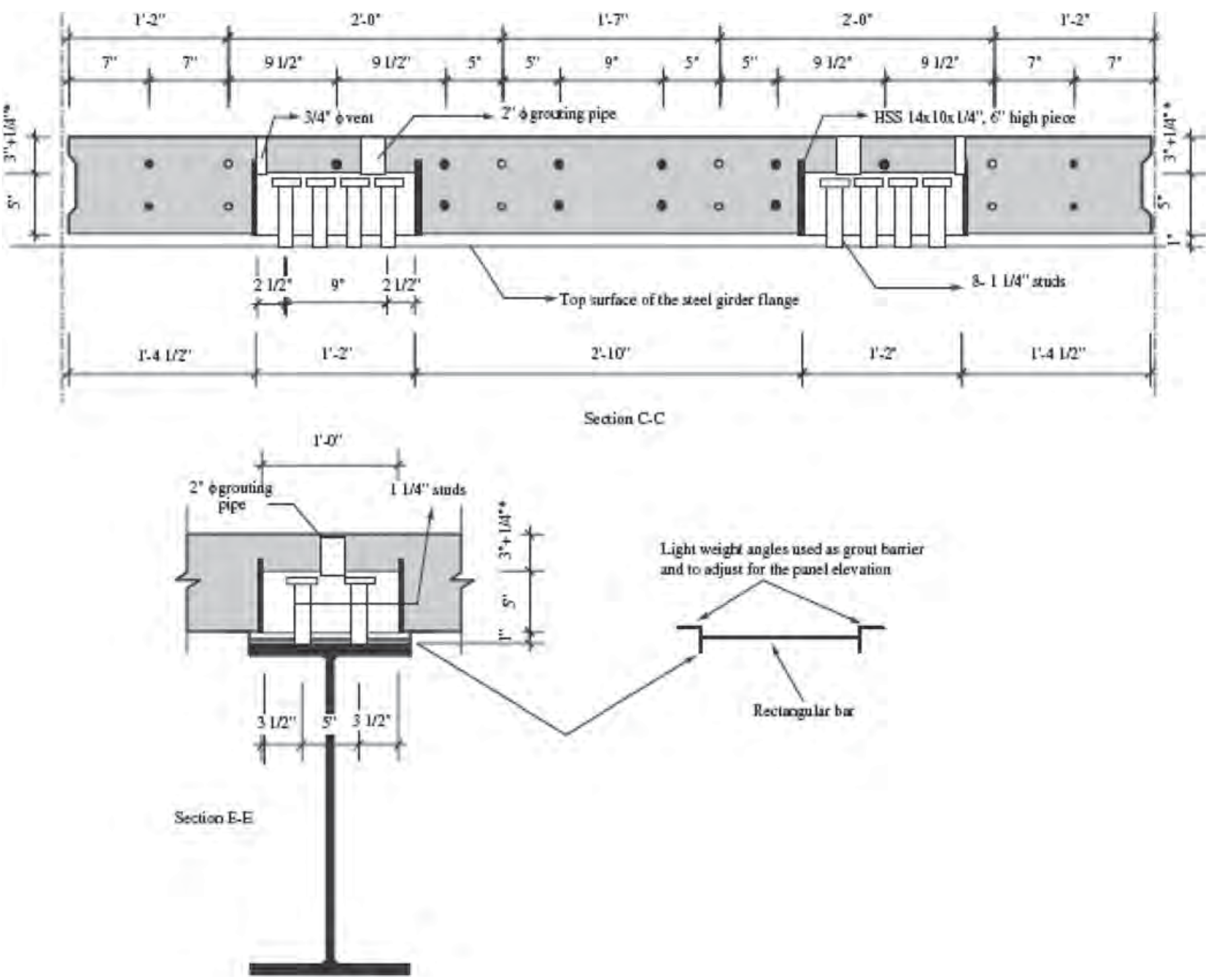


Figure 2.26. Shear connector detail for precast concrete deck panel.

Advantages

Full-depth precast deck panels offer a number of significant advantages in the ABC environment:

- Accelerated erection.
- High-quality plant production with tighter production tolerances.
- The ability to precast panels year-round without regard to weather. This is especially important if a standardized system can be developed and precast panels stockpiled for future use at a variety of sites.
- Low permeability and reduced reinforcing corrosion.
- Reduced variation in volume caused by temperature or shrinkage during initial curing.
- Reduced maintenance costs.

Disadvantages

- All concrete bridge decks should ideally be provided with an overlay or waterproofing system to act as a chloride sink and increase the life expectancy of the deck. This wearing surface requires considerable time to install in all cases, and the curing time necessitates delays in returning the bridge to service.
- Longitudinal posttensioning, which is generally not favored by a number of DOT bridge owners, may be required.
- Future maintenance inspections may be difficult.
- Precast panel components are typically very heavy and often limit the size of pieces that can be shipped to the site and erected by conventional equipment.

Innovative Opportunities

The use of full-depth concrete deck panels offers a number of significant opportunities to improve work that has been completed in the past including

- Simple methods to provide cross slope for full-width panels. This could be in the form of a "hinge" at the crown to permit panels to ship flat and be constructed on site without the need for a splice.
- Durable transverse panel connection for staged construction. One possibility for this application is the use of UHPC, which has been successfully used by the New York State DOT, as shown in Figure 2.27.
- Reduced dead load to simplify installation. The use of lightweight concrete, ultra-high-performance materials, or a waffle-slab configuration offers significant potential.
- Improved riding surface. In the past, a cast-in-place wearing surface (which offers the additional benefit of a high-density protective layer) or diamond grinding to provide a smooth ride has been used. Improvements in shimming and match casting to provide a smooth surface immediately after placement would be beneficial.

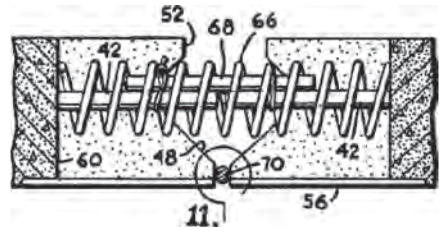


Figure 2.27. The New York State DOT UHPC joint detail.

Design Concept D-14: Concrete-Filled Steel Tube Bridge

Concept Description

The structure is an assembly of steel pipes of different diameter and is composed of three modular elements:

- Foundations;
- Substructures; and
- Superstructure.

The pipes are filled with concrete during erection for composite action.

The foundations are steel composite pipes with soil cement. The steel pipe is inserted during down-hole drilling and cement grout is mixed with soil inside and around the pipe during drilling. Base concrete is poured and the drilling head extracted. Ribs welded to the outer surface of pipe increase friction.

The columns are steel pipes filled with concrete. The steel pipe is inserted into the top section of the steel casing of foundation shaft and concrete is poured into the solution of continuity for friction connection.

The deck is made of twin concrete-filled pipe girders supporting UHPC precast deck slab panels. At the column connection, a vertical steel pipe is inserted into the top section of the column pipe, and concrete is poured into the solution of continuity for friction connection.

Design Considerations

- Quick installation of foundations, low noise level, and no removal of soil.
- Commercial circular steel tubes manufactured by cold-forming and high-frequency electric resistance welding have excellent industrial productivity.

- Thin steel pipes provide to concrete filling
  - Falsework and formwork during filling;
  - Well-distributed reinforcement;
  - High transverse confinement for enhanced ductility; and
  - Protection from aggressive agents for improved durability.
- Concrete filling provides to thin steel pipes
  - Additional compressive strength and stiffness;
  - Control of local buckling without any need for stiffeners; and
  - Enhanced ductility.
- Overall, the CFST provides
  - Rapid assembly of tubular structure;
  - Smaller cross-sectional dimensions for a given strength; and
  - Higher impact and seismic resistance.
- Ductility ratios of 6 to 8 can be expected in the concrete-filled columns.
- International design standards are available.
- Rapid robot welding of field splices with standard pipeline equipment and techniques.
- Long durability and enhanced fire resistance.
- Prestressing of the highly confined infill concrete possible on longer spans.

Design Concept

Conventional methods of driving steel piles cause noise and vibrations. To mitigate nuisance, steel pipes can be installed into bored holes and filled with concrete. The construction cost of these piles is higher, however, and removal of soil from boring and the spoil area for the removed soil cause additional costs and nuisance.

The columns are steel pipes filled with concrete. The base of the steel pipe is inserted into the top section of the steel casing of the foundation shaft and concrete is poured into the solution of continuity for friction connection.

The deck is made of twin concrete-filled pipe girders supporting UHPC precast deck slab panels. At the column connection, a vertical steel pipe is inserted into the top section of the column pipe, and concrete is poured into the solution of continuity for friction connection.

Work to Be Done in Phase II to Refine Concept

- Study of field splice robot welding with current pipeline equipment and techniques.
  - Consistent weld quality with microprocessor control of welding parameters reduces weld repairs and labor costs and virtually eliminates the risk of human error.
  - Automation in the welding process reduces dependency on experienced operators.
  - Real-time information.
- Evaluation of the main components of the orbital welding process.
  - Pipe-facing machine produces the required compound narrow bevel, covers a wide range of pipe diameters, provides precise and consistent bevels, and is self-contained, mobile, and easy to set up. Field beveling may also avoid workshop preassembly and permit geometry adjustments.
  - Hydraulic line-up clamps allow proper alignment of the two joints in preparation for the automatic welding process; copper or ceramic backing shoes are used to contain welding.
  - Automatic welding system. In conventional pipeline applications, pipe diameters range from 4 to more than 60 in. Two welding robots travel around the pipe on a guiding band, with each robot welding 180° of the pipe. Each welding robot controls the wire feeder unit and welding power source, which adjusts according to welding parameters as the welding position changes. In addition to orbital travel around the pipe, the welding robot provides three-axis movement, controlling the welding working distance, oscillation, and angle of the welding head.
  - Preliminary design of the solution.
  - Standardization of international codes.
  - Pile-column and column-deck connection detailing to be standardized.

Concept ND-1: Next-Generation Design Concepts

Next-generation concepts described in this section include new but proven design concepts as well as new material technologies that are not considered market ready for widespread use in the United States. As such, these new market technologies will not be recommended for further development under this project. Standardizing these technologies and reducing costs would entail additional research and development efforts. For instance, two girder systems are relatively common in France, even in new construction, but their continued application in the United States will be limited due to redundancy concerns. Advanced composite materials, such as FRP, have been used on several bridge renewal applications in the United States but their greater use has been impeded by high costs compared with traditional construction.

Three new technologies/design concepts are discussed:

- Concept ND-1a: Two girder systems;
- Concept ND-1b: Compliant composite web systems; and
- Concept ND-1c: Space frame segmental bridges.

Concept ND-1a: Two Girder Systems

Concept Description

The two girder system typically takes full advantage of the load distribution characteristics of a two-way slab, and engages both girders in an efficient manner regardless of how the load is applied to the superstructure.

In the design of any bridge, the ideal load fraction for which each girder is designed can be added to the load fraction for which all the other girders are designed, the sum of which should never be any larger than the actual live load that is applied to the bridge.

This ideal has been effectively achieved in the development of the load distribution factors for box girders, which have been used in the AASHTO *LRFD Bridge Design Specifications,* 4th ed., since.

This ideal is not achieved by the standard load fraction S/5.5 for which most multiple girder bridges are designed. This results in the bridge generally being overdesigned by a significant margin when compared with the actual load that can be physically placed on the bridge.

Advantages

- Reduced number of structural elements to be fabricated, shipped, and erected.
- Improved fracture toughness and superior resistance to crack propagation to meet the needs of fracture-critical design.
- Lighter steel structures and smaller volume of welding.
- Reduced paint and maintenance costs, which extends life of the bridge.
- Fast assembly; fewer field splices or bracing systems.
- Time-tested solution: most bridges in Europe (up to 80 ft wide) and many fatigue-prone railroad bridges are twin I-girder systems.
- Smaller pier caps.
- Redundancy can be enhanced with stiffer bracing, solid floorbeams, and shell action in the deck slab, for alternate load paths.
- Spaced girders enhance the advantages of full-depth precast UHPC slab panels and minimize web-slab connections.
- Fewer field operations would involve fresh concrete.
- Simple achievement of longitudinal continuity, if necessary.
- Steel tubs may replace I-girders in curved ramps.

Disadvantage

- Perceived notion of non-redundancy.

This notion is gradually being challenged by recent studies and research. Starting with the design of the Marquette Interchange in Milwaukee in 2003, the owner challenged designers with demonstrating analytically that two girder trapezoidal tub systems are not fracture critical in order to avoid including them in the Wisconsin fracture critical inventory. This has benefits in eliminating the cost of fracture critical inspections, and ultimately proved to be a political boon after the tragedy in Minneapolis.

Currently, the University of Texas at Austin is completing research that demonstrates that even a simple span two tub curved girder bridge can carry up to 135% of the standard live load with one box completely fractured.

The designers at Marquette did further parametric studies to demonstrate that while the reserve capacity of twin-plate girder bridges is not as high as tubs, there is still sufficient capacity in the superstructure, even with one girder completely fractured, to redistribute load through the deck and framing systems and to carry the design live load without collapsing. This is true even within functional limits that enable the bridge to be safely decommissioned.

Twin-box girder ramps are becoming more and more acceptable and notable for their efficiency, ease of erection, and aesthetic characteristics on highly visible, complex urban interchanges.

The next generation of box girder bridges will be single tubs, which are demonstrably more structurally efficient than two girder systems.

Field welding is used a lot in France, but is not being promoted in the United States.

Having developed a rationale based on current research in the United States, it has become apparent that European bridge engineers are implementing two girder systems more and more due to their efficiency and accelerated construction characteristics.

In most cases, the transverse members are fabricated as full moment connections, and the complete subassembly of the bridge is done in the adjacent right-of-way prior to launching into final position. Field welded girder splices and field welded transverse members fabricated as full moment connections are typically not done in the United States.

Concept ND-1b: Compliant Composite Web Systems

Concept Description

Traditionally composite bridges have been designed using steel girder systems with a composite acting precast or cast-in-place deck.

External posttensioning is a low-cost method for ensuring durability and serviceability for precast deck systems. However there continues to be a challenge associated with the time-dependent behavior of concrete under load, which creates strain–compatibility issues and potential loss of prestressing.

This concept, as shown in Figures 2.28 and 2.29, is described and includes several innovations:

- Plano-tubular webs, which are compliant with external posttensioning.

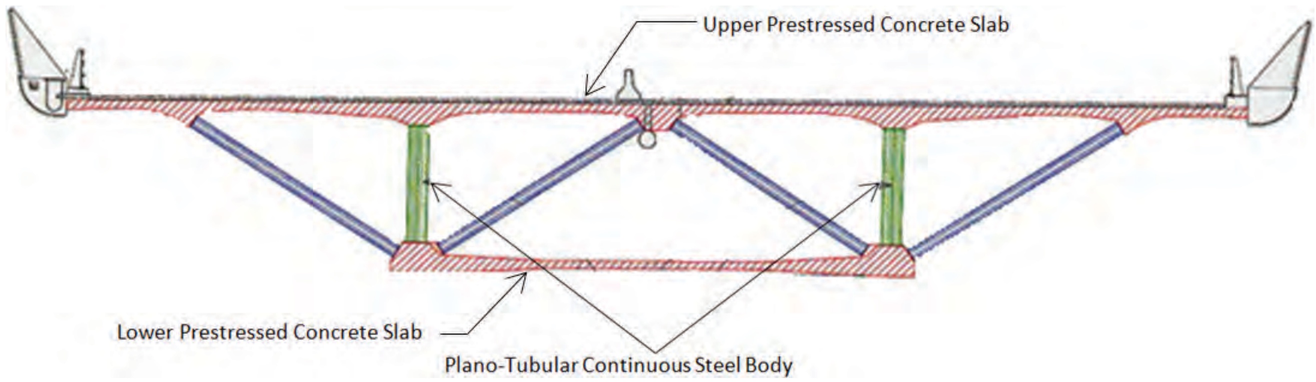


Figure 2.28. Cross section of the superstructure.

- Composite concrete top and bottom flanges.
- Construction in situ under controlled conditions.
- Incremental launching.
- A series of flat plate metal panels.
- Intermittent vertical metal tubes.
- Connection to top and bottom concrete slabs.
- The radial deformability of vertical tubes, by ovalizing, absorbs the longitudinal deformations imposed by concrete (prestress, shrinkage, creep).

Construction Sequence

The construction sequence is expedited in four separate staging areas:

- Stage 1: Assembly of the webs (Figure 2.30).
- Stage 2: Reinforcement and placement of composite slabs (Figure 2.31).
- Stage 3: Stressing and painting (Figure 2.32).
- Stage 4: Incremental launching (Figure 2.33).

Figure 2.30. Web assembly.

Figure 2.29. Plano-tubular web concept.

Figure 2.31. Placement of composite slab.

Figure 2.32. Stressing and painting.

Concept ND-1c: Segmental Composite Bridges

Concept Description

The Boulonnais bridge was constructed with conventional segmental technology, except that it incorporated a tubular steel space frame, as illustrated in Figure 2.34, in lieu of conventional precast webs. This space frame reduced weight and facilitated delivery and erection. It features

- Three viaducts, with a total length of 2 km.
- Composite truss.
- Triangulated steel tubular webs.
- High-strength concrete (50 MPa) used in the top and bottom slabs.
- Maximum span length of 110 m (variable depth).

Figure 2.33. Incremental launching.

Figure 2.34. Boulonnais bridge, France (top), and close-up of tubular steel space frame (bottom).

- Maximum pier height of 70 m.
- Erection in balanced cantilever with overhead truss.

Concept ND-2: Next-Generation Design Material Concepts

Concept Description

For more than 25 years, the FHWA, AASHTO, and NCHRP have researched and demonstrated the use of fiber reinforced polymer (FRP) composites for bridge construction. FRP composite technology can be used in new bridge construction as well as in the rehabilitation and maintenance of existing bridge inventory. This type of construction is particularly advantageous for accelerated bridge construction.

Figure 2.35. Typical FRP bridge and cross section.

FRP composites offer many advantage for building bridges, such as the following:

- Reduced weight: The reduced dead weight of the deck allows the bridge to carry an increased traffic load.
- Decreased effects from environment: FRPs do not rust and are not affected by salts and other contaminants.
- Speed in installation: Since FRP bridges can be built in a factory, they can trucked to a site and installed in considerably less time than it would take to build a bridge on site. A bridge can be installed in hours or days instead of weeks or months.

FRP compositely acting decks, as shown in Figure 2.35, represent a viable alternative to traditional systems. The initial cost is higher but the potential exists for lower life-cycle costs. FRP is rapidly deployable, causing fewer traffic and business effects.

Bridge in a Backpack

The Bridge in a Backpack is a lightweight, corrosion-resistant system for short-to-medium span bridge construction using FRP composite arch tubes, which act as reinforcement, and formwork for cast-in-place concrete. The arches are easily transportable, rapidly deployable, and do not require the heavy equipment or large crews needed to handle the weight of traditional construction materials. Researchers with the University of Maine have developed a bridge kit that could be delivered to a job site in the bed of a pickup truck and installed in a matter of days using only light-duty equipment. The kit consists of three main components: carbon- and glass-FRP composite tube arches, a self-consolidating concrete mix design, and corrugated fiberglass panels. Once on site, workers inflate the 12- to 15-in.-diameter tubes and bend them around arch forms. The crew then uses a vacuum-assisted transfer molding process to infuse the tubes with resin. The tubes, which cure in a matter of hours, function as stay-in-place forms for the self-consolidating concrete, eliminating the need for temporary formwork. They provide structural reinforcement for the concrete in the longitudinal direction, in shear, and as confinement, eliminating the need to install rebar. Over the longer term, the tubes will help protect the enclosed concrete from deterioration. To date, six Maine bridges have been built using the Bridge in a Backpack technology. Several bridge projects are planned throughout New England for 2011 and beyond.

Construction Concept C-1: Above-Deck Driven Carrier

Concept Description

Above-deck driven carriers (ADDCs) are new, modularized, lightweight equipment that can be used for rapid construction with minimal disruption to activities and environment below the structure.

The intent is to develop standard concepts for erecting highway structures using adaptations of proven technology that serve multiple functions during a bridge construction process and that can be easily adapted from project to project, are easily transportable, and are cost-effective. Lightweight steel trusses support an overhead gantry system to remove the existing structure and to transport new girders and slab panels over spans.

It is advantageous to use the system where an existing bridge deck is to be removed, where minimal disruption to traffic and the environment is desired, where traditional crane access and picks are limited, and where temporary access over waterways is restricted.

Design Considerations for Standardized ADDCs

- Use to remove existing structure from spans.
- Use to transport new girders and slab panels across spans.
- Must be easily adaptable from project to project.
- Span lengths must be adjustable.
- Must be easily transportable on both urban and rural roadways.
- Must minimize permit requirements by keeping shipped pieces lightweight, by keeping maximum widths to 8 ft, and by keeping heights (while on axles) to less than 12 ft.
- Must be mobilized with minimal erection and de-erection times.
- Using crane boom technology, assemble pieces with pin-type connections.
- Using heavy-haul applications, design to include removable (or permanently mounted) axles.
- Must not require significantly greater investment than for other standard equipment (cranes).
- Must be designed efficiently using truss concepts.
- Must be fabricated in standard lengths and cross-sectional dimensions.

Application for ABC Construction

The ADDCs can be delivered to the site in various configurations (shipped on flatbed trucks or towed using mountable axles), with delivery options weighed by contractors on the basis of project criteria. Once at the site, the ADDCs will be erected with multiple-axle configurations to allow transport over the existing structure. After reaching the destination pier, the ADDCs are raised to unload the axles, secured and supported at the pier, loaded with gantries, and are ready for demolition of existing structure or delivery of girders and slab panels.

The ADDCs can remove and replace each exterior portion of the existing structure simultaneously. Once the exterior portion of the structure is complete, the ADDCs are repositioned over the new exterior portions to allow removal and replacement of the center portion of the existing structure.

On narrow bridge structures, the ADDCs can remove and replace one-half of the existing structure through the use of counterweights on the gantries. Once half of the structure is complete, the ADDCs can then be repositioned over the newly erected half to allow removal and replacement of the remaining half of the existing structure.

Rapid Demolition of Existing Bridge Spans

Bridge decks can be saw cut for removal with minimal disruption to activities below and parallel ADDC set-up operations. Decks can be removed in panel sections. Once decks are removed, girder removal can begin.

Rapid Construction of New Bridge Spans

After demolition is complete, piers and abutments are prepared for new structures with minimal disruption to traffic below. New girders can be installed with minimal disruption to traffic or the environment below. Depending on the type of structure, new slab panels or precast barrier sections can be installed.

Repeatable Process

For shorter total bridge lengths, the ADDCs can be used to provide access from abutment to abutment. The ADDCs can also allow for complete removal and replacement of the exterior portions of an existing structure before being repositioned to remove and replace the center portion of the existing structure. On narrow bridge structures, the ADDCs can be used to remove one-half of the structure and can then be repositioned to remove the second half.

For longer total bridge lengths, the ADDCs can be used to provide access over a number of spans concurrently, to allow for complete removal and replacement of exterior portions of multiple spans of an existing structure. The ADDCs can then be repositioned forward on the next spans to remove and replace the next exterior portions of the existing structure. Once the ADDCs have replaced the exterior portions of the entire existing structure, they can be repositioned to work backward to remove and replace the center portions of the existing structure.

Erecting Longer Spans Without Significantly Increasing Cost

For multiple short-span bridge structures, the ADDCs can be used over multiple spans to remove two spans while replacing only one new span. Bridge girders can be delivered over longer spans with minimal increase in design requirements for temporary erection stresses.

Where roadways are difficult to traverse, shorter girder segments can be delivered to the site and then assembled behind the abutment and delivered over the span without increasing size or weight. Where access is possible, longer girders can be delivered to the site and delivered over the span without increasing size or weight.

Limits and Special Considerations

The ADDCs can provide rapid removal and replacement of existing structures. Due to the configuration of the gantries, special considerations and limits need to be investigated on a project-by-project basis. The use of ADDCs would be limited on highly curved bridge structures.

The weights of the existing slab panels and girders, as well as the weights of the new girders and slab panels, must be studied to verify stability of the gantry system. Longer gantry arms and moving counterweights could be used to accommodate varying loads and pick lengths.

Design Concept Sketches

The following design concept sketches are shown:

- ADDC used for different span configurations (Figure 2.36).
- ADDC used to remove and replace exterior portions of a structure (Figure 2.37).
- ADDC used to remove and replace center portion of a structure (Figure 2.38).
- ADDC used to remove first half of a narrow structure (Figure 2.39).
- ADDC used to remove second half of a narrow structure (Figure 2.40).
- ADDC rigged with multiple axles to reduce loads on structure (Figure 2.41).
- ADDC demonstrated as transportable on urban and rural highway (Figure 2.42).
- ADDC demonstrated as adjustable for multiple span lengths (Figure 2.43).

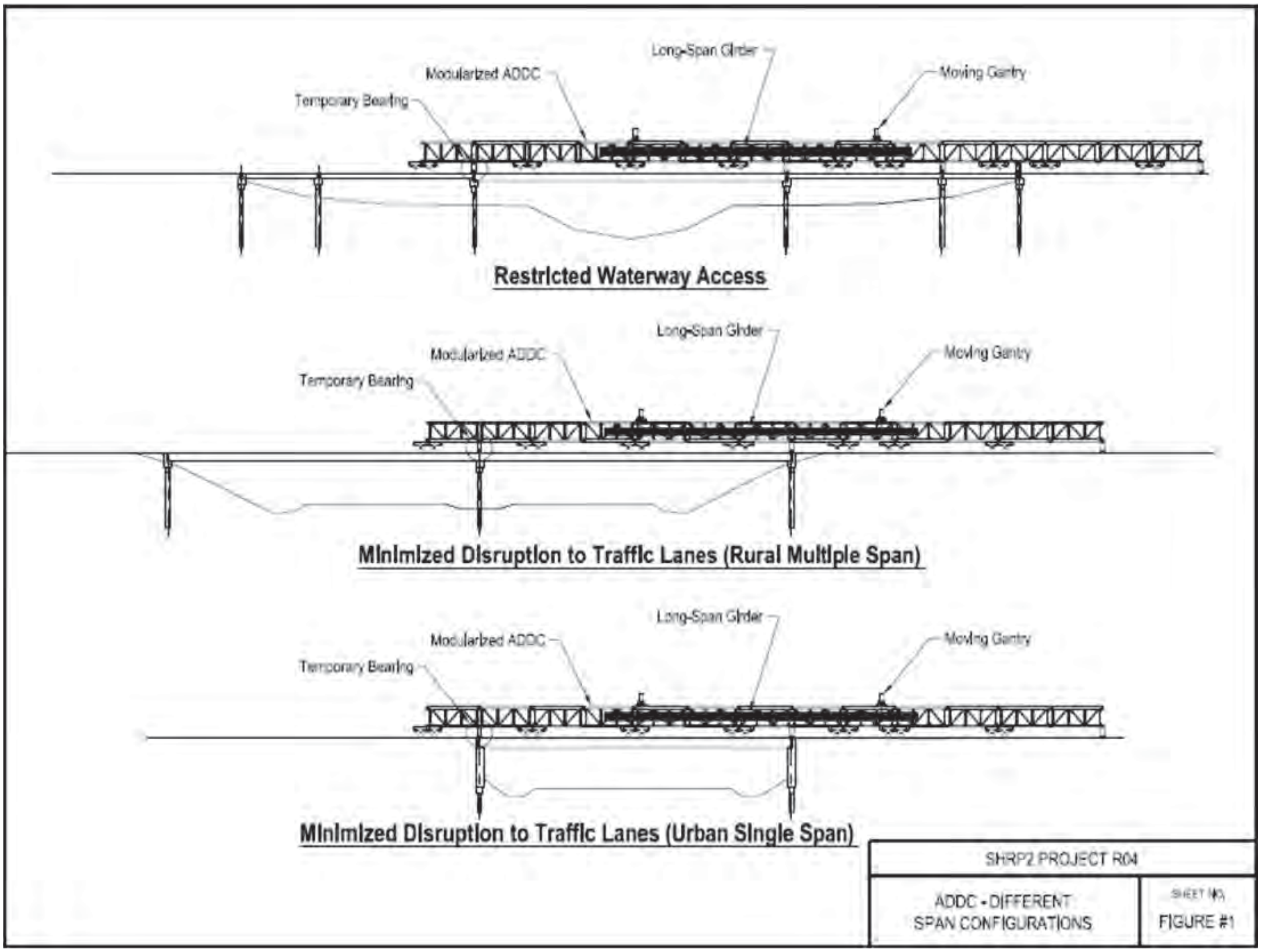


Figure 2.36. ADDC used for different span configurations.

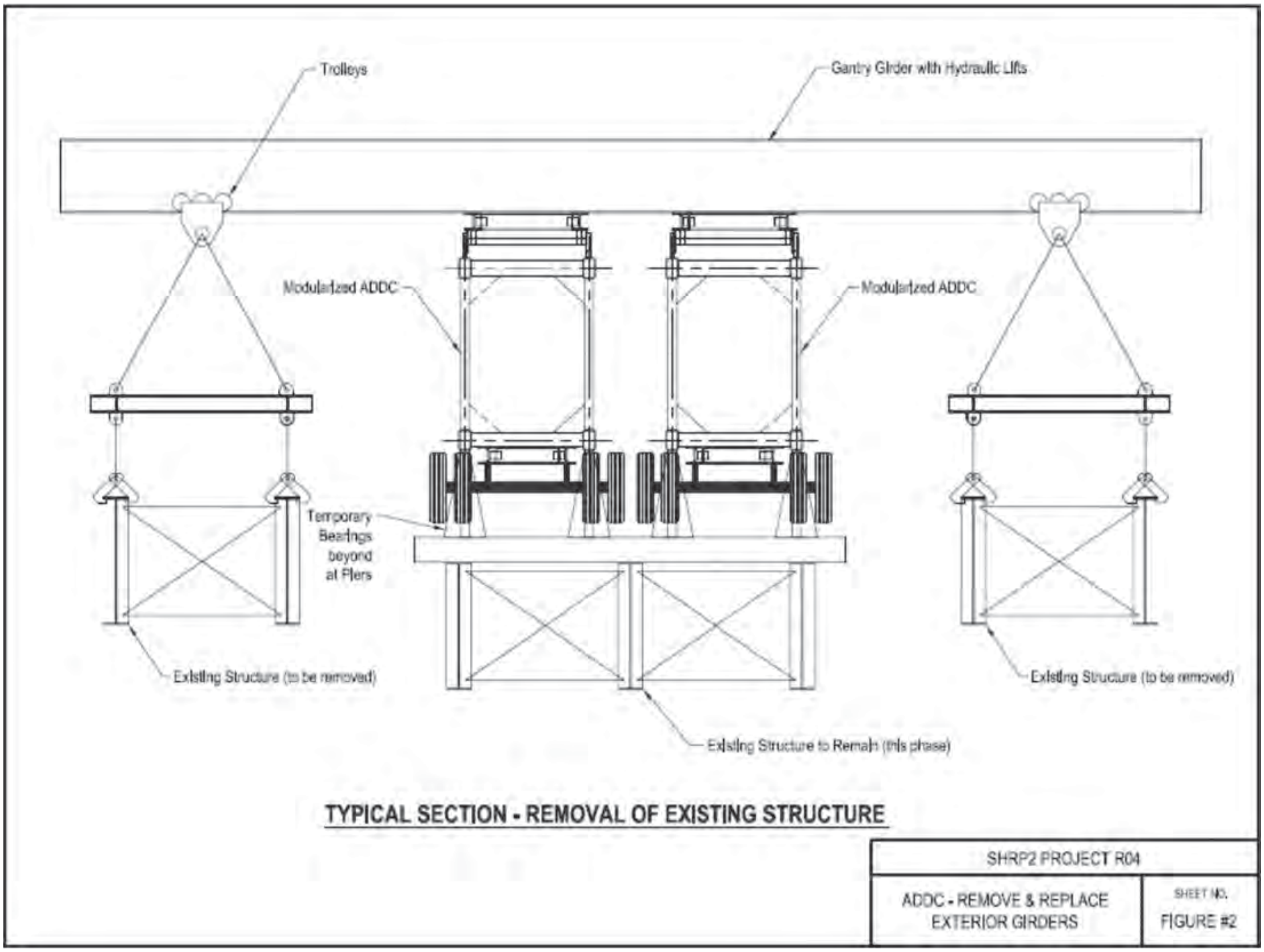


Figure 2.37. ADDC used to remove and replace exterior portions of a structure.

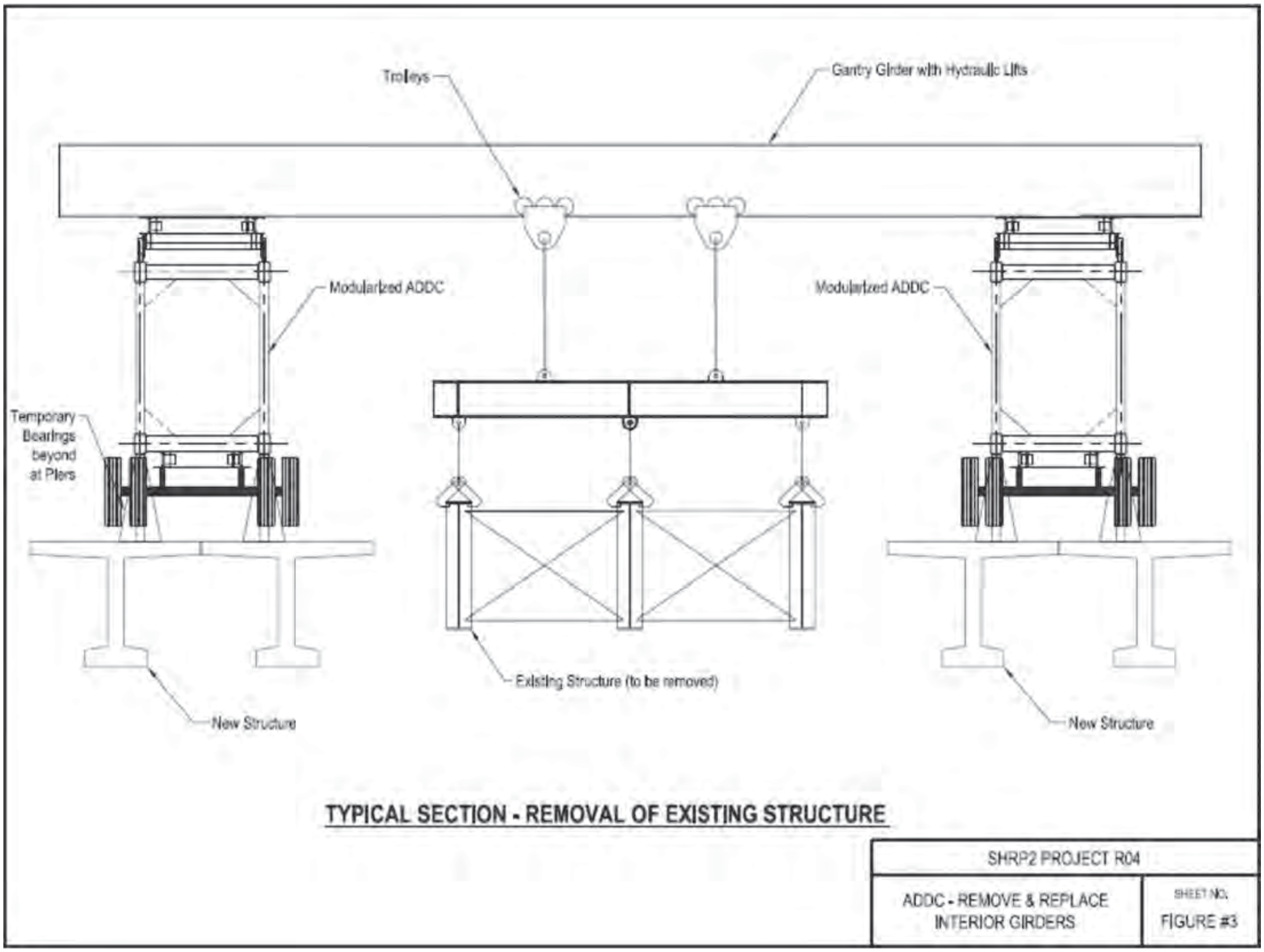


Figure 2.38. ADDC used to remove and replace center portion of a structure.

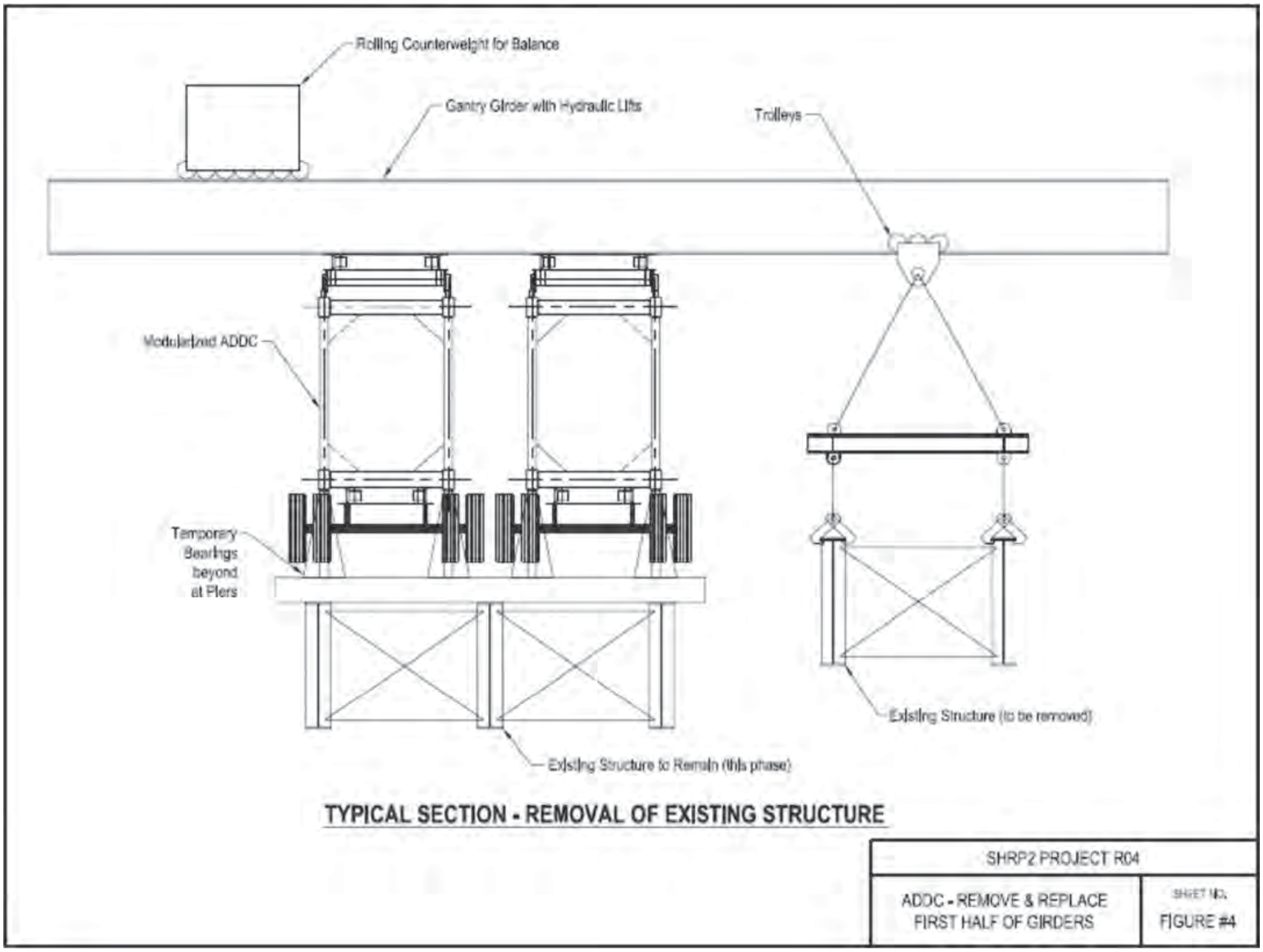


Figure 2.39. ADDC used to remove first half of a narrow structure.

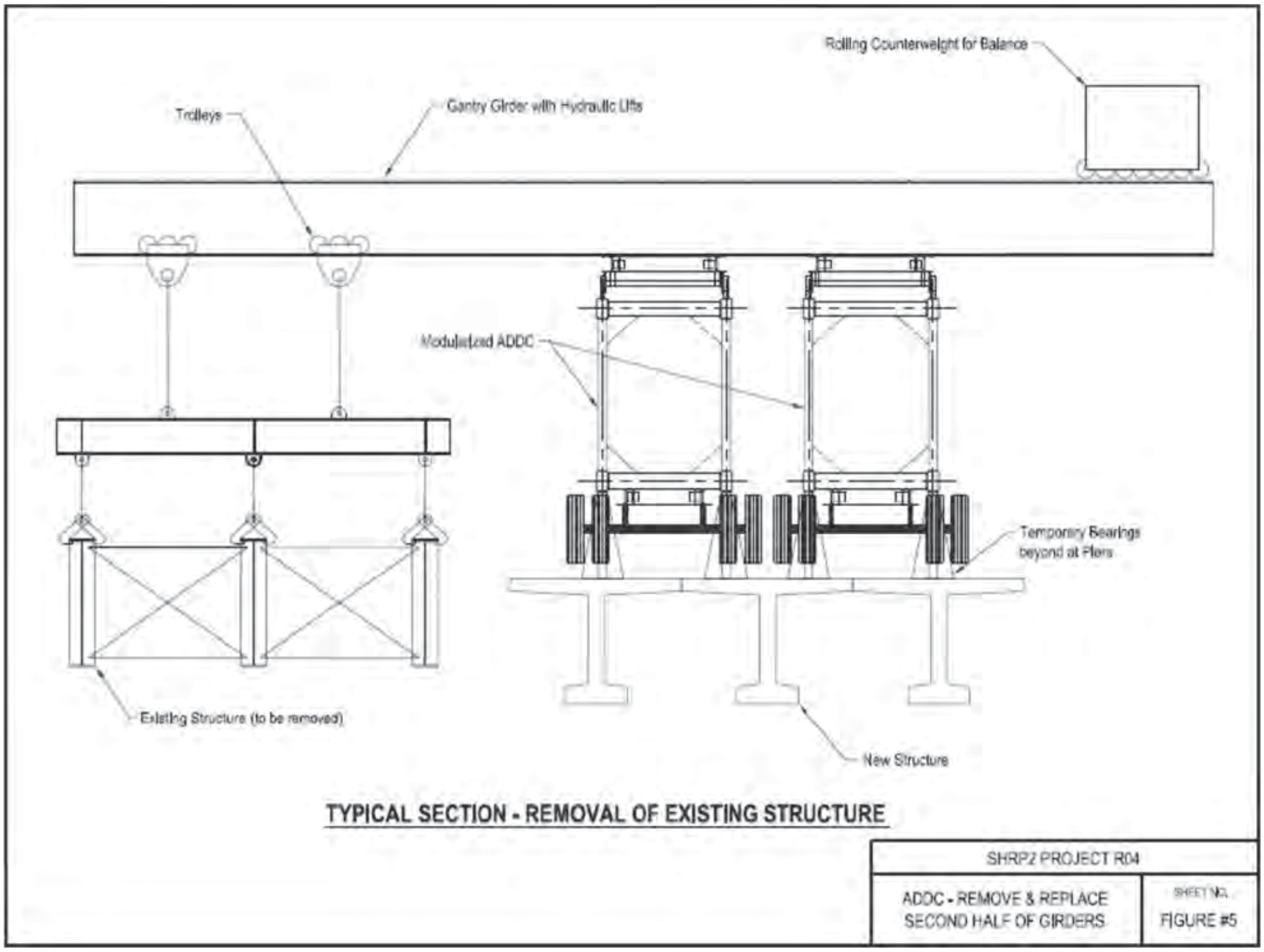


Figure 2.40. ADDC used to remove second half of a narrow structure.

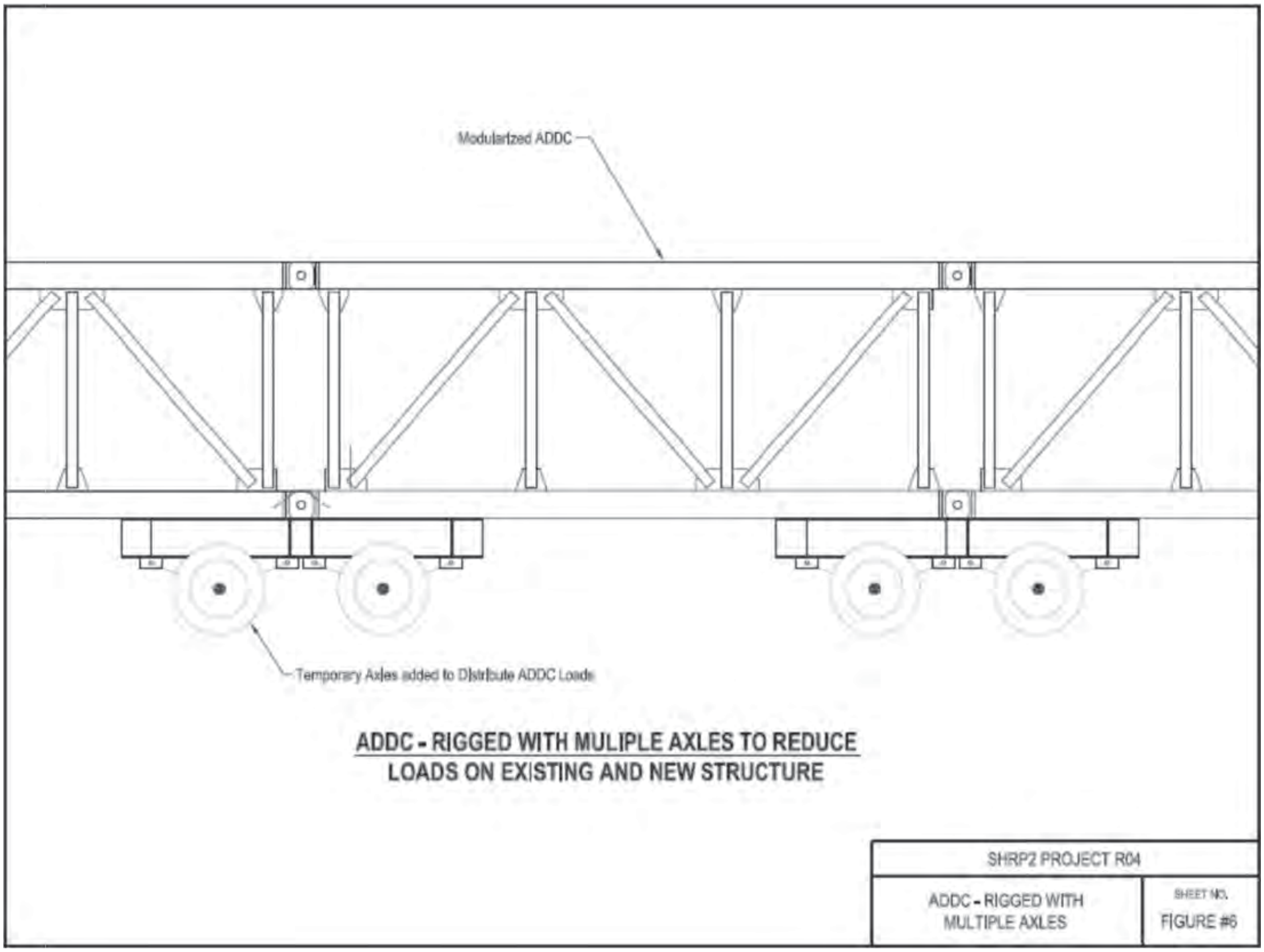


Figure 2.41. ADDC rigged with multiple axles to reduce loads on structure.

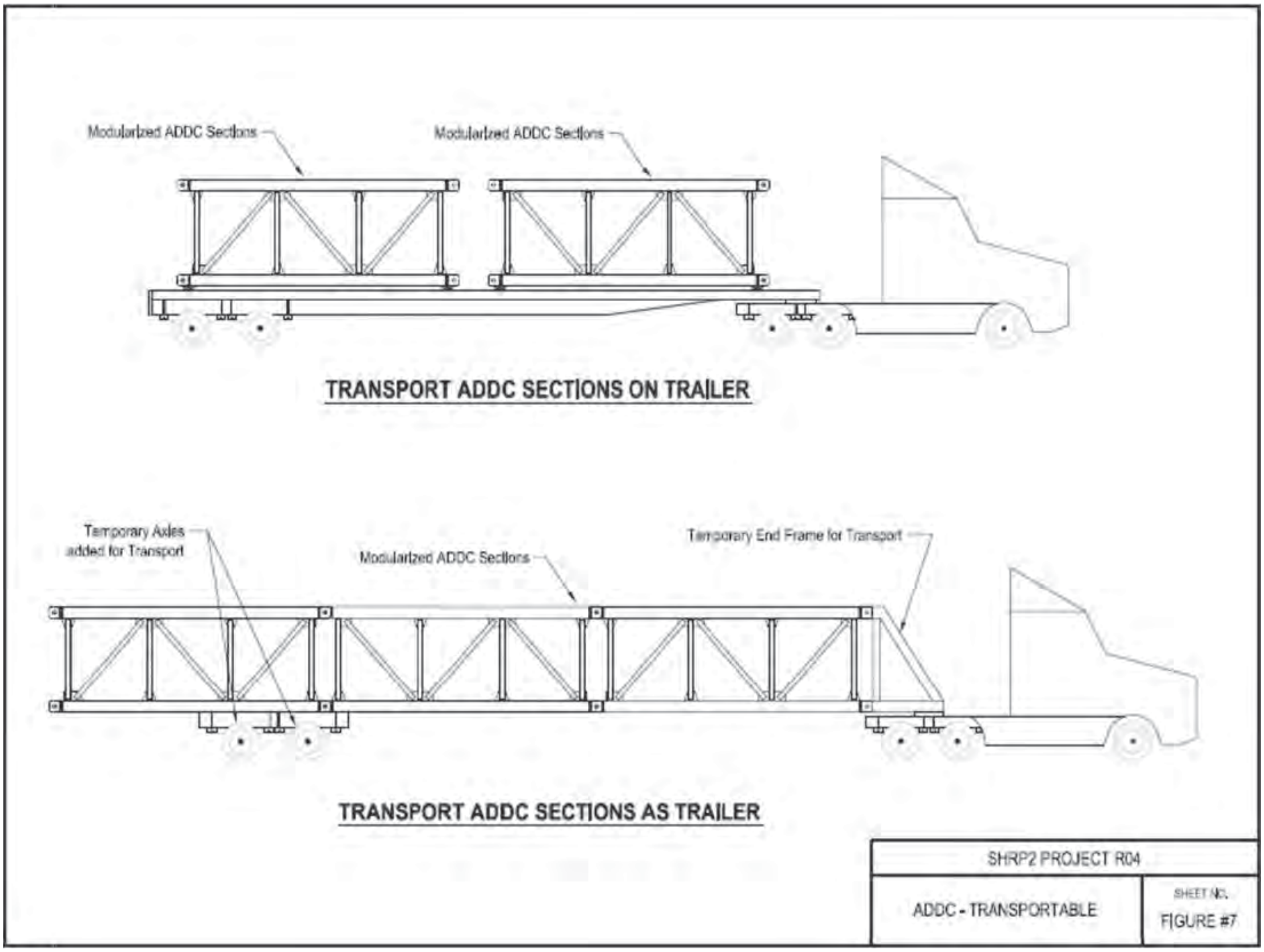


Figure 2.42. ADDC is transportable on urban and rural highways.

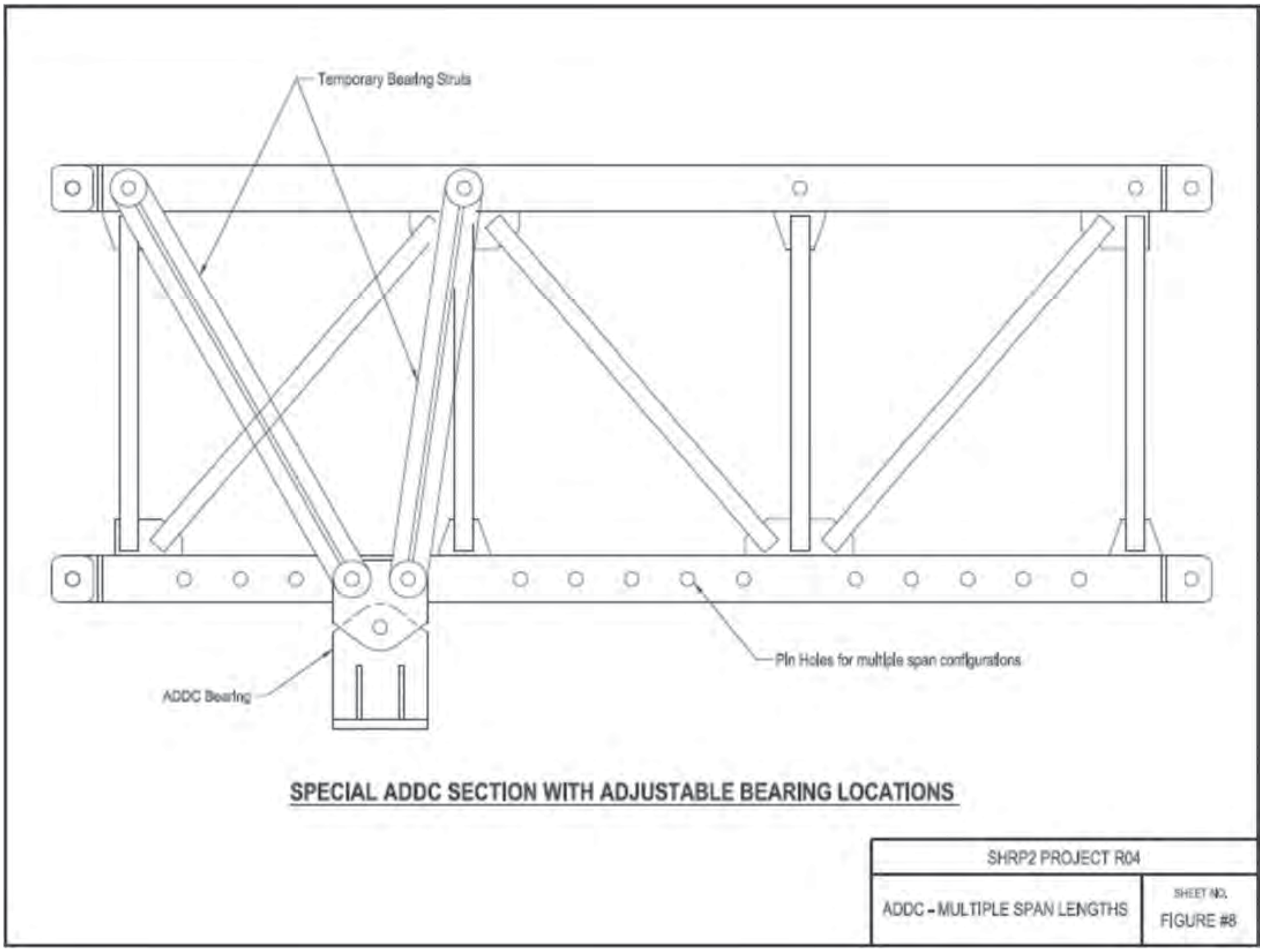


Figure 2.43. ADDC is adjustable for multiple span lengths.

Construction Concept C-2: Launched Temporary Truss Bridge

Concept Description

Launched temporary truss bridges (LTTBs) offer new, modularized, lightweight equipment that can be used for rapid construction with minimal disruption to activities and environment below the structure. The intent is to develop standard concepts for erecting highway structures using adaptations of proven long-span technology that serve multiple functions during a bridge construction process, that can be easily adapted from project to project, that are easily transportable, and that can be mobilized with minimal erection and de-erection times in a cost-effective manner. Lightweight steel trusses are used to transport girders or equipment over spans, as shown in Figure 2.44.

It is advantageous to use these systems where launching demands (the cost of extra steel, concrete, or posttensioning in each girder) outweigh the economic savings. Situations could include a launched bridge, minimal disruption to traffic or the environment is desired, traditional crane access and picks are limited, or temporary access over waterways is restricted.

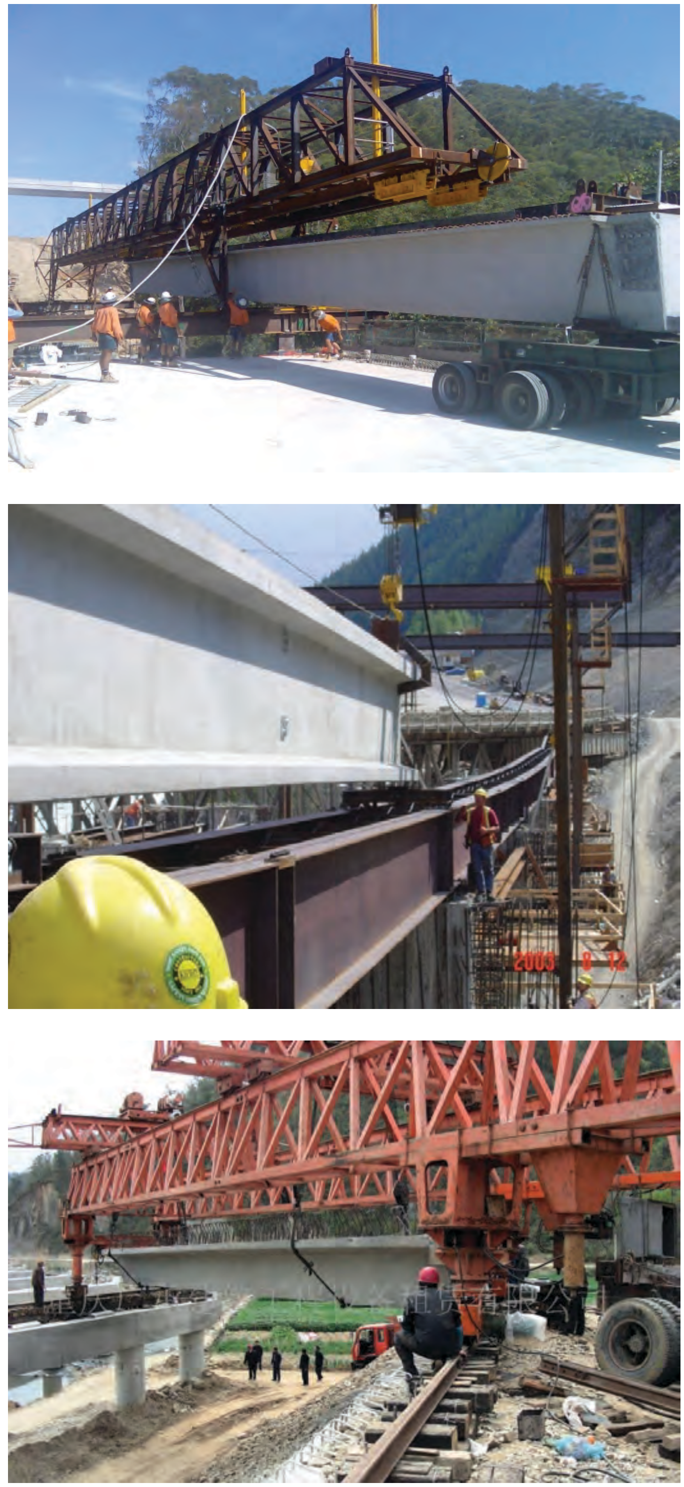


Figure 2.44. Examples of LTTB technologies that have been launched and set in place across a span.

Design Considerations for Standardized LTTBs

- Must be multifunctional.
- Can be used to transport new girders across spans.
- Can be used to transport materials and equipment across spans.
- Can be used to transport material and equipment across waterways or other inaccessible areas.
- Must be easily adaptable from project to project.
- Span lengths must be adjustable.
- Must be easily transportable on both urban and rural roadways.
- Must minimize permit requirements by keeping shipped pieces lightweight, by keeping maximum widths to 8 ft, and by keeping heights (while on axles) less than 12 ft.
- Must be mobilized with minimal erection and de-erection times.
- Using crane boom technology, assemble pieces by using pin-type connections.
- Using heavy-haul applications, design to include removable (or permanently mounted) axles.
- Must not require significantly greater investment than for other standard equipment (cranes).
- Must be designed efficiently using truss concepts.
- Must be fabricated in standard lengths and cross-sectional dimensions.

Application for ABC Construction

LTTBs can be delivered to sites in various configurations (shipped on flatbed trucks or towed with mountable axles) with delivery options weighed by contractors on a project-by-project basis. Once at the site, LTTBs will be erected and launched. After reaching the destination pier or temporary bent, LTTBs are secured and ready for delivery of girders and equipment.

Rapid Construction of New Bridge Spans

Bridge girders can be delivered over longer spans with minimal disruption by using parallel construction operations. Girders can be rolled out over spans. While cranes erect one girder, the next can be rolled into position to be ready for erection.

Minimize Traffic Disruptions Below and Crane Access Requirements from Below

Bridge girders and equipment can be delivered over longer spans, with minimal disruption to traffic below and minimal crane access required. Girders and equipment can be secured on rollers and delivered over traffic or waterways, with little or no disruption.

Larger girders or pieces of equipment can be delivered to areas to reduce the necessary crane reach, possibly reducing the crane size required for the project.

Erecting Longer Spans Without Significantly Increasing Cost

Bridge girders can be delivered over longer spans with minimal increase in design requirements for temporary erection stresses. Where roadways are difficult to traverse, shorter girder segments can be delivered to the site and then assembled behind the abutment and delivered over the span without increasing size or weight.

Where access is possible, longer girders can be delivered to the site and delivered over the span without increasing size or weight. By delivering longer girders across the span, the potential for smaller cranes on each end increases.

Design Concept Sketches

Design concept sketches are shown in Figures 2.45 through 2.48.

Construction Concept C-3: Wheeled Carriers or Self-Propelled Modular Transporters

Concept Description

The intent of this concept is to develop a standard for erecting prefabricated spans or full-length span strips from above, using adaptations of proven technology.

A wheeled carrier is used to remove entire spans or full-length span strips of existing bridges, which are then replaced with new units. The wheeled carrier is easily adapted from project to project, is easily transported, can be mobilized with minimal erection and de-erection times, and is cost-effective.

ABC Construction Considerations

- Extremely rapid removal of existing short-span bridges and replacement of new spans.
- Context-sensitive, sustainable solution. Site disruption is limited to pier retrofitting or erection of lateral piers within tight work windows.
- Compatible with irregular or inaccessible sites, steep slopes, tall piers, rivers, levees, and extreme nature of the topography.
- No interference with the area under the bridge.
- Compatible with the crossing of highways and railroads.
- Compatible with multi-span bridges.
- Complicated in continuous superstructures.
- Specialty equipment to be studied on modular bases.
- Casting facilities and storage areas required on multi-span bridges.
- Compatible with complex bridge geometry.

The movement and steering of the trolleys were governed by hydraulic motors, and the hydraulic plants were powered by diesel engines. The distance between the centerlines of the rear trolley and the front trolley was 147 ft. Longitudinal pistons shifted the rear lifting winch to the suspension points of the different types of precast spans while the front winch was fixed.

The wheeled carriers in these photographs are heavy units for railroad spans, as shown in Figures 2.49 and 2.50. For highway bridges, the spans would be lighter if made with prestressed concrete, and much lighter if made of steel girders and concrete deck slab. As a result, the wheeled carriers themselves would be lighter. The spans can also be divided into longitudinal strips to diminish the weight to be lifted and the cost of the wheeled carriers.

Construction and placement of spans can be prequalified QA/QC processes. Means-and-methods analyses and risk assessments can be performed for every major activity, and contingency plans can be identified and also prequalified.

Specific performance requirements should be identified for major construction equipment.

Work to Be Done to Refine Concept

- Statistical analysis of existing bridges (span length, span weight, etc.).
- Conceptual design of modular equipment.

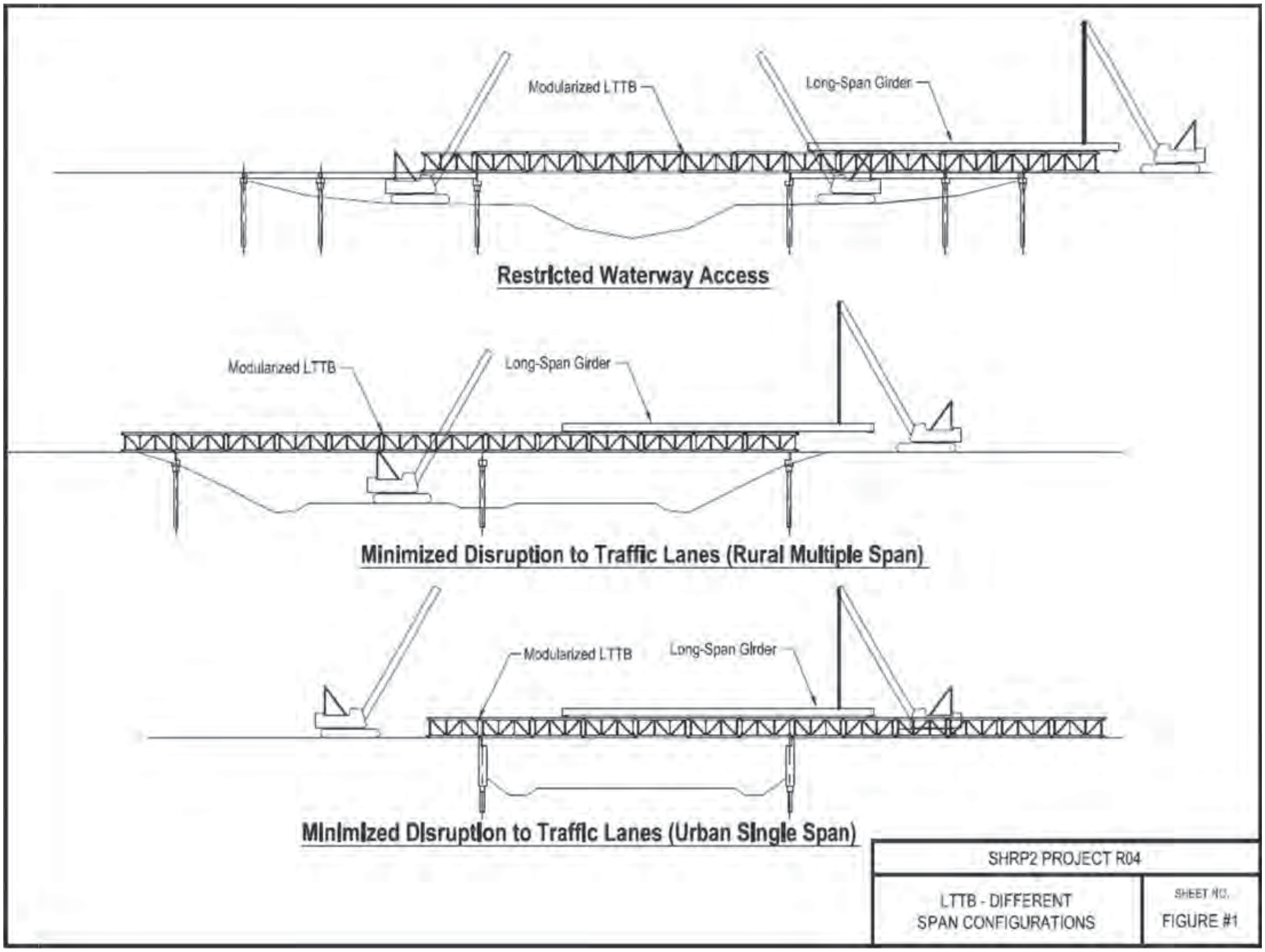


Figure 2.45. LTTB used for different span configurations.

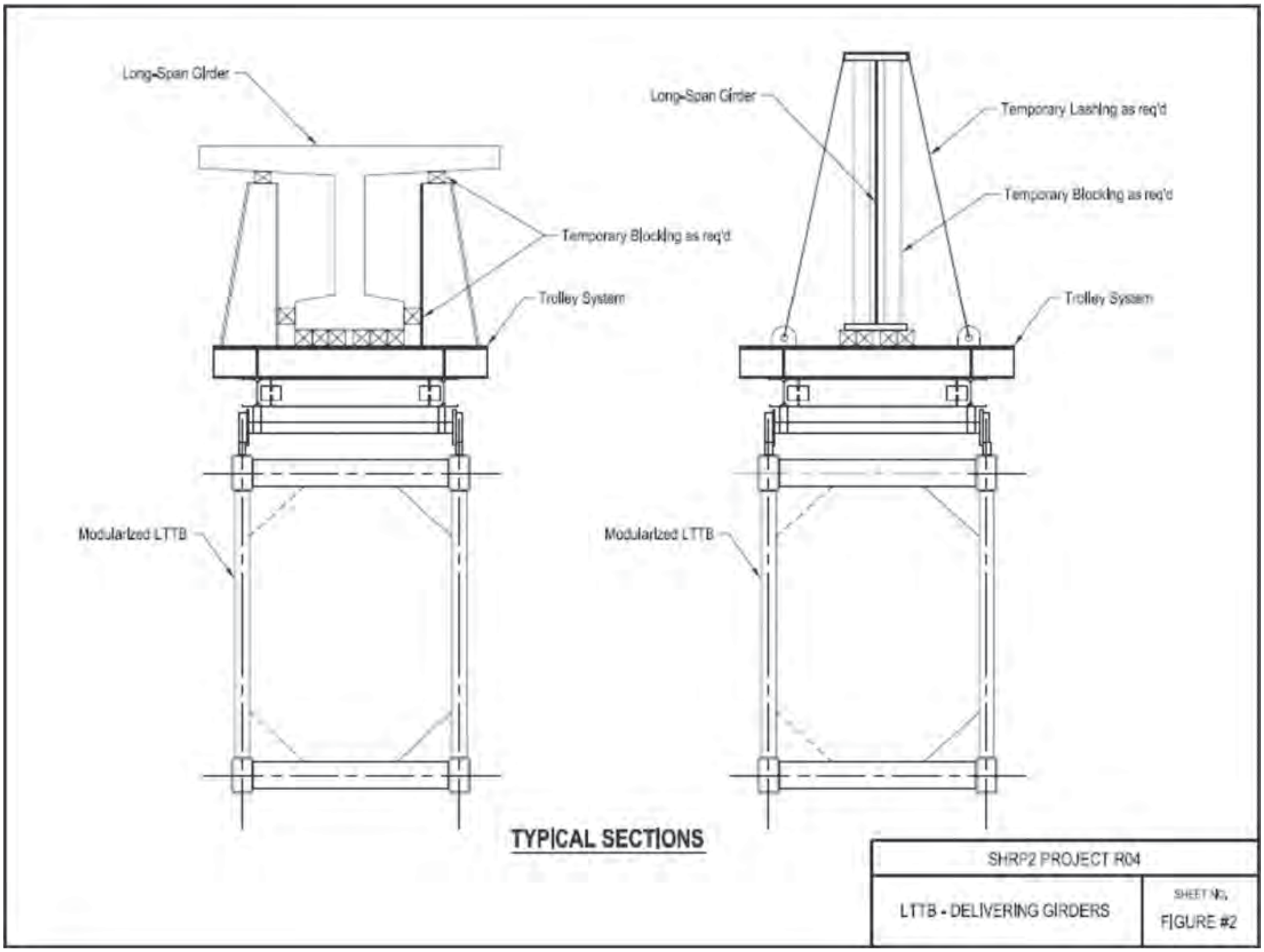


Figure 2.46. LTTB used to deliver girders across spans.

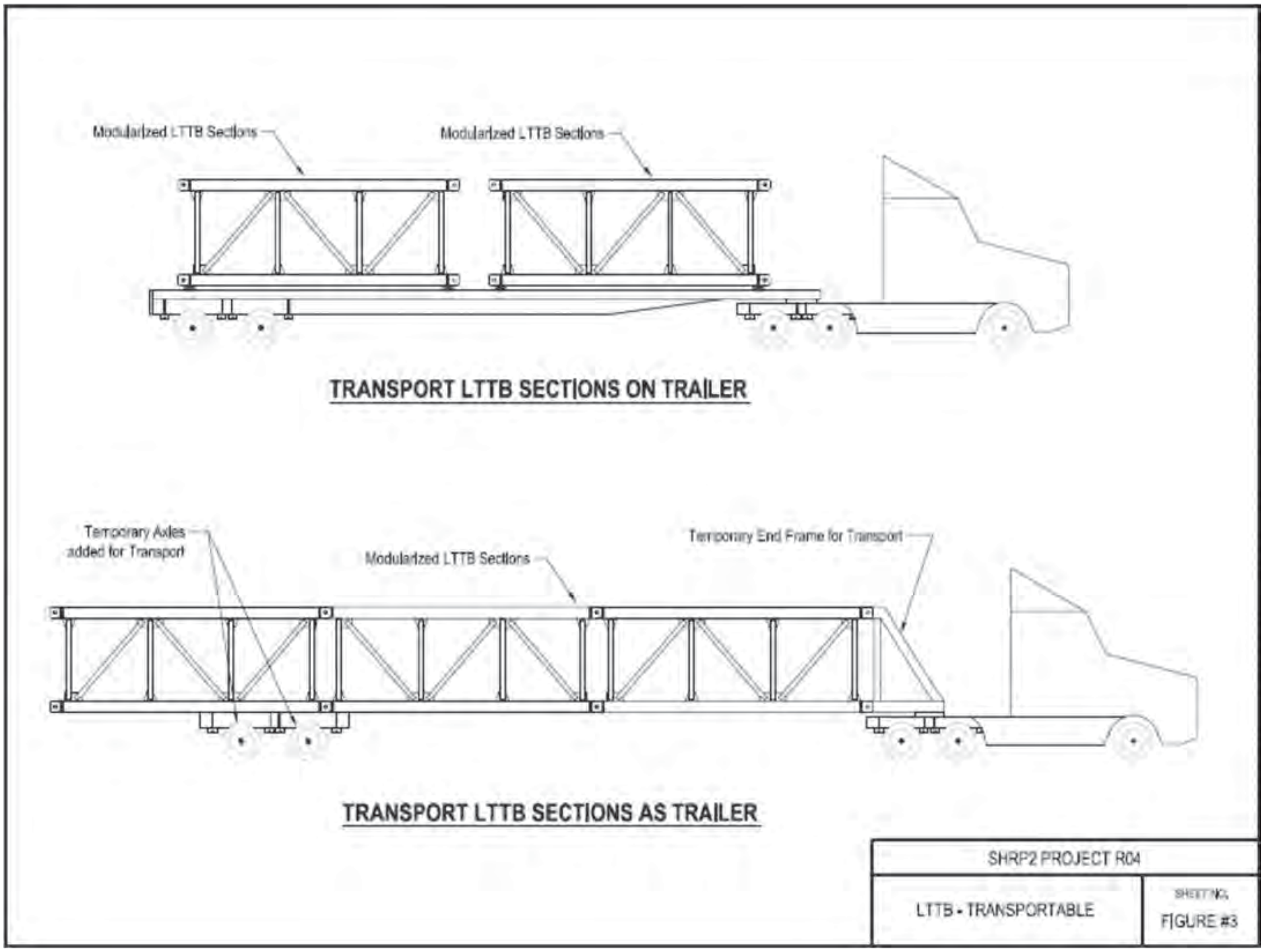


Figure 2.47. LTTB is transportable on urban and rural highways.

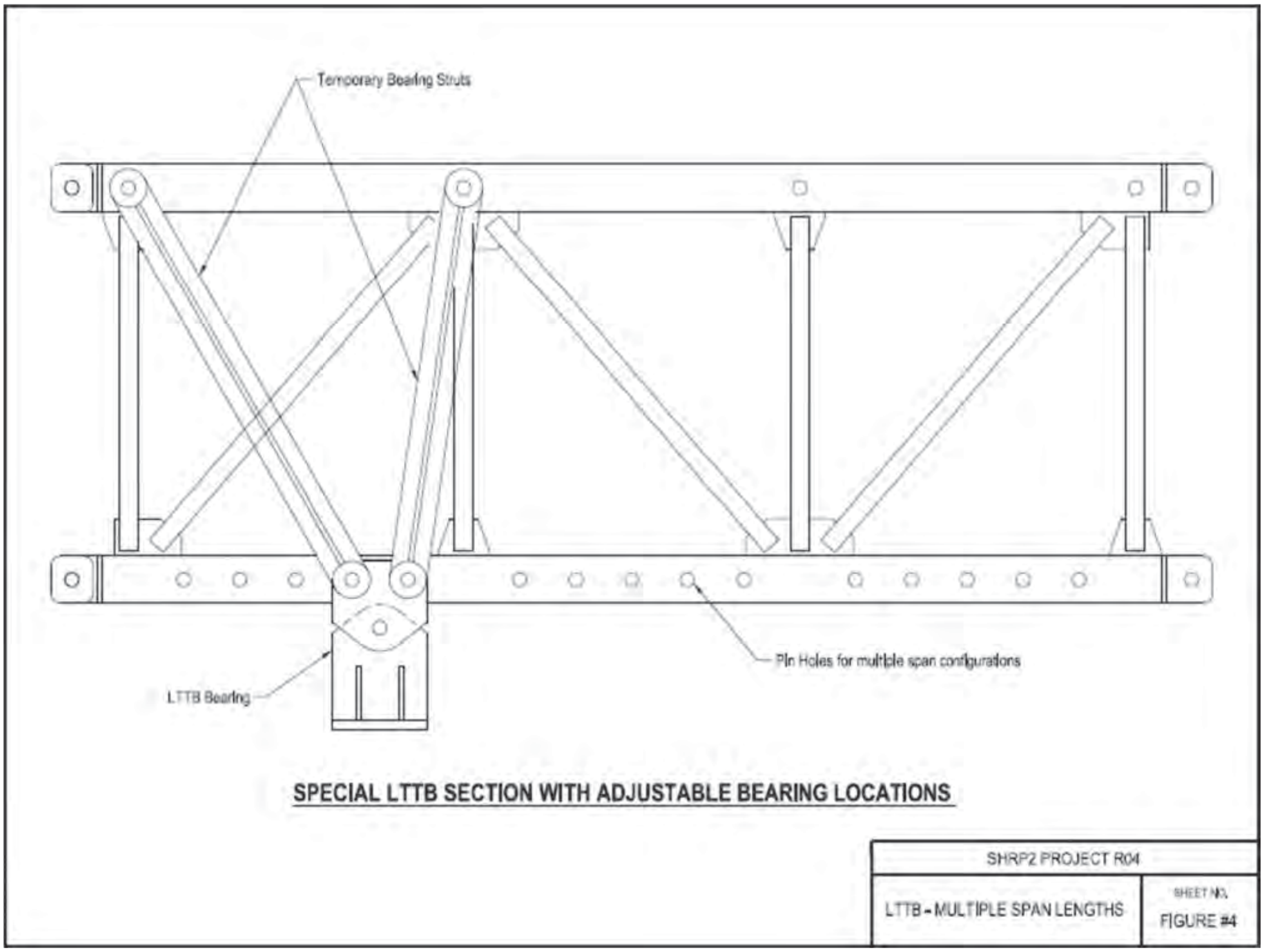


Figure 2.48. LTTB is adjustable for multiple span lengths.

Construction Concept C-4: Launching and Lateral Shifting

Concept Description

The incremental launching construction method was developed in Europe in the 1960s and is now typically used for construction of prestressed concrete and steel and steel/composite bridges, as shown in Figure 2.51. The method involves building a bridge at a single construction location in sections and launching the bridge incrementally as each section is completed.

Figure 2.49. Heavy wheeled carrier used for railroad spans.

Figure 2.50. Wheeled carrier transporting longitudinal strips.

Figure 2.51. Launched prestressed bridge.

Prestressed concrete bridges are constructed in a small casting yard behind an abutment. The first bridge segment is equipped with a light steel extension to control the launch stresses. The segment and the steel extension are launched forward onto the piers until it clears the formwork. A second bridge segment is match cast and prestressed against the first one, and the entire bridge section is launched again. This process (match casting of a new segment and launch of the entire bridge section) is repeated until completion of the bridge.

Incremental launching construction for steel girder bridges involves similar operations. In this case, however, the formwork is replaced with adjustable supports that sustain the girder segments during their assembly. The diaphragms and lateral bracing also are assembled behind the abutment. The deck slab of steel/composite bridges is cast in place on completion of the launch of the steel girders or made of full-depth precast panels.

For prestressed concrete bridges, the typical application for full-span incremental launching is on 100- to 180-ft spans and bridge lengths varying between 300 and 3,000 ft. For steel girders, the optimum span lengths vary from 100 to 300 ft. In both cases, much longer spans can be launched with the use of temporary piers.

Simply supported spans also can be launched. Such versatility is advantageous in ABC applications—from urban bridges to isolated or environmentally sensitive sites—and for widening existing structures.

ABC Construction Considerations

- Context-sensitive, sustainable solution that can cross environmentally sensitive sites with minimum impact.
- Disruption of the area under the bridge limited to pier erection in tight work windows.
- Small casting yard with no additional right-of-way.
- Improved control of noise and dust.
- Easy demolition and replacement: A launched bridge can be moved back to the abutment and demolished on the ground.
- Extreme safety for workers.
- Detours and risks to traffic can be avoided when building over highways or railroads.
- Elimination of construction clearances for the forming systems.
- Low labor demand, repetitive operations, and short learning curve.
- Parallel activities for flexible critical path and enhanced quality of ABC applications.
- Continuous production with inclement weather.
- Possible 24/7 organization for ABC applications.
- Time-tested, high-quality construction method.
- Inexpensive specialized construction equipment.
- Adaptable level of site industrialization.
- No need for heavy cranes.
- No need for heavy-haul loads in urban areas or mountain sites.
- Compatible with irregular or inaccessible sites, tall piers, steep slopes, rivers, levees, and extreme topography.
- No interference with the area under the bridge, and no impact on traveling public.
- Compatible with single-span and multi-span bridges.
- Compatible (best use) with continuous superstructures.
- Hardly compatible with complex bridge geometry.

A prestressed concrete launched bridge is built in a rigid casting cell supported on the ground. The stiffness of the forming system guarantees accurate geometry of the structure and uniform concrete cover for enhanced durability. The casting cell can be reused many times without any adjustment and architectural effects are easily and inexpensively applied to the bridge surface.

The few construction joints (usually two per span, at the span quarters) are match cast with through-reinforcement for conventional connection detailing, uniform moment capacity, and enhanced seismic behavior.

Concrete is easily pumped or fed with conveyor belts in weather- and temperature-controlled conditions. Roll compaction of the deck slab increases the strength of the concrete cover and avoids the labor demand of hand finishing. The deck surface can be easily inspected, and defects can be corrected.

Long-term cambers are minimal because prestressing is applied progressively. When the bridge is long, the reinforcement cage can be entirely prefabricated. Cage prefabrication removes the reinforcement placement from the critical path. Iron workers and carpenters work in parallel rather than in series, which improves quality and geometry control and makes correction of errors less critical.

A gantry crane places the cage into the formwork in one operation. The segment may be cast in a single stage and precast anchor blocks may be used for the prestressing tendons. The internal form of the box girder is launched with the bridge and extracted backwards into the new reinforcement cage.

In medium-length bridges, the cost of a self-extracting form may be avoided by dividing the pour into two stages: the bottom slab and webs first, and then the deck slab. In this case the cage is prefabricated only for the first casting stage. The cage also may be divided into light web segments and bar grids for the slabs to be handled with the tower crane.

In short bridges, several deck segments may be cast between two launches to further diminish the cost of forms. The cage may be assembled for the entire section to be launched to avoid interference of workers and to optimize splicing. Simple and repetitive operations result in high quality and a short learning curve for ABC applications.

A launched bridge is built on the ground. In addition to the absence of risks for workers and the environment, the casting yard can be sheltered from inclement weather to permit continuous production. Thermal treatments can be applied to setting concrete, and the bridge can be protected from excessive drying in the first curing stages.

The assembly of steel girders is simpler and more accurate when working on the ground. Adjustable saddles support the segments before bolting or welding and permit accurate cambers in the girders.

The casting yard is located immediately behind an abutment so no additional right-of-way is necessary. The yard is small and compact, containing just formwork and storage areas. The minimal dimensions of the yard allow the bridge to be built entirely under a tower crane.

Labor is concentrated in a small area, which results in easy supervision and minimized internal transportation. All materials are processed within the casting yard, and no specialty construction equipment is necessary, apart from the formwork, the launching nose, and the thrust system. Lighting and control of dust and noise are facilitated.

The length of the bridge defines the number of segments and the optimum level of industrialization of the casting yard. When the bridge is long, a high level of industrialization can save a lot of labor. When the bridge is short, a lower level of industrialization is typically used. Reinforcement is assembled into the formwork, with the bridge launched by pulling strands anchored to the abutment. The only investment is for a steel launching nose.

Labor demands can be minimized with a high level of industrialization of the casting process. Two small crews of iron workers and carpenters perform highly repetitive operations without interference in a protected work environment. When the bridge is short, the level of industrialization is lower and the labor demand therefore increases, but it is still lower than conventional construction. Labor demand is also lower in steel/composite bridges.

Worker safety is excellent for launched bridges for the following reasons:

- No construction activities adjacent to traffic;
- No risks of falling;
- No heavy loads to be handled; and
- Most of the activities carried out on the ground and under a tower crane.

Casting concrete over a sensitive environment is often difficult. Many owners require adequate protection from the construction risks, and this is particularly demanding when traffic, railroads, or environmentally sensitive areas are involved.

A great number of bridges have been launched over highways, railroads, rivers, wetlands, and lakes, in absolute safety. Launching is a preferred method for sustainable construction in sensitive environments because site disruption is limited to pier construction within tight work windows.

Future demolition and replacement also are facilitated, as a launched bridge can be moved back to the abutment and incrementally demolished on the ground.

Incremental bridge launching avoids clearance requirements during construction over highways or railroads. After erecting the piers, the construction activities take place at the deck level and mostly behind an abutment.

Avoiding additional clearance requirements by eliminating falsework may result in shorter approaches or lower grades. This is a big advantage in urban areas where the approaches connect to existing roads.

Lateral Sliding Case Study: Trenton, Ontario, Canada

An existing bridge can remain in service while a replacement bridge is constructed immediately adjacent to it, on temporary foundations, to carry traffic. Then the old bridge is demolished, and new foundations are constructed in the same place as the old bridge to carry the new bridge. Sliding equipment is then installed and ready for the lateral move.

During a short closure (weekend), such as in Trenton, the new bridge is slid into place, joints are completed, utilities are reconnected, and the new bridge put into service, as shown in Figures 2.52 and 2.53.

Parameters and considerations for slide include the following:

- 7,920 tons vertical load.
- Laminated elastomerics.
- Teflon surface.
- Stainless steel.
- 5% static friction.
- 396 tons startup force.

Figure 2.52. Lateral sliding, bearing view.

- <1% dynamic friction.
- 79 tons sliding force.
- 34 ft slide.
- 6 in. stroke.

Work to Be Done to Refine Concept

- Preliminary design of standardized launch systems for prestressed concrete bridges.
- Preliminary design of standardized modular casting cells.
- Study of standardized organization of the casting yard.
- Study of standardized organization of the assembly yard for precast segmental construction.
- Preliminary design of precast foundations for the casting and assembly yard.
- Preliminary design of standardized launch systems for steel girders.
- Study of launching precast UHPC deck slab plates onto the steel girders with shear connection at the end of launching.

Figure 2.53. New bridge put into service.

Construction Concept C-5: Jacking and Mining

Concept Description

Tunnel jacking is a term that refers to the installation of tunnels by pushing them into the ground while excavating from an open face. The tunnels, which are usually of rectangular cross section, are installed beneath a facility either that cannot be removed or that the facility owner does not wish to be removed. This technique can be used for relatively small sections (6 ft by 6 ft) up to large, full-size highway sections (80 ft by 40 ft) in lengths up to several hundred feet.

Advantages

- Builds on conventional proven pipe jacking methods.
- Uses inexpensive equipment and technology.
- Structure type is simple and inexpensive.
- Provides for accelerated construction schedule.
- Causes no interruption to existing traffic.
- Contractors favor because of self-performance.

Disadvantages

- Limited experience to date in the United States.
- Limited number of designers and contractors who are familiar with the methods.

The technique was developed from pipe jacking when the circular sections available were either too small or inefficient for the final use of the tunnel. The technique is most often used in soft ground and at shallow depth. It has been used successfully in a variety of ground conditions, including soft clays, granular material, filled ground, and mixed ground.

Experience

Tunnel jacking has been used in many parts of the world, including Europe (particularly in the United Kingdom and Germany), Australia, India, South Africa, and Canada. It has been used extensively in the Far East, particularly in Japan. It is not a technique that has been used to any significant extent in the United States to date, although a few tunnels have been jacked in California over the past 10 years or so. The I-90/I-93 Interchange (Section 9A4) on the Boston Central Artery/Tunnel Project, which is currently under construction, will be the largest and one of the most complex tunnel jacking projects to date in the world. It will, therefore, bring U.S. engineering to the forefront of this specialized technology. The preferred method of construction for shallow tunnels in soft ground is often the cut-and-cover method. This generally represents the lowest structural cost and shortest construction time solution.

However, the total cost of a tunnel is not measured only in terms of volume of concrete, excavated material, and so forth. When the ground above and adjacent to the tunnels includes rail tracks, roads, services, or other facilities, it is necessary to consider the cost of the disruption to the service provided by these facilities to obtain a true indication of the total cost of the tunnel. This can result in a significantly higher total cost than that for the tunnel construction alone.

In many instances, the owners of the facility to be crossed will not permit disruption to their services. It is in these instances that tunnel jacking should be considered as a possible solution. In many situations, tunnel jacking has resulted in virtually no disruption or effect on the overlying facility. It is a technique which allows tunnel alignments to be considered that would otherwise have been unacceptable if there are facilities, such as rail tracks or major roadways, that do not permit a closure for the period required for more traditional construction methods. This can result in considerable financial and environmental benefits by allowing the conceptual planners more freedom in the early phases of a project.

Basics of Tunnel Jacking

Figures 2.54 through 2.57 show the essential elements of the technique. A jacking pit and thrust slab are constructed at the entrance portal, and a reception pit is constructed at the exit portal. The tunnel or bridge structure is advanced by means of hydraulic jacks pushing from the rear and reacting against the thrust slab, or alternately, high-strength pulling cables are anchored at the exit portal and act in tension to advance the structure beneath the active facility overhead. A cutting shield penetrates the embankment and provides a working platform inside the tunnel for manual or mechanized excavation.

This technique evolved from pipe jacking and retains many of the same features as pipe jacking. Pipe jacking was introduced on the Northern Pacific Railroad in the late 1890s as a

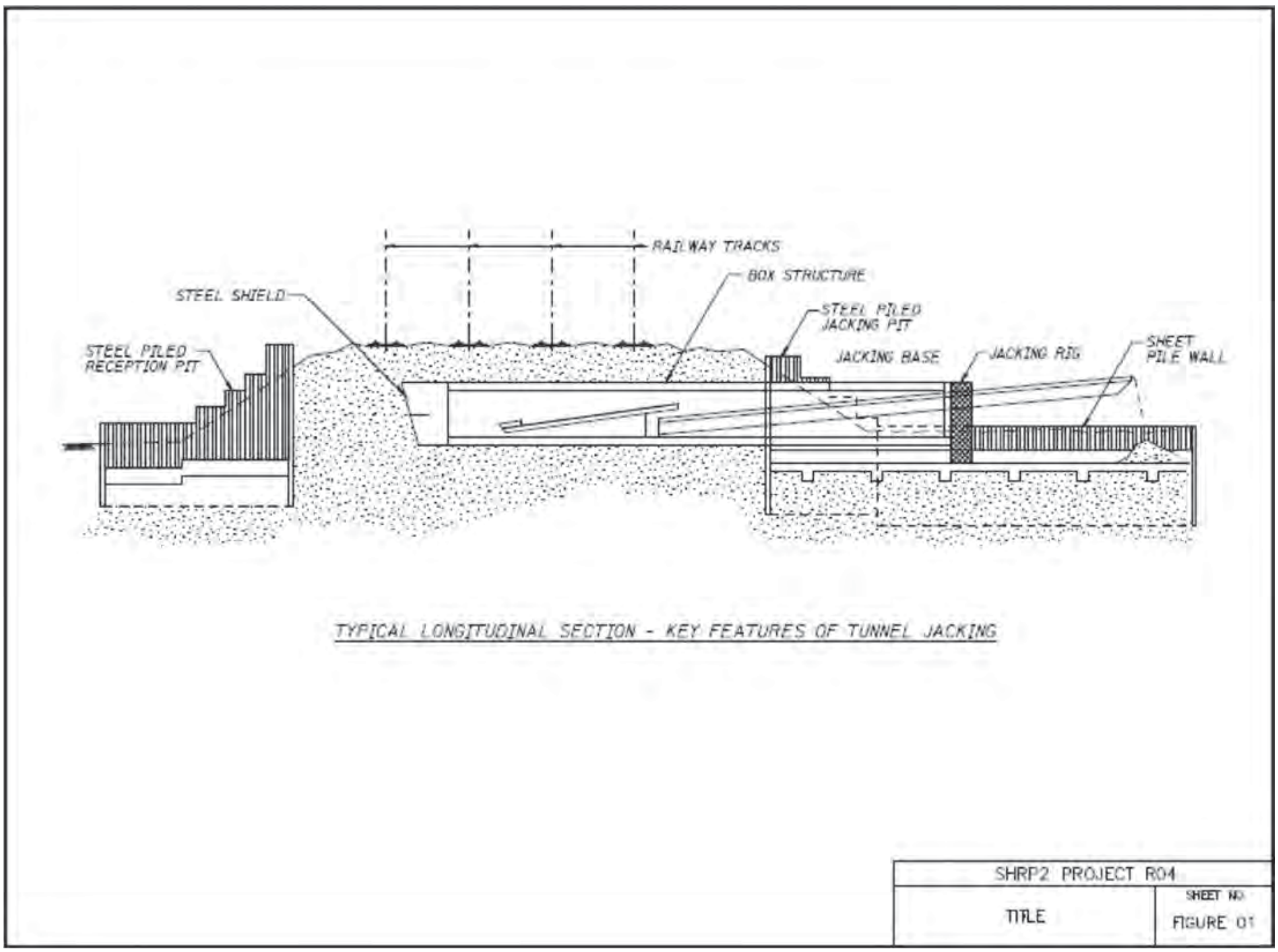


Figure 2.54. Tunnel jacking, typical longitudinal section, Example A.

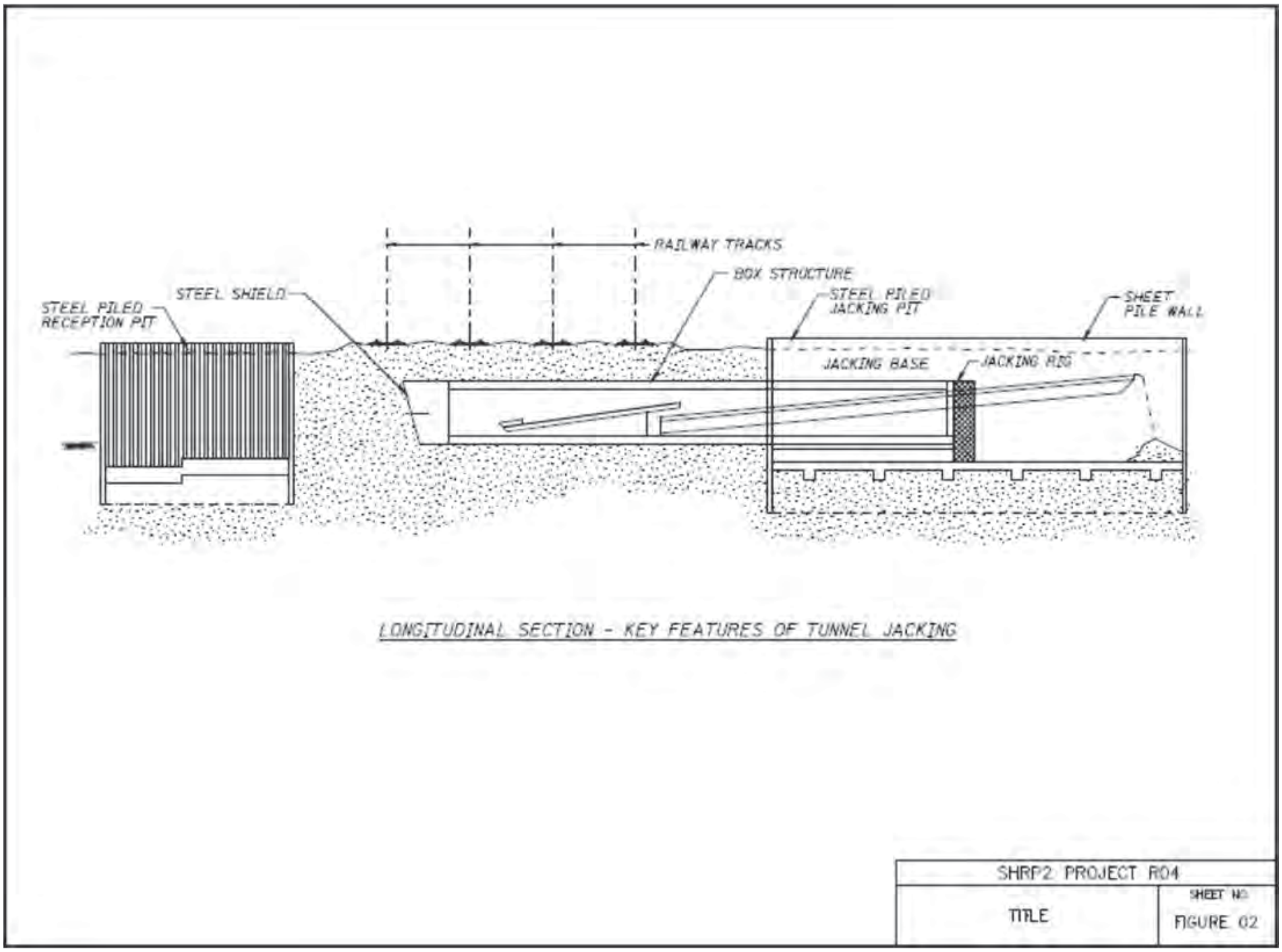


Figure 2.55. Tunnel jacking, typical longitudinal section, Example B.

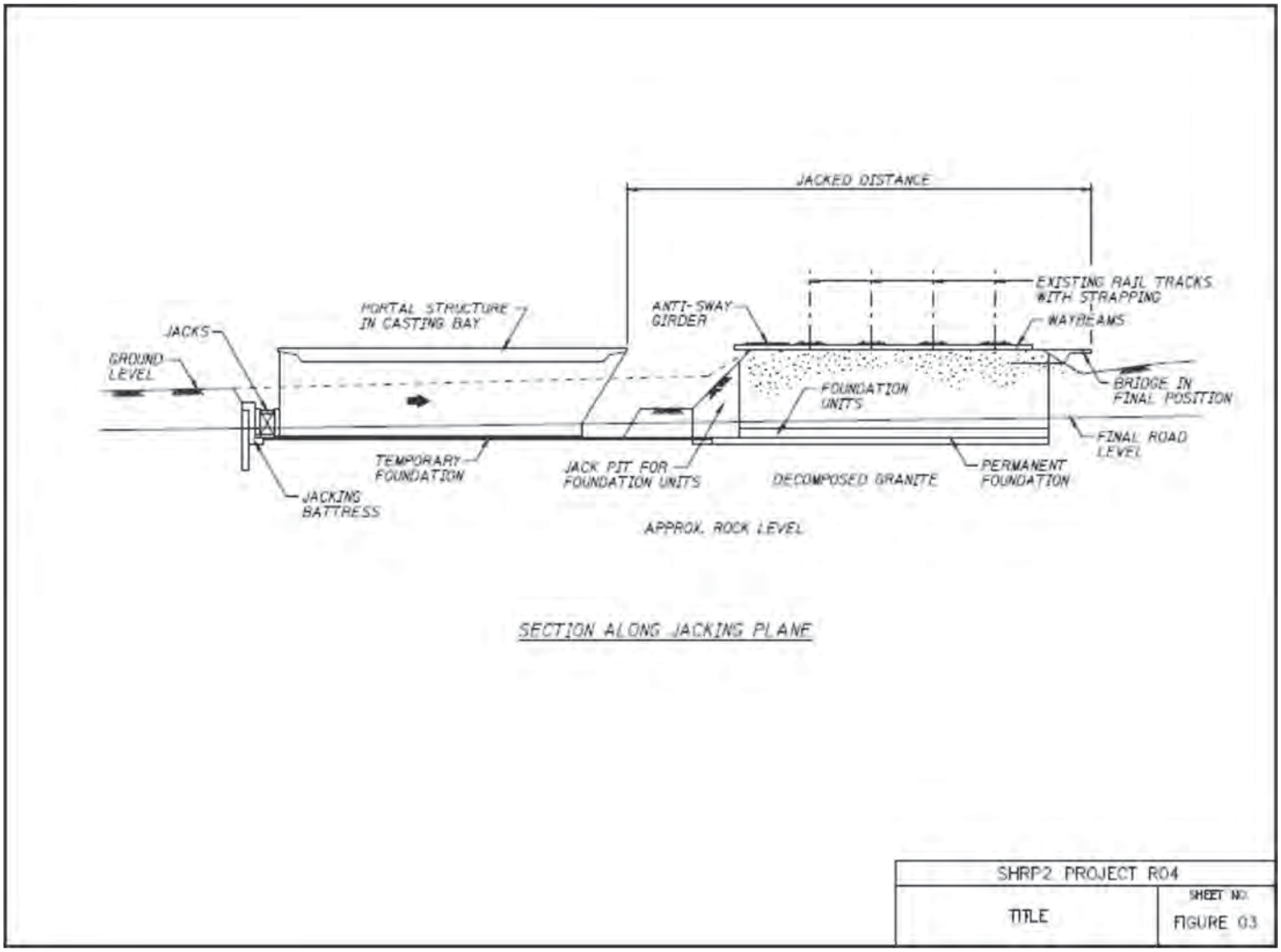


Figure 2.56. Section view along jacking plane.

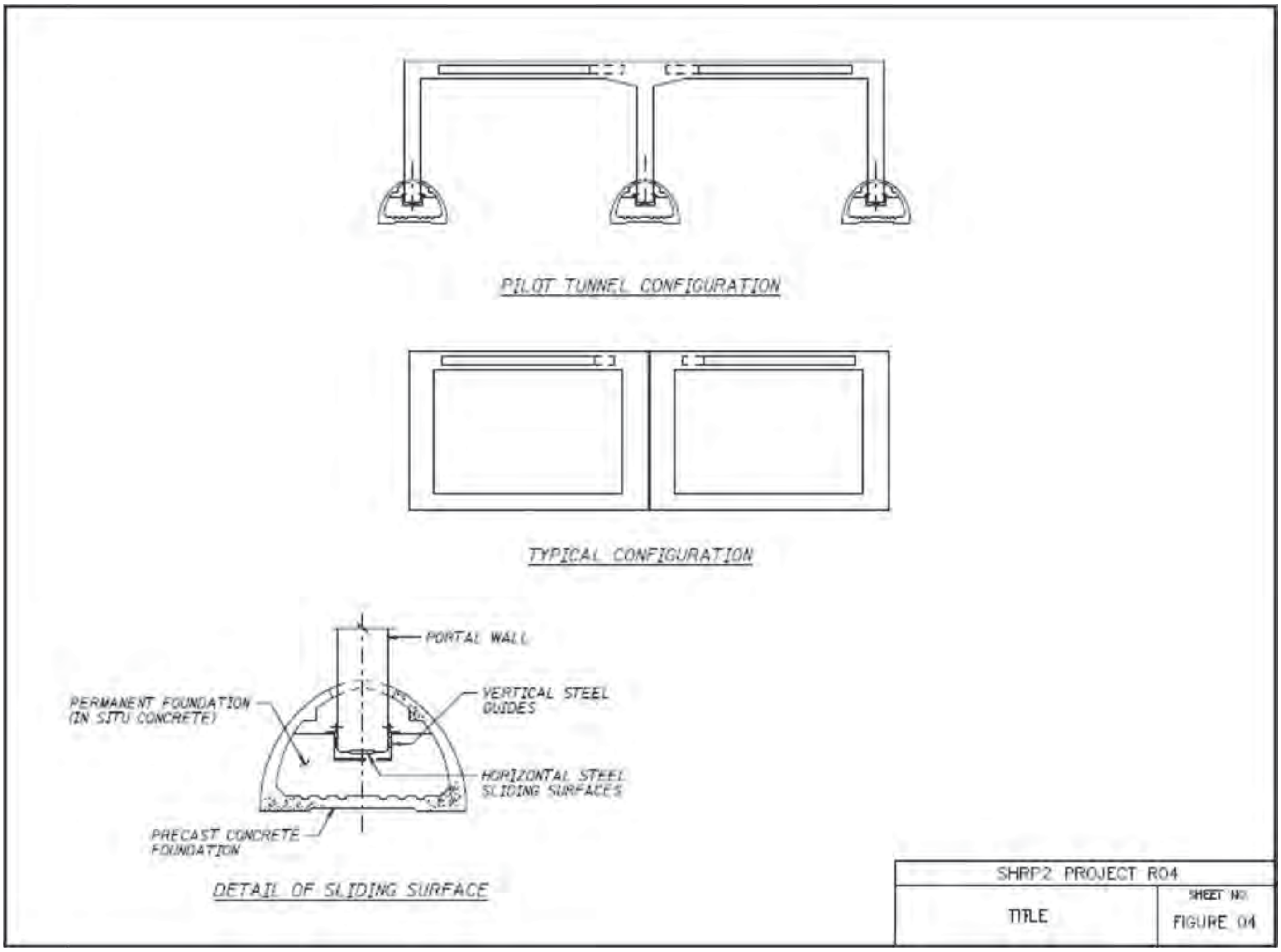


Figure 2.57. ***Typical pilot tunnel configuration and sliding surface detail.***

method for installing culverts without severe disruption to the overlying rail service.

The development from pipe jacking to tunnel jacking was made in the 1960s, when circular pipe jacked sections were found to be either too small or were inefficient for their intended purpose. At that time, contractors started to jack rectangular sections. These were typically reinforced concrete and were precast off site. In the years since those early days, the technique has developed significantly and now includes a wide variety of possible end products, ranging from 3 m square (10 ft square) rectangular pedestrian subways installed beneath busy roadways, to 25 m wide by 12 m high (80 ft by 40 ft) monolithic rectangular tunnel sections to accommodate the full width and height clearance of highway traffic beneath operating rail tracks, roads, rivers, or airport runways.

The technique has also been used to install relatively small tunnel sections within which concrete foundations can be constructed to receive a bridge superstructure. The tunnel sections can either be jacked into position with no disruption to the overlying facility operations, or slid into position during a limited possession of the overlying facility. Superstructures installed by this method have included simply supported spans, multi-span bridges, and portal frame bridges. Large skews have also been accommodated.

Experience Today

The first large-scale application in North America was in Ontario, Canada, in the early 1990s. It has been used successfully on the Boston Central Artery project and in a smaller application in Westport, Connecticut, in 2003.

Conditions for Successful Application

In selecting the most appropriate tunnel jacking solution for a particular situation, consideration must be given to a host of issues, including the following:

- The required tunnel clearance envelope.
- Any requirement for services within the completed tunnel.
- For highway tunnels, driver sight lines.
- The acceptable amount of disturbance to the overlying facility.
- The ability to re-level or to adjust the overlying facility periodically during the installation of the jacked tunnel.
- Optimum depth from the ground surface to the top of the tunnel.
- Ground conditions both for stability at the tunnel face and for the provision of required jacking force to install the tunnels.
- Maintenance provisions to the completed tunnel.
- Details of any abutting structures and tunnels.
- Architectural and aesthetic requirements.
- Health and safety of the construction staff.

Control of Face During Tunneling

A shield is provided at the front of the tunnel. In this instance, a steel shield is indicated, although shields that are substantially made from concrete are usual for larger sections. The shield is designed to allow the contractor to excavate the open face of the tunnel as jacking proceeds. Details of the shield depend on the ground conditions, size of the tunnel, and the means of excavating and removal of the soil from the tunnel face.

The shield enters the ground through an opening formed in the headwall. This is a crucial part of the tunnel jacking operation and a carefully sequenced method of progressive removal of sections of the headwall and transfer of ground support to the shield is often required. Hydraulic jacks are used to push the tunnel forward into the soil face. The jacks are located at the rear of the tunnel, and can either push off the jacking base or off a rear thrust wall.

Tunnel jacking is essentially a soft ground tunneling technique. The key to controlling surface settlements and movements is to control the “loss of ground” into the tunnel face during installation (assuming that the frictional drag has been adequately controlled). The shield is designed to support the tunnel face, to provide the means of efficiently excavating ground and to provide additional face support if the ground becomes less stable. The shield will normally have an open face, which is subdivided into an arrangement of cells. The cell size and configuration is dependent upon the ground conditions and the proposed methods of excavation.

In relatively soft soils, the shield cutting edges and cell dividers will typically be rammed into the ground by a short distance. Excavation is then carried out within each cell by removing 150 mm to 300 mm of material from the tunnel face, and once the ground ahead of the shield has been checked to ensure that there are no obstructions or hard spots, the tunnel is then jacked forward. It is the common practice of European contractors to keep the tunnel face as open as possible using vertical and horizontal dividers within the shield as a means of providing support.

Longer Tunnel Sections

For long tunnels, the length of tunnel to be pushed forward at any one time can be reduced by the introduction of an intermediate jacking station (IJS). This can result in lower jacking forces and less tendency for the ground to move forward with the moving tunnel. Longer tunnels may require several IJSs along their length.

The tunnel moves forward in a caterpillar-like motion by excavating a little of the open face, then pushing the tunnel forward by the amount of excavation. Often the shield's cutting edges will remain embedded in the ground to enhance stability of the tunnel face and to limit ground movements and resulting settlement, although in favorable ground conditions it can be possible to excavate a little ahead of the shield, particularly any internal shield dividers.

Reduction of Friction During Tunneling

The reduction of friction between the moving tunnel and the stationary surrounding soil is an area of tunnel jacking which has undergone numerous developments. Different contractors have their own preferred means and methods for controlling and reducing friction.

There are several reasons why it is preferable to reduce the friction between the tunnel units and the surrounding ground during installation of the jacked tunnels:

- Lower friction results in a lower jacking resistance. This in turn results in the need to provide fewer hydraulic rams (and their associated hydraulic power packs, hoses, control systems).
- Lower friction can result in a more uniform friction, which assists directional control of the tunnel units during installation.
- Lower friction results in less disturbance to the surrounding soil. Disturbance to the soil is a contributory factor toward surface settlement and lateral movement.

There are numerous ways in which the friction between the tunnel units and the surrounding ground can be reduced or controlled. These include

- A high level of quality control and close construction tolerances when constructing all elements of the tunnel jacking works.
- Careful control of the mining operations at the tunnel face to ensure that the ground is trimmed as required. It is normal for the shield to perform the final trim of the ground as the tunnel is jacked forward. Slight overcut to the walls (and sometimes the roof and floor, if these are to be filled with a drag-reducing material) is usually made.
- The interface between the moving tunnels and the stationary ground is lubricated using a regularly spaced array of injection points from within the tunnels.
- The moving tunnels can be separated from the adjacent ground by the insertion of a separating layer, particularly between the roof and the overlying ground. It is in this area that many developments and advances have been made over the years. Contractors have developed (and sometimes patented) different ways of providing this separation. It can be accomplished via steel plates; laminated steel, nylon, or rubber sheeting; steel cables; or pre-installed steel tubes. These separating layers often perform additional functions and can be used to reduce ground movements and to assist with directional control during installation.

Abutments Constructed Within Jacked-In Rectangular Tunnels

A development of the tunnel jacking system was made when it was found that the required finished opening size was too large for a single rectangular section. This development involved using tunnel jacking techniques as a method of providing a clear opening beneath an overlying facility. The clear opening was used to construct reinforced concrete abutments, which could subsequently be used to support a bridge superstructure that could be installed over a very limited possession.

Initially, single-jacked tunnel sections were used for the construction of the abutments. This developed into the use of multiple-jacked tunnel sections within which the finished abutments could be constructed.

Special optional features of this concept include the use of prestressing to create a monolithic section within the tunnels, the use of removable sections in which the two separate tunnels adjoin to permit the in situ concrete to flow, and the use of removable sections at the top of the upper section to allow the superstructure to bear on the pre-installed slide track and permanent bearings. This system allows the installation of large-span bridges. A two-level installation of jacked tunnels is one of the options, although the system can be used for greater height abutments using three or more tunnels, one above another. It is usual to install the lowest tunnels first.

Conclusions

Tunnel jacking is a technique that requires a clear understanding of the relationship between design and construction, tolerance of possible changed conditions within the ground, and consideration of a host of possible occurrences during construction. It can provide a solution to the problem of crossing beneath an important facility, which would otherwise be unsolvable without relocating that facility. The technique has been developed to an extent that it is now possible to jack a large range of tunnel section sizes and lengths in grounds that vary from rock to soft water-bearing deposits.

The developments have generally been made as a result of a particular need of a project, of difficulties experienced on a previous project, or because of a technical benefit perceived by specialist contractors who undertook the work. The developments have allowed tunnel jacking to grow and mature, and allow the technique to take its place along with other tunnel methods available today.

CHAPTER 3

Findings and Applications

Overview

Sixteen ABC design concepts developed by the R04 team in Phase I have been described with concept sketches and photographs in Chapter 2. They include new concepts, or adaptations of existing concepts, that are proposed as solutions to various ABC problems. Some concepts such as the modular bridge systems are truly complete systems and generally focus on the use of large prefabricated components in order to expedite construction. Many of the other concepts are at the elemental level, such as superstructure systems, deck concepts, various innovative column construction ideas, and so forth. They are not complete bridge systems, but they can be used together to form complete bridge systems.

The work of Task 6 in Phase II was to incrementally winnow the collected findings and ABC concepts from Phase I through screening, more detailed engineering and constructability evaluations, and assessment of implementation challenges. The R04 team further developed and refined the more highly rated concepts in Phase II so that they can be readily implemented by state DOTs and other bridge owners with minimal additional design effort. The most promising technologies from the Task 6 evaluations have been recommended for standardization and use in field demonstrations in Phase III. Any technology recommended must meet minimum standards of readiness for execution, suitability for ABC, a promise of durability, and value to the owner.

A key objective of SHRP 2 R04 is to produce pre-engineered ABC design standards for the recommended substructure and superstructure systems. By standardizing designs the opportunities for local or regional fabricators will be greatly increased. In many cases, standardizing designs also promotes the ability for local contractors to self-perform their precasting, thus encouraging broader acceptance of ABC and reducing costs.

To perform thorough and consistent evaluations on the recommended concepts from Phase I, all Phase II evaluations were required to follow a series of steps, as outlined below:

Step 1: Compile and summarize published and unpublished materials pertinent to the technology.

Step 2: Perform an engineering evaluation of the concept that focused on the soundness of the underlying engineering design behind any particular concept.

Step 3: Perform a constructability evaluation of the concept to evaluate issues specific to transportation of components, erection methods, equipment needs, and the suitability of the system to rapid construction.

Step 4: Discuss implementation challenges and barriers to more widespread use of the recommended technologies and any specific obstacles that may inhibit the use of a technology.

Step 5: Develop recommendations for testing for concept development and implementation.

The Phase II evaluations provided a short list of concepts recommended for standardizing and Phase III implementation. Results of the evaluations are presented in three parts in this chapter:

Part 1: Evaluation of Precast Decks and Complete Superstructure Systems;

Part 2: Evaluation of Precast Substructure Systems; and

Part 3: Evaluation of ABC Construction Technologies.

In Part 1, the results of the evaluations for precast decks and complete prefabricated superstructure systems have been presented under the following headings:

- Modular superstructure systems
  - Deck bulb tees;
  - Double tees;

- ○ Decked stringer system; and
  - ○ Decked trapezoidal box girders.
- Segmental superstructure systems
  - ○ Box girders;
  - ○ Segmental slabs; and
  - ○ Segmental voided slabs.
- Precast decks.

In Part 2, the evaluations for precast modular abutments and complete piers are presented under the following headings:

- Precast modular abutment systems
  - ○ Body;
  - ○ Wings; and
  - ○ Support options, piles, shafts, spread footing.
- Precast complete pier systems
  - ○ Whole pieces, footing, shaft, cap; and
  - ○ Support options, piles, shafts, spread footing.
- Segmental columns and piers
  - ○ Segmental columns;
  - ○ Pier caps; and
  - ○ Footings.

In Part 3, the construction concepts introduced in Phase I and subsequently carried forward for further evaluation in Phase II are presented.

Part 1: Evaluation of Precast Decks and Complete Superstructure Systems

Summary

This review of accelerated bridge construction (ABC) superstructures system is part of the SHRP 2 R04 research project Innovative Bridge Designs for Rapid Renewal, administered by the Transportation Research Board (TRB) of the National Academies. This TRB project intends to develop standardized approaches to designing, constructing, and reusing complete bridge systems that address rapid renewal needs and make efficient use of modern construction equipment.

The work of Task 6 is to incrementally winnow down the collected findings from Phase I through screening and further evaluations. The purpose of these evaluations is to provide recommended ABC concepts and techniques that can be advanced to standard plans. In this chapter, the results of the evaluations for precast decks and complete prefabricated superstructure systems have been presented under the following headings:

- Modular superstructure systems
  - ○ Deck bulb tees;
  - ○ Double tees;

  - ○ Decked stringer system; and
  - ○ Decked trapezoidal box girders.
- Segmental superstructure systems
  - ○ Box girders;
  - ○ Segmental slabs; and
  - ○ Segmental voided slabs.
- Precast decks.

This review of superstructure design concepts documents each of the concepts, provides a review of the associated research literature, and provides a review of the engineering and constructability evaluations. The review also pinpoints implementation challenges and provides suggestions to overcome those challenges. In addition, testing needs and future research are also discussed.

The engineering evaluation focuses on the soundness of the underlying engineering design behind any particular concept. This type of assessment is a more detailed review of a concept than that which was conducted during Phase I. The engineering evaluation is undertaken to evaluate the recommended concept critically and to assess the quality of the underlying research, the suitability of any proposed design approaches, the quality of proposed specifications, and so forth. The constructability evaluation is aimed at assessing issues specific to the transportation of components, erection methods, equipment needs, and the suitability of the system to rapid construction. Part of the constructability evaluation is to document the potential time savings of an ABC technique as compared with conventional construction. An evaluation matrix for each option is included in the report. Each option is evaluated based on criteria such as initial cost, durability, system simplicity, market readiness for rapid construction, ease of evaluation for overload permits, and other factors. A score for each criterion is assigned on a scale of 1 to 5 with 1 being poor, 3 being average, and 5 being very good.

Design Concept Descriptions

Modular Superstructure Systems

The intent is to develop pre-engineered standards for modular deck segments for concrete and steel bridge superstructures with spans of up to 140 ft that can be transported and erected in one piece. Longer spans, up to 200 ft, can be transported in sections and spliced on site and erected using special techniques such as girder launching. Standardizing the designs to not more than five sections (for each of the three deck segments) that will cover the span range from 40 ft to 140 ft will increase their availability through local or regional fabricators, reduce lead times, lower costs, and increase familiarity among local contractors. For short spans, these segments can be purchased and erected in a few days by county crews using conventional equipment. The deck segment concepts incorporate proven elements and details used by several states.

Four options for standardized modular superstructure systems are presented:

- Modular steel superstructure systems
  - Decked steel stringer system (two-beam steel sections with slab); and
  - Decked steel trapezoidal boxed girders.
- Modular concrete superstructure systems
  - Concrete deck bulb tees with integral deck; and
  - Concrete double Tees with integral deck.

Decked Steel Stringer System

The construction of the superstructure is a time-consuming part of cast-in-place (CIP) bridges; therefore, its prefabrication, in part or total, can significantly reduce construction time and traffic disruption. Increasingly, innovative bridge designers and builders are finding ways to prefabricate entire segments of the superstructure. Preconstructed composite units may include steel or concrete girders prefabricated with a composite deck that are cast off the project site and then lifted into place in one operation. One method of prefabrication involves constructing conventional composite stringer bridges off site and installing them rapidly on site.

Decked Steel Trapezoidal Box Girders

Steel tub girder use is becoming more commonplace in modern infrastructure design. It offers advantages over other superstructure types in terms of span range, stiffness, and durability—particularly in curved bridges. Torsional rigidity is an advantage for spans with tight horizontal curvature. In addition, steel tub girders have distinct aesthetic advantages, due to their clean, simple appearance. Bracing, stiffeners, utilities, and other components are typically hidden within the box, resulting in a smooth, uncluttered form. The steel surface area exposed to the environment is greatly reduced, since half of the web and flange surfaces are enclosed. Welded steel tub girders are more costly to fabricate than plate I-girders. It takes highly skilled workers to fabricate and erect steel tub girders, which results in a premium on labor costs even if material costs are competitive with plate girder alternatives. An example of steel trapezoidal box girders is given in Figure 3.1.

Figure 3.1. Steel folded plate trapezoidal box girder.

A folded plate bridge system offers an economical solution over welded box girders for short-span bridges. The system consists of a series of standard shapes such as trapezoidal tubs or boxes that are built by bending flat plates using a brake press. Bending plates to specified shapes is rapid and very economical when compared with welded construction. A folded plate bridge system can be constructed using light construction equipment and can provide long service life with minimal maintenance. Almost 45% of the bridges in the U.S. bridge inventory are less than 60 ft in length. Most are simple spans located on county roads. Span length for this system without a splice is currently limited to about 60 ft, reflecting the longest press brakes that are available in the industry. Folded plate girders suitable for different span lengths differ only by their cross-sectional dimensions. Span lengths longer than 60 ft can be developed using spliced sections. More specifically, varying the width of the top and bottom flanges and the depth of the web while keeping the plate thicknesses to typically ½ in. can accommodate span length requirements. The different top and bottom flange widths and web depth can easily be accommodated by changing the bend locations, so fabricators can build folded girders quickly while stocking only one or two plate thicknesses. That will assure delivery of steel bridge girders without long lead times. Plate sizes are available in 10-ft widths from four U.S. manufacturers. Sections that can be fabricated within these width limitations will aid the standardization process. Sections can also be galvanized for a small additional cost. Initially, only simple spans are planned but the design can be economically extended to continuous girders while also preserving the rapid renewal advantages. The sections can be fabricated with a precast deck in the yard prior to shipping to the site. A 60-ft folded plate girder with precast deck should weigh less than 30 tons and can be easily erected without the need for heavy cranes. In some cases, even county crews will have the capability to erect such bridges.

Concrete Deck Bulb Tees and Double Tees

Precast concrete deck girders are becoming increasingly popular as an economical solution for short-span and medium-span bridges. For local bridges, segments for short spans can be purchased and erected in a few days by county crews with conventional equipment. The top flange is designed to function as an integral deck, making them an attractive option for rapid replacement applications. Omitting the concrete topping and the transverse posttensioning will significantly reduce the on-site time for construction. The top ¼ in. can be ground after installation to achieve a smooth riding surface. Use of an overlay system is optional. The added protection from an asphalt overlay and membrane can increase service life. Cast-in-place longitudinal joints between girders can be designed to allow full moment transfer or only shear transfer. Recent advances in joint design have greatly enhanced durability and performance. The type of joint depends on the traffic exposure or the functional classification of the route. Control of camber is a key consideration for constructability and rideability. Several states, such as Washington and Idaho, have standardized deck girder sections using the bulb tee and double tee configurations to increase their availability. These girders can also be spliced in the field by posttensioning to allow longer spans than is feasible using a single transportable section.

Segmental Superstructure Systems

The segmental superstructure system consists of short sections that can be connected to each other to form the entire superstructure for a bridge. Segmental bridge sections are either precast or cast-in-place sections. The segmental superstructure system provides a number of advantages. Long spans are possible. The finished structure is durable and aesthetically pleasing. Erection methods can be easily adapted for safe and rapid construction over existing roadways, rivers, and other obstructions. Highly skewed supports are easily accommodated by this system.

Segmental bridges are generally very economical for longer spans. They are the design of choice for spanning deep valleys and wide water crossings, and across highways and existing facilities without the use of costly false work. Precast segmental construction is also typically used with long viaducts. Precast segmental technology can be readily adapted to typical highway bridges with very little effort. Sections can easily be adapted from traditional boxes to solid or voided slabs sections, or other less common sections such as channel bridge sections. Technologically, these adaptations are relatively simple.

Three types of precast segment sections are suitable for ABC applications:

- Conventional single cell box girders;
- Solid slabs and voided slabs; and
- Channel sections.

These typical cross sections can be used in simple grade separations, span-by-span viaducts, or continuous, balanced cantilever-type construction.

Conventional Single Cell Box Girders

Conventional single cell box girders have been used in dozens of applications across the United States in the past 30 years. They are typically cost-effective for longer spans (continuous versus simple).

They are commonly used for span-by-span highway or transit construction in the 125-ft to 150-ft span range. They can be extended up to 180 ft by using continuity posttensioning.

The United States is beginning to see the application of single cell box girders more and more on typical highway construction for interchanges, overpasses, and grade separations. In Minnesota, single cell box girders were used on various interstate reconstruction projects, and in Indiana, they have been employed along I-80 and I-90.

Solid Slabs and Voided Slabs

This technology represents the greatest opportunity for manufactured bridges and ABC. These solutions have been used widely in the United States and Canada over the past half century, but mostly for CIP applications. They are typically the most cost-effective and durable bridge types available for spans up to 130 ft. Examples for segmental solid and voided slab bridges are shown in Figures 3.2 and 3.3, respectively.

The use of solid deck slab panels is a standard approach for the composite deck of cable-stayed bridges with spans of 12 ft to 25 ft. Posttensioned solid slab spans are economical up to about 80 ft. Voided slab spans are economical up to about 135 ft.

This technology is readily adaptable to solid slab or voided slab segments and can be accomplished economically. It has been done elsewhere. A real market exists in the United States for a manufactured ABC bridge replacement technology. The aging U.S. interstate infrastructure drives the need for a durable replacement technology that can be applied on a large scale for "bread-and-butter" grade separation bridges.

Figure 3.2. Segmental solid slab bridge.

Figure 3.3. Segmental voided slab bridge.

Segmental Channel Sections

These sections have not been widely used in the United States, or other markets. They are limited in width (typically two lanes) without introducing transverse ribs or floor beams, include some casting issues due to their flexibility, and are mentioned only to demonstrate the feasibility of using simplified erection girders and small picks. An example is shown in Figure 3.4.

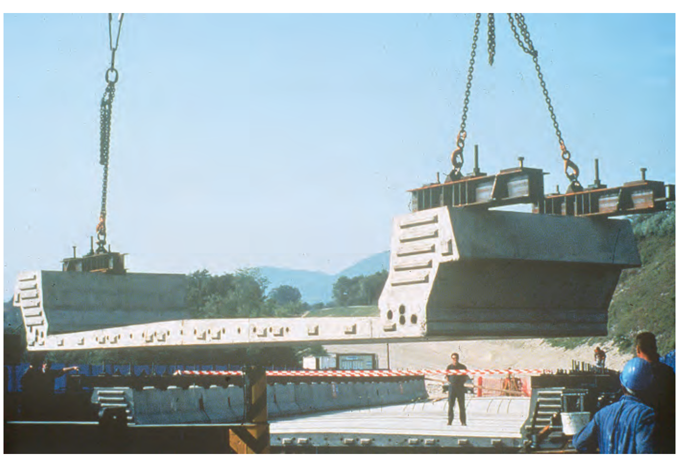


Figure 3.4. Segmental channel section.

Precast Concrete Bridge Decks

A variety of precast bridge deck technologies have been used for the past 50 years to facilitate rapid construction and ease traffic congestion.

Precast bridge deck panels can be used in place of a CIP concrete deck to reduce bridge closure times for deck replacements or new bridge construction. The precast panels are most commonly prefabricated at a casting yard, which provides optimal casting and curing conditions and which normally results in durable, long-lasting decks. For smaller projects, or those located beyond reasonable access to a PCI-certified precasting plant, a number of precast bridge deck panels have been site-cast by the bridge contractor.

Some of the challenges inherent with any bridge deck technology include: performance of deck panel joints, accommodation of a normal crown, composite action with girders, and attachment of bridge railings.

In general, precast concrete bridge decks can be classified as either partial-depth or full-depth. Partial-depth precast concrete deck panels (PCDP) are often used to provide a durable, stay-in-place concrete form system that is later supplemented by 3 to 5 in. of CIP concrete. Although these partial-depth panel systems are valuable in providing a durable bridge and have been standardized for use on both conventional and long-span bridges in a number of states, the need for a cast-in-place concrete structural topping does not make them suitable for use in an ABC environment. Therefore, the remainder of this section will focus only on full-depth concrete deck panels.

To further narrow the focus of these recommendations, the PCDP described here include those panels that span transversely between conventional prestressed concrete beams or steel girders. These pieces, typically limited in width to a size transportable without specialized permits, are usually 8 to 10 ft wide and 8 to 9 in. thick. The vast majority of bridges in the United States are two lanes wide and include moderate shoulder width of 8 to 10 ft. To eliminate the often troublesome longitudinal joint, PCDP are typically designed for the full width of the roadway, including barriers. Approximate lifting weights for full-depth concrete deck panels range from 15 to 20 tons.

Figure 3.5 schematically depicts a prestressed concrete beam bridge with full-depth precast deck panels. The construction process consists of the following:

1. Installing the panels on top of the beams. The self-weight of the panels is transferred to the beams through leveling bolts protruding through the bottom of the panels. The haunch depth is adjusted to provide the desired top-of-deck elevation.
2. The haunch areas are filled with grout or concrete between the top of the beam and the bottom of the deck. The transverse joints are filled next.

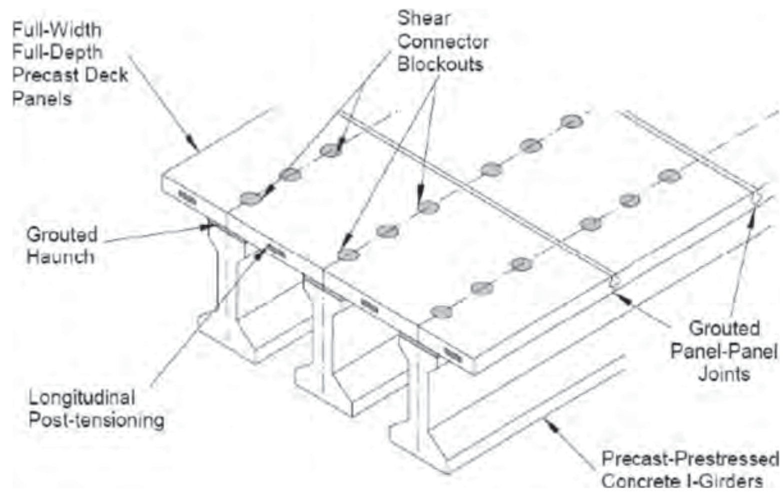


Figure 3.5. Anatomy of a precast concrete deck panel system.

3. If the deck is to be posttensioned, this operation is then performed. After the posttensioning operation is complete, the posttensioning ducts are pressure-injected with a corrosion-resistant grout.
4. The haunch is placed after the posttensioning operation.
5. To obtain composite action between the deck and underlying beams or girders, a mechanism must be used to provide shear transfer between the panels and beams. This is normally accomplished through the use of shear connector blockouts arranged to match with either shear reinforcing extended up from prestressed beams or welded studs on steel girders.
6. Once the grout in the haunch has cured, the leveling bolts are removed and the panels and beams act as a composite system.
7. Barrier rails are then cast.
8. A wearing surface may be placed if desired, but the time involved for this operation could significantly reduce the ABC benefits of the system.

The shear pockets provided in each precast panel are designed to fit over shear studs (in the case of steel girders) or shear reinforcing (in the case of prestressed beams) and the pockets are grouted to ensure fully composite action. Figure 3.6 depicts a typical shear pocket for use with a steel girder.

Reinforced Concrete Deck Panels

Precast reinforced concrete panels are only reinforced using conventional mild steel. These panels are designed per AASHTO requirements for service, strength, and serviceability. The

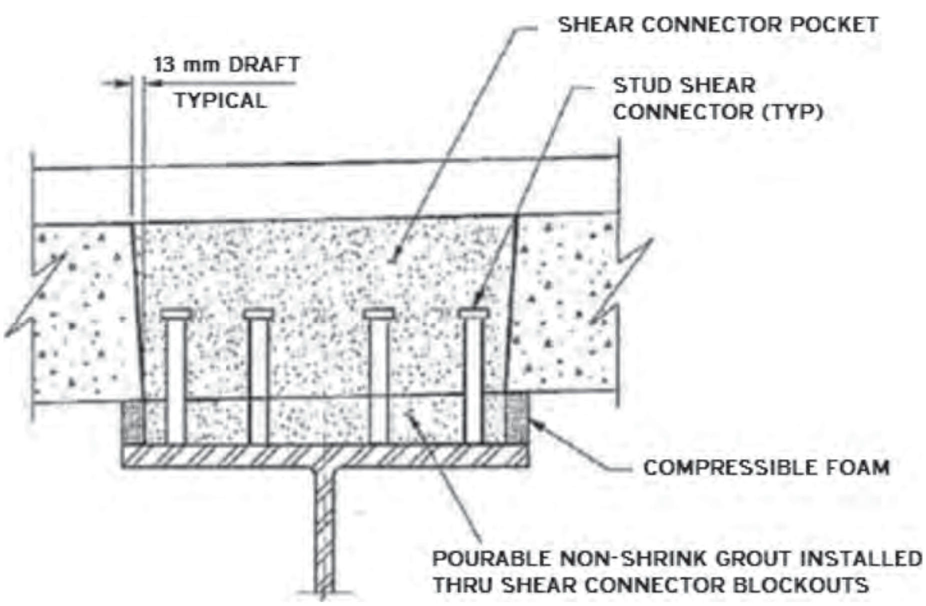


Figure 3.6. Precast deck panel with shear pocket.

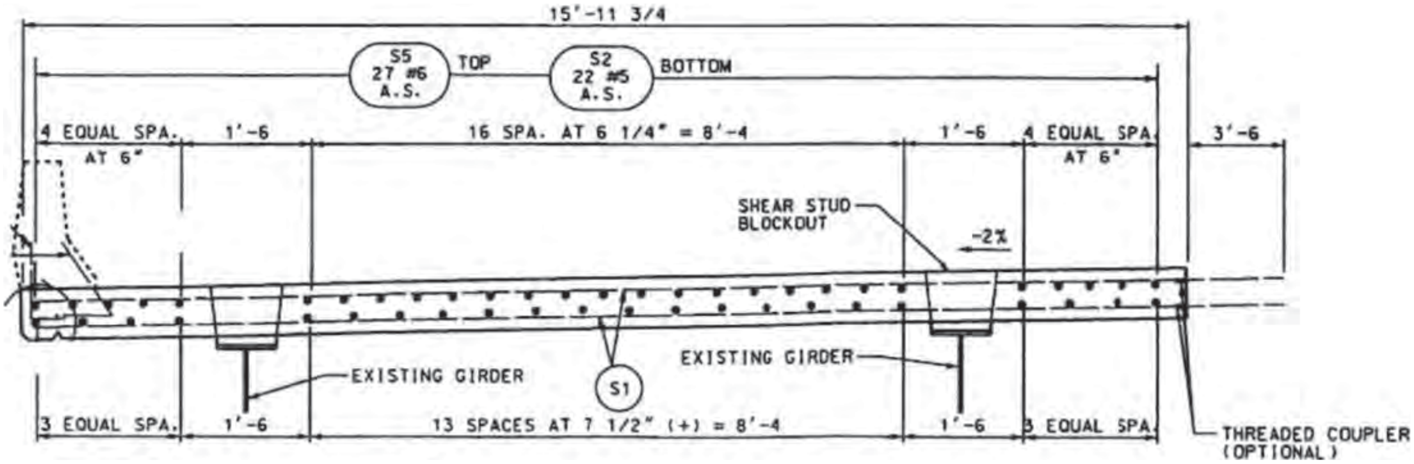


Figure 3.7. Reinforced concrete deck panels for the Utah DOT.

panels can be designed as either continuous over a series of parallel girders or simply supported over the longitudinal girder.

Figure 3.7 depicts an example of precast reinforced concrete panels used by the Utah DOT on the C-437 bridge rehabilitation of the county road over I-80 to Wanship. The precast panels are conventionally reinforced with two layers of epoxy-coated steel bars in each direction. The longitudinal reinforcement is designed to resist the negative moment over the piers resulting from the superimposed dead and live loads applied when the deck is made composite with the superstructure.

Prestressed Concrete Deck Panels

Perhaps the most common type of panel in use today, prestressed concrete panels are typically transversely pretensioned (perpendicular to the driving direction) and conventionally reinforced in the longitudinal direction. The panels are reinforced for temperature and shrinkage on both directions. Prestressed panels must be longer to meet the transfer and development length criteria of strands. The main advantage of prestressed panels over reinforced panels is that the permanent compressive force provides a crack-free deck.

Posttensioned Concrete Deck Panels

Posttensioned concrete deck panels are very similar to the prestressed panels described above, except that the main reinforcement in the panels is provided by a posttensioning system rather than prestressing. The posttensioning force is typically provided in the longitudinal direction, but can also be applied in the transverse direction for especially wide bridges or those that require staged construction. The panels are generally lightly reinforced to resist self-weight, temperature, shrinkage, and creep. Figure 3.8 depicts typical posttensioned concrete deck panels used for the recently constructed Bill Emerson Memorial Bridge, spanning the Mississippi River in Cape Girardeau, Missouri. The precast concrete deck panels are conventionally reinforced with top and bottom meshes of epoxy-coated bars. The posttensioning is provided in the longitudinal direction. The thickness of the precast panels is 10 in.

Figure 3.9 illustrates an example of deck posttensioning in both longitudinal and transverse directions for the Door Creek project on Interstate 39/90 in Wisconsin. The precast system consists of full-depth precast concrete deck panels, which were constructed off site and delivered to the site ready for placement. Because the panels were posttensioned in both longitudinal and transverse directions, ducts were placed in both directions so that they would not interfere with each other. The longitudinal posttensioning duct is located at mid-depth of the slab, while the transverse posttensioning ducts are placed above and below the longitudinal ducts. The panel thickness for the Door Creek Bridge is 8¾ in., which is similar to a typical cast-in-place deck.

One significant advantage of using posttensioned deck panels to improve an existing bridge is that they are often

Source: Missouri DOT.

Figure 3.8. Posttensioned deck panels for the Bill Emerson Memorial Bridge, Missouri.

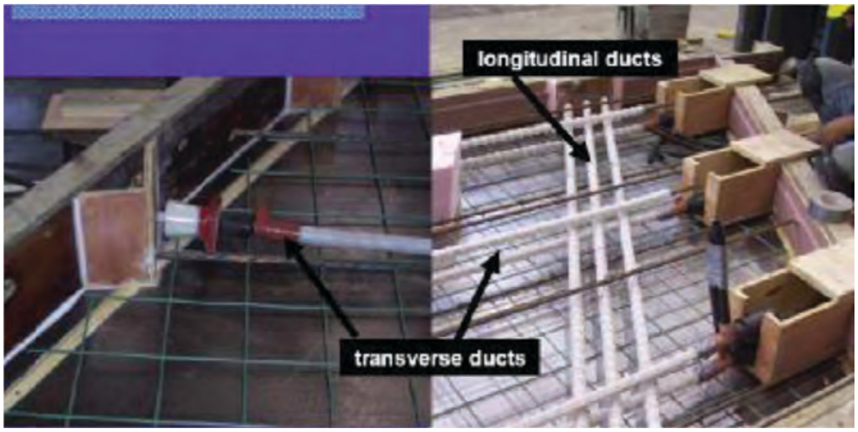


Figure 3.9. Posttensioned deck panels for the I-39/90 bridge in Wisconsin.

thinner than a CIP deck due to the existence of internal tendons in one or both directions. Thinner panels translate into a lighter-weight deck, which could improve the live-load rating of older bridges. In addition, the use of high-performance, high-strength materials allow precast decks to be significantly lighter than a conventional cast-in-place system.

Although the use of exotic fiber composite material can reduce the panel loads even further, AASHTO LRFD serviceability requirement limit how far this envelope can be pushed.

Ultra-High-Performance Concrete Panels (Waffle Slab)

An innovative precast concrete bridge deck system that is currently being developed uses an ultra-high-performance concrete (UHPC) waffle slab system that is designed to provide the equivalent stiffness of a solid concrete panel while removing much of the normal dead load. This system is currently undergoing testing at both the FHWA Turner-Fairbank laboratory and Iowa State University. A demonstration bridge project, funded through the FHWA Highways for LIFE program, was constructed in Wapello County, Iowa, in 2011.

The use of UHPC materials, which offer much higher strength and significantly improved resistance to intrusion of chlorides when compared to conventional concrete, offers future potential as a lightweight, durable bridge deck system. However, the material costs remain rather expensive and the technology is still being improved.

Ultra-High-Performance Concrete for Deck Panel Joints

The New York State DOT is currently investigating the use of full-depth precast deck panels with field-cast UHPC joints to develop the continuity in the deck panels. This solution had been previously attempted by the Ontario Ministry of Transportation on an experimental basis, but the New York State DOT has been using this solution for rapid replacement of bridge decks in high-traffic areas. The New York State DOT found that the strength and ductility properties of UHPC functioned well as a joint fill material when combined with precast deck panels.

The New York State DOT specifies that full-depth precast concrete deck panels are designed with HPC and use epoxy-coated or galvanized reinforcing steel. The agency further specifies a minimum of 40 MPa (4,800 psi) compressive strength. The panel reinforcement design was based on continuity through the joints. UHPC joint material was assumed to provide sufficient bond development to develop full continuity of the rebar just as if it were continuous through the joint.

The strength and low permeability of UHPC provides excellent protection of the rebar against corrosion and improved bond with the rebar, thereby providing short bond development lengths. Testing has shown that the bond development length of a #4 bar in UHPC is less than 3 in.

UHPC also offers excellent bond development length, resistance to freeze and thaw cycles, and high flexural strength and toughness, which provides the critical resistance to flexural loads generated by truck loads passing across the joint. Results of the New York State DOT indicate that the UHPC/HPC deck interface is bonded with no potential for leaking.

The New York State DOT conducted a demonstration project selected using field-cast UHPC joint fill on a 127-ft long, single bridge with full-depth precast deck panels supported on steel beams near Oneonta, New York, as shown in Figures 3.10 and 3.11. The UHPC joints were 6 in. wide and 8 in. deep.

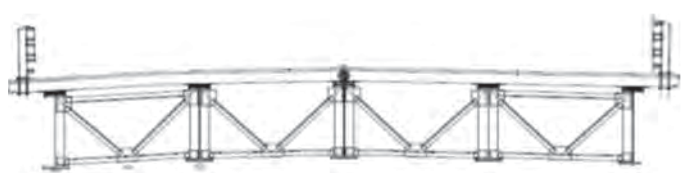


Figure 3.10. The New York State DOT Demonstration Bridge in Oneonta.

Figure 3.11. The New York State DOT precast panel joint detail for UHPC.

Following installation of the full-depth precast deck panels, the panels were adjusted and leveled for grade and a smooth, flush riding surface. UHPC joint material was transported to the joints by power buggy and then dumped directly into the joints without any vibration, which is an acceptable practice for this material. The joints were covered with form grade plywood strips and allowed to cure until reaching 100 MPa (14,500 psi) before being opened to traffic. This cure time required approximately 3 days, but could be reduced though the use of an accelerator and heat.

The field mixing of UHPC joint fill proves that this material can be batched on site and provide adequate strengths during typical field curing conditions and that local contractors can easily adapt to using UHPC in bridge projects. DUCTAL is the only supplier for UHPC. The steel fibers are from a European supplier subject to Buy America provisions.

It should also be noted that match cast and posttensioned joints are well established. They are acceptable alternatives for ABC for which designers can find information from other sources.

Engineering Evaluation

The engineering evaluation focuses on the soundness of the underlying engineering design behind any particular concept. This type of assessment is a more detailed review of concepts than that conducted during Phase I. The engineering evaluation is undertaken to critically evaluate the recommended concept and assess the quality of the underlying research, the suitability of any proposed design approaches, the quality of proposed specifications, and so forth. The engineering evaluation covers the two important aspects of modular construction: the prefabricated elements and systems, and the connections. The evaluation begins with a discussion of prefabricated elements and systems.

Modular Superstructure Systems

The intent of this project is to develop pre-engineered standards for modular deck segments for concrete and steel bridge superstructures. It is critical that only concepts that have been thoroughly vetted be advanced to the following tasks. Standardized sections need to be versatile so that they can be used in varied situations, provided they meet the decision criteria for selecting prefabricated construction. Design considerations for standardized modular superstructure systems should include the following:

- Pre-engineered standards for modular construction. Designs that can be used for most sites with minimal bridge specific adjustments.
- Optimized designs for ABC and use of high-performance materials. Simplicity and efficiency of design, availability of sections, and short lead times are key considerations.
- Segments that can be used in simple spans and in continuous spans (simple for dead loads and continuous for live loads). Details to eliminate deck joints at piers. Details for live load continuity at piers to be included for use as required.
- Use of high-performance materials: HPC/UHPC concrete, HPS or A588 weathering steel. Consider lightweight concrete for longer spans to reduce weights of deck segments.
- Deck tees and double tees with minimum 8-in. flange to function as decks with integral wearing surface so that an overlay is not required. Use of overlay is optional as part of a long-term preservation strategy.
- Limit the number of standardized designs for each deck type to five, which should cover span ranges from 40 ft to 140 ft. Consider steel rolling cycles and sections widely available.
- Segments designed to be used with either full moment connection between flanges or with shear-only connections. Each flange edge needs to be designed as a cantilever deck overhang.
- Skewed bridges: The complexity of the geometry makes prefabrication for these types of bridges a challenge. This does not preclude the possibility of using prefabrication for these structures. Prefabricated bridge replacement projects have been completed on bridges with significant skews. Attention to tolerances and field fit-up is essential for these complex structures.
- Prefabricated components can be the most cost-effective solution for any alignment. However, straight alignments allow multiple identical components, which tend to be the most economical. The alignment will affect superstructure member types. Curved alignments also typically require shorter segments in order to be transported over city streets. Initial construction costs and long-term maintenance costs are typically less for bridges on straight alignments due to their simpler construction and load paths. Preference should be given, if possible, to straightening the roadway alignment along the bridge length for lower life-cycle costs.

Modular superstructure systems are particularly suited to be used as Tier 1 concepts for weekend bridge superstructure replacements or as Tier 2 concepts, when the entire bridge may be scheduled to be replaced within a month using a detour to maintain traffic.

Decked Stringer Systems

Prefabricated decked steel stringer systems have been a very popular option for accelerated construction of bridges in this country. Their light weight, easy constructability, low cost, and availability were seen as advantages over other systems. The length and weight of each module can be designed to suit transportation of components and erection methods. Erection can be made using conventional equipment. Cast-in-place closure pours or grouted or welded joints are typically used to connect adjacent units in the field. The modules can be made to different widths to fit site and transportation requirements. They can be fabricated with square or skewed ends.

Steel stringers/girders with precast decks have become increasingly common in steel bridge construction. Advantages include the following:

- Improved efficiency with lighter steel beams (shored construction).
- Uses standard rolled shapes and welded plate girders.
- Economical or average construction costs.
- Can be fabricated with exact camber and skew to meet existing site requirements.
- Top of deck can be textured for riding surface.
- Easy and rapid erection and construction.
- Suitable for use as continuous spans.
- Durable, since deck is cast in controlled conditions.

Inverset-type concrete deck and steel composite systems have had a successful track record as an economical alternative for rapid superstructure replacement. Many such bridges have been built over a weekend, which demonstrates their constructability even in congested urban locations. Some examples summarized here are provided to illustrate their suitability of the system to rapid construction:

- In Virginia, the I-95 bridge over James River had 102 superstructure spans replaced in just 137 nights, with no impact to rush-hour traffic. Full-span-length prefabricated superstructure segments, up to 114 ft long, were fabricated at a nearby casting yard and transported using conventional flatbed trailers. Each prefabricated segment consisted of three steel plate girders with an 8¾-in. deck. During the night, the old segments were removed, and cranes then installed the new prefabricated superstructure segments.
- Also in Virginia, the US-15/29 bridge over Broad Run was completed with road closures on three weekends to replace the 12 superstructure segments, one span per weekend. Each segment consisted of two rolled steel beams made composite with a concrete deck.
- In New Jersey, each of three bridges along Route 1 was replaced during a weekend closure. Each superstructure span consists of five full-length segments with two steel girders and a 9-in.-thick composite concrete deck (Inverset) system.

It is common to cast these units in an inverted position at a prefabrication yard so that the deck is in compression in the final condition. Such a casting method may not always be feasible if the contractor is self-performing the precasting at the bridge site. Even if the deck is precast under conventional shored conditions, this modular system will provide the benefit of shored construction where the dead load is carried by the composite section. It should be noted that the beams are designed for non-composite dead loads in consideration of future deck replacement. One advantage of decked steel modular systems is that they allow the replacement of the deck, while the steel stringers can be reused.

Use of preconstructed composite units is relatively new. Accordingly, the performance of these systems is not well documented. There is less experience with and less literature discussing decked stringer systems than exist for other conventionally built systems. Although no literature was found on problems specifically associated with decked stringer systems, it is assumed that they could experience problems similar to those suffered by full-depth deck panels and prestressed concrete multi-beam superstructures. One key issue could be the longitudinal and transverse joints between the units and the durability of these joints. Durability of these joints can be achieved through proper detailing, as discussed in this section. Different top slab elevations of adjacent units could also be another key issue, which could be adjusted through a leveling procedure.

Decked Bent Plate Steel Box Girder System

A bent plate bridge system requires steel plates to be cold formed or bent in a press brake. Cold forming involves plastic deformation of the metal surface on the outside of the bend. Cold forming results in strain hardening of the material and this in turn affects mechanical properties. The extent to which the plastic deformation can take place without exceeding the limits of material ductility controls the minimum radius of bend. As the ductility and fracture toughness decrease in the areas subject to plastic deformation, it will be necessary to determine the radius of bend and the material properties and steel types suitable for bent plate girders. In the Japan bridge specifications, an allowable bending radius for cold bending is set at $5t$, where t = plate thickness, to ensure proper fracture toughness after cold bending. Bent plate connections have been successfully used on highly skewed bridges in the United States over a long period.

Generally, low carbon content is a prerequisite to good formability in bent plate applications. Steel plates are more readily formed with the bend axis transverse to the rolling direction of the plate. Though not warranted for bridge girder applications discussed here, it should be noted that heat treatment can eliminate all traces of cold working in steel plates. Engineering considerations pertinent to cold-formed steel applications are the effects of strain hardening and consequent increase in strength with reduction in fracture toughness and ductility. In the past, fracture and cracking concerns have impeded the adoption of cold bending for bridge structures. The recent availability of high-performance steel (HPS) with high fracture toughness has largely eliminated this issue and has provided a significant impetus for cold-formed applications. Though it is important to recognize and account for the effects of cold working on girder behavior, it should not be a limiting factor anymore for the use of bent plate girder systems as an economical and rapidly constructible bridge alternative.

ASTM A709 Grade HPS 50W is contained in A709-01 and is produced by using conventional hot-rolling up to 4 in. thick in lengths similar to Grade 50W steel. HPS is produced to a lower carbon content than grade 50W. The fracture toughness of high-performance steel is much higher than conventional bridge steels. The brittle-ductile transition of HPS occurs at a much lower temperature than conventional Grade 50W steel. This means that HPS 50W remains fully ductile at lower temperatures where conventional Grade 50W steel begins to show brittle behavior. The fatigue resistance of high-performance steel is controlled by the welded details of the connections and is not a concern with bent plate structures. HPS has the ability to perform without painting under normal atmospheric conditions. HPS steel has enhanced atmospheric corrosion resistance that is even better than the conventional grade 50W steel did. The same guidelines and detailing practice for conventional weathering grade steel should be followed to assure successful applications of HPS steel in the unpainted conditions. The cost-effectiveness of HPS has been demonstrated by the design and construction of HPS bridges in many states.

A steel/concrete composite bridge had been proposed in Japan with cold-formed steel U-girders, which are filled with concrete and partially prestressed near the intermediate supports of a continuous bridge. This U-girder is cold formed from a single steel sheet. Laboratory tests were performed on cold-formed U-girder models to investigate bending behavior (Nakamura, 2002). These models were about one-fourth of the preliminary designed bridge, with a span of 60 m. Bending tests were carried out to investigate the static bending behavior of the girder models in the positive and negative bending moment areas. The girder model at the span center behaved as a composite beam. In all the cases, the bending strength of the girder models was higher than the calculated yield moment, and nearly reached or exceeded the plastic moment. The U-girder section is therefore regarded as the compact section, and the plastic design can be applied for the proposed bridge. The proposed bridge system with cold-formed steel U-girders has sufficient bending strength and good deformation and rotation capacity, and it is feasible and economical.

Advantages of Bent Plate Steel Box Girder System

- Economy of steel section compared with I-girder system: The elimination of cross frames reduces steel weight. The box girders can be designed as continuous for live loads. Composite action is achieved with the precast deck.
- Economy of construction: Precasting of the deck and elimination of cross frames eliminates time consuming and expensive components of the structure.
- Reduced weight and increased underclearance: Total superstructure weight and depth are reduced when compared with concrete superstructures or I-girder steel systems. Vertical underclearance is increased. This avoids overloading substructures.
- Allows future widening: Superstructure widening can be done quickly and effectively without disrupting traffic. Fascia girders can be produced to accept future widening.
- Cost competitive: Cost competitive with concrete or other steel superstructure systems.
- Torsional rigidity: Allows the use on spans with tight horizontal curvature.
- Aesthetics: Clean appearance for very visible structures.

Disadvantages of Bent Plate Steel Box Girder System

- Specification support: AASHTO LRFD Specifications do not address the design of cold-formed structures. However, the *AISC Steel Construction Manual* does address cold bending.
- Fatigue resistance: There are potential problems of fatigue resistance at the longitudinal bend locations. This can be alleviated through the use of HPS. Direction of plate rolling needs to be considered in cold bending applications.
- Inspection access: The optimum box depth is structurally less than ideal for physical access for maintenance crews. Because the section does not include welded details, internal access would not be necessary. Weep holes at the bottom can be provided for drainage as necessary.
- Press brake limitation: Maximum length and thickness of plates that can be cold bent are limited by press brake capabilities. Inquiries made by HNTB show that current manufacturing capabilities include cold bending a ⅝-in. steel plate up to 54 ft long. This could allow bent plate box girder spans up to 100 ft with a single splice at midspan. Press brakes can bend plates over 1 in. thick, but in smaller lengths.

Conceptual Design for Composite Bent Plate Box Girder System

A preliminary load and resistance factor design (LRFD) was performed for a bent plate box girder system. The concrete deck width and thickness were taken as 8 ft and 8 in., respectively, and the concrete compressive strength was assumed as 4,000 psi. A 2-in.-thick asphalt wearing surface was assumed to be incorporated in the dead weight calculations. The material for the steel plate was 0.5-in.-thick ASTM A709 Grade HPS 50W. In addition to its more ductile performance at low temperatures compared with conventional steel, the enhanced fracture toughness of high-performance steel results in less microcracking during the cold forming process, thus improving fatigue performance.

The preliminary design of the bent plate system is based on the current AASHTO *LRFD Bridge Design Specifications,* 4th ed., box-section flexural members section (AASHTO LRFD, Section 6.11). Although reasonable results were achieved, it should be noted that current AASHTO LRFD specifications do not specifically cover cold-formed steel members.

The conceptual design was performed for a simply supported bridge with a span length of 60 ft. The box girder was dimensioned to carry the factored weight of components, the wearing surface, and the HL93 design live load; including a 33% dynamic load allowance for the Strength I limit state. The factored loads that were used in the evaluation are given in Table 3.1.

To perform a parametric analysis, the available sheet plate width was taken as 10 ft. This also corresponds to the maximum width that can be bent at the shop. Since the deck thickness and width were known, by keeping the top flange of the box girder constant at 2 in. by 6 in., it was possible to directly calculate the exact location of the plastic neutral axis for different values of the box girder depth (d) and the inclination angle. Trial analyses were performed for three different depth values: 24 in., 36 in., and 48 in. The lower and upper boundaries for the inclination angle were chosen as 30° and 90°, respectively. The conceptual cross section is shown in Figure 3.12.

Results from the trial analyses are summarized in Table 3.2.

The relationship between the ratio of flexural capacity to flexural demand and the inclination angle is also shown in Figure 3.13.

Table 3.1. Load Effects on 60-Ft Trapezoidal Box

	DC	DW	LL	1.25DC + 1.50DW + 1.75LL
Shear (kips)	30.3	3.9	100.0	219
Moment (kips-ft)	454.5	58.5	1,356.0	3,029

Note: DC = dead load of structural components and nonstructural attachments; DW = dead load of wearing surfaces and utilities; and LL = vehicular live load.

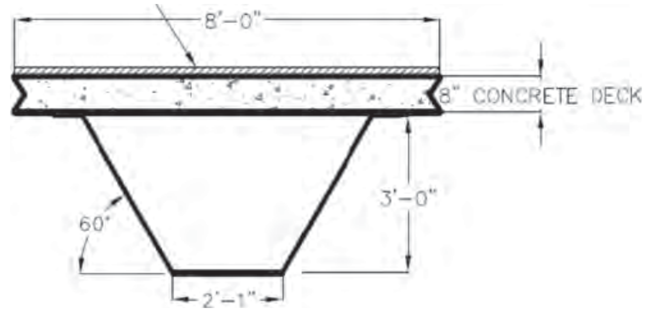


Figure 3.12. Decked steel bent plate box section—conceptual design.

Trial analyses showed that the ultimate capacity of the system is always governed by flexure rather than shear. However, even with a depth of 24 in. and a relatively shallow inclination angle of 30°, the bent plate box girder system is capable of carrying the factored dead and live loads. Depending on the deck geometry and the material availability (the maximum width of the sheet plate that can be bent at the shop is 10 ft), it is possible to improve the flexural performance by increasing the depth of the box girder, although an increase in depth from 24 in. to 36 in. resulted in a greater gain in strength, when compared to an increase in depth from 36 in. to 48 in. In addition, the rate of strength gain also seems to diminish at higher inclination angles.

Table 3.2. Parametric Analysis Results

d (in.)	Inclination Angle (degree)	Capacity-to-Demand Ratio
24	30.00	1.14
	40.00	1.30
	50.00	1.39
	60.00	1.45
	70.00	1.48
	80.00	1.50
	90.00	1.51
36	50.00	1.69
	60.00	1.82
	70.00	1.90
	80.00	1.94
	90.00	1.95
48	70.00	2.10
	80.00	2.17
	90.00	2.20

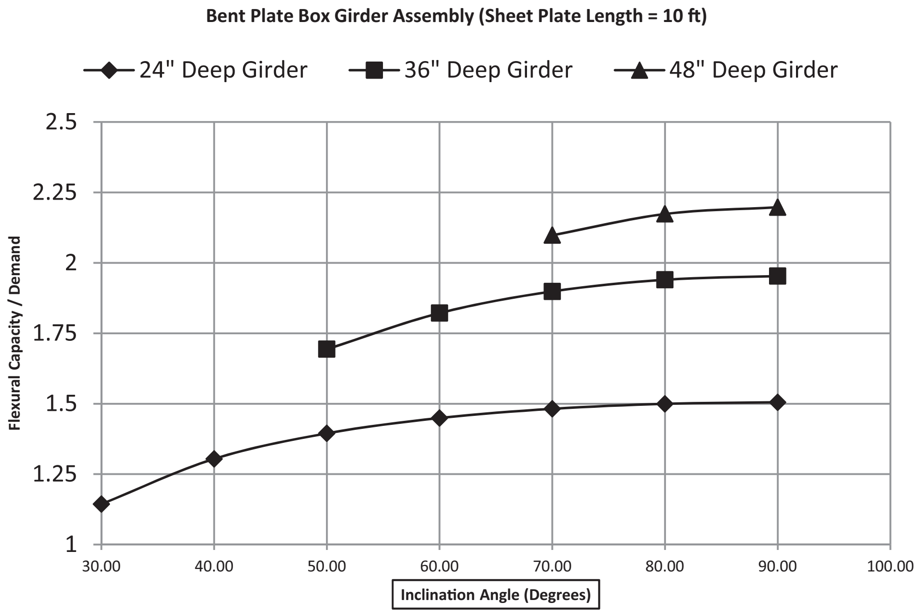


Figure 3.13. Capacity-to-demand ratio vs. inclination angle.

AASHTO LRFD Specifications also include limits for the cross-sectional proportions, most of which are based on the thickness of the steel plate. Since the steel plate thickness is fixed at 0.5 in., it is possible to calculate upper-bound values for the depth of the box girder, and also for the upper flange width:

- Per AASHTO LRFD 6.11.2.1, maximum depth for the box girder is 75 in.
- Per AASHTO LRFD 6.11.2.2, maximum width for a single upper flange is 12 in.

Although the specifications require the thickness of the flanges to be at least 10% more than the web thickness (AASHTO LRFD, 6.10.2.2–3), it is not possible to satisfy this clause, since the box girder has an overall uniform thickness.

Per AASHTO LRFD, 6.11.2.1.1, the inclination of the web plates to a plane normal to the bottom flange should not exceed 1 to 4. This requirement is met in the conceptual design trials.

Concrete Deck Bulb Tees and Double Tees with Integral Deck

These superstructure systems have been used by various DOTs including those of Washington State, Idaho, and Utah. These states have developed standards for these systems, which is indicative of the use in each state. The Utah DOT has developed the *Precast Bulb Tee Girder Manual.* The purpose of this manual is to provide guidance with the design and detailing of precast prestressed concrete bulb tee girders. The manual discusses the design, detailing, fabrication, and handling of precast prestressed girder bridges. The girders may be pretensioned or posttensioned. Three families of bulb tee girders have been developed by the Utah DOT. The girders are based on the Washington State DOT WF Series girders. The girder depths range from 42 in. to 98 in. in 8-in. increments.

The Washington State DOT deck bulb tees have widths up to 6 ft and 6 in. minimum thickness. The Washington State DOT standard details cover square ends and skewed ends, up to a maximum 30° skew. Welded and grouted joint details are shown between the units and an asphalt overlay is required. The typical unit widths for the double tee units vary between 8 ft and 11 ft. The Washington State DOT and the Precast/Prestressed Concrete Institute Northeast have developed standards for these types of beams. As indicated in these standards, double tee with integral deck units are currently feasible for spans up to approximately 90 ft using concrete strengths of 10 ksi.

This modular superstructure concept involves deck bulb tee beams and double tees with a welded plate connection between the deck units for bridges subject to light-to-moderate traffic, and with full moment deck closure pours for bridges subject to heavy truck traffic. Both options are acceptable from an engineering viewpoint; however, the welded plate connection option should be limited to bridges that carry light truck traffic and are located in the regions of low seismicity, where seismic design of bridges is not required. The option with full moment deck closure is recommended for bridges that are expected to carry moderate-to-heavy truck traffic and are located in zones that require bridges to be designed for seismic design loading. The option with full moment connection is market ready, but the option with welded plate connection required further research, testing, and code development. Headed reinforcing bars, as indicated in NEXT D Beams Standards by the Precast/Prestressed Concrete Institute Northeast, also could be used with closure pours. However, this type of connection requires further testing and the design requirements for this type of connection need to be codified.

The concept that uses welded plate connections between adjacent deck bulb tee units requires further research and testing for suitability and the required minimum capacity of the connection. Because this type of a connection is a shear-only connection and does not provide moment transfer between adjacent units, the ability of one beam to transfer its load to the adjacent beam is limited, thus reducing superstructure redundancy, especially when one beam is damaged or deteriorated. With this type of connection between the adjacent units, as the beam loaded under traffic would have limited ability to transfer the load to adjacent beam, AASHTO live-load distribution factors would not be applicable because the deck is not fully continuous in the bridge cross section. The welded plate connection also reduces the redundancy of the structure if a beam failure occurs and does not provide an efficient alternate load path to transfer the force to adjacent beams or an adjacent portion of the deck. Deck slabs on each beam would behave as cantilever beams and would need to be designed as such for the applicable design loadings. The weight of the traffic barrier would also not get distributed to multiple beams, but would rather be supported primarily by the fascia beam, which could require additional prestressing strands in the beam that supports the barrier. With increasing spans, as the deck tee beam depth increases, the steel channel diaphragms should be replaced with truss-type, steel cross frames. As the load distribution to beams would be significantly different than those used for conventionally constructed bridges, design criteria to be included in the design codes for this type of construction need to be developed based on testing and further research.

Because the deck slab does not transfer moment with the welded plate connection, this option reflects reduced structural capacity under seismic loading compared to conventionally constructed bridges that have a fully continuous deck slab in cross section. Additionally, the welded plate connection with grouting detail is not covered by AASHTO LRFD Specifications for seismic loading and therefore, seismic details and design criteria for the connectivity between adjacent deck tee elements need to be developed, tested under seismic loading, and codified.

When the adjacent deck tee units are connected to provide full deck continuity in cross section, the design is more suitable for bridges subject to moderate and heavy truck traffic, and the design covered by the current AASHTO LRFD Specifications is applicable. Deck continuity in cross section also offers improved load distribution under traffic loading and superimposed dead loads and redundancy. As the deck slab is continuous to transfer moment and shear and axial loads, the structure capacity under seismic loading would be comparable to that of a structure constructed using conventional techniques. With the increase in span length, as the beam depth increases, steel channel diaphragms should be replaced with truss-type, steel cross frames.

Both of these concepts are also suitable for multi-span bridges made continuous for live loads. For deck tees, continuity for live loads could be achieved by splicing the deck longitudinal rebars over the piers and extending the beam prestressing strands into the cast-in-place concrete diaphragms by bending them up 90° into the diaphragms. Continuity diaphragms and the deck slab in the area of continuity diaphragms are placed by using a closure pour. This detail would be similar to that provided in the NEXT D Beam Sample details developed by the Precast/Prestressed Concrete Institute Northeast. Live load continuity reduces the number of deck joints, which in turn reduces initial construction cost and eliminates the maintenance costs associated with these joints.

Overlays for Modular Superstructure Systems

The combination of high-performance concrete and high-quality construction will provide a long service life for these systems. Some owners may, however, have concerns about the long-term durability of bare decks. Use of an asphalt overlay with a membrane could be a desirable option in such situations to provide enhanced durability. In most cases, the overlay can be installed in a day prior to opening the bridge to traffic, or the overlay can be done during night lane closures at a later point. If the bridge is constructed and opened during cold-weather months, the asphalt overlay can be installed when warm weather returns and the asphalt plants open. Asphalt overlay will also provide an improved ride quality. Use of an asphalt overlay may be required in bridge widening and for multiple simple spans, to even out the roadway profiles.

European practice is to always use an asphalt overlay with a membrane as a protective system for bridge decks. Their experience indicates that keeping water away from bridge decks significantly improves service life. The preservation strategy for bridges with an asphalt overlay would be to replace the overlay on an as-needed basis. For bridges without an overlay, a new bonded concrete overlay or topping slab may be added to compensate for any deck deterioration. This project should investigate the substitution of steel rebars in the deck slab/top flange with fiber reinforced polymer (FRP) bars to achieve a longer deck life. The FRP bars may cost two or three times more than steel, but the overall cost impact would not be much. Several FRP reinforced bridges are in service and have performed well.

Connections for Modular Superstructure Systems

The ease and speed of construction of a prefabricated bridge system is paramount to its acceptance as a viable system for rapid renewal. Additionally, as discussed, connections between the modular segments can affect the live-load distribution characteristics, seismic performance of the superstructure system, and also the superstructure redundancy. The designers need to develop a structure type and prefabrication approach that can be executed within the time constraints of the project site and also achieve the desired structural performance. Connections play a critical role in this approach. Connections of the modular units are important elements for accelerated bridge construction, as they determine how easily the elements can be assembled and connected together to form the bridge system. Often the time to develop a structural connection is a function of cure times for grouted connections, and the time it takes to make a welded or bolted steel connection.

The connection details between the modular segments and their load transfer capabilities are critical with respect to several design issues:

- The amount of wheel load transferred from a loaded beam to an adjacent unloaded beam must be considered to determine appropriate design loads for the girders. The amount of load that can be transferred depends on the ability of the joints between the girders to transfer forces, as well as the sectional properties (e.g., torsional stiffness) of individual girders and the system as a whole.
- The development of precast concrete components for bridges located in seismic areas is complicated by increased requirements on structural continuity, increased ductility, and increased development length for the reinforcement. These requirements make the design of connections between the precast components more difficult than the connections used in low and moderate seismic regions. Full moment connections are preferred in seismic areas. Recommendations for joint details are provided in this section.
- Questions can arise about the redundancy of these modular superstructure systems when the connections between the units, such as a grouted joint, do not provide full moment transfer capabilities. This needs to be considered and accounted for in the design and detailing procedures. Full moment connections will provide the same level of redundancy as cast-in-place construction.

Joints are prone to deterioration and are considered the weakest link in any structure, thereby reducing a structure's effectiveness and long-term performance. The number of joints and the type of joint detail is crucial to both the speed of construction and to the overall durability and long-term maintenance of the final structure. The use of cast-in-place concrete closure joints should be kept to a minimum for accelerated construction methods due to placement, finishing, and curing time. Durability of the joint should be achieved through proper design, detailing, joint material selection, and construction procedures.

The following are design considerations for connections between deck segments:

- Achieve durability at least equal to that of the deck.
- Joint designs should consider truck traffic severity to achieve durability.
- Joint details suitable for heavy/moderate/light truck traffic sites.
- Achieve acceptable ride quality (similar to CIP decks).
- Does not require overlays (overlay use is optional).
- Does not require posttensioning.
- Details can accommodate slight differential camber.
- Can be opened to traffic in a matter of hours or days.
- Preferably avoids the need for placement and removal of formwork, requiring access from below.

Connection Details for Prefabricated Bridge Elements and Systems (Culmo, 2009) has introduced three classifications for connection details:

- Level 1: This is the highest classification level. It is assigned to connections that have been used on multiple projects or that have become standard practice by at least one owner agency. Level 1 details are typically practical to build and will perform adequately.
- Level 2: This classification is for details that have been used only once and were found to be practical to build and have performed adequately.
- Level 3: This classification is for details that are either experimental or conceptual. Some Level 3 details have been researched in laboratories, but to the knowledge of the authors, have not been put into practical use on a bridge. Also included in the Level 3 classification are conceptual details that have not been studied in the laboratory, but are thought to be practical and useful.

These standardized designs will use primarily Level 1 details. Level 2 details will be considered only where they are appropriate and can be justified through a critical evaluation.

Joint Types

- Match cast and posttensioned joints are well established and are acceptable alternatives for ABC. Designers can find information on these from other sources.
- Passively reinforced joints (full moment connections suitable for heavy truck traffic sites).
- Welded and grouted joints (shear-only connections suitable for moderate to light truck traffic sites).

Two alternates may be considered for passively reinforced joints for modular construction at heavy truck traffic sites:

- Full moment connection using ultra-high-performance concrete (UHPC) joints.
- Full moment connection using high-performance concrete (HPC) joints.

In addition to these two full moment connections, a welded and grouted joint option is available for modular systems for sites with light truck traffic, such as local roads. These three options are evaluated from the standpoint of rapid renewal requirements and structural behavior and durability considerations.

In precast construction, continuous connections exist when both moment and shear are transferred through the joint. Connections that just transfer shear work as hinges.

For ductility and redundancy, AASHTO LRFD Article 1.3.3 notes that the requirements for ductility are satisfied for a concrete structure when the resistance of a connection is not less than 1.3 times the maximum force effect imposed on the connection by the inelastic action of adjacent components. For nonductile connections the ductility factor shall be at least 1.05. Systems with nonductile connections should be classified as nonredundant. For system redundancy and ductility, this project will recommend the use of continuous connections for ABC as the preferred approach. The hinge-type connection is an available option for low-traffic sites.

Heavy Truck Traffic Sites: Full Moment Connection Using Ultra-High-Performance Concrete (UHPC) Joints

The term "ultra-high-performance concrete" (UHPC) refers to a class of advanced cementitious materials. When implemented in precast construction, these concretes tend to exhibit properties including compressive strength above 21.7 ksi, sustained tensile strength through internal fiber reinforcement, and exceptional durability as compared to conventional concretes (Rosignoli, 1998a). The specific UHPC investigated in this study is a product of a major worldwide construction materials manufacturer and supplier. It is currently the only product of this type that is widely available in the United States in the quantities necessary for large-scale infrastructure applications. European and Asian markets currently have multiple suppliers, and a similar situation will likely occur in the United States as the market for this type of advanced cementitious product develops.

The Composition

During the summer of 2009, the New York State DOT completed two bridge projects using the UHPC closure pour concept. The New York State DOT was interested in full-depth precast deck panels and deck bulb tee prestressed girders for use in constructing and reconstructing bridges. In both bridge types, the precast concrete elements needed to be connected together at the deck level via a permanent, durable connection. This connection is heavily stressed both structurally and environmentally, meaning that the long-term performance of the bridge is dependent on acceptable performance of the connection. Conventional construction practices for connection details can result in reduced long-term connection performance as compared to the joined components. UHPC presents new opportunities for the design of modular component connections due to its exceptional durability, bonding performance, and strength. The properties of UHPC may make it possible to create small-width, full-depth closure pour connections between modular components. These connections may be significantly reduced in size compared with conventional concrete construction practice, and could likely include greatly simplified reinforcement designs.

Use of advanced cementitious composite materials such as UHPC in connection design presents new opportunities to advance the use of modular components with the following advantages:

- Passively reinforced joint only 6 in. long. No posttensioning needed. Figure 3.11 shows UHPC joint detail used by the New York State DOT.
- Full moment connection suitable for heavy truck traffic sites, but can also be used under less-severe traffic situations.
- The placement and curing of UHPC can be performed by using procedures similar to those already established for use with some HPCs. The fluid mix is virtually self-placing and requires no internal vibration.
- UHPC can provide significant durability improvements to bridge decks due to the high strength, extremely low permeability, high resistance to freeze thaw, and improved connection details inherent in the system. Research demonstrates that UHPC exhibited almost no permeability and was not susceptible to chloride ingress.
- In the New York State DOT detail, the shorter development length of reinforcing bar in UHPC allowed a narrower joint, which reduced the total shrinkage. Tests done by the New York State DOT show that a 5-in. development length was sufficient for #6 rebars. This allowed a full moment connection to be made using a 6-in. closure pour and straight rebars. The agency successfully completed a project in 2008 using a UHPC joint.
- In tests made at Michigan Tech Transportation Institute, the UHPC showed compression strength of 28,000 psi, compared with 4,000 psi for normal concrete. Tensile cracking strength was above 1,000 psi, compared with 400 psi for normal concrete. In testing for resistance to road salts and chlorides, UHPC withstood these chemicals at a rate 100 times greater than that of normal concrete.

- The compressive strength gain behavior of UHPC is an important characteristic of the concrete. UHPC does not have any compressive strength for nearly 1 day after casting. Then, once initial set occurs, UHPC rapidly gains strength over the course of the next few days until over 10 ksi of strength is achieved in about 3 days. No special curing is needed for the joint material (though steam curing is beneficial when applied). Regardless of the curing treatment applied, UHPC exhibits significantly enhanced properties compared with standard normal strength and HPCs.
- Level any differential camber between adjacent beams before placing the joint. Slight differences in camber (<¼ in. can be tolerated).
- Installation time of about 3 days includes erecting, placing, closure pours, and curing.
- FHWA is conducting additional testing on UHPC joints. The results will provide improved guidance for design, as discussed in this chapter.
- DUCTAL is the only supplier for UHPC, and the steel fibers are from a European supplier subject to Buy America provisions.

Testing of UHPC Connections

In conjunction with the New York State DOT, researchers at the FHWA Turner-Fairbank Highway Research Center (TFHRC) are investigating whether the exceptional durability, high strengths, and superior bonding characteristics of UHPC lend themselves to the development of a new generation of connection details applicable to modular bridge components. The TFHRC's ongoing research program into the use of UHPC in highway bridges has recently begun focusing on deck-level connections between modular precast components. A physical testing program has been initiated (Transportation Pooled Fund Project TPF-5[217], titled Ultra-High-Performance Concrete Connections Between Precast Bridge Deck) in which subassemblages of full-scale precast bridge deck panels are connected via UHPC closure pours and then cycled under repeated truck wheel loadings. The test program has six specimens, with variables including joint orientation, slab thickness, reinforcement configuration, and reinforcement type. None of the specimens include any pre- or posttensioning. Four of the six test specimens will simulate 8-in.-thick precast deck panels, while the remaining two will simulate 6-in.-thick top flanges on deck bulb tee girders. Cyclic testing has been completed. All specimens performed well through the more than 2 million cycles of 2-to-16-kip loading and the more than 5 million cycles of 2-to-21.3-kip loading. No specimens showed any evidence of leakage along the joint interface. No joint interface debonding was observed. Also, the cracking behavior of the specimens demonstrated that individual structural tensile cracks in conventional concrete were interrupted and replaced by multiple tight-width cracks in UHPC. Static testing will begin next month. The final report is scheduled to be complete by the end of June 2010. The performance of the specimens tested to date has met all benchmarks. Test results to date, along with two New York State DOT bridges constructed in 2009, demonstrate the potential viability of using UHPC as a closure pour material.

UHPC early age behavior and its compressive strength gain behavior are important material characteristics for ABC applications. Results indicated that UHPC does not have any compressive strength for nearly 1 day after casting. In tests, UHPC didn't begin setting for approximately 22 hours. Once initial set occurs, UHPC rapidly gains strength over the course of the next few days until over 10 ksi of strength is achieved 2 days later. At that point, the rate of strength gain decreases, but the strength gain continues until over 18 ksi of compressive strength is achieved by 28 days. Cure time for UHPC connections will limit their suitability for weekend replacement projects.

Heavy Truck Traffic Sites: Full Moment Connection Using High-Performance Concrete (HPC) Joints

HPC denotes high-strength concrete that must have other characteristics specified to ensure durability, including permeability, deicer scaling resistance, freeze-thaw resistance, and abrasion resistance. These characteristics are particularly suited for connections with the following advantages:

- Alternate passively reinforced joint with HPC, no post-tensioning.
- Full moment connection suitable for heavy truck traffic sites, but can be used under less-severe traffic situations.
- The lapping of steel may be achieved with overlapping looped bars or short straight bars whose development is improved by the geometry of the joints or by external means such as confining spirals.
- The greater widths (up to 3 ft) that are typical for these types of joints, relative to UHPC or welded/bolted joints, may increase the likelihood of shrinkage cracking and may require erecting and removing formwork from below.
- The interface between the precast deck and the cast-in-place closure is of particular concern since cracks can develop due to shrinkage. A penetrating sealant should be applied to the top surface of grouted joints after curing to enhance durability.
- Level any differential camber between adjacent beams before placing the joint. Slight differences in camber (<¼ in. can be tolerated).
- Installation time is about 3 days including erecting, placing closure pours, and curing.
- Researchers are investigating the use of a small closure pour with headed reinforcing bars.

Moderate-to-Light Truck Traffic Sites: Shear-Only Connection by Welding and Grouting

Connections that transfer shear only may be adequate for local roads. This detail has the advantage of reduced construction time, lower cost, and easy adaptability to all types of modular systems. Concerns about joint performance and durability have limited their use in ABC applications.

- Use a welded tie connection combined with a grouted key. The 6-in.-long by ½-in.-thick steel ties are normally spaced 5 ft on center along the edge of the beam. They are welded to angles embedded in the beams and anchored with studs.
- This connection is primarily designed as a shear-only connection. There is no intent to make this connection a deck moment connection. Each flange edge needs to be designed as a cantilever deck overhang.
- The Texas DOT researched transverse welded connections for adjacent precast members and found that when combined with a grouted shear key, the connection is sound and durable. The Washington State and Idaho DOTs have also used a welded joint detail for precast members.
- Any differential camber should be leveled before welding. Connection can be made even if there is slight camber differential between the beams.
- Installation time is about 2 days, including erecting, welding, and grouting. Multiple spans can be built in the same time frame with larger construction crews.
- Use of fiber reinforced grouts can enhance joint performance. Some welded joints have not worked too well under certain applications. The Utah DOT has had some issues with leakage. More study of welded joints is recommended.

The states use a welded tie connection combined with a grouted key. The ties are normally spaced 5 ft on center along the edge of the beam. This connection is primarily designed as a shear-only connection. There is no intent to make this connection a deck moment connection; therefore each flange edge needs to be designed as a cantilever deck overhang. Some designers have concerns with the long-term fatigue behavior of the welded tie connections. Therefore, at this time, it is recommended that these welded tie details be used for bridges with lower truck volumes.

Several issues need to be considered when using welded plate connections. First, the bottom portion of the welded plate connector is not protected from the weather on the underside of the connection. If a bridge is to be constructed in a corrosive environment, the designers may want to consider the use of stainless steel plates and rods. Second, it is important that the grout placed in the keys between the beams completely fills the void. The most common keyway failure results from inadequate filling of the keys. If voids are present, the mechanical interlock of the key is lost.

Posttensioning is a well-established and acceptable alternative for ABC for which designers can find information from other sources. This toolkit focuses on innovative materials such as UHPC and advances their use for ABC connections. Use of high-performance lightweight concrete is a viable option to reduce the weight of prefabricated elements and systems.

Segmental Superstructure Systems

Segmental precasting for highway bridges is a well-established design and construction technology that offers many benefits, including:

- Manufactured solution that is highly adaptable to many demands;
- Speed of manufacture;
- Quality control;
- Speed of erection;
- Small segments that are easily handled, shipped and erected; and
- Repetitive erection technology.

This concept includes a combination of both design and construction concepts for ABC. Both the design of greatly simplified segmental superstructures and the erection equipment for smaller spans is addressed.

The adaptation of segmental technology to manufactured small-scale bridges is technically and economically feasible. It will require a paradigm shift on the part of owners and contractors. The net result will be a greatly simplified technology from the perspective of both design and construction. Much simpler segments, much lighter lifts, and greatly simplified equipment and erection technology will result in enhanced production and erection and lower costs. The adaptation of this technology will also result in high-quality, durable, and low-maintenance bridges with a longer life expectancy.

Segmental technology has been thoroughly proven in the United States, and current standards cover most aspects of both the design and construction technology.

AASHTO and American Segmental Bridge Institute (ASBI) have published standards on precast segmental construction. Several publications are also available that further discuss segmental construction in detail. References to these standards and publications can be found in the References.

Channel Sections and Slabs

The engineering underlying the concept of precast segmental bridges with channel sections and solid or voided slabs is well developed. Channel sections are frequently used for channel bridges, railway bridges, and light rail transit (LRT) systems.

In-place voided slabs were commonly used during the early period of prestressed concrete bridge development within the span range of 80 to 115 ft and with span-to-depth ratios of up to 20 to 25.

Adapting the precasting plant to different deck widths would also be simpler and less expensive. Extruding wet concrete along longitudinal reinforcement with a movable form is a typical construction method for voided slabs of buildings. Slab depths up to 2 ft with a total length of about 70 ft are typically achieved; the maximum length of the precast slabs in this case is dictated by transportation requirements. In the case of precast segmental bridges, the extrusion lines may be lengthened to optimal values for the casting process. Rectangular or circular voids would be easily achievable in the slab segments.

If embedded forms with a few large shear keys are used to divide the segments during the extrusion process, after segment separation and removal of the forms, the contact surfaces would require only sandblasting and application of bond enhancer.

The webs and slabs of the cross section may be particularly thin because of the industrial casting process, the reduced surface exposed to the atmosphere, and the smooth exterior surface that avoids gathering water. In addition to pleasant aesthetics, the span-to-depth ratio of voided slab bridges can be particularly low due to a better transverse distribution of live loads and the presence of a wide bottom slab.

The bridge would be composed of two types of segments: the standard voided slab span segments and solid segments at the piers and abutments. The two types of segments may be cast in two separate extrusion lines or alternatively in the same line. External prestressing tendons may be deviated with steel saddles bolted to the internal surface of the cells; the end anchorages may be embedded into the abutment segments.

The longitudinal prestressing tendons would possibly extend the full length of the bridge, anchored on the end faces.

Transverse prestressing and the use of high-performance, lightweight concrete for the precast segments may further increase durability and diminish the weight of segments or increase their length if transported vertically. Transverse pretensioning and posttensioning are both possible, although pretensioning would increase forming costs. The use of unbonded monostrand tendons (also frequent in building construction) would add durability and further lighten the section. Dead anchorages could be alternatively used on the opposite sides of the bridge to diminish the cost of transverse posttensioning.

The following aspects of the construction process require additional research and investigation.

- Treatment of construction joints and reliability of stress transfer;
- Details and technology of concrete stitches between precast segments;
- Optimization of cross-sectional design;
- Optimization of longitudinal and transverse prestressing;
- 3D solid modeling and analysis of stress dispersal;
- Modular support girders (discussed later in this chapter); and
- Incremental launching erection of precast segmental bridges.

Precast Concrete Deck Panels

As noted for modular systems, the intent of this project is to develop pre-engineered standards for ABC systems that offer the greatest potential for future advancements. In fact, there are already a number of standardized precast concrete deck panel systems that have been developed by various state DOTs.

The issues involved with advancing precast concrete deck panels to greater acceptance in the industry lie not in developing the details, but rather taking the details that already exist and have been used with some success and addressing the most critical deficiencies found to date.

Design considerations for standardized precast concrete deck panel systems should include the following:

- Pre-engineered standards for precast deck panels should address the most common bridge sites without considering site-specific geometry, and so forth.
- Advanced materials, including high-performance concrete and UHPC, when cost-effective.
- Optimize designs for ABC and use of high-performance materials.
- Designs should be established for a range of common bridges widths (36, 40, and 44 ft) and girder spacings (8 to 11 ft).
- Designs should prioritize panels that span the entire roadway width and can thus be installed without a centerline joint.
- Designs should simplify posttensioning (PT) details and should include only longitudinal PT.
- Eliminate skewed and curved bridges from consideration for standard precast deck systems.
- Lightweight concrete should be evaluated for use in posttensioned systems to ensure that creep behavior is acceptable.

Challenges to Broader Use of Precast Deck Panel Systems

Although precast concrete deck panels have been used for more than 40 years, a number of challenges to the wider use of this technology remain. The most critical challenges are presented in the following paragraphs.

Bridge Deck Joints. The transverse panel joint is a fundamental part of virtually all precast bridge deck systems. However, longitudinal panel joints are typically used only for certain projects, such as those in which the bridge is so wide that the individual panels are simply too large to be economically transported to the site and installed using moderately sized equipment, or those that are constructed in stages and require the shifting of traffic from one lane to another during replacement of an existing deck. In many heavily traveled bridge replacement scenarios, a structure is closed during overnight or weekend hours for bridge deck replacement and then reopened to traffic in the morning. Essentially, if a single panel can be used to cover the entire width of the bridge, it is strongly recommended that the longitudinal joint be avoided.

A bridge deck is subjected to considerable exposure to deicing salts and thus any cracks that permit the intrusion of chloride-laden water will cause the bridge deck accelerated deterioration. When compared to a cast-in-place bridge deck, which is often placed continuously from end to end of a bridge, a precast concrete deck panel system by its very nature provides a multitude of opportunities for water intrusion.

There are several components of a successful precast concrete deck panel system, including

- A smooth riding surface and effective load transfer between panels;
- Effective filler material for panel joints; and
- A durable, reliable posttensioning system.

It should be noted that a precast concrete bridge deck that provides a smooth wearing surface is desirable not simply as a comfortable ride for the traveling public. In fact, one of the fundamental causes of deterioration of a precast concrete bridge deck occurs if adjacent panels do not provide a completely flush-fitting and effective load transfer system at a transverse joint. Repeated wheel loads passing across these joints, and amplified by the dynamic impact effect, will eventually cause one or more reflective cracks to develop either immediately at the bond line between a panel and the interstitial grout or in the panel concrete itself.

These reflective cracks permit the ingress of water, and the cyclic freeze-thaw effect will eventually widen these cracks. This self-propagating crack development pattern continues and continual maintenance of the deck will be required to slow the deterioration of concrete panels.

Another component of successful joint performance is the application of the appropriate filler material. In most cases, a cementitious grout product is used; however, there has been limited use of neoprene materials to date. Ongoing research has been working toward the improvement of these joints by evaluating different shape joints and filler material. As yet, only limited success has been achieved, and considerable work remains.

A variety of systems that use posttensioning in the longitudinal direction have been developed and tested by bridge owners. These systems are designed to provide a crack-free, durable deck that remains in compression throughout its life cycle.

Posttensioned Connections Between Panels. Posttensioning is a well-established and acceptable alternative for ABC for which designers can find information from other sources.

Suggested AASHTO LRFD Code Improvements. In Phase III of the ongoing project, the research team will develop and propose AASHTO–formatted design specifications to assist owners and designers with wider implementation of accelerated construction.

One specific issue that could significantly improve the climate for ABC using precast deck panels is related to maximum stud spacing requirements.

National bridge design standards, including both the AASHTO standard and AASHTO LRFD Specifications, provide a requirement for maximum stud spacing of 24 in. for steel girders. These shear studs are placed in clusters to accommodate shear pockets in the precast panels. In case of concrete beams, most bridge owners prefer that the stirrups placed for vertical shear are extended into the bridge deck to provide composite action. As mentioned before, it is very cumbersome to clear concrete around the shear studs or shear reinforcement. It is recommended that the AASHTO provisions for shear connector spacing be evaluated for suitability with accelerated construction. At least in the case of prestressed concrete beams, the roughed surface on the tops of the beam flange transfers substantial horizontal shear though friction alone. It is evident from several studies that this shear friction theory can be used to satisfy the horizontal shear requirement, which would allow the precast deck panel and beam to contribute to the composite action.

AASHTO currently requires that these stud clusters and their corresponding panel blockouts be provided at a maximum 24-in. spacing. This requirement frequently leads to congestion and significant discontinuity of the mild reinforcing in the panels. An effort is under way through the PCI Bridge Committee, and supported by ongoing and future research into the 24-in. requirement, to modify this requirement to allow shear pockets at a 48-in. maximum spacing.

If this amendment is incorporated into AASHTO LRFD, the fabrication and placement of precast deck panels will be much simpler and thus less expensive. In addition, the future removal of precast deck panels will be considerably improved; approximately half of the shear studs to be worked around would be eliminated during the removal of each panel for bridge widening or replacement.

AASHTO Chapter 9, which deals with precast concrete deck panels, should be carefully revisited. The research team envisions code language that is more definitive and less open to varying interpretations than the current code. The following recommendations should be considered for inclusion in future interim specifications:

- Guidelines for stress requirement on the panels for stage construction or full construction. It is anticipated that stress requirement for stage construction is higher than that for full construction.
- Example detailed content to address both transverse and longitudinal joints.
- Guidelines for the use of reinforced concrete or prestressed concrete for a variety of span configurations and beam/girder spacings.
- Reconsideration of resistance phi factors to account for the higher level of quality control available in the closely monitored fabrication environment when compared with cast-in-place concrete design.

As noted previously, the recent NCHRP 12-65 study developed an extensive list of proposed AASHTO LRFD Section 9 modifications. These recommendations were presented to the appropriate AASHTO committee. Currently, there are ongoing discussions among the AASHTO committee members as to whether to provide the proposed modifications in the AASHTO construction specifications or AASHTO LRFD Bridge Design Specifications.

Members of the current R04 research team believe that the inclusion of the proposed modifications in AASHTO LRFD Specifications are valid and should be strongly considered for inclusion in the AASHTO code. A more detailed evaluation of these code provisions will be made during Phase III of the project.

Constructability Evaluation

Constructability evaluation is aimed at assessing issues specific to transportation of components, erection methods, equipment needs, and the suitability of the system to rapid construction. ABC Designs need to be optimized to meet the transportation and erection requirements for prefabricated construction.

Modular Superstructure Systems

Constructability Issues and Considerations

- Usually length ≤140 ft, weight ≤100 tons, width ≤8 ft for transportation and erection using conventional construction equipment.
- Design for sections that can be transported and erected in one piece, for lengths up to 140 ft, may be feasible in certain cases. Provide one method of erection. (Spans longer than 140 ft may be erected by shipping the segments in pieces, splicing on site, and using a temporary launching truss for erection.)
- Segments designed for transportation and erection stresses, including lifting inserts. Sweep of longer beams should not be an issue for erection because there is an opening between the beams.
- Able to accommodate moderate skews. For rapid renewal, it would be more beneficial to eliminate skews altogether by making the bridge spans slightly longer and square.
- Provide standard details for durable connections between deck segments that can also be rapidly constructed.
- Segments that can be installed without the need for cross frames or diaphragms between adjacent segments. Improves speed of construction and reduces costs. Use of diaphragms is optional and based on owner preference.
- Deck segments when connected in the field should provide acceptable ride quality without the need for an overlay. Deck segments to have ¼-in. concrete overfill that can be diamond ground in the field to obtain desired surface profile.
- Control of camber for longer spans will be important for modular superstructures. Control fabrication of concrete sections, time to erection and curing procedures so that camber differences between adjacent deck sections are minimized. Leveling procedure to be specified to equalize cambers in the field during erection.
- Edge sections of deck with curb piece ready to allow bolting of precast barriers.

Transportation and Erection Issues

Transportation and erection (crane capacity) limitations could present challenges that need to be considered in the standardization process. Every state has requirements for shipping of oversize and overweight loads that can limit the permissible size of the elements. These limitations could influence the maximum span lengths of the new bridge systems suitable for ground transportation. The width that can be transported without special permit is generally limited to 8 ft. Optimizing the weight of these bridge systems through design or the use of new lighter and durable materials will allow the transportation and erection of larger and longer bridges. High-performance materials that are lightweight and durable are most suited for prefabrication in large sizes.

Camber and Riding Surface Issues

One of the greatest construction difficulties is eliminating the differential camber between the girders. It is important to develop an adequate means of removing the differential camber between the girders on site. Differential camber in prefabricated elements could lead to fit-up problems and riding surface issues. If the differential camber is excessive, the contractors in some states will apply dead load to the high beam to bring it within the connection tolerance.

To the traveling public, the smoothness of the riding surface is a significant riding comfort issue. This is also an important factor for durability and maintenance, as vibrations from an irregular surface can affect the structural steel components of a bridge. Due to irregularities in the riding surface that can occur at longitudinal and transverse joint locations between modular components, it may be necessary to plane the deck surface through diamond grinding.

LRFD Article 2.5.2.4, Rideability, requires that

> [t]he deck of the bridge shall be designed to permit the smooth movement of traffic. . . . Construction tolerances, with regard to the profile of the finished deck, shall be indicated on the plans or in the specifications or special provisions.
>
> The number of deck joints shall be kept to a practical minimum. . . . Where concrete decks without an initial overlay are used, consideration should be given to providing an additional thickness of 0.5 in. to permit correction of the deck profile by grinding, and to compensate for thickness loss due to abrasion.

Achieving Ride Quality with Prefabricated Superstructure Segments

- While the application of an overlay helps overcome finite geometric tolerances, it also requires another significant critical path activity prior to opening a structure to traffic.
- Today's availability of low-permeability concretes and corrosion-resistant reinforcing steels allows owners to forgo the use of overlays on bridge decks.
- With prefabricated superstructure construction, the challenge is to develop methods that achieve the final ride surface without the use of overlays. Control of cambers during fabrication and equalizing cambers or leveling in the field is intended to achieve the required ride quality.
- An attractive option is diamond grinding decks with sacrificial cover to obtain the desired surface profile. Such a method can be faster and more cost-effective.
- For continuous or multiple simple spans, beam cambers may affect ride quality to a point where an asphalt overlay system may be recommended (see the discussion below).

Control of Camber During Fabrication and Equalizing Cambers in the Field

- Differential camber of beams can lead to dimensional problems with connections.
- Schedule fabrication so that camber differences between adjacent deck sections are minimized. Measure camber on each deck section immediately after transfer of prestress forces. (The Washington State DOT requires that at transfer of prestress, the difference in camber between adjacent deck sections of the same design must not exceed ¼ in. per 10 ft of span length or a maximum difference of ¾ in., whichever is less.)
- Equip all deck sections with leveling inserts for field adjustment or equalizing of differential camber. The inserts with threaded ferrules are cast in the deck, centered over the beam's web. The Washington State DOT specifies a minimum tension capacity of 5,500 lb for the inserts. After all adjustments are complete and the deck sections are in their final position, fill all leveling insert holes with a nonshrink epoxy grout.
- The welded joint details can accommodate minor differential camber. If the differential camber is excessive, the contractors in some states will apply a dead load to the high beam to bring it within the connection tolerance. A leveling beam also can be used to equalize camber.
- Have available a leveling beam and suitable jacking assemblies for attachment to the leveling inserts of adjacent beams. Adjust the deck sections to the tolerances required. More than one leveling beam may be necessary.
- If the prescribed adjustment tolerance between deck sections cannot be attained by use of the approved leveling system, shimming the bearings of the deck sections may be necessary.
- See Figure 2.21 in the previous chapter for an image depicting a New York State DOT bridge leveling procedure for adjacent beams.

Assembly Plans

It is common for designers to require the submission of erection plans for conventional construction projects. This is normally limited to the erection of beams and girders. Bridges built with prefabricated elements require special erection and assembly procedures due to the larger number of elements that need to be erected. The New Hampshire DOT required the contractor to submit an assembly plan for its first fully prefabricated bridge project. The assembly plan is similar to an erection plan; however, it also includes information such as grouting and grout curing procedures, timing and sequence of construction, and temporary shoring of substructure elements during each phase of construction. It is recommended that projects built with prefabricated elements contain specifications requiring the submission of a detailed assembly plan.

Deck Bulb Tees and Double Tees with Integral Deck

The girders may be pretensioned or posttensioned. Posttensioned girders are often used for long spans in which shipping limitations preclude the use of pretensioned girders. Posttensioned girders are often cast in two or more pieces that are connected in the field by splicing. Posttensioning can also be used to simplify girder shipping. Spliced girder technology can be used to create multi-span bridges. The girders are spliced with reinforced concrete closure pours.

Accurate predictions of the deflections and camber are difficult to determine since modulus of elasticity of concrete (E_C), varies with stress and the age of concrete. The effects of creep on deflections are difficult to estimate. An accuracy of 10% to 20% is often sufficient. Leveling of beams to offset camber differences can be carried out in the field, as discussed in this report.

The durability of grout also should be tested, as the loss of grout would accelerate corrosion of the welded plate connections and could result in a potential loss of connectivity between the beams. The loss of grout and connectivity would also accelerate the deterioration of beams and the substructure units. The current AASHTO LRFD Specifications do not address the welded connectivity between the adjacent deck tee beam units and this type of construction. Construction specifications for this type of construction, including tolerances, should be separately developed. This type of construction is suitable for bridges with no skews or small (10° to 15° AASHTO-permitted) skews. For larger skews, this type of construction may lead to fit-up issues with welded connection plates due to differential deflections between beams. Leveling beams could be used to achieve the intended top of deck elevations and to improve the connectivity of the welded plates. The beam cambers must be strictly maintained in the fabrication shop to limit the differential between the theoretical camber and actual camber to approximately ¼ in. This type of construction is suitable for bridges with constant cross slopes and on tangent alignments. It is not suitable for bridges on a curved alignment, on significant skews, or for bridges supporting a flared roadway. The grout and closure pour concrete properties should be carefully selected to avoid cracking and provide durability.

It is estimated that the addition of concrete diaphragms for live-load continuity would add approximately 3 days to construction time. Given several weeks of construction duration using ABC, this additional time is acceptable.

Precast deck tee sections with weights up to 100 tons and widths up to 8 ft could be transported using conventional equipment and erected using conventional cranes. With the use of precast sections, the superstructure construction time could be reduced to weeks or days depending on project needs.

Lifting locations for deck tees should be located such that cracking during transportation is minimized. Criteria for lifting locations and details are not presently addressed in the AASHTO LRFD construction specifications and need to be developed.

Segmental Superstructure Systems

Bridge length, the span length, and the nature of the obstruction to overpass significantly influence the construction method. Most bridges for ABC applications will be short or medium-length bridges. Erection methods commonly used for long viaducts require some adaptation and simplification.

Simple erection girders and light crane picks are the preferred technology for erection of short and medium-length precast segmental bridges with solid or voided slabs or with channel sections. If the access below for cranes is limited, or the traffic effects are significant, the segments can be erected from either end with light cranes and simply rolled into place on erection girders.

This technology can be implemented by smaller local contractors on a few overpass locations, or even more economically, on a larger number of ABC sites. Manufacture and erection of segments are processes that can be self-performed by small to intermediate-size contractors when the technology is simple and repetitive.

Precast segmental highway bridges cover span lengths ranging from 120 ft to 160 ft at the lower limit and 390 ft to 460 ft at the upper limit. The longest precast balanced cantilevers constructed in Europe are now in the 600- to 700-ft range. Below 120 ft, the use of precast girders and in-place deck slabs is generally more economical, although the quality of in-place deck slabs is lower than that of precast segments and the construction duration is generally longer. Achieving longitudinal continuity is also complicated and time-consuming.

For ABC applications, segment length and weight need to be manageable for available cranes, weight limits, and the typical height and width of handling and transportation requirements. Lengths of up to 12 ft are often transportable on public roads without excessive restrictions.

Application of ABC techniques to modular short-or medium-span precast segmental bridges may pose new technological challenges in relation to the bridge length. When the bridge length permits amortization of the investments and of the mobilization and demobilization costs of a launching gantry, the viaduct can be rapidly built with an under-slung or overhead gantry without new technological challenges. When the bridge is just a few spans, however, simpler erection means should be used to avoid the costs of an erection gantry and to simplify and accelerate mobilization and demobilization of the erection site.

The simplest erection method for a precast segmental bridge is supporting the spans with ground falsework. In addition to the high labor cost and the long construction duration, however, the area under the bridge must be accessible for the entire bridge length, which is often incompatible with crossing highways, railroads, rivers, and environmentally sensitive sites.

Support girders may be used to support the segments and diminish the impact of construction on the area under the bridge. The support girders would be simplified erection gantries, with some of the standard features of the latter removed to diminish the investment and the mobilization and demobilization costs. A support girder for short bridges would have the following:

- Simplified self-launching capability. The girder can be pulled along support rollers with a small winch anchored to the abutment. The truck used for transportation of the support girder may also be equipped with a special winch.
- Overhead or under-slung configuration. Both configurations should be compatible with channel bridge sections and solid or voided slabs. In the under-slung configuration, the support girder may pose clearance problems when overpassing railroads or highways. This may require lifting the vertical profile of the bridge to avoid conflicts or erecting the span in a raised configuration and lowering it onto the bearings when the support girder is removed.
- Modular composition to fit different span lengths.

For the scale of projects targeted by this research for ABC implementation, it is expected that the size, complexity, and cost of erection equipment, scaled down as it is for the size of the project, will not present a large initial cost, or a disincentive to cost-effective and competitive solutions. In fact, it may represent a reduction in cost due to the small size and simplicity of the segments and corresponding equipment necessary for erection.

As an alternative, a precast segmental bridge with channel sections or a voided slab bridge can be assembled behind an abutment and positioned onto the piers by incremental launching. This construction method would offer many advantages, including the following:

- Safety for traffic: no work adjacent to traffic, no erection equipment between the piers, no construction clearances for support girders, no drop of materials;
- No detours of traffic when overpassing highways;
- No speed limitation on vehicles and trains;
- Safety for workers: bridge built entirely on the ground;
- High quality and easy inspection: bridge erected behind the abutment;
- Context-sensitive solution: minimal disturbance to environmentally sensitive sites;
- Easy crossing of rivers: only interference is pier erection within tight work windows, no reduction of the hydraulic section, no consequences from floods;
- Compatible with hard-to-access sites: rivers, channels, wetlands, highways, railroads, deep valleys, steep slopes, piers of any height;
- Compatible with transverse shifting and ABC replacement of bridges in service;
- Small erection yard with no additional right-of-way;
- Low labor costs: labor used entirely on production, minimized access problems, minimized crew transportation, minimized use of ground cranes;
- Industrialization of the erection process easily adaptable to bridge length; and
- Inexpensive and adaptable erection equipment with rapid mobilization and demobilization.

Incremental launching construction of prestressed concrete bridges has seen hundreds of applications worldwide. Its application to short and medium-length precast segmental bridges with channel sections or voided slabs would be an innovative evolution of a time-tested construction method. The risks of innovation would be mitigated by the positive 50-year history of the launch techniques and the absence of serious accidents. Ample research is available on construction of precast segmental bridges. The American Segmental Bridge Institute specifically addresses this type of construction. Books and manuals are also available.

Joints

In modern segmental construction, segments are typically match cast in a precasting plant and glued on site with epoxy joints. Dry joint technology is not applicable to ABC construction in which slabs and channels might be considered, as external posttensioning is not feasible.

With epoxy joints, the segments to be erected are first guided into position to ensure that no damage occurs to the concrete. The segment is offered up to the previously erected segment on a dry run, with the joint prestressing bars already in place to ensure that everything fits together and matches. The segment is then moved back and the epoxy applied to the joint surface, after which the segment is pulled into position and the joint prestress fully installed.

The temporary joint prestress normally consists of bars. These are quick to install and hold the segments in place until the permanent prestressing tendons are installed and tensioned. To keep the epoxy thickness uniform over the joint, the temporary prestress is applied with an average compressive stress of 30 to 45 psi.

The joint prestressing bars are either internal or are arranged outside the concrete. Internal bars are usually left in place and grouted to become part of the permanent prestress for the deck. External bars are positioned above the two slabs and anchored on temporary steel or concrete blocks stressed down to the segment. Permanent anchor blisters within the box cell are often used to avoid holes in the slabs and to save labor during erection. The advantage of using external bars is that they are easy to destress and reuse.

When the segments are erected, they join up to the adjacent segments with the same horizontal and vertical angle deviation that existed when they were match cast against each other. This is facilitated by the presence of shear keys on the webs and alignment keys on the slabs, which guide the segment into position.

Small inaccuracies in individual segments or misalignment across joints accumulate when long sections of deck are constructed, and construction and surveying tolerances may require corrections to the segment alignment during erection. In superstructures erected by balanced cantilever assembly, small misalignments are corrected within the cast-in-place joints at mid-span. With the span-by-span method and continuous span structures, misalignments are corrected with short mortar or concrete stitches typically located at the joints of the pier diaphragms.

Simple spans do not pose any problem for small geometric discrepancies. When it is necessary to adjust the alignment of segments during erection, fiber or polyethylene (PE) shims are inserted into the joints to increase the thickness of the epoxy joint along one edge. Although this technique achieves only a small adjustment to the deviation at every joint, the effect is magnified as subsequent segments are built on.

Segmental Construction for ABC

Using precast segmental construction technology with solid or voided slabs and channel sections for short and medium-length bridges for ABC applications should not pose major technical challenges. However, several aspects of standard precast segmental construction should be revised in relation to the specific requirements of these types of bridges and cross sections. The relatively short length of several bridges for ABC applications, in particular, suggests the use of specialized joints between segments to facilitate amortization of the investment associated with precasting facilities.

For ABC construction, the development of standard, simple sections that can be mass produced in relatively small and inexpensive fabricating plants should be readily achievable. Construction of solid slab, or even voided slab, segments that are posttensioned is even less costly than the construction of long casting beds with anchor bulkheads and deviation anchors for typical prestressed girder fabrication. The use of external tendons avoids problems of durability from leaking joints. However, external tendons are hardly compatible with solid slabs and channel bridge sections because both types of cross sections are devoid of cells for cable containment.

The costs of these precast segmental solutions would be governed by the amortization of investments (i.e., by the entity of the investment and by the number of elements to be built in the precasting facility). In turn, the entity of the investment would be governed by the construction deadlines and the optimal level of industrialization of the casting process. Segmental precasting typically requires a precasting or fabrication facility, transportation, and erection equipment. The following subsections present the requirements in more detail.

Figure 3.14. Long-line match casting of a solid slab.

Precasting or Fabrication Facility

This includes storage areas for loose materials, cage prefabrication templates, casting cells for short- or long-line match casting of segments, gantry cranes and straddle carriers for handling of segments, coverings of the working areas, and storage areas for segments. Production requirements and the time available for bridge construction dictate the number of casting cells and the dimensions of the storage area.

Figure 3.14 shows a long-line match cast segmental setup for solid deck slabs. A very limited amount of special equipment and technology is required. The process involves the casting of every other segment, the removal of the bulkhead forms, and the infill match-casting of the intermediate segments.

Transportation

The cost of transportation depends on the weight of segments. The number of transportation units depends on the bridge construction schedule and the distance between the precasting plant and the erection site. In the case of long distances, storing segments close to the erection site may diminish the number of transportation units but requires double handling of segments. Figure 3.15 shows the handling and transportation of a segmental solid slab.

Erection Equipment

Three different techniques are typically used for erecting a precast segmental bridge: the span-by-span assembly, the balanced cantilever assembly, and the progressive placement of segments with the help of temporary stays or props. An

Figure 3.15. Segmental solid slab handling and transportation.

example illustrating the erection of a segmental solid slab is shown in Figure 3.16.

With the span-by-span method, all the segments for a span are positioned before the prestressing tendons are installed and the complete span is lowered onto the bearings. This method is used for both simply supported spans and continuous superstructures. The adjacent spans of continuous bridges are joined together with concrete stitches to avoid propagation of the geometry tolerances of segmental precasting. After the stitch concrete has hardened, prestressing tendons are tensioned to make the deck continuous.

With span-by-span erection and epoxy joints, a typical 130-ft span is usually erected every 2 or 3 days. With an under-slung gantry and dry joints, an erection rate of up to a span a day is achievable. Overhead or under-slung gantries support a complete span of segments during erection; after application of longitudinal prestressing, the gantry releases the span onto the bearings and launches itself forward to erect the next span. Span-by-span erection with launching gantries is typically used for long bridges and spans shorter than 160 ft; for longer spans it tends to be more expensive than balanced cantilever erection because the erection gantries become very heavy. The cost of launching gantries, including investment and mobilization and demobilization, requires long viaducts for amortization; when the area under the bridge is accessible, therefore, short precast segmental bridges are typically erected onto ground falsework.

Figure 3.16. Segmental solid slab erection.

The balanced cantilever method involves erecting the segments as a pair of cantilevers about each pier; the pairs of opposite segments are prestressed with tendons that cross the entire hammer. This method is primarily suited to long spans; long viaducts with shorter spans are better processed using the span-by-span method. Segments can be positioned with a launching gantry or a lifting frame supported by the deck itself. Standard cranes can sometimes work on the deck although this causes significant load unbalance on the piers. Ground cranes are used only when the area under the bridge is accessible and the piers are short.

With the progressive placement, a lifting frame or ground crane raises and places the segments in one direction from the starting point, passing over the piers in the process. Progressive placement is usually the most time-consuming erection technique because of the single work location; however, the specialty equipment can be particularly inexpensive, especially when ground cranes can erect the segments along the entire length of the bridge.

Only one bibliographic reference has been found on the incremental launching erection of precast segmental bridges. The reason may be that the high investments of segmental precasting require long viaducts for amortization while the incremental launching construction is typically addressed to shorter bridges. ABC applications of precast segmental construction would be addressed to numerous short or medium-length bridges rather than a long viaduct, so the financial break-even point would be different. A combination of precast segmental construction and incremental launching erection might merge the advantages of two construction methods that have amply demonstrated their capabilities in the respective typical fields of application.

The precast segments are typically fabricated in a precasting facility by short-line match casting. With this technique, the new segment is moved to the end of the casting cell and the next segment is match cast against it. The geometric relationship between the two segments is achieved by rotating the front segment in 3D space before match casting the new segment. This permits the application of cambers and plan and vertical curvatures.

The segments can also be fabricated close to the bridge; segment transportation is thus minimized but on-site processing of loose materials is necessary. Long-line match casting is often used in this case because of the less stringent geometry tolerances. With long-line match casting, two foundation beams support the entire segmental span during construction. On completion of construction, the span is dismantled and reassembled at the final erection site with any of the above construction methods.

Precast segmental construction of box girder bridges is a well-established technique that does not require additional research within the purposes of this study.

Precast Concrete Deck Panels

Many of the challenges of constructing a bridge deck using precast concrete panels are not unlike those described for the other modular superstructure systems presented earlier in this section. However, the flexibility to use precast concrete deck panels (PCDP) for both new construction and deck replacement projects offers some unique challenges as well. These issues will be briefly discussed in the following paragraphs.

Constructability Issues and Considerations

- For a typical bridge, the panel is usually ≤50 ft in length, ≤50 tons in weight, and ≤8 ft wide for transportation and erection with conventional construction equipment.
- Panels typically span entire bridge width with transverse joints.
- Panels will be transported and erected in one piece. A piece typically weighs less than other controlling items on the project.
- Panels, including lifting inserts, must be designed to withstand both transportation and erection stresses.
- Panel flatness must be considered. Proper handling and storage of panels prior to arrival on site is critical.
- Precast panels for skewed bridges are problematic. Rectangular panels are typically much easier to form in a production environment. Small skews (less than 10°) may be accommodated in precasting.
- Large skews may require an additional step in construction to cast a closure pour at the end of the bridge. For accelerated construction, it would be more beneficial to eliminate skews altogether by making the bridge spans slightly longer and square.
- Provide standard details for durable connections between deck panel segments that can also be rapidly constructed.
- Deck panels absolutely must provide a smooth riding surface. A joint that is not flush will be subjected to significant impact loads and will suffer premature deterioration.
- Deck panels are often designed with an additional ¼-to-½-in. cover that can be ground after installation to provide smooth ride.
- Edge sections of deck with curb pieces must be fabricated for bolting or posttensioning of precast barriers.
- Panels are typically cast flat to simplify casting and installation in the field.
- Shear connectors must be installed to coordinate with pockets cast into the deck panels.

Transportation Issues

Limitations on the transportation, including crane capacity, of precast concrete deck panels could present challenges to the standardization process. Every state has established a maximum size and weight for pieces to be shipped without the need for a permit. Given their general shape, precast deck panels will often times be difficult to ship because they must lay flat. These limitations could influence the maximum bridge width that could be transported by ground. In most states, the width that can be transported without special permit is generally limited to 8 ft. High-performance materials that are lightweight and durable are most suited for prefabrication in large sizes.

Erection Issues: End Pour for Skewed Bridges

Another challenge during the construction of precast concrete deck panels is the end closure pour. This detail occurs in two specific situations:

- At the end of skewed bridge; and
- For bridges that incorporate an integral abutment.

In normal (square) bridges, the panels can be laid end-to-end along the bridge and posttensioned parallel to the beams or girders with no additional complications because the panels are all rectangular with square corners. The precast operation can be very economical, as all panels use the same forms and essentially the same details. However, if the bridge contains one or more skewed ends, the end panel must either be specialty formed and precast (for small skews only) or be cast-in-place with an end closure pour. In the case of a special precast end panel, the cost of forming this piece can be significantly higher than for the normal production panel.

One particular complication can arise when replacing an existing bridge deck with even minimal skew and attempting to salvage the existing abutment backwall. In this situation, the skewed end panel must fit precisely between the final normal panel and the backwall without creating a tapered expansion joint opening. The contractor must accurately measure the skew angle of the existing backwall prior to casting the skewed end panel. A deviation of even a small amount from the correct skew angle may make it impossible to place the final precast deck panel and working under an overnight closure is not the preferred time to learn that the final panel will not fit.

In the case of an integral abutment bridge with a longitudinally posttensioning deck, a cast-in-place closure pour is required to create a monolithic connection to the abutment, which provides for moment transfer.

In either of these cases, the need for appropriate curing time of the closure pour can significantly affect the time savings expected in accelerated construction and somewhat retards the purpose of rapid construction.

Erection Issues: Crown from Flat Panels

A review of the literature and surveys of bridge owners demonstrates that the vast majority of projects completed to date using precast deck panels have not had a crown built into the deck, but rather are constructed flat. For a situation in which a single panel can be used to construct the full width of the bridge, a moderate crown can be built in by thickening the slab along the centerline. However, this method of positive deck drainage is not very practical for wider bridges or where the cross slope for the bridge exceeds the normal 2% value. In addition, where panels are thickened at the center, a significant additional dead load is imposed on the supporting superstructure.

In most cases, wider precast decks that cannot be constructed as a single panel are designed as a two-panel system with a longitudinal joint supported on a center girder. In new construction, the center girder can be elevated to provide the proper support geometry. However, in the case of a redecking in which the existing girders are to remain in place, a possible solution is the inclusion of an exceptionally tall beamline haunch that is constructed of CIP concrete. A challenge for future development is the need to construct a durable longitudinal joint for this scenario that can be quickly constructed without the need for a CIP closure pour.

Panel Flatness and Riding Surface Issues

As discussed in the section on constructability issues for modular superstructure systems it is important to develop an adequate means of removing the differential camber between the girders on site to ensure comfortable rideability for users.

Where concrete decks without an initial overlay are used, consideration should be given to providing an additional thickness of 0.5 in. to permit correction of the deck profile by grinding, and to compensate for thickness loss due to abrasion. AASHTO LRFD Article 2.5.2.4 requires that the deck of the bridge be designed to permit the smooth movement of traffic. This section is primarily concerned with the serviceability of the deck and not necessarily with the structural performance. Construction tolerances, with regard to the profile of the finished deck, should be indicated on the plans or in the specifications or special provisions.

The number of deck joints should be kept to a practical minimum; however, the maximum panel width of 8 ft will likely govern this criterion.

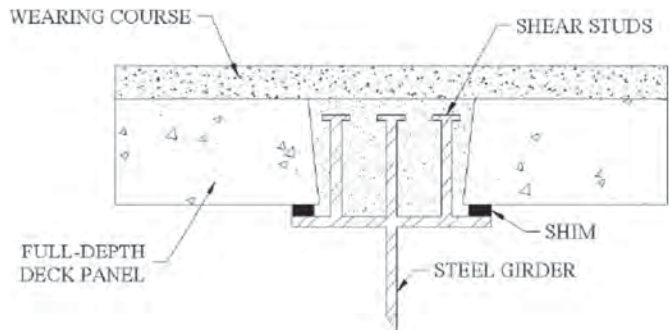


Figure 3.17. Shear pocket connection for steel girder bridge.

Constructability Issues and Considerations

To develop the largest possible load carrying capacity, precast concrete deck panels are made composite with supporting beams or girders using shear connectors. In the case of steel girders, this composite connection is achieved through shear studs welded to the top flange. With concrete beams, the composite connection is achieved by reinforcement protruding from the girders. In either case, the physical connection between panels is made through a variety of shear pockets, as shown in Figure 3.17. Typically, rectangular openings in the deck panels correspond with clusters of studs or reinforcing steel. These pockets are grouted after any deck posttensioning is completed to minimize any force transfer to the girders and subsequent loss of effective posttensioning force in the process.

Installation and Removal of Concrete and Shear Connectors

Two different concepts to streamline the installation of shear studs for use with precast concrete deck panels exist.

Shear studs used in composite steel bridge construction are typically ¾-in. or ⅞-in. diameter. Research has been conducted within the past 10 years to development a much larger super stud with a 1¼-in.-diameter. This new stud offers approximately twice the strength and a much higher fatigue capacity than a conventional ⅞-in.-diameter stud. The use of these larger-diameter studs would require far fewer studs to be installed along the length of the girder with cast-in-place concrete and provide far more room within the shear pocket when used with precast deck panels. Not only would this system increase bridge construction speed and future deck replacement, but it would also reduce the potential for damage to the studs and girder top flange during future deck removal.

The number of studs or stirrups in each pocket is usually based on AASHTO design requirements. The maximum shear stud spacing (or distance between shear stud blockouts) permitted by AASHTO LRFD is 24 in. There is some opinion in the industry that this limit is an arbitrarily safe "rule of thumb" limit imposed by AASHTO to assure complete composite action and avoid fatigue conditions. In most circumstances, this spacing is based on the fatigue capacity of the studs, and not the ultimate capacity. With precast panels it is beneficial, however, to place the shear connector blockouts at the largest spacing possible. This allows for fewer blockouts in the panels, which in turn increases panel strength for shipping and decreases manufacturing time and cost. Research and tests using both static and cyclic loads have been used to support the claim that 48-in. shear pocket spacing is adequate. An effort is under way through the Precast/Prestressed Concrete Institute (PCI) to gather support for a revision to the LRFD code that would permit the use of 48-in. stud spacing under certain conditions.

The replacement of an existing composite concrete deck, whether consisting of cast-in-place concrete or not, is complicated by the need to work around existing shear connectors. Past studies have shown that one of the most time-consuming parts of rapid deck replacement is the clearing of concrete around the existing studs. The bridge construction specifications used by bridge owners state that it is necessary to protect the shear connectors during deck removal and replacement. This is especially true for prestressed beam bridges in which a particularly desirable method for replacing any shear reinforcing that is damaged during deck removal does not exist. In addition, the potential for extensive damage to the beam concrete must be carefully monitored by contractor and inspector alike. Figure 3.18 shows an example of exposed shear studs following concrete removal.

In the case of a steel girder superstructure, it is frequently acceptable to simply cut the existing shear connectors off within a ½ in. of the top flange to accommodate precast panel installation. Shear studs are installed in clusters to match the location of stud pockets in the precast panels to be installed.

The recent development of ever-larger bulb tee beams has allowed for the efficient use of precast concrete spans well in excess of those available with AASHTO sections of years past. However, with these advantages come significant complications. With their wider and thinner top flanges, an effective and time-sensitive deck removal procedure that can be performed without damaging the underlying beams will present an even greater challenge for future generations of bridge engineers.

Figure 3.18. Clustered shear studs following concrete removal.

Uniform Bearing of Panels on Beams and Girders

Fabrication variations and differential camber among bridge beams and girders can cause the bearing of the full-depth deck panels on the girders to be uneven. Full-depth deck panels should be leveled and bear evenly on the girders to ensure optimum performance and long life. If the panels are not leveled, extensive spalling of the transverse joints and a poor riding surface may result, while placing the panels directly on the girders leaves voids that can cause the panel and joint to crack and spall.

To alleviate this problem, the void between the girders and panels must be filled completely with grout to provide a solid, uniform bearing surface. Leveling bolts or shims are commonly used to level the panels and temporarily support them above the girders.

Bridge girders with a large amount of camber, such as long-span prestressed concrete beams, require that the full-depth panels be leveled to produce the proper bridge profile, as illustrated in Figure 3.19. There are two common ways of achieving this level condition:

- Leveling the full-depth panels on the girders so that the panels align with the bridge profile. This allows for a thin wearing course to be used, which promotes rapid construction.

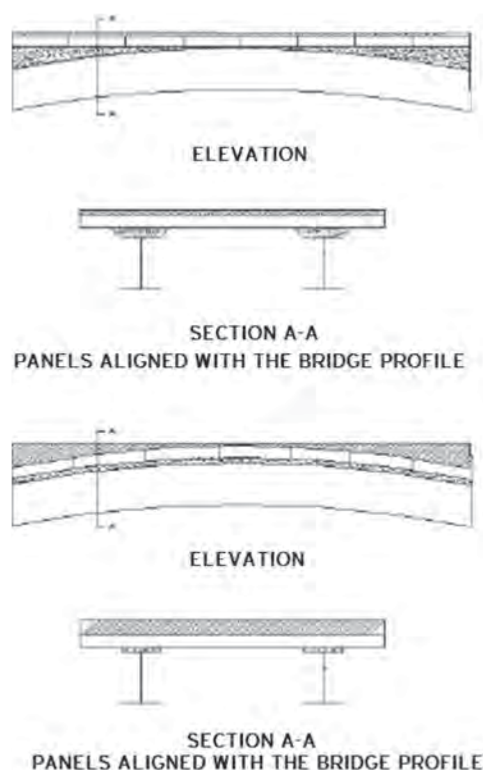


Figure 3.19. Two methods for leveling precast deck panels.

When this method is used, the clear distance between the bottom of the panels and the top of the girders can become very large near the ends of the bridge, making it difficult to level the panels and form the haunch region.
- Leveling the full-depth panels with the girder profile a short depth above the girder. This minimizes the distance between the bottom of the panels and the top of the girders, making the leveling easier, but the difference between the girder profile and bridge profile must be made with a varying depth wearing course, which requires a longer cure and adds significant additional dead load to the bridge.

Posttensioned Connections

Posttensioning is a well-established and acceptable alternative for ABC for which designers can find information from other sources.

Wearing Surface

A bridge deck constructed from full-depth concrete deck panels often exhibits an extremely rough surface because of the grouted joints and shear pockets. A typical full-depth panel deck is shown in Figure 3.20. This type of wearing surface is not acceptable on many bridges, especially those with large volumes of high-speed traffic, so a high-performance concrete wearing surface is often added to provide additional safety, rider satisfaction, and improved durability. A wearing surface offers another advantage in that it eliminates potential impact effects on any deck joints that are not absolutely flush.

Placing a wearing surface will have a significant effect on the time required to replace a bridge deck. The alternative to a placing a wearing surface would be to diamond grind the surface immediately prior to opening the bridge to traffic.

Splicing of Tendons During Overnight Closure Operations

For heavily traveled bridges or for bridges without a reasonable detour that would permit full closure of the bridge, the replacement of an existing deck with precast concrete deck panels is normally performed during overnight and weekend bridge closures. In these cases, the existing deck is removed and replaced to the extent permitted by closure duration each night. There are examples of many successful projects that have been completed this way, including the SR-433 Lewis and Clark Bridge and the Route 64 bridge over Lake Pomme du Terre in Missouri.

Figure 3.20. Rough surface texture of full-depth precast panel deck.

For precast concrete deck panels that incorporate longitudinal posttensioning, the need to splice onto in place tendons and continue adding panels each night complicates the problem. The posttensioning has to end at a specific location, which depends on the number of panels that can be completed within the closure duration. Thus, the posttensioning bulkhead is established at a particular panel location corresponding to the end of an overnight of work. If the posttensioning needs to be continuous for subsequent panels, the panel and stressing design will need to incorporate splicing and resumption of stressing in order to maintain a uniform posttensioning force in all panels. In addition, the contractor may need to make provisions for an unplanned panel installation endpoint in the event of a mechanical breakdown or unexpected weather delay.

Potential Time Savings During Construction

The time required to construct a typical cast-in-place concrete deck is considerable and normally lies on the critical path to opening the bridge to traffic. In addition to the actual time to construct the deck and place the concrete, most owners require a 7-day wet curing period for all bridge decks.

Potential time-related considerations for precast deck systems include the following:

- Panels are precast and delivery is scheduled when needed on site.
- Panels are erected directly from the delivery truck.
- Panel systems may require grinding or overlay in order to achieve a smooth ride and to eliminate the potential for salt intrusion.
- Construction staging (and replacing half of the deck at a time) can be achieved with a longitudinal joint.
- Precast barrier sections have been developed. They can be installed using bolts or posttensioning.
- CIP concrete placement may be delayed by hot weather or rain.
- Removal of existing concrete decks is very time-consuming and requires working around existing shear connectors without damaging the supporting members, which is challenging.

To illustrate part of the constructability evaluation, the team compared the time required to construct a bridge deck

Table 3.3. Time Durations for CIP Decks and Precast Decks

Cast-in-Place Deck		Precast Concrete Deck System	
Task	**Duration (Days)**	**Task**	**Duration (Days)**
Install walers and plywood deck forms between girders	12	Install neoprene haunch strips for each beam line	2
Install overhang jacks, brackets, forms, walkway, and handrail	8	Erect precast deck panels (8 per day)	7
Install curb forms and screed rail	4	Assemble ducts and place closure pour concrete	4
Tie main reinforcing steel, including vertical barrier rail reinforcing	7	Thread posttensioning tendons	2
Assemble and adjust screed rails	2	Perform posttensioning	2
Assemble and test deck finish machine	1	Grout posttensioning duct	1
Place deck concrete and curing system	1	Grout shear pockets and girder haunch	2
Wet curing period	7	Install precast concrete barrier rail sections	5
Strip deck forming	7	Place high-density concrete overlay	3
Underdeck patching	2	Wet curing period	3
Slipform barrier	3	Install strip seal expansion joint gland	3
Install strip seal expansion joint	3		
Total Time	**57 days**	**Total Time**	**34 days**

for a typical DOT grade-separation bridge deck, as summarized in Table 3.3. The contractor is assumed to have considerable experience with DOT bridges, but not necessarily with posttensioned deck panels. The following basic data were assumed:

- Bridge consists of four spans: 70 ft, 100 ft, 100 ft, and 70 ft, for total length of 340 ft.
- Bridge width = 44 ft (two 12-ft lanes plus two 8-ft shoulders and barrier rail).
- Bridge superstructure consists of steel beams with shear studs installed in the field.
- Shear studs are assumed to require the same amount of time for each alternative. Although the CIP deck alternative may have a greater number of studs to be installed, it is assumed that the greater precision required to match shear pockets in the panels is roughly equivalent.
- CIP deck alternative consists of single 8-in.-thick high-performance concrete deck with epoxy-coated reinforcing steel. Barrier rail will consist of slip-formed, jersey-style barrier rail. During the 7-day curing period, a number of operations can take place that do not require the placement of heavy loads on the bridge deck (screed rail removal, curb form stripping). Given the length and width of the bridge, the contractor will be allowed to place the deck concrete in a single, continuous operation.
- Precast deck alternative consists of 8-in.-thick precast panels, with longitudinal posttensioning. Transverse joints are spaced at 8 ft centers. In order to increase durability, the precast deck panels will be overlaid with a 2-in.-thick high-density concrete overlay. The barrier rail will consist of precast barrier units installed after the deck panels are installed, but prior to the placement of the overlay.
- Bridge deck will be constructed in a single operation. No staging of traffic is required and a detour is available.
- Abutment is non-integral, so both bridge deck alternatives will have a strip seal joint.

This simplified example illustrates the potential time savings for a routine bridge project. A total time savings of 23 days, or nearly 40%, is available through the use of a precast concrete deck panel system. Even given some fairly conservative assumptions for construction durations, and notwithstanding that a contractor can proceed much faster simply by applying more labor for those items that are not simply a waiting period, the potential time savings are significant.

There are a number of variations for bridge deck replacements that cannot be performed as simply as closing the bridge to traffic and pouring a CIP deck in one operation. Examples include the following:

- Staged replacement of a two-lane bridge deck where no reasonable detour exists. The existing bridge is removed in a series of transverse strips. In this case, as many deck sections of the existing bridge are removed as can be replaced each night.
- Staged replacement of a four-lane deck where traffic can be shifted to other lanes during overnight operations. With a

longitudinal joint along the centerline, it may be possible to remove half the deck, replace it with precast panels, switch traffic to the new deck, replace the other half the deck with precast panels posttensioned longitudinally, and finally connect the halves with transverse posttensioning.

- Long viaduct deck replacement that cannot be completed in a single, continuous concrete placement. In these types of situations, the contractor is required to place deck concrete in a particular sequence to avoid damaging previously placed sections. Typically, each positive moment region in the bridge is placed in a separate operation (with 7 days of cure time at each step) and the same process is repeated for the negative moment regions.
- Horizontally curved bridges would require precast panels that are tapered in order to accommodate the varying radii from the interior side of the curve to the exterior side of the curve.
- Bridges with a superelevation transition. Any time of precast operation is best suited for repetitive operations where the mass production of the precast panels can be leveraged to obtain the best possible bid price from the precaster.

Implementation Challenges

Most of the systems described in this report, though proven, are only occasionally used for bridge replacement projects. Even relatively simple-to-deploy solutions such as precast deck panels, prefabricated modular bridges, or movement solutions such as sliding, rolling, launching, movement using SPMTs, and so forth, are used for a very small fraction of ongoing bridge work. This section addresses issues pertaining to implementation and what should be done in the development stages to overcome the impediments to ABC implementation.

Modular Superstructure Systems

Precast concrete girders have seen widespread acceptance as an economical construction alternative over the last 50 years. In recent times, deck girder systems and prefabricated composite stringer systems have gained some traction in rapid construction situations. More needs to be done to achieve greater penetration of prefabricated systems and components to minimize on-site construction and change the current cast-in-place construction culture. While most agencies are aware of ABC technologies, very few practice it on a large scale. According to the surveys in Phase I, many ABC techniques are ready for implementation, yet DOTs are hesitant about using ABC techniques because of certain concerns, including higher initial costs and longevity of connections. The objective of the R04 project is to develop standardized approaches to designing, constructing, and reusing (including future widening) complete bridge systems that address rapid renewal needs and efficiently integrate modern construction equipment. Pre-engineered standards for proven systems will make available a toolbox of ABC systems to designers who may be new to this method of construction to help overcome the initial resistance to trying a new approach that they may perceive as being risky or complex.

Advancing the state of the art to overcome obstacles to ABC implementation and achieve more widespread use of ABC is a goal of this research. Key findings from the Phase I outreach efforts of owner and contractor concerns and impediments to ABC implementation pertinent to superstructures are as follows:

1. There is a cast-in-place (CIP) construction culture among contractors. Contractors like to keep as much work for themselves as possible to keep crews employed and maximize profits. Precast options may require work to be subcontracted out and reduces the control of the prime contractor.
2. The largest impediment to increased use of ABC appears to be the higher initial costs. Reducing cost was a priority with most owners.
3. ABC is perceived as raising the level of risk associated with a project. It is also perceived by some contractors as being too complex. Proven superstructure and substructure systems that reduce overall risks would be quite attractive to owners and contractors.
4. There are concerns about the durability of joints and connections in precast elements.
5. There are concerns about seismic performance of precast elements and connections in seismic regions.
6. Lack of familiarity with ABC methods is a concern. States are looking for design manuals and other aids that could help them to design and implement ABC. Training could be beneficial.
7. Standardizing components is good but also offers challenges in getting the industry and the states to come together in a regional approach to ABC. Developing ABC standards that could be adopted regionally is one goal.
8. There is a need for design considerations for structures to be moved, for acceptable deformation limits during movement, and for better specifications.
9. ABC designs should be adaptable to a number of placement options to be cost competitive. A majority of contractors are not receptive to owners requiring that a specific method of construction be used in ABC contracts.
10. Lack of access for equipment and the need for large staging areas unavailable in urban locations are hindrances to large-scale prefabrication. Use of smaller elements that can be assembled on site for superstructures and substructures will overcome mobility issues. The modular concept of building bridges could overcome this concern.

11. Contractors would be more willing to make equipment purchases if bridge construction became more standardized or industrialized, and was based on certain methods of erection to speed the assembly. Standardization increases the prospects for repeated use of the same equipment.

Any obstacles to implementing these modular superstructure systems will depend on how effective these systems will be in addressing these owner and contractor concerns about ABC.

Item 1

There is a cast-in-place (CIP) construction culture among contractors. Contractors like to keep as much work for themselves as possible to keep crews employed and maximize profits. Precast options may require work to be subcontracted out and reduces the control of the prime contractor.

One of the main reasons for the lack of support for ABC among contractors is the reluctance to subcontract out much of the work to precasters and other specialty subcontractors for transportation and erection, which is seen as reducing the general contractors' profit and control of project operations. Contractors want to keep their own crews busy. With precast decks, there have been problems with unions; the ironworkers' union has complained because there was no deck to reinforce. This is a valid concern that will not go away with innovative designs or standardization of ABC concepts. Ways to work with the contracting industry need to be found to make this transition to ABC happen. The solution is to introduce the CIP industry to precast technology and demonstrate its profitability.

Self-Performing of Precasting by General Contractor

This is an approach to ABC in which the designs allow maximum opportunities for the general contractor to do its own precasting at a staging area adjacent to the project site or in the contractor's yard with its own crews. The more the contractor performs using its own crews, the more profits it will realize. There is no reason to go to a precaster, unless the precast is posttensioned or prestressed, which would need to be fabricated in a certified plant. In this regard, the prestressed deck girders would need to be fabricated by a precaster. However, contractors have always obtained their precast prestressed beams from precasters. Using ABC would not signal a significant change except for the integral deck that is cast with the beam. With regard to the decked steel girder systems, contractors can perform the fabrication themselves because the deck is made of conventionally reinforced concrete. The casting of the deck can be done off-line by contractor crews at a staging area and then transported for erection. The casting of the deck can be done under fully shored conditions in which the beams are ground supported, which is advantageous for ease of construction, worker safety, and enhanced structural resistance of the system, since it avoids buildup of non-composite stresses. The weight of modular systems should be kept under 100 tons to allow for erection by conventional cranes. Lightweight concrete mixes can be used to lighten sections. Self-performing is perhaps even more significant for substructure components and is discussed further in the substructure evaluation report.

Items 2 and 3

The largest impediment to increased use of ABC appears to be the higher initial costs. Reducing cost was a priority with most owners.

ABC is perceived as raising the level of risk associated with a project. It is also perceived by some contractors as being too complex. Proven superstructure and substructure systems that reduce overall risks would be quite attractive to owners and contractors.

Standardizing modular superstructure systems for ABC is aimed at increasing their availability through local or regional fabricators. Doing so will greatly increase their availability to owners and contractors and reduce lead times, which should result in more widespread use and thus reduced costs. Standardizing also makes these systems more familiar to engineers, owners, and contractors, thereby reducing complexity and the level of risk associated with the project. Contractors will be more willing to offer competitive bids on a system they have experience with and that they perceive to be proven and easily constructible. Repeated use of a standardized ABC design also allows the owner or engineer to iron out kinks in the system through continuous improvement, leading more to a better design than to a onetime, customized solution. Repeated use of these systems will also encourage contractors to provide suggestions about how the constructability could be further improved, which will lead to further reductions in overall risks and cost.

The modular superstructure systems will be developed to achieve cost and risk reductions through the adoption of the guiding philosophy for all ABC concepts advanced in this project. This philosophy is stated as follows:

As light as possible
- Simplify transportation and erection of bridge components.

As simple as possible
- Fewer girders, splices, or bracings.

As simple to erect as possible
- Fewer workers on site;
- Fewer fresh concrete operations;
- No falsework structures required; and
- Simpler geometry.

Items 4 and 5

There are concerns about the durability of joints and connections in precast elements.

There are concerns about seismic performance of precast elements and connections in seismic regions.

The quality and durability of joints and connections between prefabricated components has been a significant concern that has impeded the greater use of ABC. Western states have also had concerns about shear-only grouted joints and their performance during a seismic event. With this in mind, modular superstructure systems have placed maximum emphasis on developing durable connection details between prefabricated elements. Full moment connection using ultra-high-performance concrete (UHPC) or high-performance concrete (HPC) is being recommended as the preferred connection for details that are strong, durable, and seismically sound. Standardizing these connections with proven, easy-to-construct details will go a long way in overcoming the past concerns with performance of joints and connections.

Items 6 and 7

Lack of familiarity with ABC methods is a concern. States are looking for design manuals and other aids that could help them to design and implement ABC. Training could be beneficial.

Standardizing components is good but also offers challenges in getting the industry and the states to come together in a regional approach to ABC. Developing ABC standards that could be adopted regionally is one goal.

Standardizing ABC systems will bring about greater familiarity with ABC technologies and concepts and will foster greater regional cooperation, which will help achieve region-specific customization that accommodates regional practices and industry needs. Pre-engineered standards to be developed in this project will emulate cast-in-place construction but will be optimized for modular construction and ABC. These standards can be inserted into project plans with minimal additional design effort to adapt to project needs. Using these standardized designs will serve as a training tool to increase familiarity about ABC among engineers. Formal training courses should also be developed to provide background information on ABC and the application of design specifications.

Items 8 and 9

There is a need for design considerations for structures to be moved, for acceptable deformation limits during movement, and for better specifications.

ABC Designs should be adaptable to a number of placement options to be cost competitive. A majority of contractors are not receptive to owners requiring that a specific method of construction be used in ABC contracts.

Modular systems can be erected using conventional equipment for most span ranges. Longer spans may be erected using specialized erection methods or movement technologies. The intent of ABC construction technologies being developed in this report is to develop standard concepts for erecting highway structures by using adaptations of proven long-span technology that can also be easily adapted from project to project. This project will develop conceptual design of equipment suitable for ABC use. Another important class of ABC projects includes those that require the movement of large components or bridges completed using various movement techniques. These movement techniques include self-propelled modular transporters (SPMTs); bridge sliding, skidding, and rolling using various sliding surface movement methods; and incremental launching. These ABC construction concepts will provide the contractor a range of construction options, from conventional erection to specialized erection and movement techniques, to make ABC projects cost competitive.

Design considerations for structures or components to be moved and acceptable deformation limits during movement are topics that will need further clarification during the development of the design standards. Additional specification language may also need to be developed to guide practitioners.

Items 10 and 11

Lack of access for equipment and the need for large staging areas unavailable in urban locations are hindrances to large-scale prefabrication. Use of smaller elements that can be assembled on site for superstructures and substructures will overcome mobility issues. The modular concept of building bridges could overcome this concern.

Contractors would be more willing to make equipment purchases if bridge construction became more standardized or industrialized, and was based on certain methods of erection to speed the assembly. Standardization increases the prospects for repeated use of the same equipment.

Moving complete bridges by using wheeled carriers requires large staging areas, which may be in short supply in congested urban areas. Modular systems allow the superstructure to be built in place with smaller components, thus overcoming the mobility issue. In short, modular systems allow a more versatile option to ABC not limited by space availability at the bridge site.

Standardized designs will allow for the repeated use of modular superstructure systems, which will make contractors more willing to invest in equipment on the basis of certain methods of erection to speed assembly. Repetitive use will allow contractors to amortize equipment costs over several projects, which is an important component to bring overall costs in line with conventional construction.

Segmental Superstructure Systems

There is an unfortunate perception in many parts of the country that voided slabs have been plagued by performance and durability problems. This is not the case, considering that the United States has not used voided slabs as they are defined elsewhere in the world. True voided slab systems are regularly and successfully designed and constructed as a least initial cost solution in Canada and Europe, where they are known for superlative strength, redundancy, and durability in highly aggressive environments. Voided slabs, as defined in U.S. practice, generally mean box girders that have been used for short and intermediate spans. Almost always, they have been implemented (unfortunately) with inadequate detailing, poor workmanship, and a lack of proper posttensioning. All of these factors have combined to create a negative perception of this technology in many quarters.

Short and medium-length precast segmental bridges for ABC applications do not pose particular implementation challenges. Segmental precasting is a well-known construction method, the use of concrete stitches and long-line match casting have been amply tested, and the engineering underlying channel or ribbed sections and solid or voided slabs is also proven technology.

Short and medium-length precast segmental bridges may be erected with support girders or by incremental launching, as shown in Figure 3.21. The support girders would be simplified versions of self-launching gantries, which are amply tested machines. Incremental launching is also a time-tested construction method, applied for 50 years all over the world. The first launched bridge was composed of precast segments joined with concrete stitches. Multi-cellular box girders and ribbed slabs have also been launched.

Figure 3.21. Segmental channel beam erection.

In both cases, therefore, the technical obstacle is not the risk of innovation. Simplifying existing machines and learning how to use a construction method described in tens of publications and several domestic and international codes involve minimal risk.

The technical obstacle to the adoption of these technologies is the poor knowledge of the developments achieved by the international bridge industry and the inertia of the U.S. bridge industry in exploring construction techniques that are amply consolidated elsewhere. Inertia can be won with incentives for contractors to erect short and medium-length segmental bridges with support girders and incremental launching. These incentives include the following:

- Merging several bridges into one contract to facilitate amortization of the initial investment for designing and implementing a modular system of segmental bridges.
- Relaxing the design requirements where possible, including simple geometry, constant radii of plan and vertical curvature, and constant width. Move all transitions out of the bridges.
- Educating DOTs, contractors, and designers with itinerant courses given by specialists in bridge erection machines and incremental launching of bridges. The courses should provide continuous education credits.
- Financing experimental projects to spread information on costs, quality, and technology.

Testing Needs

UHPC Joints for Modular Superstructures

Continue to monitor the Turner–Fairbank Highway Research Center's (TFHRC's) ongoing research program into the use of UHPC in highway bridges, specifically the research into deck-level connections between modular precast components. In conjunction with the New York State DOT, researchers at the TFHRC are investigating the use of UHPC for a new generation of connection details applicable to modular bridge components [Transportation Pooled Fund Project TPF-5(217), UHPC Connections Between Precast Bridge Deck]. The findings from this research will be pertinent to the UHPC connection details to be developed for this project.

Precast Decks

To address the challenges described in the preceding sections on engineering and constructability evaluations, a number of research and testing needs have been identified. To promote the wider use of full-depth precast concrete panels, these challenges will need to be addressed to produce a durable and easily constructible deck for a wide variety of situations.

A number of these important topics are currently under investigation by researchers around the country. These needs can be addressed through a combination of theoretical analysis, numerical analysis, or laboratory testing. Identified research needs include the following:

- Evaluation of transverse and longitudinal joints;
- Guidelines for a closure pour for skewed and integral abutment bridges;
- Practical, durable solutions for providing crown in precast decks;
- Shear stud or shear steel configuration to make girders composite with decks;
- Guidelines for intermediate closure pour in the case of stage construction; and
- Specific modification to AASHTO LRFD Chapter 9.

Evaluation of Transverse and Longitudinal Joints

Additional research is needed to evaluate joint details. This research should focus on eliminating posttensioning to the greatest extent possible through the use of advanced materials such as ultra-high-performance concrete. The New York State DOT and FHWA are currently involved in research into UHPC joints for modular superstructures and precast deck panels. There is considerable evidence that the incorporation of longitudinal posttensioning in precast deck systems can result in a crack-free and leakproof deck. As noted previously, many bridge owners and contractors are inexperienced with posttensioning. As with any unknown technology, contractors will increase their bid prices to help them mitigate uncertainties.

The research focus should be to fully understand the complex forces initiated in a joint by the applied dynamic wheel loads. This research can take two forms: either finite element analysis of a joint or by testing small specimens in a laboratory.

Two primary forces exist in a typical transverse bridge deck joint: shear and bearing. The application of force is fatigue in nature. In a longitudinal joint, the stress field becomes even more complicated through the addition of flexural forces. If a longitudinal joint is over a girder, negative flexural moments in the joint are predominant. If a joint is not properly detailed to accommodate these flexural loads, there is considerable chance that a crack may occur in the joint, which eventually permits the intrusion of water through the deck and promotes the rapid deterioration of the joint.

In the event that this research reliably concludes that posttensioning cannot be eliminated completely, additional research should focus on developing a system in which the panels are lightly reinforced for self-weight to overcome handling and installation stresses, while the main reinforcement in both the transverse and longitudinal directions is provided through posttensioning in nature.

Guidelines for a Closure Pour for Skewed or Integral Abutment Bridges

Additional guidelines are required for the design and construction of skewed PCDP bridges. As mentioned previously, if the bridge skew is severe, it is difficult to design and cast panels to match the skew. It is far simpler to design panels in a skewed fashion when only reinforced-concrete design concepts are used. Due to the skewed alignment at one end of a particular panel, the prestressing or posttensioning will introduce additional eccentric forces into the panels. In situations that are not time-critical, it may be desirable to design the panels as tangents and provide cast-in-place closure pours. In addition, further investigation is needed for rapidly constructed closure pours.

Practical, Durable Solutions for Forming Crowns in Precast Decks

Reliable design details are needed to provide a practical solution for crown formation using two flat panels. Current solutions either provide additional dead loads to the structure or provide a longitudinal joint that is susceptible to water leakage. Additional hardware can be built into the panels to provide rotational capability and eventually a crown to the deck panels.

Recommendations

Recommendation of concepts considered suitable for standardization is based on a critical evaluation of factors that were uniformly applied to each concept via an evaluation matrix included with each evaluation report. Twenty-one evaluation criteria, as given below, were used to rank these concepts. Each criterion was given a score from 1 to 5, where 5 = very good and 1 = poor. The maximum score possible was 105. The score for each superstructure concept is as follows:

- Concrete deck bulb tees = 84
- Concrete double tees = 83
- Decked steel stringer system = 88
- Decked steel trapezoidal box girders = 72
- Segmental concrete superstructure systems = 78

Certain important criteria about the appropriate superstructure system should be considered before a decision is made to choose prefabrication as the best course for bridge construction. A matrix of criteria for selecting modular and segmental superstructure systems is given in Table 3.4.

Modular Superstructure Systems

Modular superstructure systems composed of both steel and concrete girders are recommended for advancement to the subsequent tasks in which pre-engineered standards will be

Table 3.4. Selection Matrix for Modular and Segmental Superstructure Systems

Criteria	Decked Steel Stringer	Decked Bent Plate Steel Box	Concrete Deck Bulb Tee	Concrete Double Tee	Segmental Systems
Spans <140 ft	X	X	X	X	X
140 ft < Span < 250 ft (with field splicing)	X		X		X
Spans >250 ft					X
Tier 1 ABC: Can be completed over a weekend	X	X	X	X	
Tier 2 ABC: Can be completed in a few weeks	X	X	X	X	
Tier 3 ABC: Accelerate larger projects, saving weeks or months	X	X	X	X	X
Light weight is a priority due to site access	X	X			
Able to be constructed by local contractors	X	X	X	X	
Bridge has multiple similar spans/Long viaducts	X		X	X	X
Bridge has continuous spans	X		X	X	X
Should allow future widening	X	X	X	X	
Bridge has curvature		X			X
Bridge has underclearance issues		X			X
Spans with limits on falsework or ground access for construction					X

prepared for these systems. Deck bulb tee, deck double tee, and decked steel stringer systems received the highest scores, as these are proven systems for rapid renewal. Even two lower-scoring alternatives have specific advantages for certain sites. Each modular system is expected to see a 75- to 100-year service life due to the quality of its prefabricated superstructure, the use of high-performance concrete, and the attention given to connection details.

Precast Concrete Deck Girders (Bulb Tee/Double Tee)

Conventional precast concrete girders have been well established for bridge construction in the United States for more than 50 years. There is wide acceptance for them among owners and contractors because they are easy and economical to build and to maintain. In most cases, the girders are used with a cast-in-place (CIP) deck built on site. For ABC applications, the key difference is that the girders will have an integral deck, which eliminates the need for a CIP deck. The precast deck girders combine all the positive attributes of conventional precast girder construction with the added advantage of eliminating the time-consuming step of CIP deck construction. Contractors familiar with conventional precast girder construction should have no difficulty in adapting to these newer deck girders installed using an ABC approach. Deck girders are a proven system, having been standardized for use by several Western states. The team expects that the deck girder bids will be very competitive when compared with the girder and CIP deck systems and may come in even lower for sites in which constraints to deck casting operations may exist.

Decked Steel Stringer System

Similar to the concrete deck girder system, the decked steel stringer system is also a proven concept shown to be quite economical and rapidly constructed. Many states are familiar with the Inverset system or some variations of it. Standardizing generic designs for commonly encountered spans and skews will provide a big boost to this modular concept, gaining quick acceptance and more widespread use. Length limitations resulting from press brake capacities will mean that the splicing of bent plate boxes will be required to reach spans in the range of 60 ft to 100 ft or more. As for the precast deck girders, the recommended connection will be the full moment connection for all the same reasons previously discussed.

Decked Bent Plate Box Girder System

The bent plate trapezoidal box is another innovative alternate design concept for steel girder systems that will be a good complement to the stringer systems in certain situations, such as curved ramps and bridges, bridges with limited underclearance, and bridges where aesthetic considerations become deciding issues. At the present time, this system is recommended only for simple spans. A single sheet can be used for cold bending trapezoidal boxes up to about 60 ft. Spans up to 100 ft can be accommodated by splicing sections. This system is currently envisioned for short spans with no end continuity. Further research and development will be needed to validate a design approach for the negative bending regions.

Connections

The option with full moment deck continuity is recommended to provide a more durable structure with redundancy as compared with the option with a welded plate connection combined with a grouted key. For bridges subject to moderate-to-heavy traffic and located in zones in which design is required for seismic loading, it is recommended that a full moment deck slab closure pour be used between adjacent modules or precast sections. For multi-span bridges, additional efficiency could be achieved by making the beams continuous for live loads by installing cast-in-place concrete diaphragms at the piers using closure pours.

The welded plate connection between the deck girders, though not recommended as a standard option, may be used by agencies as a lower-cost detail for bridges subject to light traffic, bridges carrying local roads, or bridges located in low seismicity zones in which seismic design is not required. As the design of prefabricated sections with connection plates and grout is not covered by AASHTO LRFD specifications, structure evaluation for permit loading would be difficult at the present time. Evaluation requirements for permit loading need to be developed in addition to the other requirements indicated in this report.

Overlay

An overlay is not considered necessary when the girder and connections will be constructed with high-performance materials, which should provide good durability without an overlay or concrete topping. The deck segments will have ¼-in. overfill, which will be diamond-ground in the field to provide adequate rideability. All modular systems also allow for future widening. The design of modular systems and precast girders could be standardized for various span lengths and commonly used beam spacings for efficiency. Spans in the 200-ft to 250-ft range could be constructed by splicing two separate girders of transportable lengths in the field.

Segmental Superstructure Systems

Segmental precasting of box girder bridges is a well-established construction method that offers many benefits on suitable projects. The transfer of precast segmental technology to channel sections and solid or voided slabs should not pose particular technical challenges and would result in new structural solutions for ABC applications. Posttensioned slab spans can provide economical, low-maintenance ABC systems for spans up to about 150 ft.

Adapting Segmental Slab Systems for Routine Bridges

Segmental slab systems, based on years of experience and the application of proven technology in the United States, can be scaled back and applied on typical grade separation projects for ABC with significant cost and schedule benefits.

The technology can be adapted to a variety of widths and span arrangements as follows:

- Solid slabs are economical up to 80 ft in length (for simple spans) and can be extended to 100-ft spans with continuity in the longitudinal direction.
- Voided slabs are cost-effective between span lengths of 90 ft and 150 ft with continuity.
- Voided slabs must be transversely prestressed prior to shipping. Solid slabs do not require transverse posttensioning unless the deck width and pier configuration require it.
- Longitudinal posttensioning is applied in the field.
- Joints are match cast in simple long-line forms with epoxy applied in situ.
- Manufacture of segments does not require expensive forms, beds, or exotic detailing and can be readily self-performed by any competent contractor.
- Erection equipment consists of simple erection girders that can be delivered on flat beds and erected or dismantled overnight.

The critical issues for the success of this technology are as follows:

- Simple and effective detailing;
- Quality workmanship; and
- Proper design of prestressing and posttensioning.

The proper execution of the above parameters should result in competitive solutions on a bridge-by-bridge basis, and does not necessarily require a minimum number of segments to be cost competitive. The increasing number of smaller projects using conventional segmental technology across the country supports this point.

Larger projects and multiple bridges will result in only a more competitive solution.

Finally, significant life-cycle and performance benefits will also accrue, including the virtual elimination of cracking, watertight deck systems, reduced maintenance, and enhanced life expectancy. Overlays can be applied to any deck system to further extend durability and life expectancy.

Precast Concrete Deck Panels

Precast concrete deck panels (PCDPs) are a proven technology awaiting an engineering solution for several of the bridge renewal challenges. PCDP is somewhat unique from two different perspectives when compared with the other superstructure elements presented in this report. First, full-depth precast concrete deck panels, whether posttensioned or used with a variety of cast-in-place joint materials, are the only elements presented that are truly applicable to both the complete replacement of existing bridges and also in the rapid replacement of a deteriorated bridge deck where the underlying superstructure remains in serviceable condition.

Second, precast concrete deck panels have been used for projects across the country for at least 40 years and have a record of proven performance. The advancement of precast concrete deck panel technology is not as much about the development of a new system as it is about promoting an existing system as worthy of consideration in Phase III of the current project. This system will not reach its full potential until owners, designers, researchers, and contractors can use this technology for development of a precast deck system that addresses the need for the following:

- Durable, long-lasting details that provide composite action with beams or girders.
- Rapid construction for present-day applications.
- Reliable removal for future deck replacement projects.
- Ride quality, which translates into a longer-lasting deck through the elimination of impact loading from wheels passing across the joints.

A number of systems associated with precast concrete deck panels have been developed, tested, and implemented with a wide range of success. These systems include reinforced concrete, pretensioned concrete, and posttensioned concrete panels with a variety of joint materials. The research team has evaluated a variety of these systems and identified key features to be incorporated into a system for future standardization on a national or regional basis. Recommendations for implementation include panel and connections to beams and girders, panel joints, and posttensioning, all of which are covered in the rest of this section.

Panels and Connections to Beams and Girders

Full-depth precast concrete deck panels offer significant advantages in construction over conventional cast-in-place concrete. Advantages and recommendations include the following:

- Reinforced concrete panels are most suitable for mass production and minimize the need for special hardware or casting yards.
- Panels up to 8 ft wide and up to approximately 50 ft long will permit the largest possible pieces without the need for special transportation permits and large cranes for installation.
- A minimum panel thickness of 7½ in. for girders spaced up to 11 ft, and 8 in. for girders spaced up to 12 ft.
- Fully composite panel connection to the superstructures using shear pockets. Shear pockets should be grouted after any posttensioning (if used) is applied.
- The use of larger-diameter shear studs should be further studied with an eye toward future implementation.
- The current effort to modify the 24-in. maximum spacing requirement for shear connections up to 48 in. should be strongly considered. Research under NCHRP 12-65 provides supporting lab testing results. This modification would simplify both new construction, as well as the future removal of precast deck panels when a deck replacement is required.
- Flowable, self-leveling, freeze–thaw durable, nonshrink grout mix should be used. The grout material, if stored on the construction site, should be kept protected from environmental factors such as humidity and rain.

Panel Joints

To create a durable maintenance-free bridge deck system, precast concrete panels must use the most-reliable joints possible. Given that most precast deck panel installation projects will likely include full-depth panels that span the entire roadway width, only transverse joints will be discussed in these recommendations.

- Ultra-high-performance concrete joint filler. Although UHPC has not been widely used for precast concrete deck panel joints, the advantages offered by its very high strength, low permeability, and bonding capacity with precast concrete panels make this material highly suitable for this application.
- Transverse, shear key joints shall be used to connect adjacent precast deck panels. It is critical that all joints be designed, detailed, and constructed to be completely flush and provide full shear transfer across the joint. Flush joints are essential to eliminate impact loading due to truck wheel applications and the consequent problems with water intrusion and long-term deterioration.
- For staged construction in which only a partial bridge deck is constructed overnight, and in which longitudinal posttensioning is provided with staged construction, the end panel of every night's work shall be provided with tendon splicing devices ("dog bones") that allow anchoring of the longitudinal posttensioning at the end of one work period and the resumption of posttensioning at a later stage.
- Nonposttensioned connections that use bulged structural tubes for spliced reinforcing connections, as developed during NCHRP 12-65, should be considered for further development and implementation. These details offer the advantage of eliminating the need for posttensioned joints while creating a durable load transfer connection.
- Posttensioning is an acceptable alternative to UHPC for ABC construction.

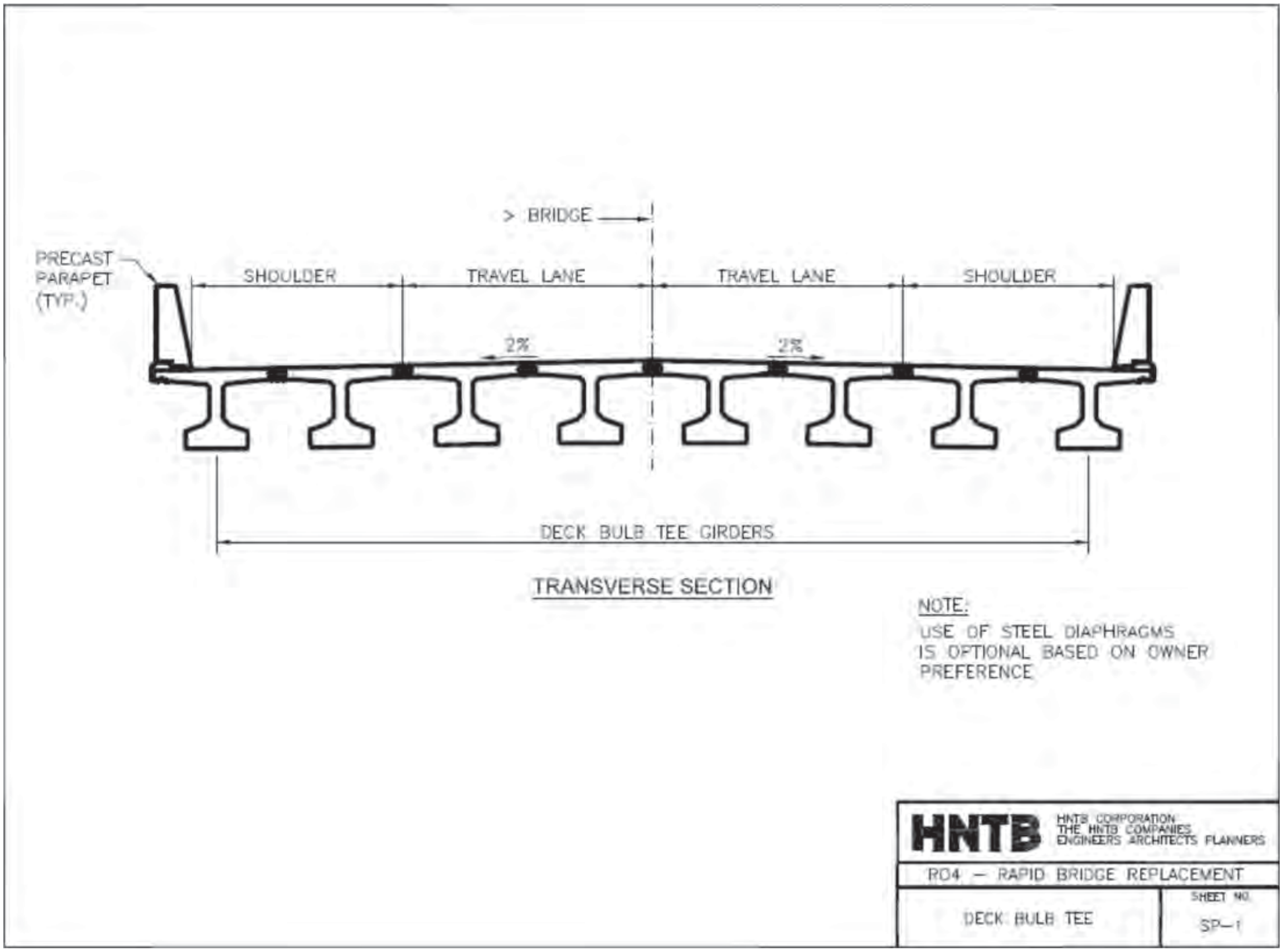


Figure 3.22. Deck bulb tee girders.

Posttensioning

If posttensioning (PT) is used, the following additional recommendations should be considered:

- PT force should be applied to only the precast deck to obtain the greatest effective prestressing force in the concrete deck. Therefore, the panel-to-girder connection should not be constructed until the PT is tensioned and anchored.
- The minimum average effective stress on concrete due to PT should be at least 250 psi.
- PT tendons should be uniformly distributed across the slab width and spaced no further than four times the structural composite slab thickness.
- Maximum jacking stress in PT reinforcement should not exceed 80% of the specified minimum of the guaranteed ultimate tensile strength (GUTS) of the posttensioning steel.

Exhibits

Different types of precast deck panels are shown in Figures 3.22 through 3.27.

Part 2: Evaluation of Precast Substructure Systems

Overview

In this section, the results of the evaluations for precast modular abutments and complete piers are presented under the following headings:

- Precast modular abutment systems
  - Body;
  - Wings; and
  - Support options, piles, shafts, spread footing.

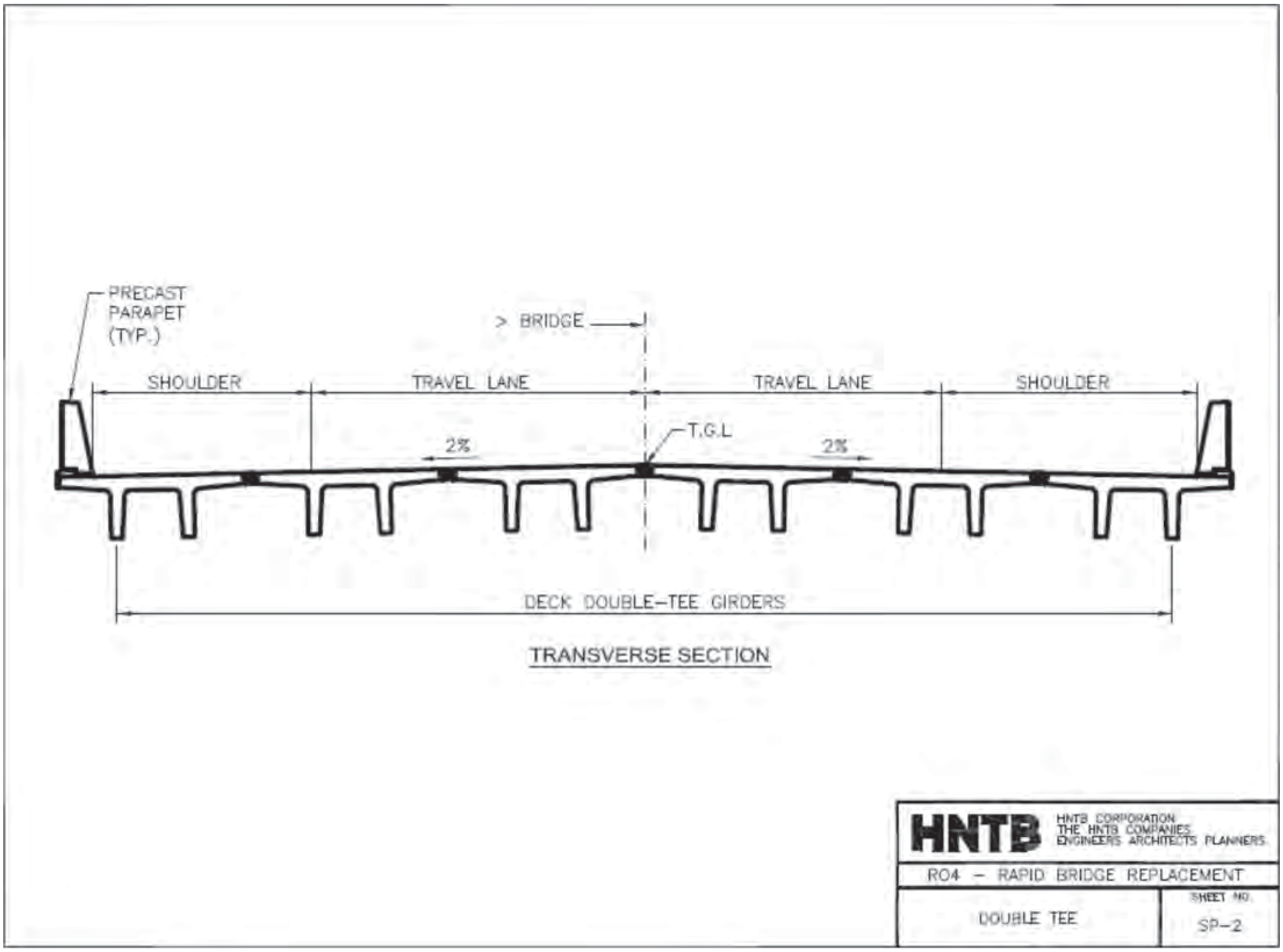


Figure 3.23. Deck double tee girders.

- Precast complete pier systems
  - Whole pieces, footing, shaft, cap; and
  - Support options, piles, shafts, spread footing.
- Segmental columns and piers
  - Segmental columns;
  - Pier caps; and
  - Footings.

This review documents each of the concepts, provides a review of the associated research literature, and provides a review of the engineering and constructability evaluations, as well as pinpoints implementation challenges and provides suggestions to overcome those challenges. In addition, testing needs and future research are also discussed.

The review shows that the precast modular abutment and precast pier design concepts described in this report are worthy of promotion to Phase III implementation. With the results from this research project, and those in the future, the codification (development of code specifications) process of precast substructures will begin. Codification will lead to standardization, and standardization will give the design engineers a measure of comfort and liability protection. These tools will lead to greater designer acceptance. With designer acceptance, precast substructures will be proposed as solutions more frequently and owner acceptance will increase. With DOTs committed to the concept, contractors will begin to embrace the concepts as well.

As is proven with all new technologies, as acceptance and use increases, the costs will decrease and the product will evolve. The reduced costs will provide owners the tangible financial incentive necessary to pursue precast substructures for projects on a greater scale. This research project, as well as future projects, will provide the necessary stepping stones for this evolution. The precast substructure systems are design concepts on their way to codification, standardization, and implementation.

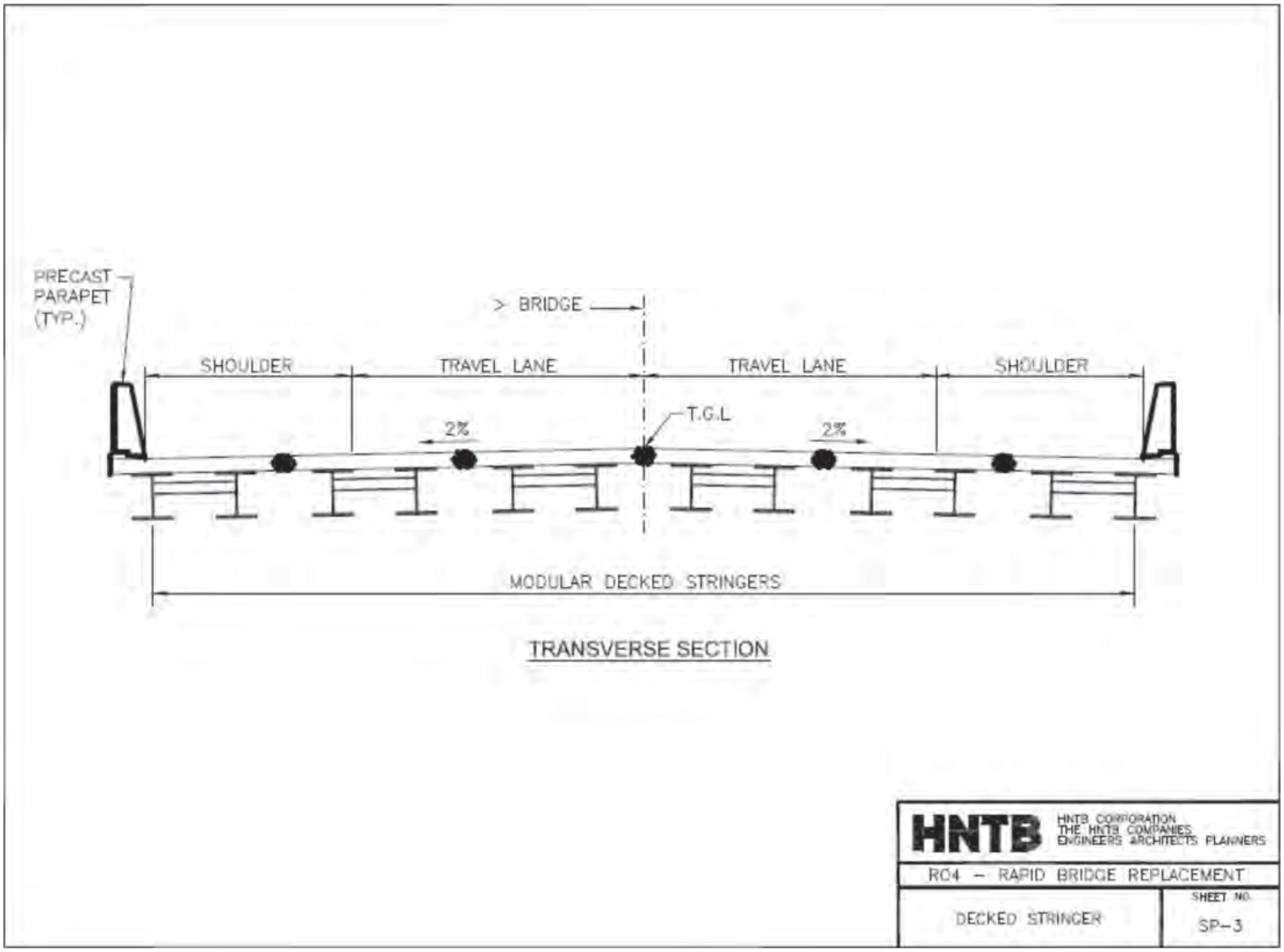


Figure 3.24. Modular decked stringers.

Design Concept Descriptions

Precast Modular Abutments

Precast modular abutments are composed of separate components fabricated off site, shipped, and then assembled in the field into a complete bridge abutment. Precast modular abutments have been constructed in several states. The current schematic details employed by the Utah Department of Transportation (DOT), have been used by the Utah DOT and are proven and complete.

The Utah DOT precast modular abutment details include a stub-type abutment on drilled shafts or piles and a cantilever abutment on spread footings. The Utah DOT makes use of an integral connection of the superstructure to the substructure. Since not all states use integral abutments, standards should be created for both integral and non-integral abutments. Also, non-integral abutments would be easier to reuse. In significant seismic zones, easily detachable seismic restraint devices may be used to connect the abutment to the superstructure to prevent the superstructure from losing vertical support during an earthquake. The abutment consists of standard-length cap sections, or wall sections for cantilever abutments, with a precast backwall attached by grouted splice sleeves. An example of a precast modular integral abutment can be found in Figure 3.36 and alternate connection ideas can be found in Figure 3.42.

The individual precast components should be designed to be shipped over roadways and erected using typical construction equipment. The precast components are made as light as is practical. Voids are used vertically in the cap section, or wall section, to reduce shipping weights and to allow for larger elements to be used. These voids are also used to attach drilled shafts or piles to the cap for stub-type abutments. Once the components are erected, the voids and shear keys are filled with high early strength concrete. Wingwalls are also precast, with a formed pocket to slide over wingwall drilled shaft

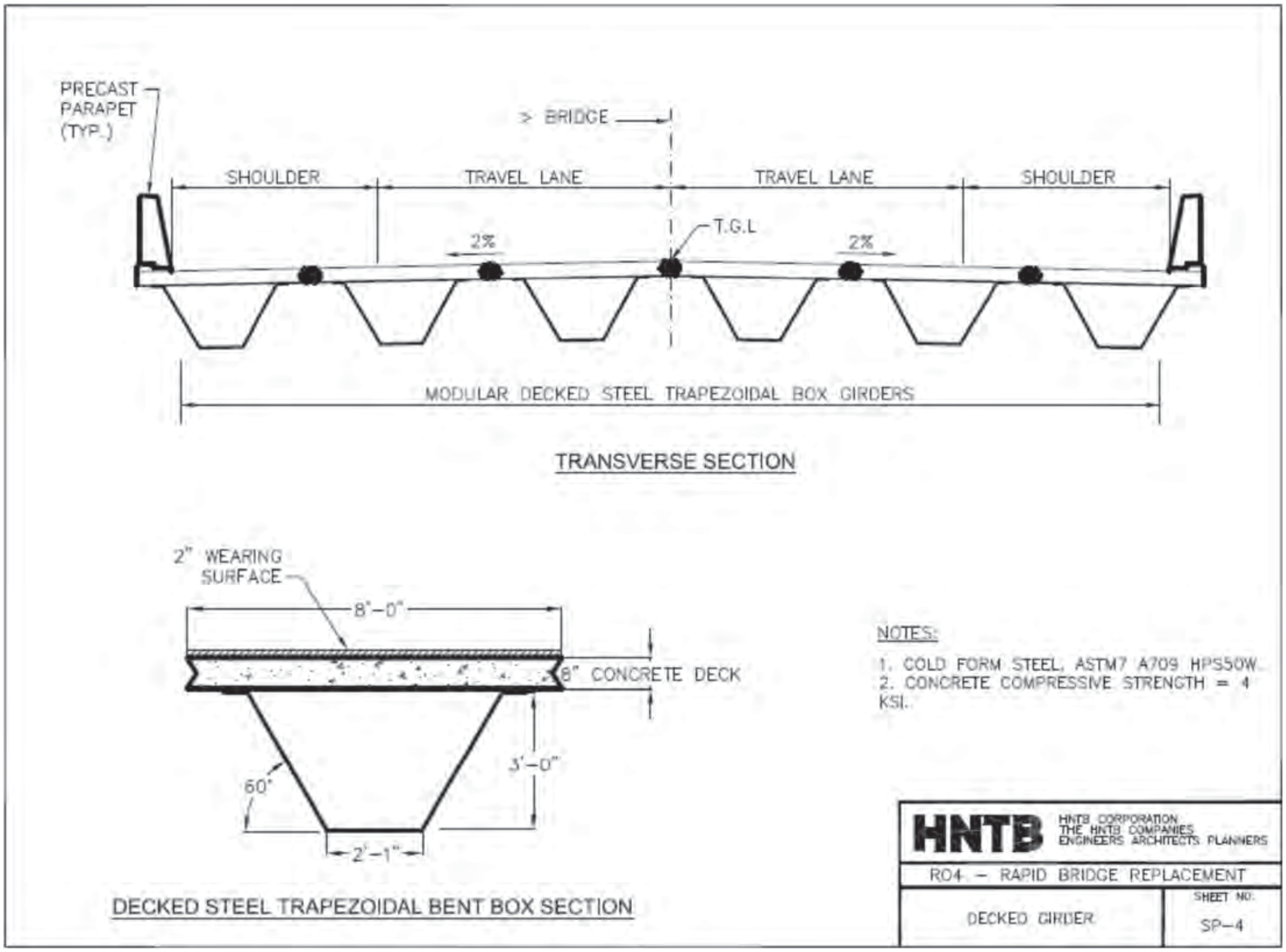


Figure 3.25. Modular decked steel trapezoidal box girders.

reinforcing. Once in place over the wingwall drilled shaft, the wingwall pocket is filled with high early strength concrete.

Precast Complete Piers

Precast complete piers are composed of separate components fabricated off site, and then shipped and assembled in the field into a complete bridge pier. While the Utah DOT includes standard schematic details for complete piers, they have not been used in their current state. Similar concepts are envisioned by the PCI Northeast Bridge Technical Committee and have been studied by the University of Alabama at Birmingham. Thus, the Utah DOT precast complete pier standard schematic details is a good place to begin.

Piers with single-column and multiple-column configurations are available. Foundations can be drilled shafts with precast footings or precast spread footings. Attached to the foundation by grouted splice sleeve connectors is a precast column. Precast columns are an octagonal shape, the top of which is connected by grouted splice sleeves to the precast cap. The precast cap is a standard rectangular shape. Many states employ the use of integral piers, so it is recommended that standards for both non-integral and integral piers be created. Also, non-integral piers would be easier to reuse. In significant seismic zones, easily detachable seismic restraint devices may be used to connect the pier to the superstructure to prevent the superstructure from losing vertical support during an earthquake. An example of a precast complete pier can be found in Figure 3.37, and alternate pier ideas and connections can be found in Figures 3.40 through 3.44.

Like the precast modular abutment, the components of the precast complete pier should be designed to be shipped over roadways and erected using typical construction equipment. The precast components are made as light as practical. Precast spread footings can be partial precast or complete precast components. A grout-filled void beneath the footing is used to transfer the load to the soil, avoiding unexpected localized point loads. Column heights and cap lengths will be limited by

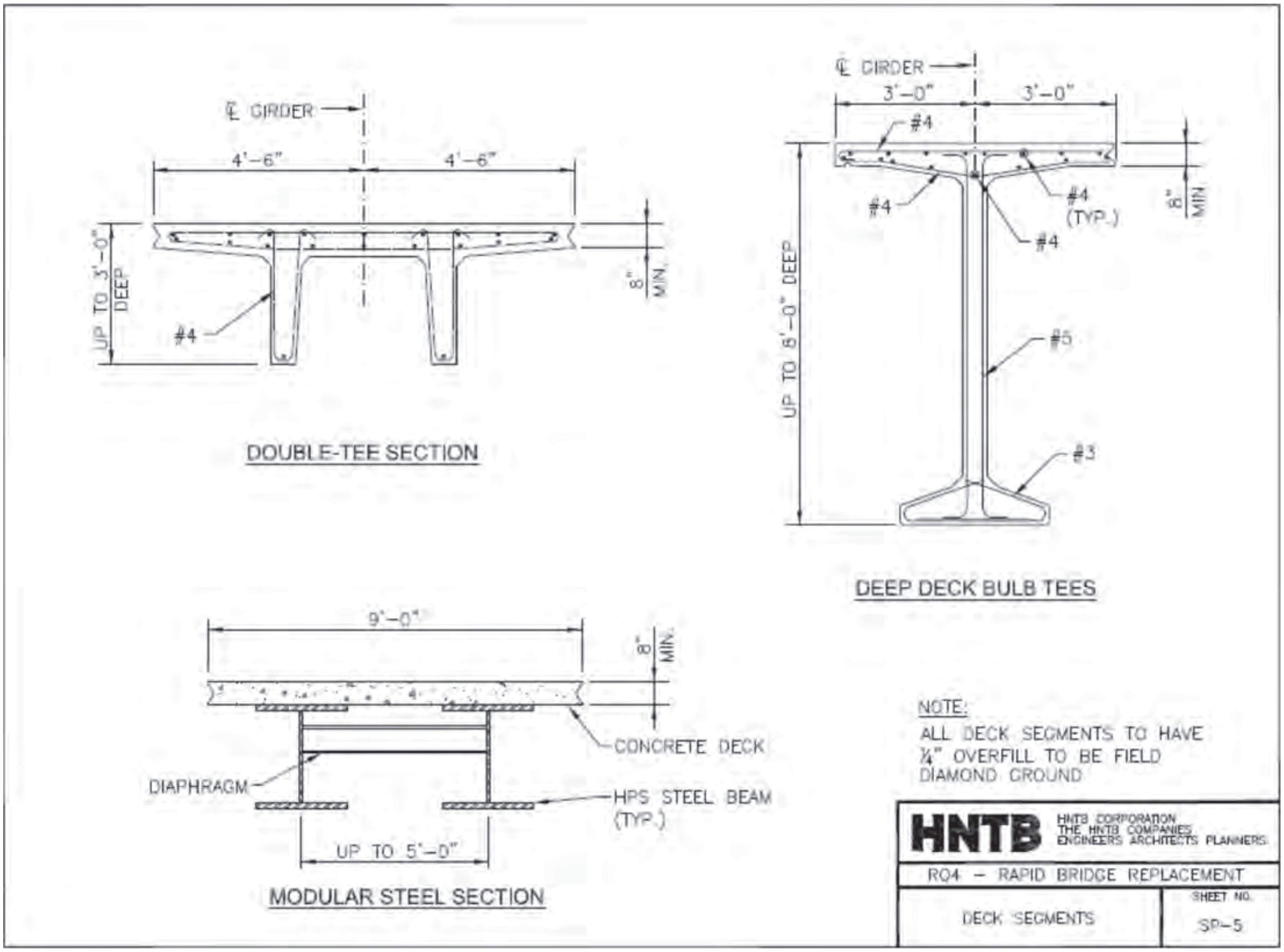


Figure 3.26. Deck segments.

transportation regulations and erection equipment. Alternatively, the cap length limitation can be avoided by using multiple short caps combined to function as a single pier cap. Precast bearing seats can also be used.

Segmental Columns and Piers

While it is preferable in precast concrete construction to have the columns fabricated in full-height segments, many times the need arises to fabricate them in several pieces for ease of transportation to the site and placement in the bridge substructure. Usually weight controls the maximum size of each segmental component for the columns.

Segmental columns consist of components of varying length, based on the demands of the design, that are stacked vertically until the designed height for the columns has been reached. Once they are in place, these column segments may be vertically posttensioned together and to the foundation for stability. For large-size piers, it is much easier to handle and erect these discrete components as compared with whole columns of equal heights.

Concrete segmental column components can be thin-walled hollow segments to reduce weight. They can be match cast to ensure proper alignment, as well as full contact of the concrete at the joints. The segments can also be mass-produced with a thin layer of mortar bed between segments. The mortar bed should be designed to resist the actual loads from design, to provide a thorough closure of the joint, and to be designed with proper creep and shrinkage characteristics. Shims can be used to maintain vertical alignment.

Engineering Evaluation

Precast Modular Abutments

Many pilot projects have been completed with precast modular abutments. The initial success of these projects has proved the viability of the design and construction procedures of precast substructures. Although these systems have been used,

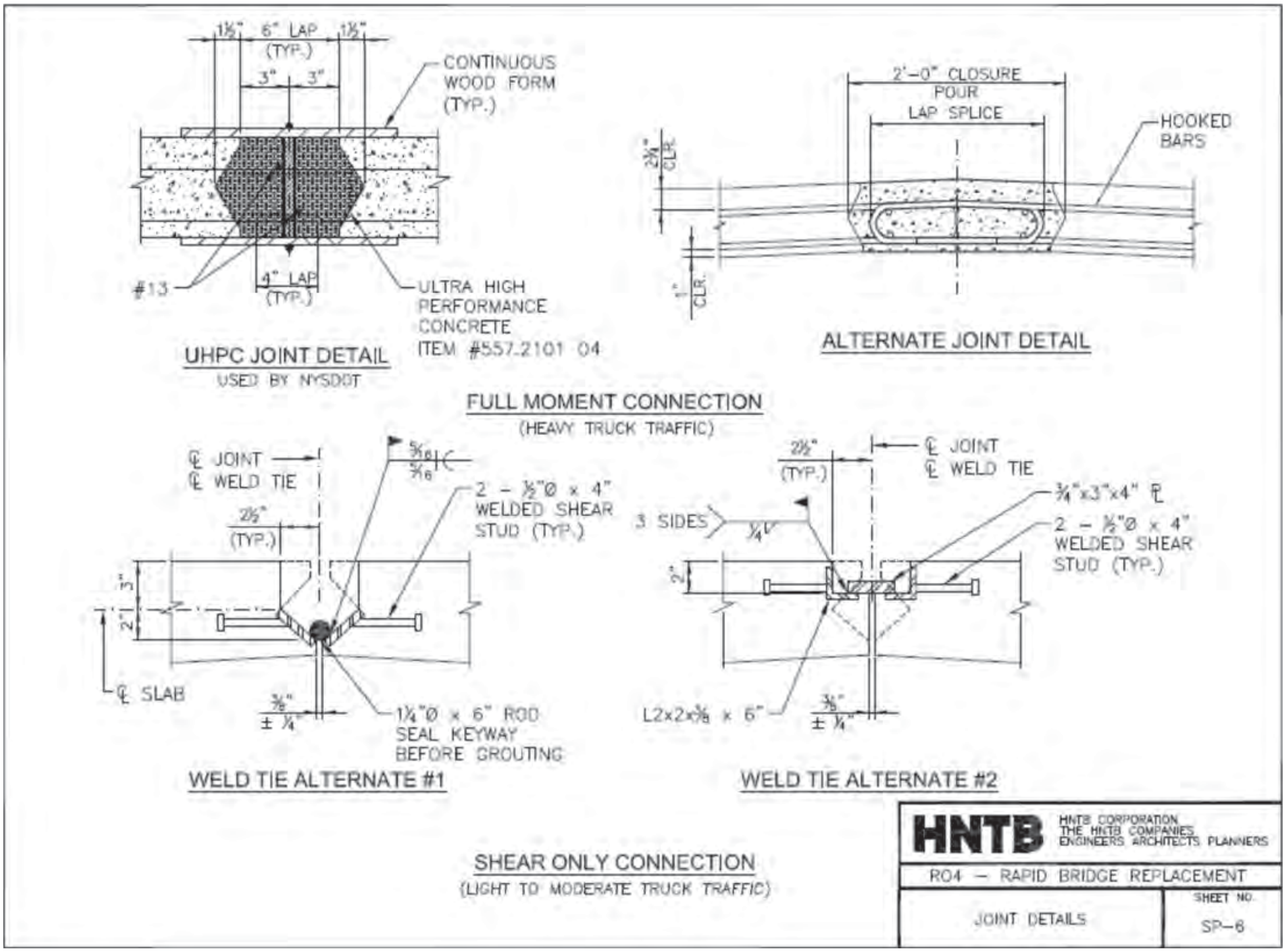


Figure 3.27. Joint details.

additional testing such as seismic response and strength tests of connections are recommended to gain confidence from designers nationwide.

Over the next several years, these structures will undergo the scrutiny of maintenance inspections. The evolution of these systems depends greatly on these inspections and on how durable these structures prove to be.

Research into the use of modern materials for the purpose of increasing ease of construction and improving durability is also recommended. Use of high-performance concrete, early strength concrete, and self-consolidating concrete may reduce section size and weight, improve strength, and ease construction.

The Utah DOT *Precast Substructure Elements Manual* (Utah DOT, 2010c) and the PCI Northeast Bridge Technical Committee's *Guidelines for Accelerated Bridge Construction* (PCINE, 2006) provide adequate guidance for the design of precast substructures, and the Utah DOT's standard schematic details with guidance on their use provides adequate direction on the development of design details for precast substructures. In addition, NCHRP Project 12-74 and its report, *NCHRP 681: Development of a Precast Bent Cap System for Seismic Regions* (Restrepo et al., 2011), and FHWA's *Connection Details for Prefabricated Bridge Elements and Systems* (Culmo, 2009) provide guidance on available connections. This report provides a short summary as well.

Currently, a design can be developed by using existing codes and design guides. However, additional information and commentary is suggested for the LRFD specifications concerning emulation design, mechanical couplers, and current seismic criteria. Any additional information in the LRFD specifications would contribute to a designer's confidence in the design process.

Precast Complete Piers

As with precast modular abutments, pilot projects have been completed using precast complete piers. Most precast complete piers are constructed by posttensioning the precast cap to the precast column. Additionally, precast caps on cast-in-place columns and complete segmental piers have been used and have proven to be successful on many projects. The initial success of these projects has proved the viability of the design and construction procedures of precast substructures. Although these systems have been used, additional testing for seismic response and strength of connections is recommended to gain confidence from designers nationwide.

The future of these systems also will be tested through their ability to prove durable and to withstand the scrutiny of maintenance inspections.

As in precast modular abutments, research into the use of modern materials to ease construction and improve durability is recommended. Using high-performance concrete, early strength concrete, and self-consolidating concrete all may reduce section size and weight, improve strength, and ease construction.

The Utah DOT precast substructure details presented in Figure 3.37 make use of the grouted splice sleeve connectors for mechanical connections in which 100% of the splice is at one location. Currently, this requirement would require special detailing for high seismic regions. The grouted splice sleeve connector does not depend on the surrounding concrete cover to develop its strength as with typical lap splices. The Utah DOT and other state DOTs are currently pursuing funding for additional research into the seismic behavior of these mechanical connections and funding to develop AASHTO code specifications.

While these details include the grouted splice sleeves, other connection methods are available. The designer should refer to the FHWA's *Connection Details for Prefabricated Bridge Elements and Systems* (Culmo, 2009) and NCHRP Project 12-74, and its report, *NCHRP 681: Development of a Precast Bent Cap System for Seismic Regions* (Restrepo et al., 2011), for additional information.

Here, too, designs can be developed using existing codes and design guides, but they would be greatly enhanced through additional information and commentary in the LRFD specifications concerning emulation design, mechanical couplers, and current seismic criteria.

Segmental Columns and Piers

Many projects—whether pilot projects or those completed for convenience and ABC purposes—have been successfully completed with segmental columns and piers. Successful completion of such projects has proven the viability and advantage of such applications. Although the application of segmental columns has been successful in the past, a few issues have arisen during the experience.

Some issues such as tendon corrosion are similar and apply to all types of segmental construction, whether horizontal or vertical, that took place prior to 2000. These issues are related to the location of the region within the columns where the posttensioned tendons were applied, the types of grout used for filling out the duct, poor workmanship, some inefficient detailing of the anchorages, and inadequate levels of protection that were required to protect the tendons and anchorages from moisture.

Match casting the segments is the preferred way of fabricating column segments because it avoids most of the issues discussed. Match casting results in an almost-perfect joint fit between the two adjacent components. It provides for tighter joints between components, better distribution of stresses across the joint, and easier erection on site, which accelerates on-site construction. Segments can be match casted vertically or horizontally.

While vertical casting of columns segments provides better quality of components, it usually results in a more costly product. It requires several handling and movement operations for the components. In addition, formwork depth is limited for long column segments.

Horizontal match casting has also been used efficiently. It avoids the repetitive handling and movement operations by casting the segments in line next to each other for the available length of the bed. Also, the length of the formwork does not impose any limitation to the length of the segments to be cast. But casting the segments horizontally has disadvantages. Some of the most popular cross-section shapes, such as the circular shape, can be difficult to cast horizontally and are therefore more costly. In addition, horizontally cast components can result in finishes that vary from rough to smooth within the same segment, which could be aesthetically unacceptable.

Segments can be produced in mass with a thin layer of mortar between segments. The number of segments that could be produced at one time with this procedure is limited only by the length of the bed and the number of formworks available. Although it is usually cheaper to fabricate the segments this way, many contractors prefer the match cast procedure of segment fabrication. This is mostly because there is a need to include connection hardware in the ends of the segments. In addition, erection of such segments requires formwork at each joint before the grout can be placed, which adds to the labor and time to erect these segments. The latter, combined with the additional time needed for the grout to cure before the placement of additional segments, reduces the advantage of the accelerated intent of the project.

Grouted joints may unintentionally be built with a nonuniform bearing surface. In such instances the column components may suffer stress concentration resulting in edge crushing, cracking, gaps, chlorite penetration, and ultimately corrosion of the mild steel and of the posttensioning.

Shear along the joint surface of the column is adequately resisted in most instances by shear friction. Shear keys can be used to enhance the shear resistance of the column at the joint location.

In segmental columns, posttensioning is designed to resist flexure in the column. Adequate posttensioning is usually required to avoid the opening of joints due to service load flexure. This requirement may impose excessive demand for initial compressive stress in the segments, thus diminishing available ductility, which is very important in seismic applications. Special care should therefore be applied when designing these systems in seismic regions to avoid crushing of the concrete from the imposed reversed cyclic loading.

Due to their integrity during transportation and erection, solid shapes would be preferred, where possible, to hollow thin-walled shapes that could be fragile and more easily damaged during handling. In addition, in instances in which segments are located within the limits of potential plastic hinges, solid shapes would be more suitable to handle the deformation demand, as well as to accommodate the confinement reinforcement that is required in such regions.

The segmental nature of this type of construction makes these applications very suitable for the use of innovative modern materials to make hybrid columns. For example, engineered cementitious composites (ECC) could be used to fabricate certain segments to be included within the overall column at potential hinge regions, thus maintaining the integrity of the component and the system considerably better than regular reinforced concrete can.

Small to moderate-size columns frequently used in standard applications around the country would be more suitable for standardized segmental construction than are larger-size or unique columns and pylons that are used sporadically or only in special projects. The benefits of standardization and precasting are obtained through multiple uses of the standards.

Segmental column applications have been used in many projects to date. These projects were intended to test the advantages of the systems in the field. Other applications were made to accelerate construction time in congested areas or were used where casting the concrete in place would have been difficult. Currently, there is a knowledge database of such system applications, including use in the field as well as testing in the lab.

While no specifications currently address segmental column design, such design could be made with existing applicable codes and guides such as the AASHTO LRFD Bridge Design Specifications and pertinent ASBI publications. It is recommended that AASHTO publications include appropriate language that would directly address the design of such systems as well as provide references of previous applications, connection details, and research performed to date.

Constructability Evaluation

Precast Modular Abutments

While specialized equipment and innovative erection methods can be effectively used with prefabricated complete superstructures, currently it is best to design the substructures so that their components can be shipped and erected using typical equipment. The benefits of using specialized equipment and innovative erection methods would have to be great to justify the additional cost. For precast modular abutments, the benefits would not justify the cost.

The designer should always consider state and local shipping size regulations, as well as erection load limits, for typical cranes. Consideration should also be given to the pavement capacity of local streets, if necessary. The following can be used as a guideline for sizing precast concrete substructure elements on design–bid–build projects:

- The width of the precast component and any projecting reinforcing should be kept below 12 to 14 ft.
- The height of the precast component and any projecting reinforcing should be kept to 12 ft or less for vertical clearance at existing bridges.
- The weight of each precast component should be kept below 100,000 lb to keep the size of cranes needed reasonable. Weights of 60,000 lb should be anticipated.

For ABC projects, the designer should work with both the fabricator and contractor to size the elements on the basis of the fabricator's equipment, the contractor's equipment, and the load limitations of local shipping routes. An assembly plan showing all elements and connections should be a requirement for all prefabricated bridge projects.

The R04 team estimates that an abutment with drilled shafts or spread footings can be built in about 6 days, from the beginning of the first concrete pour to the completion of an appropriate cure time. Alternatively, the team estimates that a precast abutment can be assembled on the site in about 3 days, from the beginning of the foundation construction to the completion of an appropriate cure time for closure pours and grouted connections. This results in a 50% reduction in construction time for the abutments.

Precast Complete Piers

Although specialized equipment and innovative erection methods can be effectively used with prefabricated complete superstructures, currently it is best to design precast complete piers so that their components can be shipped and erected using typical equipment. The benefits of using specialized equipment and innovative erection methods would have to be great to justify the additional cost. For precast complete piers, the benefits would not justify the cost.

As is the case with precast modular abutments, the designer should always consider state and local shipping size regulations, as well as erection load limits for typical cranes when designing precast complete pier components. If necessary, consideration should also be given to the pavement capacity of local streets. The following can be used as a guideline for sizing precast concrete substructure elements on design–bid–build projects:

- The width of the precast component and any projecting reinforcing should be kept below 12 to 14 ft.
- The height of the precast component and any projecting reinforcing should be kept to 12 ft or less for vertical clearance at existing bridges.
- The weight of each precast component should be kept below 100,000 lb to keep the size of cranes needed reasonable. Weights of 60,000 lb should be anticipated.

For design–build projects, the designer can work with both the fabricator and contractor to size the elements on the basis of the fabricator's equipment, the contractor's equipment, and local shipping routes.

The R04 team estimates that a cast-in-place pier with drilled shafts or spread footings can be built in about 3 weeks, from the beginning of the first concrete pour to the completion of an appropriate cure time. Alternatively, the team estimates that a precast complete pier can be assembled on site in about 1½ weeks, from the beginning of the foundation construction to the completion of an appropriate cure time for grouted connections. This results in a 50% reduction in construction time for bridge piers. The total substructure construction time savings increases with each additional pier. Since most bridges have multiple piers, the benefits of complete precast piers become clear.

Segmental Columns and Piers

Segmental columns and piers should be designed with as few components as possible. Fewer segments would allow for more accelerated construction and would improve the durability of the system, with fewer joints. However, the size of the columns combined with transportation and erection equipment limitations often dictate the number of segments that are needed for a particular application. Highway and waterway transportation limits must also be considered. Navigable waterways that could allow barging have fewer limitations than highways. The design should carefully evaluate at least one possible route that would potentially accommodate the transportation of components. Sizes of each component should be designed with consideration for all limitations, as well as what erection equipment is readily available to local contractors. However, if larger equipment would be needed for particular applications, the design could suggest the type of equipment to be used for that particular application.

Construction of the substructure may expend 60% to 70% of the total construction time required for a project. If properly designed, segmental columns could be very versatile and easy to erect, thus drastically reducing the time of construction. It is estimated that such systems could take a few days for smaller projects to about 2 weeks for larger projects. This would mean a time reduction of about 30% to 50% of the time needed to erect the same column as a cast-in-place application.

The use of segmental columns instead of single-piece precast column segments needs to be considered during the design phase so that the project will run smoothly during the construction phase. Smaller multi-column piers can be efficiently constructed with single-piece columns, whereas taller single-column and wall-type piers lend themselves to the segmental approach. Most routine bridges with column heights under 25 ft should allow for the fabrication and erection of the column as a single piece. It is not expected that segmental columns will see widespread use in ABC applications.

The size and weight of the precast element has much to do with this decision. When the column sections would start to exceed 100,000 lb, the availability of cranes and transporters needs to be considered. Segmental columns can keep the crane sizes and precast element weights reasonable and economical.

Implementation Challenges

Designer Perspective

A designer must provide a safe design. Safety is at the forefront of each and every design. It is natural for a designer to be comfortable with a design, or design process, that has been in use for many years and to be equally uncomfortable with the opposite.

A typical, or standard, design will help alleviate a designer's reluctance to use precast substructures. The precast modular abutment and precast complete pier can be standardized in a manner similar to how cast-in-place standards were created. Standardization should start with typical roadway widths and common skews, with limits on abutment and column heights that take the emulated design to their strength limits.

The Utah DOT precast substructure details presented in Figure 3.36 make use of the grouted splice sleeve connectors for mechanical connections in which 100% of the splice is at one location. The AASHTO LRFD specifications currently allow this only for structures in regions of the country with low seismicity (Zones 1 and 2). This requirement would require special detailing for highly seismic regions.

These details include the grouted splice sleeves, but other connection methods are available. The designer should refer to FHWA's *Connection Details for Prefabricated Bridge Elements and Systems* (Culmo, 2009) and NCHRP Project 12-74, and its report, *NCHRP 681: Development of a Precast Bent Cap System for Seismic Regions* (Restrepo et al., 2011), for additional information.

The Utah DOT and other state DOTs are currently pursuing funding for additional research into the seismic behavior of these mechanical connections and funding to develop AASHTO code specifications. This research project, and those in the future, will begin the codification (development of code specifications) process. Codification will lead to standardization, and standardization will give the design engineers a measure of liability protection.

The designer must also provide an economical design. One might think that given the size of the proposed project, the economic benefit of a precast substructure system can be readily determined. However, there is an unknown factor that the designer may not take into account when determining the cost of a precast substructure system. This unknown is the risk premium a contractor will factor into its bid.

Currently, a design can be performed using existing codes and design guides. However, clarification is needed in the LRFD specifications for how precast substructures would be handled. Additional information and commentary would contribute to a designers comfort in the design process.

Many designers are reluctant to propose or implement segmental column applications, even in projects in which the benefits of their use are readily obvious. There are no published documents that provide design standards for such applications. While engineers could combine creativity and their engineering judgment to implement such applications, doing so could open their work to unnecessary liability. In addition, as a result of the lack of design standards, such applications may require additional design time and adequate time to perform an independent peer review of the design. It is usually very difficult for the designer to be awarded additional time from the owner to design such applications.

All of the above are even more applicable for special designs such as seismic applications. While several studies have shown the adequacy of different segmental column applications, there is still some uncertainty as to the behavior of these columns with opening and closing of joints, behavior and serviceability of the posttensioning tendons, and available ductility when subjected to seismic reversible loading.

Current testing has demonstrated that emulation design of precast substructures is sufficient for use of precast substructures for most regions. Testing of emulation design for use in high seismic zones will be required for complete acceptance on a nationwide scale. NCHRP Project 12-74 has taken the first steps toward this goal. Refer to the section on connections for additional information.

Owner Perspective

Despite the success of prior projects, owners are still reluctant to pursue prefabricated substructures for bridge projects without a financial incentive to do so. This financial incentive will need to be actual dollars added to the DOT's budget, not just savings of user costs, which although substantial, do not provide adequate incentive in most cases. To encourage the use of ABC, such as precast substructures, perhaps the federal government could pay the states a substantial percentage of the additional cost of the ABC approach to remove the initial financial disincentives. Currently, there are two critical financial disincentives that contribute to owner reluctance. These are the initial cost increase and the unknown future life-cycle costs.

The initial cost increase can be rationalized if the user cost is included in the analysis, but in many cases this rationale is not adequate enough, since user cost cannot be recovered. The life-cycle costs depend on the long-term performance of the precast substructure. Thus, durability becomes a major concern. While precast elements by themselves are viewed as durable, it is the durability of the connections of these elements that needs to be confirmed. Although many of these connections have been used in commercial buildings for many years, an owner will want to see long-term, time-tested performance of the connections in bridge applications before confidence is gained.

Outside of these costs, other peripheral costs, such as the cost for future widening, the cost for future demolition or removal, and the cost for load ratings for overload permits, are also present. The precast substructure is designed to emulate cast-in-place construction. Thus, an analysis for overloads should not be more difficult than that of cast-in-place construction. Additionally, widening and removal of precast structures should not be more difficult than that of cast-in-place structures.

While segmental columns have successfully been implemented in several projects, there are still concerns about the durability of the components. The latter, combined with some findings of corroded tendons of segmental pylons in prime bridges such as the Sunshine Skyway Bridge in Florida, make owners more uneasy about these applications. Using actively applied preloading on the components and counting on that load to ensure the integrity of the overall system poses some uncertainty. Single columns especially are by default nonredundant elements, and the possibility of tendons corroding without being detected is a dreadful thought to many owners.

This research project, as well as those in the future, will begin the standardization process, and standardization will lead to repeatable, cost-effective, and constructible projects. The cost-effectiveness will give the owners the unassisted financial incentive they need to pursue precast substructures for their projects.

Contractor Perspective

The time savings of precast substructures over cast-in-place substructures could be as much as 50%, which is significant. The more piers involved, the greater the total time savings.

Contractors make money when their forces are busy and gainfully employed. Some contractors see an issue with the use of a subcontractor to supply the precast elements because it cuts into work contractor forces normally perform. Being able to do more projects per season may be the answer to keeping contractor forces working and turning a profit for the contractor. Keeping the details as simple as possible and allowing contractors to self-perform the precasting instead of subcontracting will also be attractive features. There is no reason to go to a precaster, because the more the contractor does itself, the more money it makes.

Self-Performing Substructure Precasting by General Contractors

This is an approach to ABC in which the designs allow maximum opportunities for the general contractor to do its own precasting at a staging area adjacent to the project site or in the contractor's yard with its own crews. Unless a precast is posttensioned or prestressed, there is no need for a precaster. Substructure components are made of conventional reinforced concrete that can be precast by the general contractor without the need to apply for any plant certification. Contractors have precast pier caps to the side of the project site in the past. Several states, including Nevada and West Virginia, do not have an established precast industry nor certified precasters, so precast girders come from elsewhere and incur an extra shipping expense. These states have contractors who are very good with CIP. Contractors self-performing the precast elements will overcome such impediments. The solution is to introduce the industry to precast technology and demonstrate its profitability.

Substructure components need to be designed to allow contractors to self-perform the precasting, giving special consideration to the following:

- Components that are simple enough to fabricate.
- Components that allow some tolerance for erection.
- Maximum repetition of components to reduce formwork cost.
- Component weights preferably not exceeding 50 tons, to allow easy transportation and erection. Heavier components can be used depending on contractor capabilities and site conditions.
- Substructure components that do not need prestressing or posttensioning in the field.
- Connection details that are easy to construct.

The use of precast elements for substructures has been impeded by the weight of components and by hauling. In addition, precast substructure elements do not provide much tolerance for field installation. Limitations on the cranes that can be used at a site are a major concern with prefabricated elements. A reasonable upper limit for such elements is 100 tons. Site access is a large problem, and the use of larger cranes is also more expensive. Trucking and lifting can be an issue with larger precast elements. Lightweight concrete mixes can be used to lighten sections, enabling geometrically larger section placements by the same cranes.

Designing for self-performing of precasting by the contractor requires closer collaboration between designer and contractor. States are often restricted in what they can do to engage the contractor during the design phase, which makes ABC more difficult. Some states have used construction manager/general contractor (CM/GC) contracting to improve this early communication without going to design–build. ABC is perceived as being more risky by contractors. Closer collaboration between designer and contractor will greatly reduce the risk premium that is often built into ABC bids.

Rapid construction requires the designer to spend a lot more time on site and to be instantly available in the design office to work with the contractor and DOT construction personnel to provide speedy responses. A close partnership between designer, owner, and contractor is a key to success for ABC projects. Such a collaborative approach is particularly important when precast substructures are involved and when field adjustments are far more likely to be required to accommodate site conditions and foundation issues.

Since precast elements are designed for transport over the road and erected with typical construction equipment, transportation and erection should not pose a problem. Particular attention needs to be paid to weight limits that may govern the constructability of large substructure components. An increase in transportation costs will occur if precasting is done off site, and an increase in time and cost for crane rental could occur.

Most precast substructure elements will require a crane for placement. Cranes require a level surface capable of withstanding the anticipated ground loads. The project site should lend itself for easy crane placement. If not, increased costs for crane platforms will add to the erection costs.

A contractor's reluctance to participate in this type of project stems from not knowing the risks involved and the ability to price that risk. The contractor is in business to make a profit. Take profit away and the contractor will follow. Utilizing past experience, a contractor will bid on a typical cast-in-place project with a good understanding of the tasks involved, how much those tasks cost, and how fast those tasks can be performed. When presented with a new process or procedure, the unknown risks become a factor to be included in the bid items and the actual profit margin will remain unknown until the end of the project.

Another reason for contractor reluctance is the potential requirement for investment in new equipment. This reluctance can be greatly reduced if the state departments of transportation were fully committed to the new construction techniques.

New technologies, such as grouting, posttensioning, and so forth, may be necessary to erect the segmental columns. The contractor may not be fully familiar with these technologies and they may not be available locally. Investment in training for new skills and technologies may be required, or it may be necessary to outsource these activities to outside subcontractors that may become future competitors.

Precasters may need to make initial investments in buying new beds for casting segmental components. They have to be trained in the new technologies. However, the sporadic nature of such projects at the current time does not justify the initial investments for acquiring the new equipment and skills if future similar projects are not a certainty.

Despite all of the issues enumerated above, contractors have shown interest in such future projects. During the focus group meetings, representatives from contractors and owners showed keen interest in this type of construction. Some comments indicated there is a clear advantage for using such applications in locations with seasonal work limitations and that the reduction of curing times would be very favorable. Preference was shown for the use of posttensioning bars instead of strands and tendons. Some concerns were expressed about the limitations of knowledge in posttensioning operations from some contractors, which could limit the number of contractors that could do the work.

Large design–build projects could potentially be a perfect opportunity to apply precast elements, mostly because the designer and contractor will work side by side to design and erect the precast systems.

This research project, as well as those in the future, will begin the standardization process; and for the contractor, standardization will lead to repeatable structures that are biddable, constructible, and profitable.

Connections

Background

Cast-in-place concrete structures are built with construction joints that usually involve lapped reinforcing bars. The principle of emulation design is to substitute an alternate connection that mimics or emulates the standard lap splice. For emulation design in seismic regions, the goal is for a prefabricated system to be comparable to a cast-in-place system in performance such as energy dissipation, ductility, stiffness, strength, and similar reliable failure modes.

During Phase II of this study and prior to the evaluation of technical information related to ABC connection details, previous Phase I information was updated and additional information was identified. Specifically, the sources of information for this report section include the following:

- Phase I report;
- *Connection Details for Prefabricated Bridge Elements and Systems* (Culmo, 2009) (hereafter referred to as "FHWA Connections Manual");
- NCHRP 12-74 project report, *Development of a Precast Bent Cap System for Seismic Regions* (Restrepo et al., 2011);
- *Guidelines for Accelerated Bridge Construction,* (PCINE, 2006); and
- Additional contacts with academia, state DOTs, and other pertinent federal agencies.

Most states believe additional testing of connection details is important. In particular, they are concerned with connection effectiveness (strength and stiffness) and durability. States in seismic regions noted that there is inadequate precast connection performance information.

Many state DOTs are just starting to consider using ABC concepts and are only beginning to investigate experimental testing (laboratory or field). A popular beginning point for states has been to investigate the use of full-depth deck panels and their connection to each other and to bridge girders. A general theme noted is that the testing research for connections is just scratching the surface and there are still testing needs. A few states indicated that while they would like better information, particularly with regard to long-term performance (durability), they will still proceed to implement an ABC concept when a situation calls for it.

Figure 3.28 shows a pier substructure with 10 connection types labeled (AP, A, B, C, D, E, F, G, H, and I). Detailed discussions for substructure connection types I, G, F, and E are provided in this section. The connections discussed were previously chosen from the FHWA Connections Manual and determined by the SHRP 2 R04 team to be the most useful for implementation. Subsequently, these connections (or similar types) were studied to identify previous laboratory test information. If such information was available, a brief description of the information is provided and a summary of additional laboratory testing needs is also provided. A discussion on the need for construction specifications is also included.

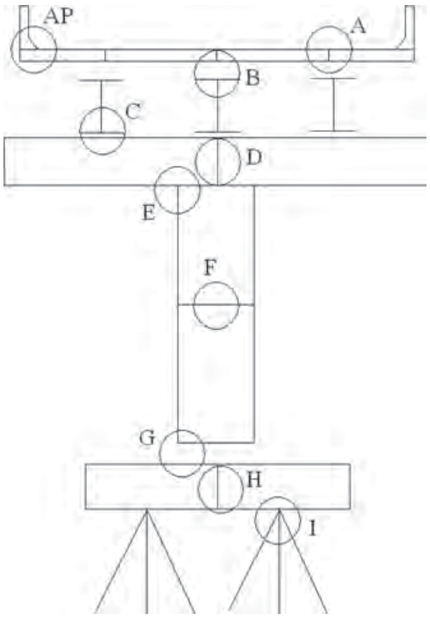


Figure 3.28. Precast component connection types for pier substructure.

Connection Type I

An important aspect of project acceleration is expediting the construction of bridge foundations and substructures. In traditional foundation construction, driven piles are frequently embedded in a cast-in-place footing. However, in an ABC environment, the use of precast footings could greatly speed construction. A reliable method of connecting driven piles to precast footings needs to be established. Additionally, the technology could be extended to not only driven piles (in groups) connected to a pile cap but more generically driven piles connected to integral abutments or bent caps (such as for pile bent construction).

The recently published FHWA Connections Manual includes numerous concepts for the connection of concrete columns to cap beams, prestressed piles to footings, and details to connect steel H-piles and pipe piles to concrete footings or bent caps. Some details are provided and generally consist of forming a void to confine the piled head, which is then backfilled with concrete. The size and configuration of these blockouts tends to vary widely.

The majority of details proposed at this point for connecting steel piles to pile caps involve leaving an individual void for each pile or a larger void to capture the entire pile group. Some suggested details are provided for both piles subjected to uplift and those without uplift. It is undetermined to what level testing has progressed to provide the current recommendations.

To date, very few prefabricated footing systems have been constructed. The Northeast PCI Bridge Technical Committee has developed conceptual details for the connection of a spread footing to steel piles. The major issues with a pile-to-footing connection are whether or not there is anticipated uplift on the piles or if there is a need to provide moment capacity in the pile connection. Uplift capacity can be achieved by welding reinforcing steel to the pile end and embedding the reinforcement in a closure pour (note that weldable reinforcing steel is required for this connection). Moment capacity is achieved by embedding the pile top at least 12 in. into the footing. An example for the pile supported precast footing is shown in Figure 3.29.

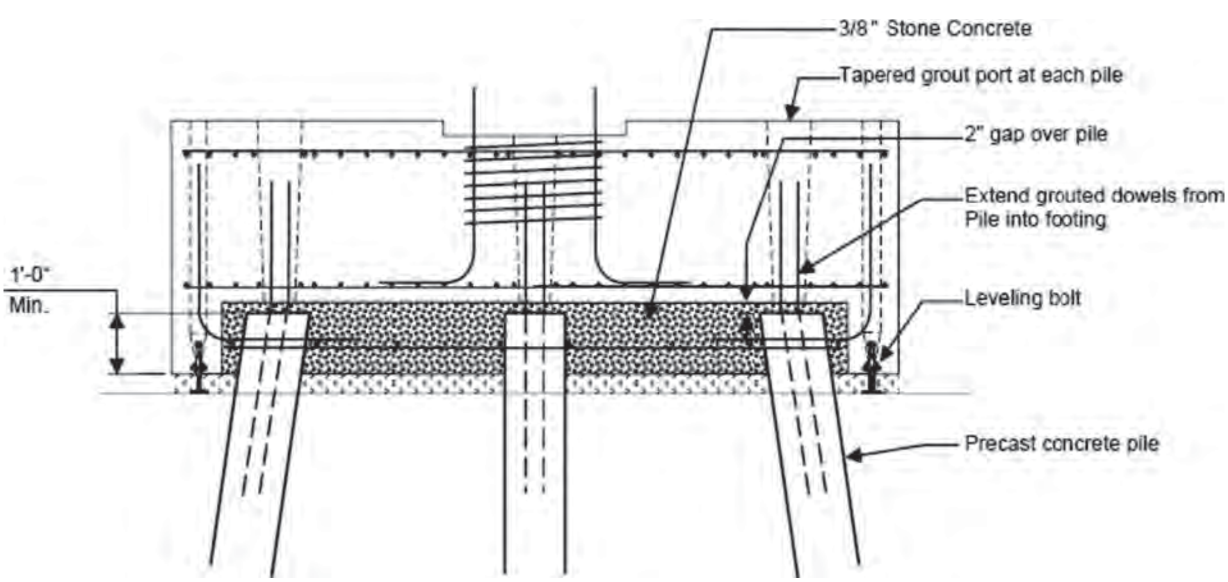


Figure 3.29. Pile-supported precast footing with uplift.

No literature could be identified that documented previously completed testing of these details. It is anticipated, however, that these details will perform just as well as cast-in-place footings.

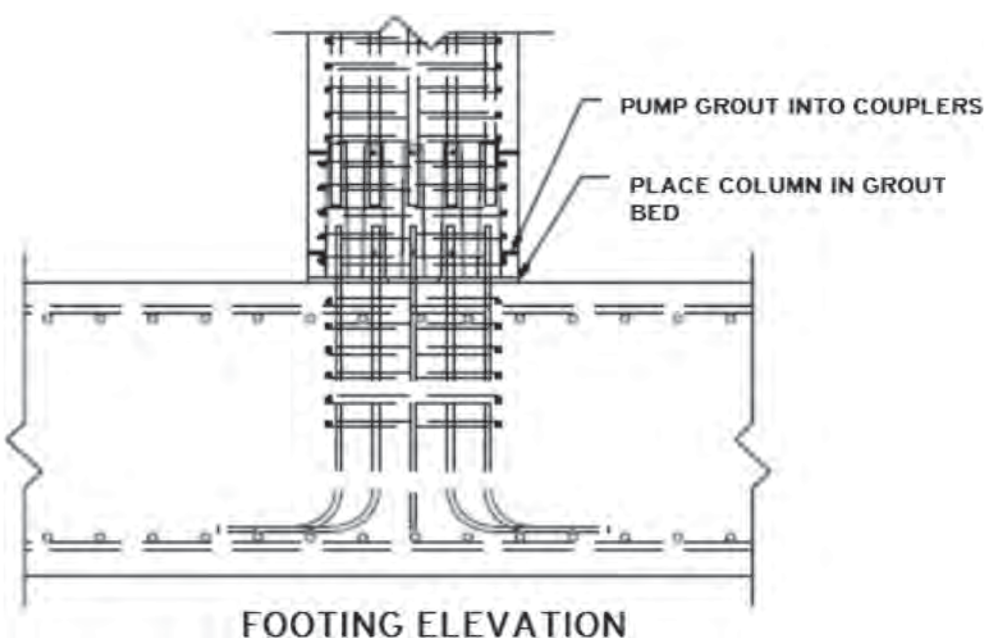
Connection Type G

A review of the FHWA Connections Manual for footing to bottom-of-column precast connections was completed. The noteworthy Type G detail in the manual is Detail 3.1.4.2 B (Utah, Precast Column to Precast Spread Footing, FHWA Connections Manual, page 3-59), as shown in Figure 3.30.

It should be noted that the specific Detail 3.1.4.2 B had not been implemented per information contained in the FHWA Connections Manual, and specific literature could not be identified that documented previously completed testing of these connection details. However, literature related to other connection details (precast column to precast spread footing) was identified and evaluated. Details for connection of precast column to cast-in-place footings or pile caps follow one of two general approaches.

- The first approach involves temporarily supporting the column base segment with mild steel reinforcement projecting from the bottom and casting the footing around the column. This approach has been applied in practice in Washington, but no research was found in the literature regarding laboratory or field testing of such a connection. There is some literature related to analytical modeling, which did contain suggested specific experimental research needs. The requirement for temporary support introduces obvious construction and alignment issues. While strength would not appear to be at issue with this detail, durability concerns, especially in terms of moisture penetration, and seismic concerns may be raised.
- The second general approach involves the use of proprietary grouted coupler systems to splice reinforcement (either mild steel or posttensioning bars) at the joint between the column and footing. While tensile strength testing has been performed for specific coupler systems, and strength performance of similar details applied at column-to-pier cap tests are encouraging, little research was found in the literature to quantify behavior of the joint detail in terms of below-grade durability. This detail has been applied in low seismic regions, but durability in general, as well as development issues for seismic applications. should be investigated. In terms of constructability, careful tolerance control is required to ensure adequate alignment of spliced reinforcement.

A third approach that involves casting a socket in the footing to receive the column base has been tested in the laboratory and has been applied widely in building construction in Europe. The annular cavity between the socket and column base is filled with a flowable epoxy grout once the column is appropriately aligned. This detail has proven robust in laboratory testing under cyclic loading and would likely provide a high degree of protection against moisture ingress.



Source: Culmo, 2009.

Figure 3.30. Column-to-footing connection using grouted splice coupler.

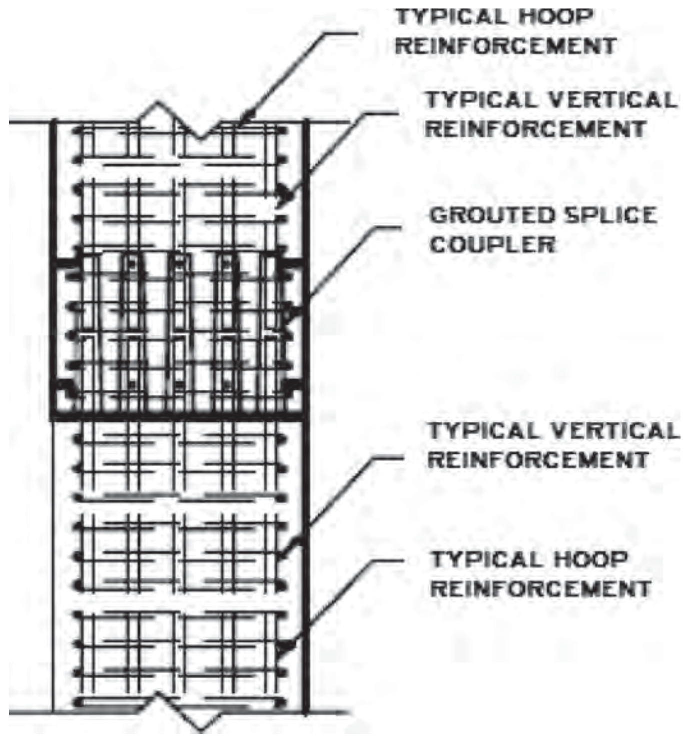
Seismic performance of grouted sleeve couplers is a research area that needs to be further explored before the couplers' use in high seismic zones can gain acceptance. The use of these connectors is common in the precast building market and is being increasingly promoted for bridge construction using precast elements as well. Notable projects such as the Mill Street Bridge replacement in New Hampshire, the Edison Bridge in Fort Meyers, Florida, and many others have made use of this simple connection made in the field by grouting a coupler connecting two bars in lieu of a lap splice or other mechanical system. The use of these couplers is commonly noted in the FHWA Connections Manual. Additionally, the Utah DOT depicts such a mechanical joining concept for its development of statewide ABC standards.

There are no design issues with the use of these couplers in nonseismic or low seismic regions. The underlying problem is the current AASHTO prohibition on splicing 100% of the reinforcing steel in a single plane for Seismic Zones 3 and 4. The requirement is that no more than 50% of the bars be spliced in a single plane and the remaining bars are at least 24 in. away. This would require deeply embedded couplers, and the likelihood of constructability problems would potentially be greater. The question is whether such a prohibition is warranted.

These coupler systems have been extensively used in Japan and other high seismic locations around the world. The field performance has been reported to be excellent. Some states are still reluctant to accept the concept of splicing all the bars in a single plane. Their additional concerns are that the stiffness of the coupler assembly will produce short plastic hinge lengths and inadequate ductility. Recommended testing program of various axial-flexural tests would need to be conducted with multiple bars joined in a single plane to assess the ductility and overall seismic performance of precast elements connected with grouted splice sleeves.

Connection Type F

Connection types listed in the FHWA Connections Manual were reviewed. The most mature Type F connection identified is Detail 3.1.2.1 B (Utah DOT, Precast Pier Column Section to Precast Pier Column Section, FHWA Connections Manual, page 3-45), as shown in Figure 3.31.



Source: Culmo, 2009.

Figure 3.31. Column-to-column connection using grouted splice coupler.

Detail 3.1.2.1 B is listed in the FHWA Connections Manual as "Under Development." It appears, however, that this connection has been used since publication. Published literature that documents experimental evaluations of Detail 3.01.2.1 B does exist. Generally, the experimental descriptions indicate that the evaluations were more of a constructability nature. In some cases, however, the constructability was at least partially evaluated using strength type tests. Although very little information on the specific application of Detail 3.1.2.1 B was identified for connection Type F, similar details have been applied in other connection types and experimental results do exist for these other connection usages. Concerns about future inspection of this type of connection exist.

Connection Type E: Pier Column–to–Pier Cap Connection

Considerable research has been conducted to develop reliable, constructible precast connection details. However, implementation in seismic regions has been limited, primarily due to uncertainty in the seismic performance of these connections.

New research has been conducted under NCHRP 12-74, Development of Precast Bent Cap Systems for Seismic Regions, to validate the expected performance of various precast substructure column-to-cap connection types for use in all seismic regions in the United States. The research used a CIP control specimen designed in accordance with the *AASHTO LRFD Bridge Design Specifications, 3rd ed.,* 2004 with 2006 Interims, and the 2006 *Recommended LRFD Guidelines for the Seismic Design of Highway Bridges.* The control specimen was classified as seismic design category (SDC) D and designed, detailed, and tested as such. The precast specimens used the same design and testing basis as for the CIP control specimen. The performance of the precast specimens was then compared with that of the CIP control specimen. Many connection types were tested, including the grouted duct connection and the cap pocket connection. The grouted duct connection and the cap pocket connection were tested for seismic performance; however, the grouted splice sleeve was not tested. Descriptions of the grouted splice sleeve connection, the grouted duct connection, and the cap pocket connection are discussed below.

Grouted Splice Sleeve (or Grouted Reinforcing Splice Couplers)

The Utah DOT precast complete pier details make use of the grouted splice sleeve connectors. According to the FHWA Connections Manual, grouted splice sleeves are produced by several manufacturers. They are hollow-cast steel sleeves similar to a pipe. The sleeve, preferably used in the vertical direction, is cast into the end of one element and a protruding reinforcing bar is cast in the end of the adjacent element. The elements are connected by inserting the protruding bars from one element into the hollow end of the coupler in the other element. The joint between the pieces is then grouted, and grout is pumped into the couplers to make the connection. An illustration of a grouted splice sleeve connection is shown in Figure 3.32.

Illustration

These connections have been thoroughly tested and can develop as much as 125%, 150%, and even 160% of the specified yield strength of the reinforcing bars. However, only limited testing has been performed to determine their behavior under seismic conditions. The Utah DOT precast complete pier details make use of grouted splice sleeve connectors in which 100% of the splice is at one location. The AASHTO specifications allow for splicing 100% of the longitudinal bars with mechanical splices at one location for low to moderate seismic zones. Additionally, NCHRP 12-74 has recommended that the grouted splice sleeve connection be used for limited-ductility connection applications only.

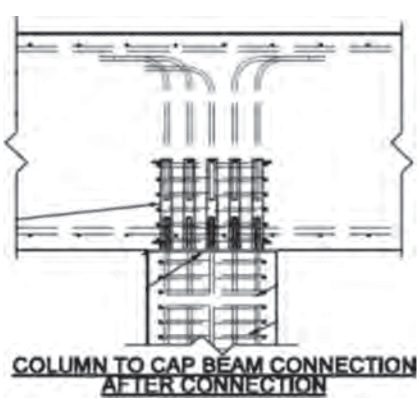
The FHWA Connections Manual suggests the splicing limitation of this connection may be overcome with special detailing. The Utah DOT and other state DOTs are currently pursuing funding for additional research into the seismic behavior of these mechanical connections and to develop AASHTO code specifications relating to their use.

Grouted Duct (or Grouted Posttensioning Ducts)

Several states have experimented with the use of grouted duct, also known as grouted posttensioning ducts, for connections between precast concrete elements. These connections are similar to grouted reinforcing splice couplers (grouted splice sleeves) in that reinforcing bars or threaded rods are inserted into a sleeve made up of standard posttensioning duct. The difference is that the duct is nonstructural; therefore, additional confinement reinforcing is required around the pipe to develop a significant connection. The posttensioning duct is much larger than a grouted coupler; therefore, tolerances are not as strict. A photograph of a grouted duct connection is shown in Figure 3.33.

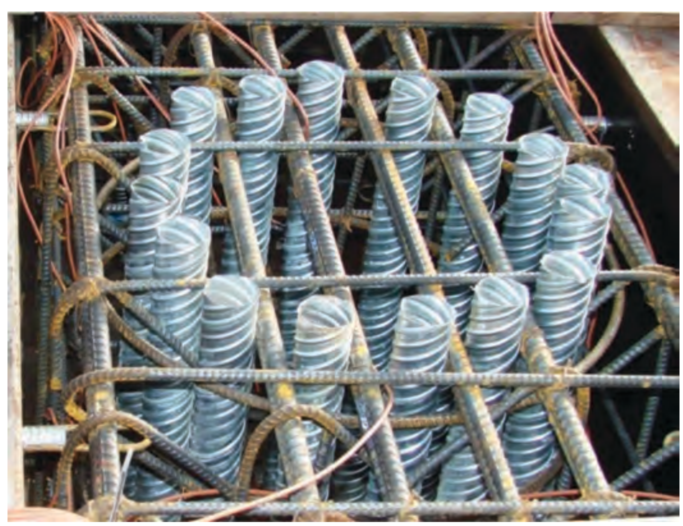
While the FHWA Connections Manual indicates that this connection was not recommended for high seismic areas at the time of publication, it does concede that research is ongoing (NCHRP 12-74) about precast connections for use in all seismic regions. More recently, NCHRP 12-74 has concluded that a grouted duct connection can be used for all seismic regions.

The findings of NCHRP 12-74 concluded that the grouted duct specimen satisfied the performance goal of the seismic design, achieving an extensive drift without appreciable



Source: Culmo, 2009.

Figure 3.32. Grouted splice sleeve.



Source: NCHRP 12-74 (Restrepo et al., 2011).

Figure 3.33. Grouted duct connection.

Source: NCHRP 12-74 (Restrepo et al., 2011).

Figure 3.34. Cap pocket connection.

strength degradation and exhibiting extensive plastic hinging of the column, limited joint distress, and essentially elastic behavior of the bent cap. Additionally, the report found that the emulative performance is concluded on the basis of closely matching overall behavior to the CIP control specimen, including lateral force–displacement response; plastic hinging; joint shear stiffness; level of joint distress; pattern of joint cracking; strain patterns of bent cap and joint reinforcement; integral behavior between the bedding layer, column, ducts, and bent cap; and minor bar slip.

Cap Pocket

The cap pocket connection uses a medium-diameter steel pipe to create a void in the bent cap to house the column bars and serve as a stay-in-place form and equivalent joint hoop reinforcement. Normal-weight concrete is placed in the bent cap void and bedding layer to anchor the column bars. The cap pocket connection studied under NCHRP 12-74 used a single 18-in. nominal-diameter 16-gauge steel pipe. Photographs of a cap pocket connection are shown in Figures 3.34 and 3.35. The report concluded that the cap pocket connection can be used for all seismic regions.

Source: NCHRP 12-74 (Restrepo et al., 2011).

Figure 3.35. Cap pocket connection close-up.

The findings of NCHRP 12-74 concluded that the cap pocket (full ductility) specimen satisfied the performance goal of the seismic design, achieving an extensive drift without appreciable strength degradation and exhibiting extensive plastic hinging of the column, limited joint distress, and essentially elastic behavior of the bent cap. Additionally, the report found that the emulative performance is concluded for the cap pocket (full ductility) specimen on the basis of closely matching overall behavior to the CIP control specimen, including lateral force–displacement response; plastic hinging; joint shear stiffness; strain patterns of bent cap longitudinal reinforcement; integral behavior between the bedding layer, column, pipe, and bent cap; and minor bar slip.

Connection Use

On the basis of the results of NCHRP 12-74 and the information provided in the FHWA Connections Manual, the team believes the grouted splice sleeve, the grouted duct, and the cap pocket connections emulate cast-in-place construction and can be used for precast substructure column-to-cap connections for various seismic regions in the United States. Table 3.5 is a summary for precast substructure column-to-cap connection use expected on implementation of the recommendations of NCHRP 12-74.

Note that NCHRP 12-74 has recommended the grouted splice sleeve for limited-ductility applications only. While the grouted duct and the cap pocket can be used for both limited-ductility and full-ductility applications, the detailing requirements between limited and full ductility may differ. Refer to NCHRP 12-74 for design and detailing examples of the grouted duct and cap pocket connections, as well as for design flowcharts. Also note that NCHRP 12-74 tested a limited-ductility cap pocket connection in addition to the full-ductility cap pocket connection. However, the performance of the limited-ductility specimen did not match the expressed intent of Article 4.7.1 of the 2009 *AASHTO Guide Specifications for LRFD Seismic Bridge Design.*

While the results from NCHRP 12-74 tested precast substructure column-to-cap connection types, the team

Table 3.5. Connection Types for U.S. Seismic Regions

Column-to-Cap Connection Type	Seismic Design Category
Grouted splice sleeve[a]	A, B, C
Grouted duct	A, B, C, D
Cap pocket[b]	A, B, C, D

[a] NCHRP 12-74 has recommended use for limited-ductility applications only.
[b] NCHRP 12-74 tested both a limited-ductility and a full-ductility cap pocket connection.

recommends that additional testing be conducted for foundation-to-column connections. It also recommends that the grouted splice sleeve connection be further studied and tested for use in high seismic regions. Grouted splice sleeves should be subjected to characteristic cyclic loading (both seismic and non-seismic). Although more expensive, some specimens used in the cyclic and durability testing programs should be full-size specimens. In scaled specimens, it is frequently difficult to get the reinforcement in the desired locations, obtain the desired cover, and so forth.

Proposed construction specifications for precast systems and connections could be developed specifically for projects in which accelerated bridge construction is one of the project goals. The construction specifications could be based on the testing results as well as on field observations of precast assembly erection from the demonstration project. The construction specifications could describe both materials (e.g., hydraulic cement, grout, connection hardware) and methods (grouting, placement, testing).

Girder Connections at Piers for Seismic Regions

The superstructure of a bridge is intended to remain elastic during an earthquake, and seismic superstructure designs are similar to non-seismic designs. Therefore, the precast concrete superstructure designs developed for non-seismic areas can be implemented in the seismically active western United States. Unlike the superstructure, the substructure can experience large inelastic deformations during an earthquake. Often, the frame action in the transverse and longitudinal directions is the primary mechanism for resisting seismic loads. Engaging the superstructure and substructure in the longitudinal direction is necessary to mobilize the frame action. Proper seismic design of precast piers entails a detailed evaluation of the connections between precast components (as discussed previously), as well as the connection between superstructure and the supporting substructure system.

Monolithic action between the superstructure and substructure components is the preferred approach for the design of seismic-resistant precast concrete bridge systems. Plastic hinges are expected to form at the ends of the columns during an earthquake. Ductile action of the bridge during an earthquake event is achieved through plastic hinging in the columns. Plastic hinges may be formed at one or both ends of a reinforced concrete column. After a plastic hinge is formed, the seismic loads are redistributed until the second plastic hinge is formed. If there is no monolithic action between the superstructure and the bent cap in precast construction, either the girder seats or the column tops will act as pinned connections. This requires that for stability in the longitudinal direction, the column bases will need to be fixed to the foundation supports, which places substantial force demands on the base of the columns and the foundations, particularly in moderate to high seismic zones. Developing a moment connection between the superstructure and substructure makes it possible to reduce moments at the base of the column.

An experimental research program at the University of Washington has developed and evaluated details for a precast concrete bridge bent substructure system with satisfactory seismic performance and suitability for rapid construction. The proposed system uses a small number of large bars grouted into ducts much larger in diameter to achieve the connection between components so that the substructure can be rapidly constructed. This product innovation, developed through Highways for LIFE programs, is intended to create a design methodology and details and includes laboratory testing to ensure that the detail can be deployed in varied applications of ABC in seismic regions.

Testing Needs

Several different areas of testing needs have been identified during this research project. Some of these needs are currently being addressed by researchers around the country; a few other areas will need to be examined in the future.

The use of self-consolidating concrete has great potential when producing precast substructure elements. Several states have done research on the use and properties of this material. Regional differences in aggregate and other concrete elements will likely require some localized research to develop strength, shrinkage, and creep characteristics for the local application of self-consolidating concrete. The strength component is critical for substructure applications. The shrinkage and creep issues are less of a concern than they would be for a superstructure beam made from self-consolidating concrete. Local precast suppliers will need to work with their DOTs to establish mix designs that meet the structures' strength requirements.

Joint systems have been tested for strength-related needs on grouted splices and grouted metal sleeves. The results of this test have shown good strength performance for these joints. Extending this research into the performance of these joints under seismic loads is needed. To use the ABC approach in seismically active regions, an extensive testing program is needed. This testing should show how the different joints react to cyclic loads and determine if the needed joint plasticity can be achieved for the precast substructure elements to be prescribed in seismically active regions of the county. NCHRP Project 12-74, Development of Precast Bent Cap Systems for Seismic Regions, has taken the first steps toward this goal. Refer to the earlier section on connections for additional information. However, the team believes that testing the grouted splice sleeve for high seismic applications would be beneficial to expand the connection toolbox. In addition, testing various foundation-to-column connections for high seismic regions is also needed.

The acceptance of precast elements, such as prestressed concrete girders, by state DOTs has been well established for more than 50 years. Most state DOTs report that they are comfortable with precast concrete in individual elements. Issues arise when the precast elements are assembled, as problems have been documented in joints between elements and in posttensioning systems, when they have been employed. Further proof testing in an accelerated environment may be needed to show the longevity of the currently proposed joint systems. One recurring query from the DOTs was, Will the precast system last as long as the tried-and-true cast-in-place alternate? Proving the longevity of the precast joint systems will do much in moving the precast system toward implementation in many states.

Summary and Recommendations

Precast modular abutments, precast complete piers, and precast segmental piers are design concepts worthy of promotion to Phase III implementation. Sufficient design guidance and details exist and can be used to form the basis of nationwide standard designs. Standard drawings should be created for both integral and non-integral abutments and piers. Standardization should start with typical roadway widths and common skews. Standard details for alternate connections should also be included. The precast substructure elements should be designed so that they can be shipped and erected using typical equipment.

Over the last 10 years, many successful segmental pier projects have been completed. Sufficient design guidance and details now exist and can be used for ABC Designs. The use of segmental columns over the possible use of single-piece precast column segments needs to be considered during the design phase. Smaller multi-column piers can be efficiently constructed with single-piece columns, whereas taller single-column and wall-type piers lend themselves to the segmental approach. Most routine bridges with column heights under 25 ft should allow for the fabrication and erection of the column as a single piece. It is not expected that segmental columns will see widespread use in ABC applications. Standardizing their designs for ABC use is not recommended.

This project takes the approach that for ABC to be successful, ABC Designs should allow maximum opportunities for the general contractor to do its own precasting at a staging area adjacent to the project site or in the contractor's yard with its own crews. This is particularly true for substructure components that have traditionally been constructed by contractor crews. Substructure components are made of conventional reinforced concrete and can be precast by the general contractor. Components will be designed to allow the contractor to self-perform the precasting by paying special consideration to the following:

- Components that are simple enough to fabricate.
- Components that allow some tolerance for erection.
- Maximum repetition of components to reduce formwork cost.
- Component weights preferably not exceeding 50 tons.
- Substructure components that do not need prestressing or posttensioning.

The R04 team recommends testing and research be promoted for developing specifications and commentary concerning emulation design of current cast-in-place elements by using mechanical couplers and required seismic criteria. Codification will provide designers with needed knowledge and liability protection, freeing them to propose precast substructure solutions in greater numbers. The benefits of precast substructures can be emphasized to owners, and with increased exposure to solutions from designers, owners can be better informed, leading to a technical acceptance of the concept.

This research project begins the standardization process, and standardization will lead to repeatable, cost-effective, and constructible projects. The eventual cost-effectiveness will give the owners the incentive they need to pursue precast substructures for their projects. As long as the owners are committed to precast substructures, contractors will be committed as well. When the tipping point is reached, the risks will be known and contractors can bid on precast substructure projects with the knowledge and experience needed to reduce or eliminate the risk premium associated with current precast substructure projects.

Exhibits

Different types of substructure configurations and details are shown in Figures 3.36 through 3.45.

Part 3: Evaluation of ABC Construction Technologies

Overview

The objective for the SHRP R04 project, Innovative Bridge Designs for Rapid Renewal, is "to develop standardized approaches to designing, constructing, and reusing complete bridge systems that address rapid renewal needs and efficiently integrate modern construction equipment."

While other parts of this Phase II report have focused on prefabricated substructure and superstructure components, the primary focus of this part will be on the application of proven and new construction technologies used for the rapid completion of bridge projects that employ either conventional

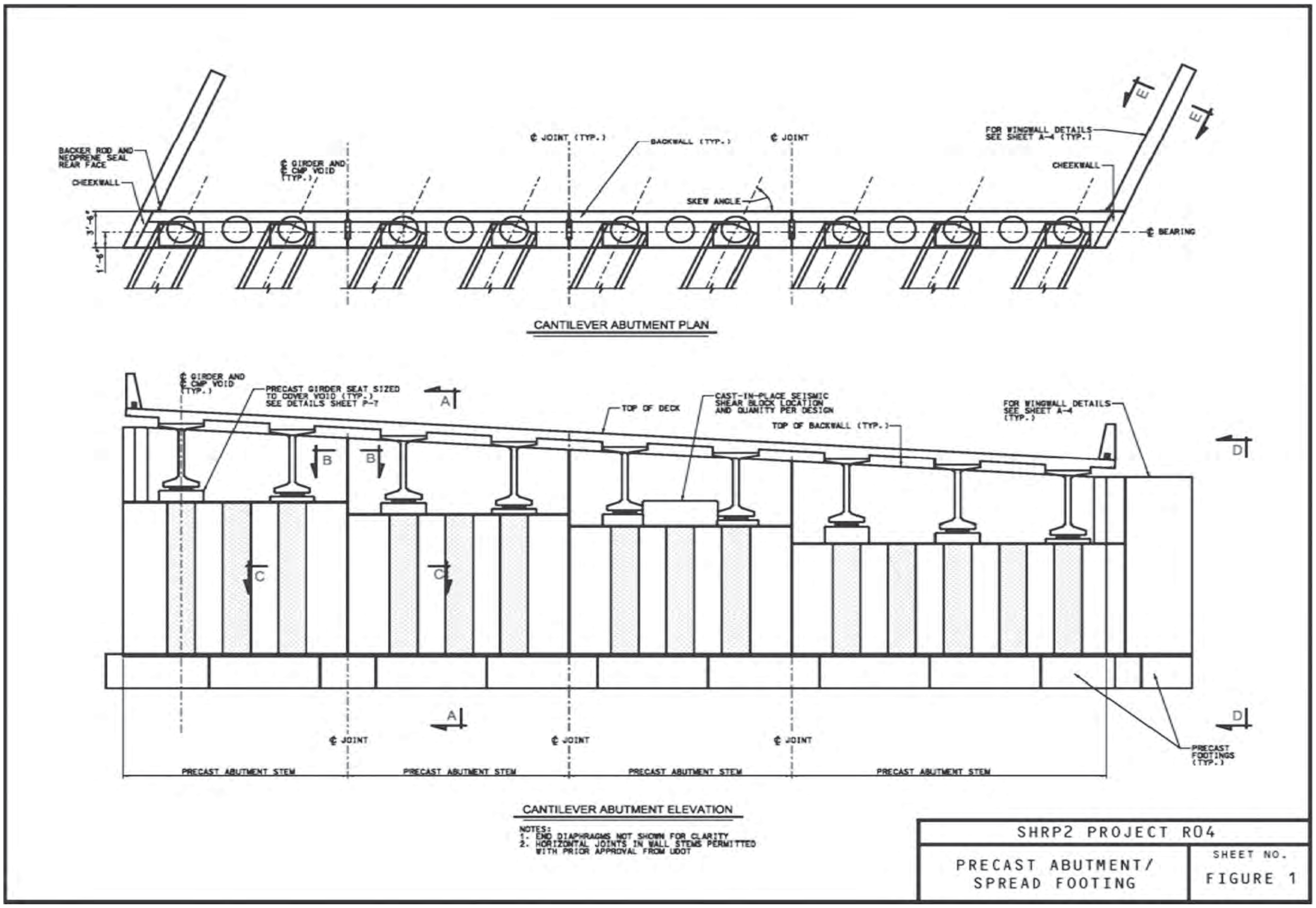


Figure 3.36. Precast modular integral abutment.

designs or ABC Designs similar to those presented in this report.

Costs associated with investment in these technologies will not be addressed nor will they be evaluated at this time, as the intrinsic soft costs driving the need for accelerated or specialized construction equipment and techniques may outweigh the additional project hard costs. Therefore, the opinion of the research team is that the needs of an individual project should be used to evaluate and select the type of construction equipment and techniques used and that the additional hard costs should be used as a final evaluation tool to decide whether to recommend a renewal project as an ABC project. With this said, the ABC type designs presented in this report have been developed with the intent that the prefabricated modular components could be erected using conventional equipment when adequate site access above, below, and surrounding the bridge project are available. The ABC technologies reviewed in this part offer an alternative means to erect conventional or ABC bridge substructures and superstructures when site access for conventional equipment is limited or restricted, or in situations in which the prefabricated lengths and weights of modularized components exceed the capacities or limits of conventional equipment.

The construction concepts introduced in Phase I and subsequently carried forward for further evaluation in Phase II are as follows:

C-1: Above-deck driven carriers (ADDCs);
C-2: Launched temporary truss bridges (LTTBs);
C-3: SPMTs and other wheeled carriers; and
C-4: Launching, sliding, and lateral shifting.

These ABC Construction Technologies can be grouped into two categories for use:

- Bridge erection systems: Concepts C-1 and C-2 are technologies in which the erection equipment is designed to deliver individual components of a proposed structure in a span-by-span process. The C-1 and C-2 technologies are intended to be easily transportable, lightweight, and modular systems. The use of this type of equipment to deliver

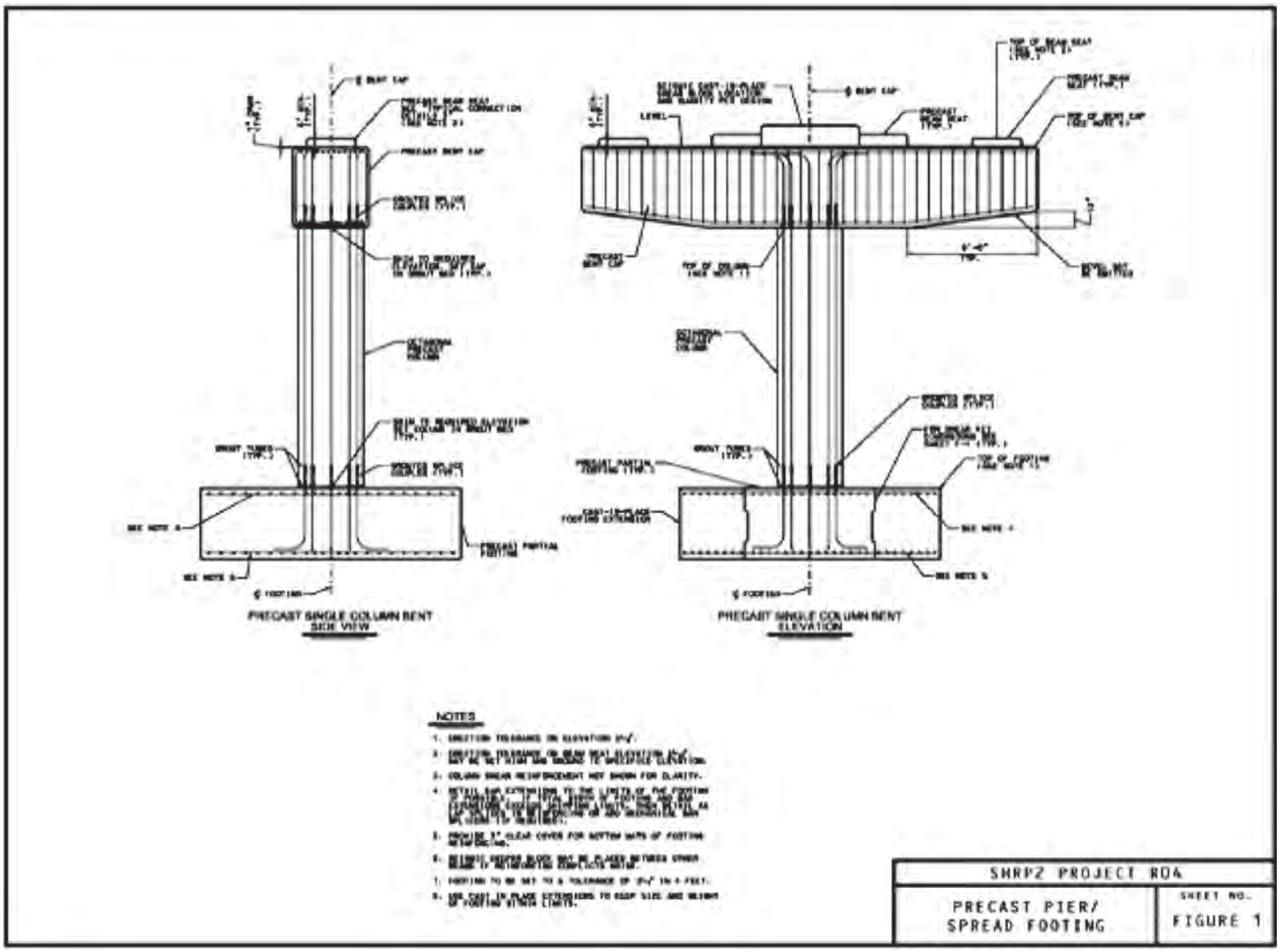


Figure 3.37. Precast complete pier (see Figure 2.6 for illustration).

fully preassembled structures is not practical (although it is possible on a very small scale).
- Bridge movement systems: Concepts C-3 and C-4 are technologies in which the erection equipment is designed specifically to lift and transport large complete or partial segments of preassembled structures.

The ultimate goal for this report would be to demonstrate that ABC technologies in both categories provide a means for a more rapid construction time period; create a safer working environment for both the contractors and the traveling public; minimize disruption to the surrounding natural environment, residential neighborhoods, and business districts; and generate a method for better quality control over the final constructed product.

Before setting the goals and expectations for the ABC technology vision too high, the team recognizes that not all bridge projects requiring rapid renewal may fit into one of the two ABC categories and thus may not justify or allow for the use of specialized equipment for a more rapid removal of existing structures or installation of new structures.

A case could even be made that given the proper project criteria, use of conventional equipment would be the first choice for constructing a bridge designed with ABC modularized components.

With this perspective, all bridge renewal projects can be categorized into one of the four design and construction project types, as follows:

1. Bridge Designs built with ABC Construction Technologies
   a. Designed assuming the use of precast or modularized substructure and superstructure components.
   b. Detailed using the ABC standardized component details presented in previous parts of this report.
   c. Constructed using ABC Construction Technologies.

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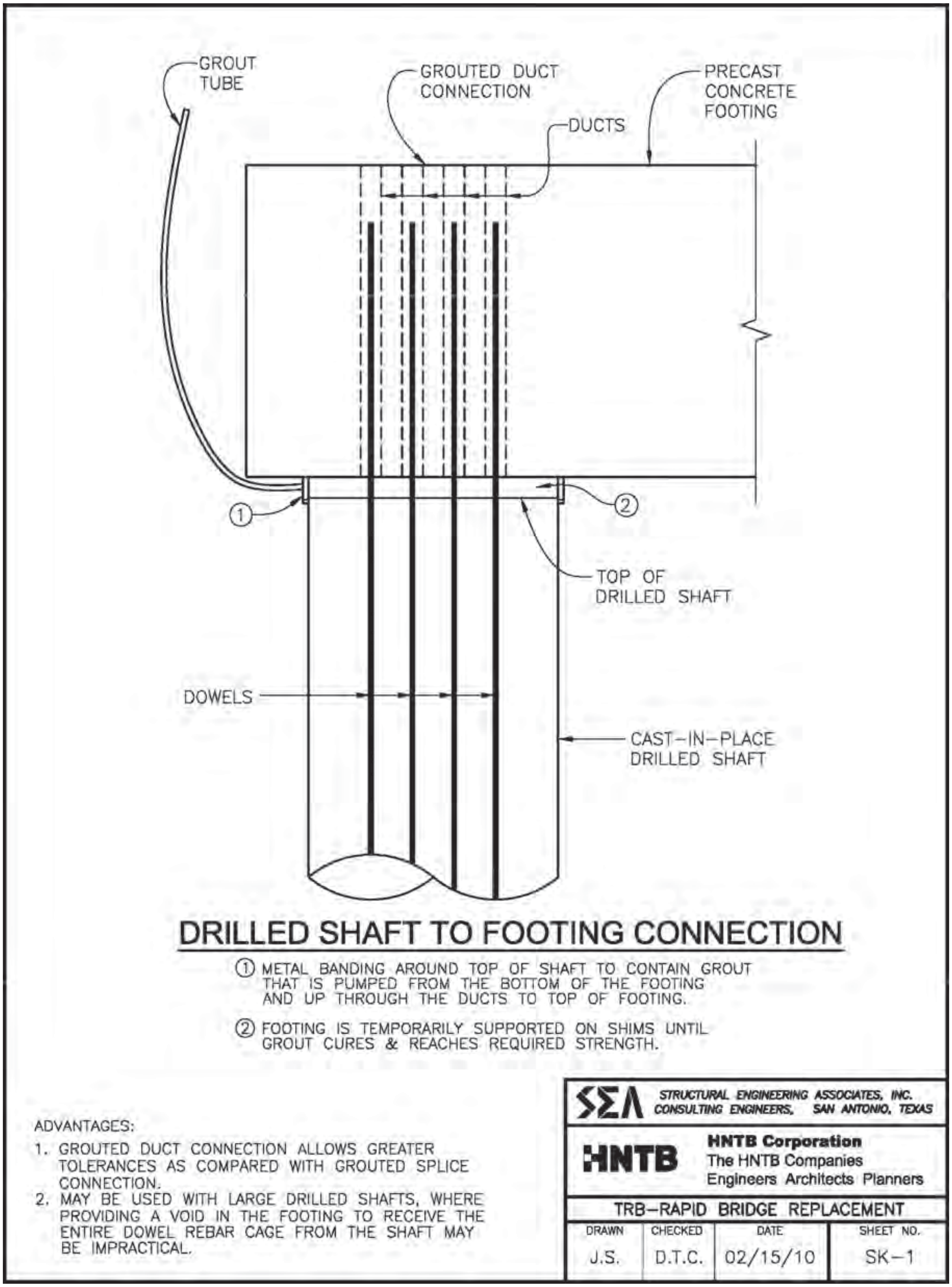

Figure 3.38. Alternate drilled shaft–to–footing connection.

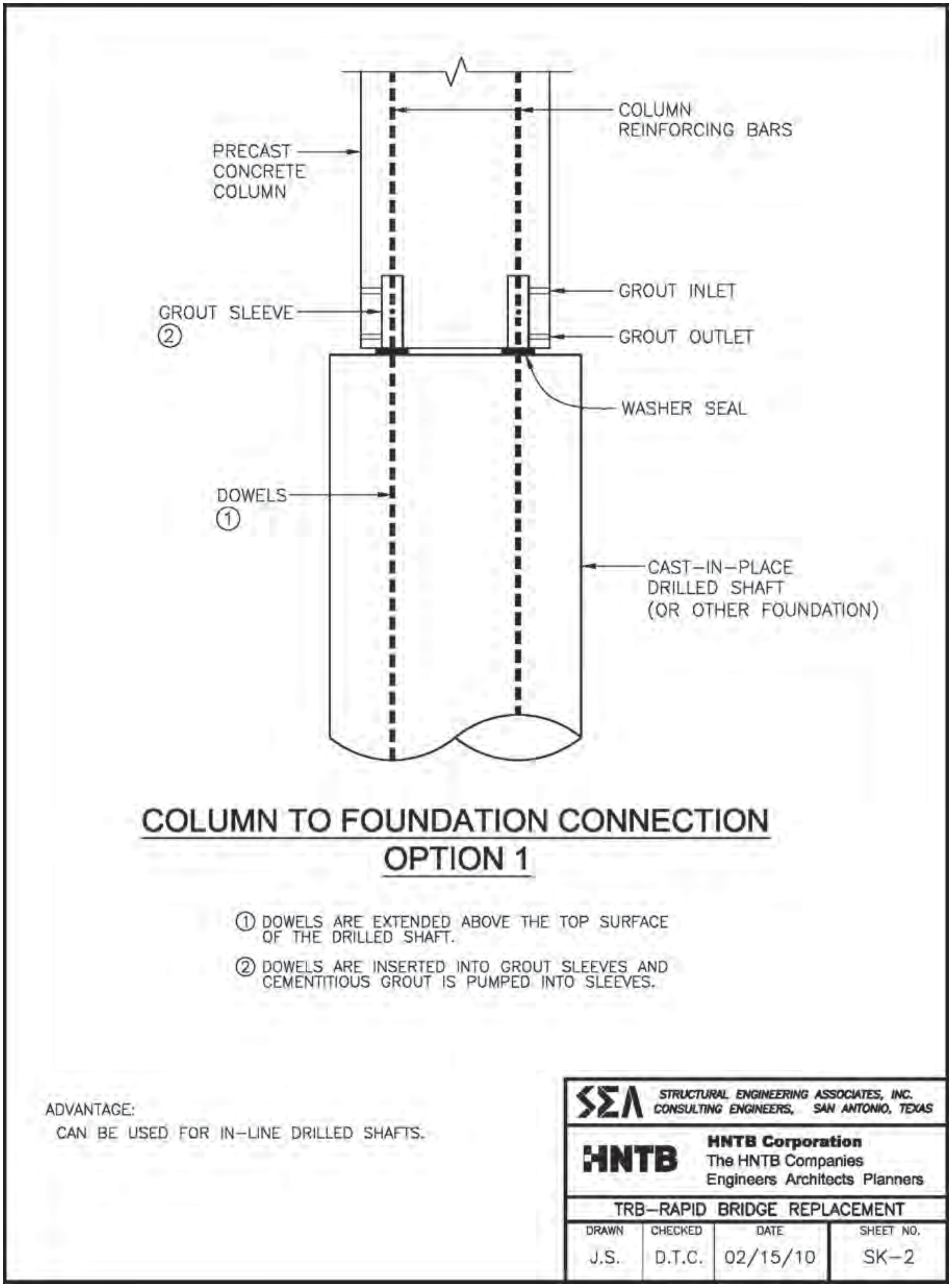

Figure 3.39. Alternate column-to-foundation connection—Option 1.

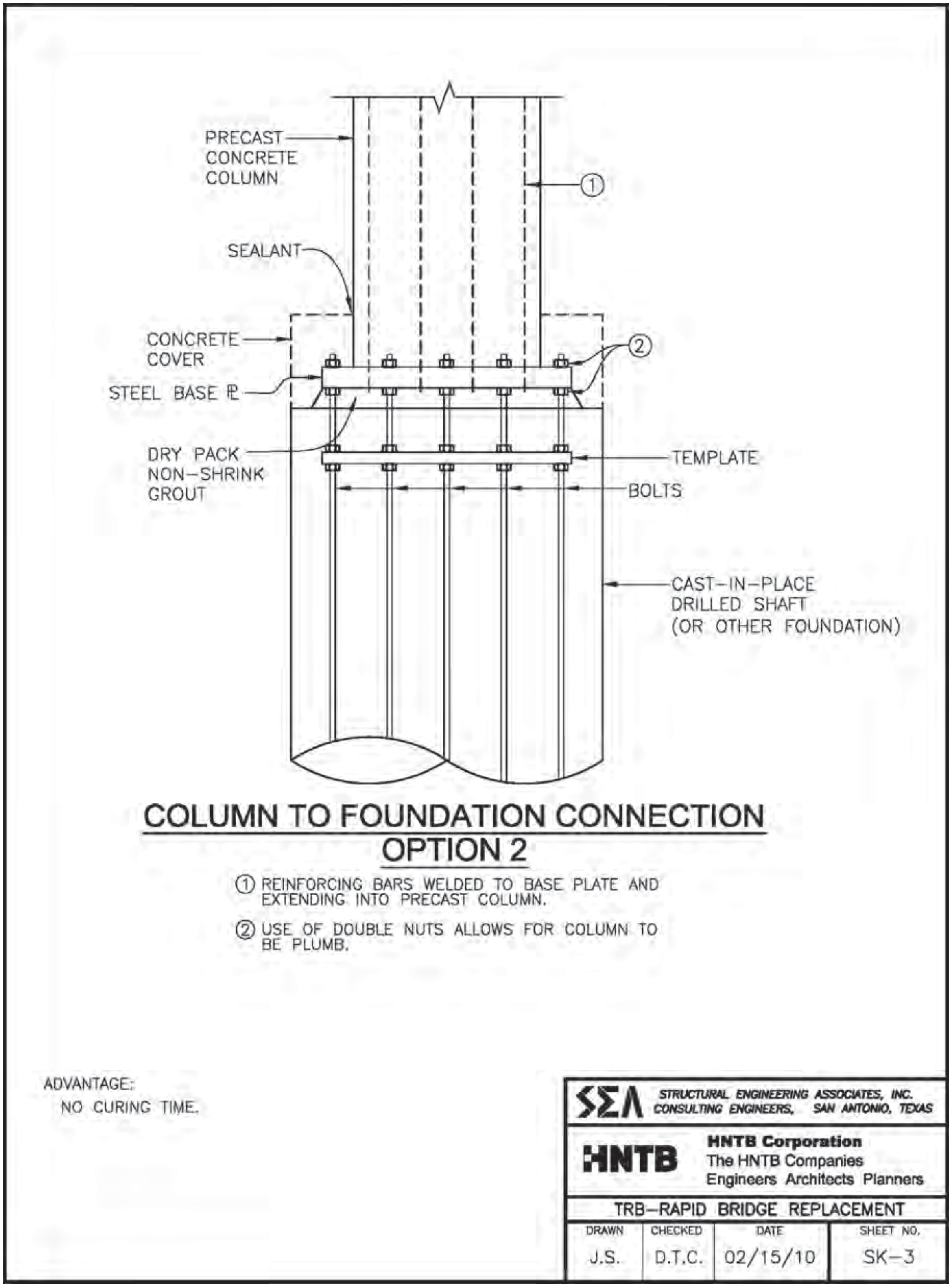

Figure 3.40. Alternate column-to-foundation connection—Option 2.

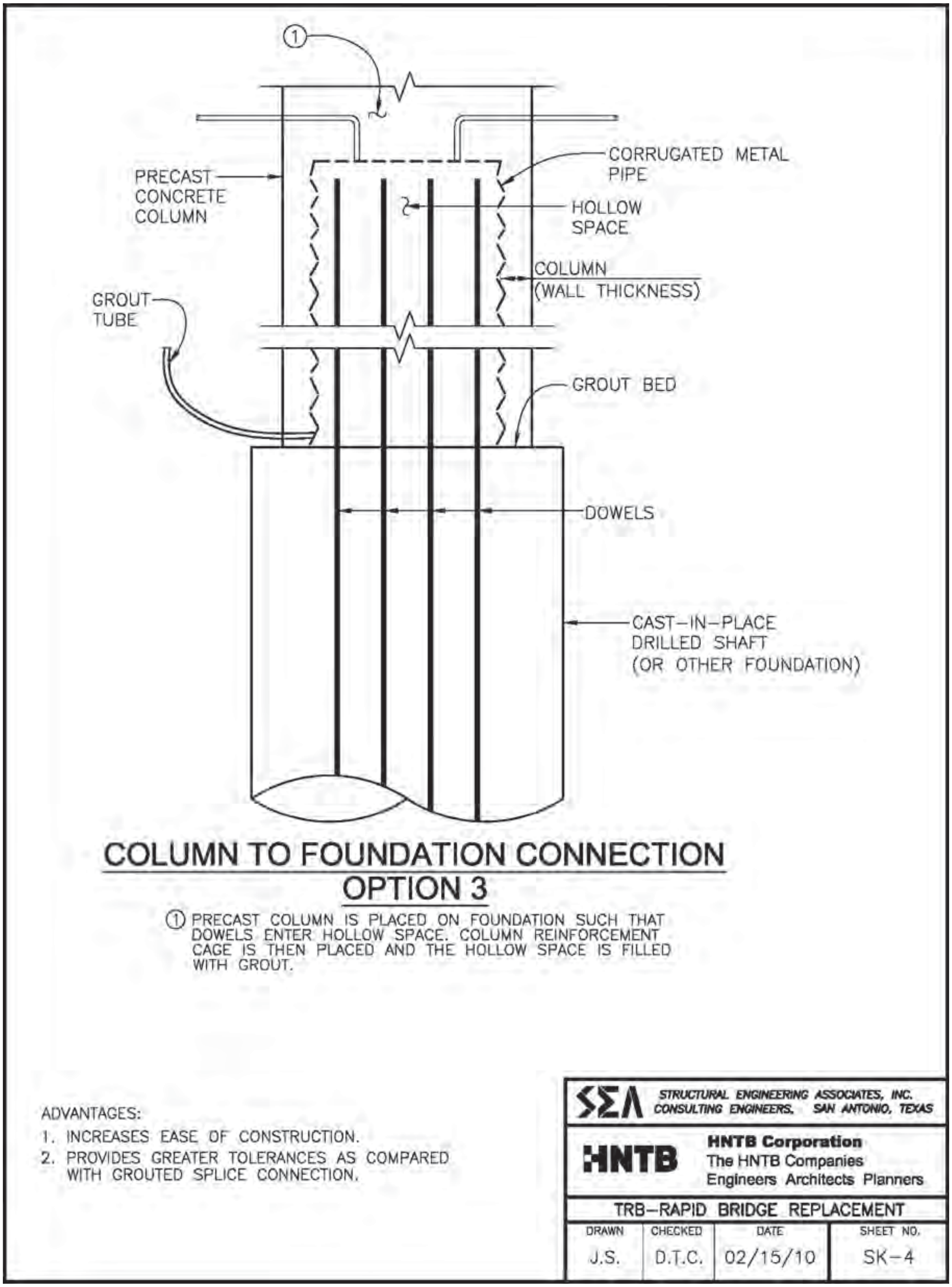

Figure 3.41. Alternate column-to-foundation connection—Option 3.

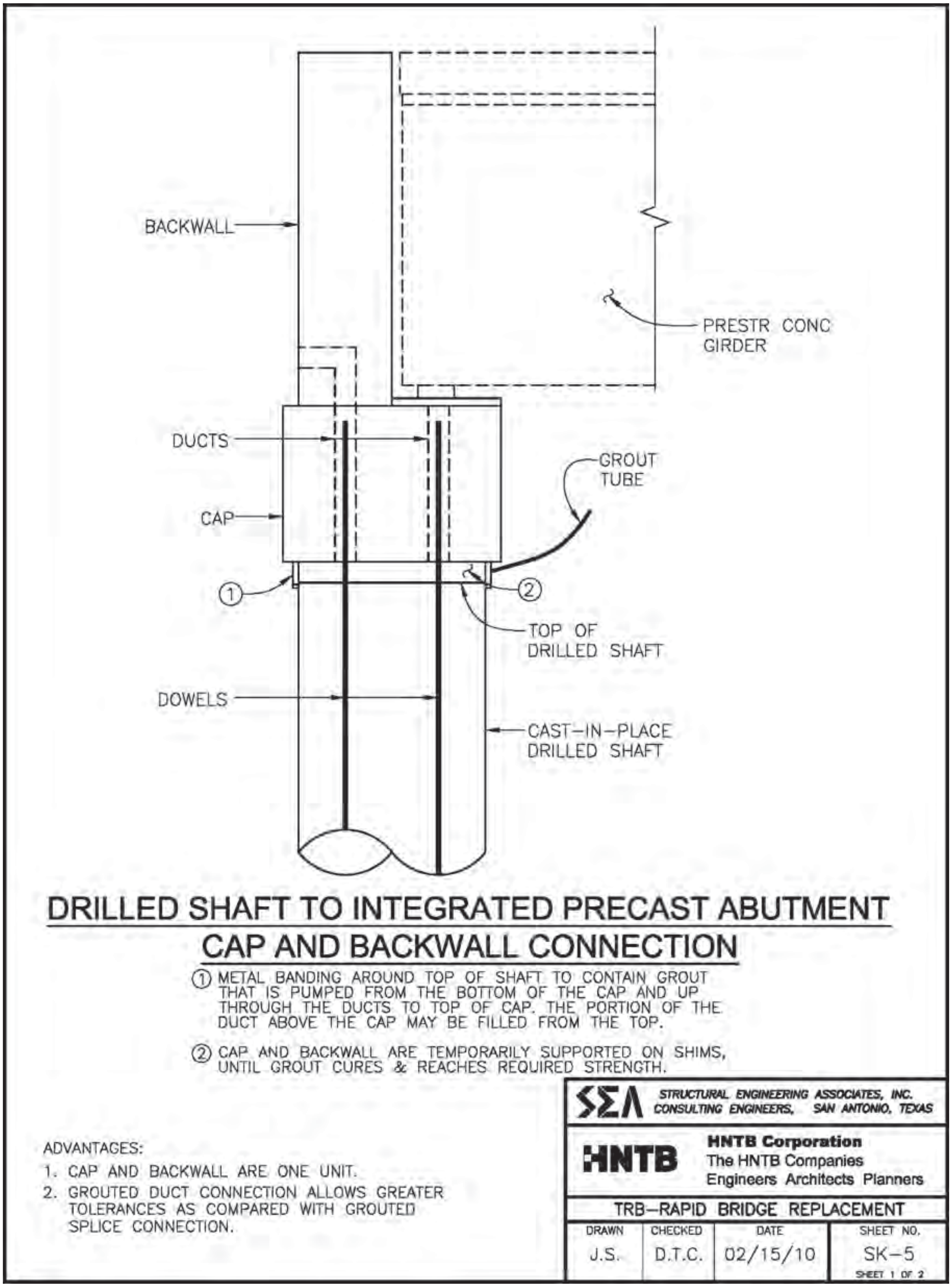

Figure 3.42. Alternate drilled shaft to integrated precast abutment cap and backwall connection—Option 1.

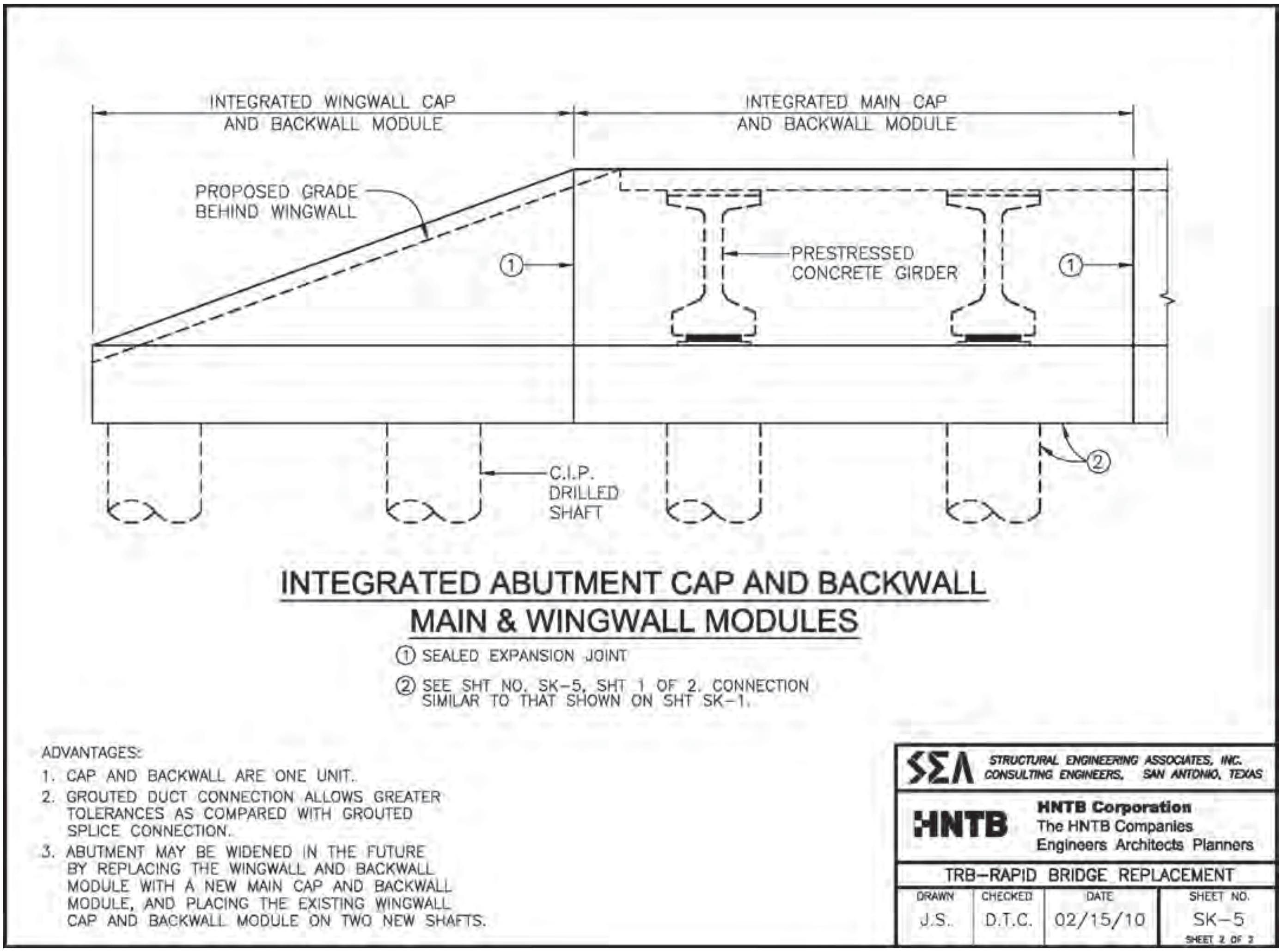

Figure 3.43. Alternate drilled shaft to integrated precast abutment cap and backwall connection—Option 2.

(continued from page 129)

2. Bridge Designs built with conventional construction
   a. Designed assuming the use of precast or modularized substructure and superstructure components.
   b. Detailed using the ABC standardized component details presented in previous parts of this report.
   c. Constructed using traditional equipment and processes currently accepted by the vast majority in the contracting community.
3. Conventional bridge designs built with ABC Construction Technologies
   a. Designed assuming the use of traditional structural systems.
   b. Designed assuming the use of well-proven standard DOT details.
   c. Constructed using ABC Construction Technologies.
4. Conventional bridge designs built with conventional construction
   a. Designed assuming the use of traditional structural systems.
   b. Designed assuming the use of well-proven standard DOT details.
   c. Constructed using traditional equipment and processes currently accepted by the vast majority in the contracting community.

To properly evaluate the correct use of ABC Construction Technologies, the era in which the bridge construction industry originally exploded must be considered. This was a time when many smaller cranes dominated project sites, when significant detours, lane reductions, lane shifts and disruption to the surrounding environment were standard practice, and when months to years were acceptable periods for project completions. Today, and looking beyond, traffic volumes have ballooned beyond original estimates to a point in which detours, lane reductions, and shifts are typically unsafe; in which disruption to the surrounding natural environment, residential neighborhoods, and business districts may have heavy social and economic impacts; and in which

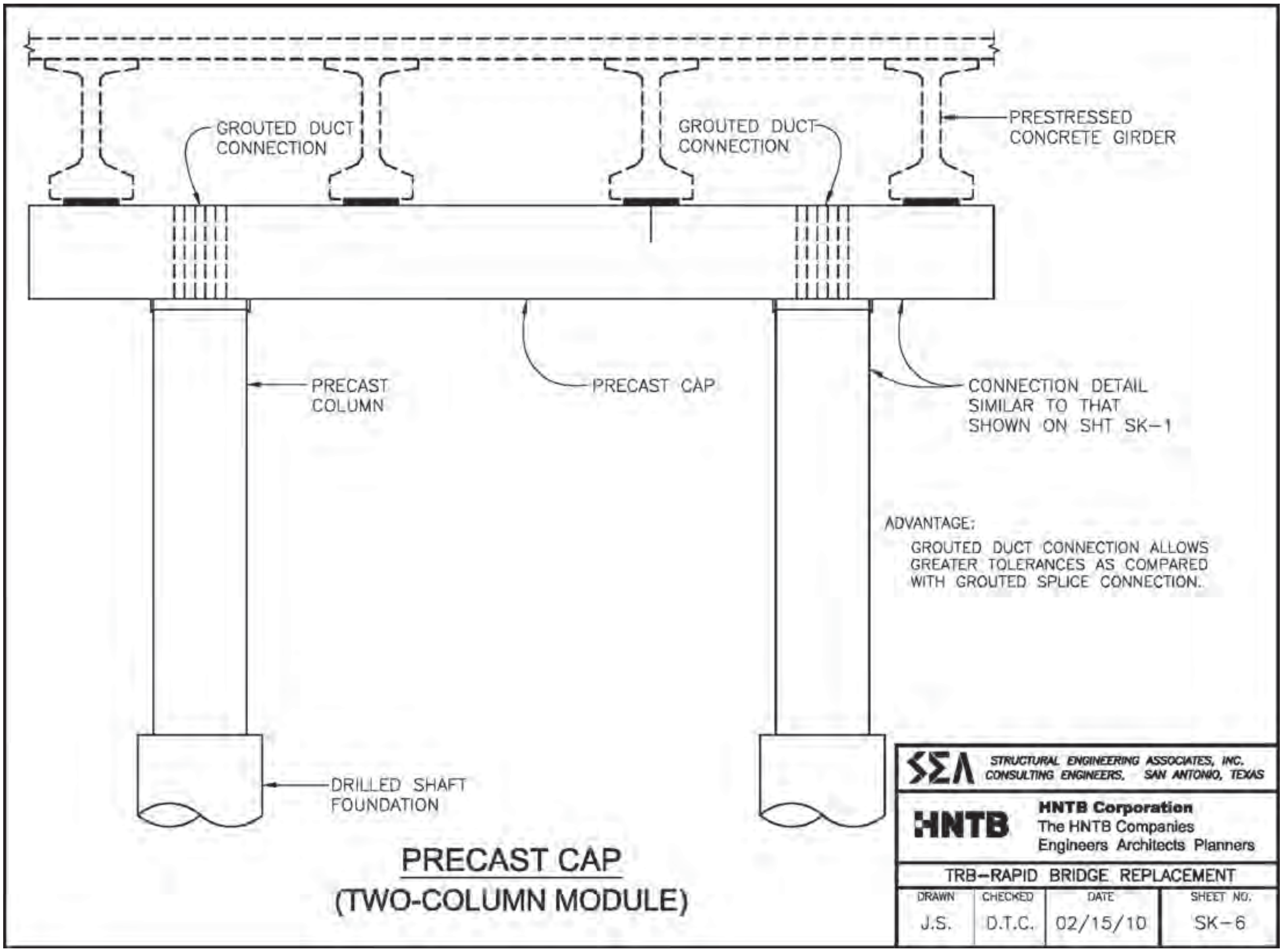

Figure 3.44. Alternate precast cap (two-column module).

more innovative construction equipment and structural systems are available to provide more rapid renewal and less disruptive construction.

The key for owners and their engineering consultants is to define the goals of a renewal project, survey the limits and constraints that could affect the design and construction of the project, evaluate the impact of those limits and constraints, and finally develop a list of design criteria that will be used to prepare plans and specifications for construction.

Goals

The Phase I goals for the Task 6 evaluations of ABC Construction Technologies are as follows:

- Develop standard concepts for erecting highway structures using adaptations of proven long-span technologies that can also be adapted from project to project.
- Document the potential time savings of an ABC Construction Technology as compared with more conventional construction techniques and equipment.
- Continue to examine the various innovative methods of erection to see if there are better ways to employ these in accelerated bridge construction.

After reviewing the Phase I goals, the Phase II evaluation approach for ABC design concepts was modified to better suit the evaluation criteria for ABC Construction Technologies. The evaluations included

- The development of a matrix of questions for owners and their consultants that would guide them in selecting a construction technology that best fits a project's needs and limits.
  - The matrix would consider all four design and construction project types.

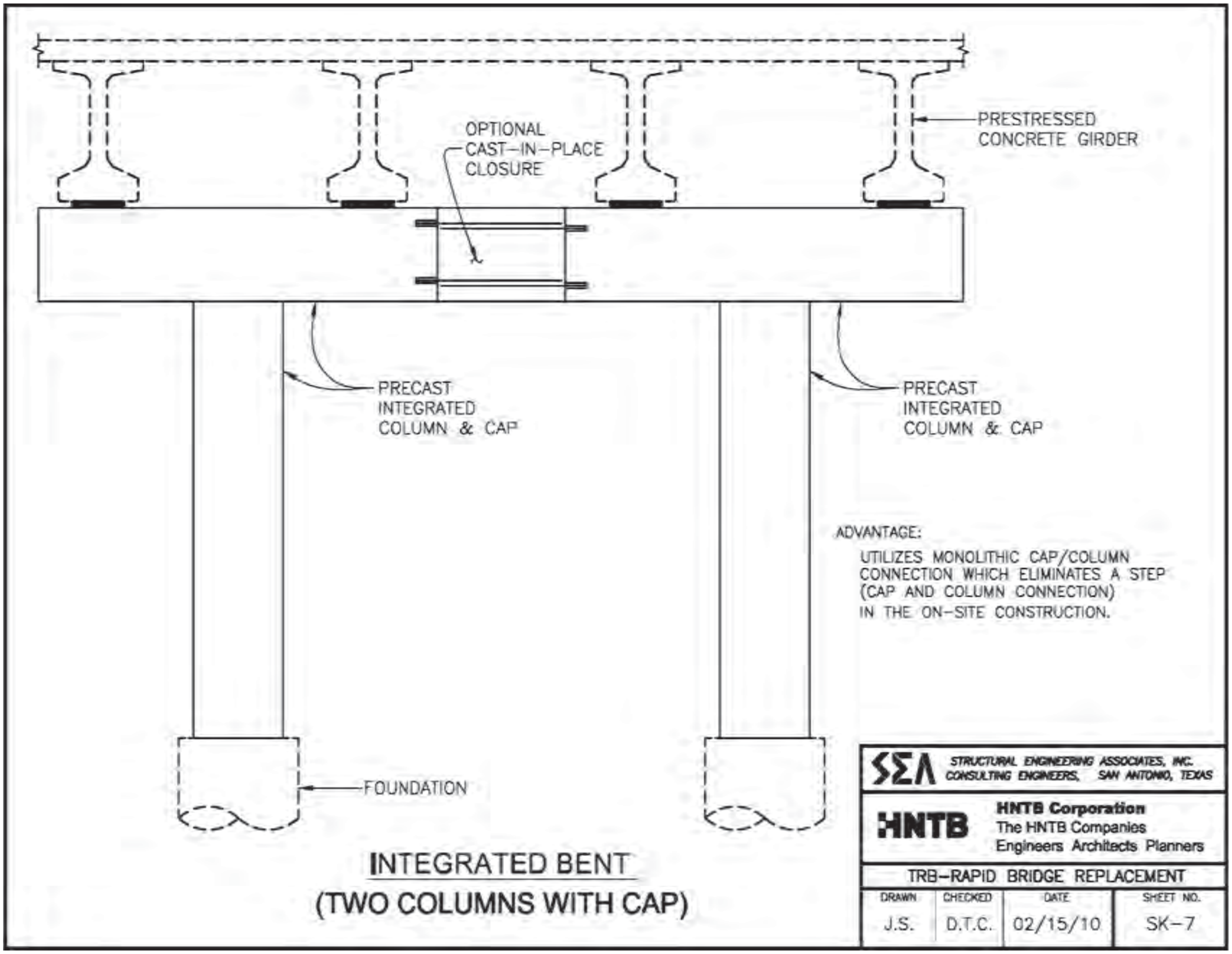

Figure 3.45. Alternate integrated bent (two columns with cap).

   - The matrix would factor in the site and traffic variables and constraints.
   - The matrix would factor in the surrounding environment.
   - The matrix would be independent of the structure type.
   - The matrix would consider the project construction time period.
- The development of a checklist of items that must be addressed during the design and construction phases of a project.
   a. These lists would be tailored to each of the four specified ABC Construction Technologies evaluated in this part.
   b. These lists would address design considerations and expectations for both the engineer of record and the contractor's construction engineer.
- The development of a set of standard conceptual details that define terminology or demonstrate the possibilities and limits of the four specified ABC Construction Technologies.

Review of ABC Construction Technologies

Above-Deck Driven Carriers

Above-deck driven carriers (ADDCs) are designed to deliver individual components of a proposed structure in a span-by-span process with minimal disruption to the activities and the environment below the structure.

Current ADDCs exist in two forms, and both perform a similar function. An ADDC rides over an existing bridge structures and then delivers components of the new bridge spans by using hoists mounted to overhead gantries with traveling bogies. As shown in the examples below, the ADDC equipment can be quite specialized as in the case of the RCrane Truss system used by railroads to replace existing short bridge spans. Some systems, like the Mi-Jack Travelift overhead gantry, require specific site adaptations to align their wheel set with the centerlines of the existing girders that support the heavy

Figure 3.46. RCrane Truss.

moving loads. An example of ADDC equipment is shown in Figure 3.46.

A modified ADDC concept would be a combination of the RCrane Truss and the Mi-Jack Travelift to create pairs of lightweight steel trusses supporting an overhead gantry system. This lightweight equipment could then be used on structures in which the existing bridge deck or girders are insufficient to support the heavier wheel loads of current ADDC equipment. This construction technology would be multifunctional, would be easily transportable both on urban and rural road systems, and would be mobilized with minimal erection and de-erection time.

The trusses of the modified ADDC would be modularized into lengths that are easily trucked over both primary and secondary roads (either shipped on flatbed trucks, or towed with the mountable rubber-tired bogies). Once assembled at the project site, the system would be equipped with several rubber-tired bogies that would be spaced to reduce and more evenly distribute the localized equipment dead load. Once the modified ADDC is rolled out over across the bridge span(s), temporary jack stands would be lowered at the piers and abutments and would bear on the deck where blocking had been added below, from the pier up to underside of the bridge deck. By bearing at the piers and abutments, the modified ADDC prevents overloading of the existing bridge structure during the delivery of the bridge components, as illustrated in Figure 3.47.

This ABC Construction Technology would be applicable when an existing bridge or set of twin bridges is planned to be widened and when portions of the existing bridge are to be replaced, as shown in Figures 3.48 and 3.49. In an extreme case, with several movements, the technology could be used to replace an entire bridge.

Although the modified ADDC concept is intended for use on more typical ABC bridge components, a heavier, more specialized version of the concept could also be used for those cases in which the component lengths or weights exceed the limits and capacities of conventional equipment.

Advantages of ADDCs include the following:

- Minimizes disruption to traffic and the environment at lower level of bridge project.
- Can be used where conventional crane access is limited by site constraints.
- Allows for faster rates of erection due to simplified delivery approach of components.
- Component delivery occurs at the end of the existing bridge, which minimizes disruptions at the lower level of the project site.
- Reduces the need to work around existing traffic and decreases the need to reduce lanes, shift lanes, and detour lanes, which in turn improves safety for both the workers and the traveling public.
- Can be used to deliver prefabricated modularized components of ABC substructures and superstructures.

Additional advantages of modified ADDCs include:

- The dual truss concept will allow either overhead gantries or under-slung hoists.
- The lightweight trusses can be driven over existing bridge decks.
- The temporary loads generated while delivering the bridge components are introduced directly into piers, eliminating potential overloading of the existing bridge structure.

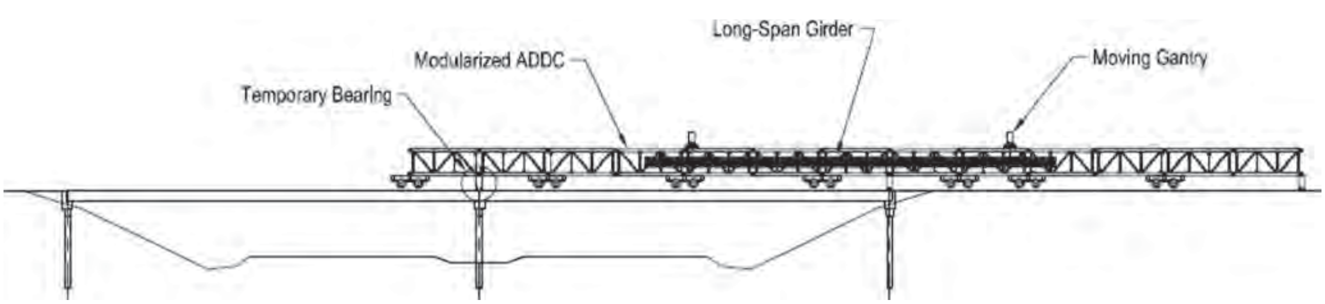

Figure 3.47. Modified ADDC rolled across span and delivering a bridge component.

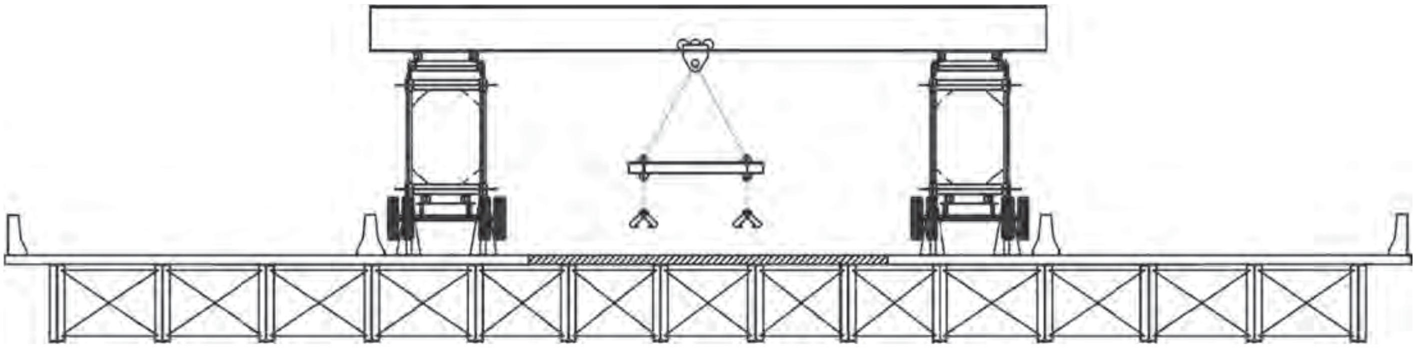

Figure 3.48. Modified ADDC concept used for widening the inside edges of a set of existing twin bridge structures.

Launched Temporary Truss Bridges

Launched temporary truss bridges (LTTBs) are designed to deliver individual components of a proposed structure in a span-by-span process with minimal disruption to activities and environment below the structure. Examples of LTTBs are given in the next two figures.

Currently, LTTBs exist in many forms; however, the basic technology is the same for each. LTTBs are launched across or lifted over a span or set of spans and, while acting as temporary bridges, are used to deliver the heavier components of a span without inducing large temporary stresses into those components. As shown in the examples below, the pieces of LTTB equipment are designed and modified on the basis of varying sets of criteria from project to project. The equipment can be quite specialized, depending on the needs of the project, and can require extensive modifications from project to project based on changes in span lengths and component weights.

With the industry moving forward with more-standardized bridge designs of different span lengths and component sizes and weights, the idea behind a modified LTTB is to create a set of standardized lightweight steel trusses that could be assembled to a specific length for a given project. The truss design and details would follow the quick connect concepts used in crane boom technology and would allow site modifications with relatively minimal effort. The lightweight equipment could then be used to bridge new spans to deliver components for a new bridge structure. This construction technology would be multifunctional, would be easily transportable both on urban and rural road systems, and would be mobilized with minimal erection and de-erection time.

The trusses of the modified LTTB would be modularized into lengths that are easily trucked over both primary and secondary roads (either shipped on flatbed trucks, or towed with mountable wheel-tired bogies). Once assembled at the project site, the lightweight equipment would then be launched from span to span or could be lifted into position with cranes, as

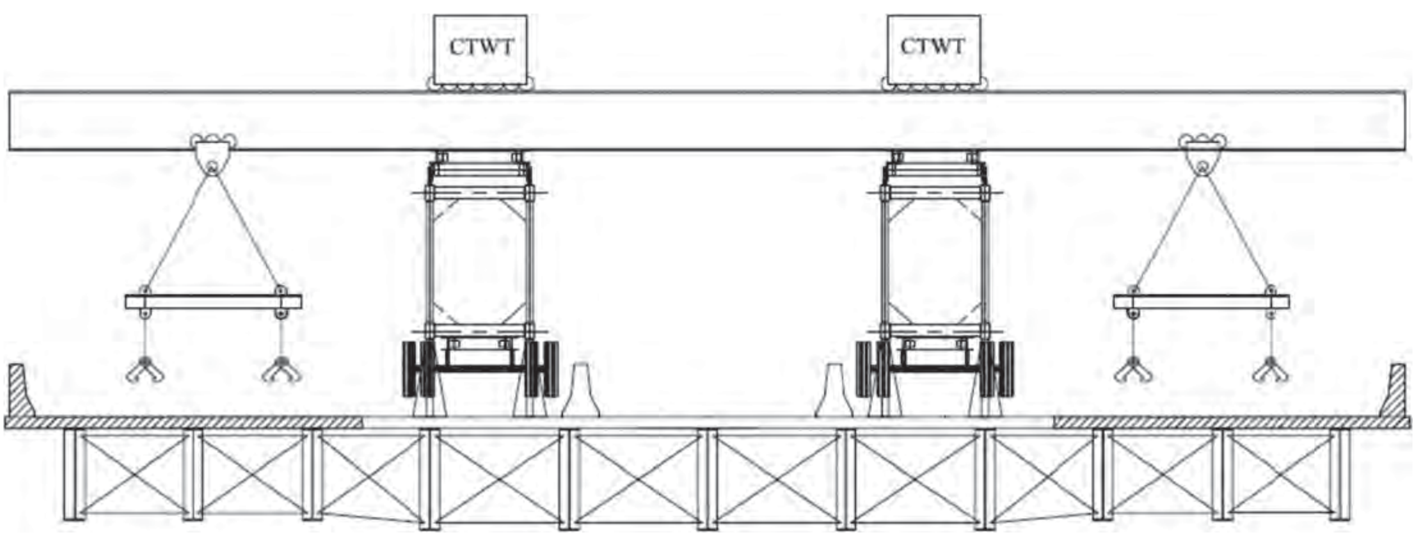

Figure 3.49. Modified ADDC concept used for widening the outside edge of an existing bridge structure.

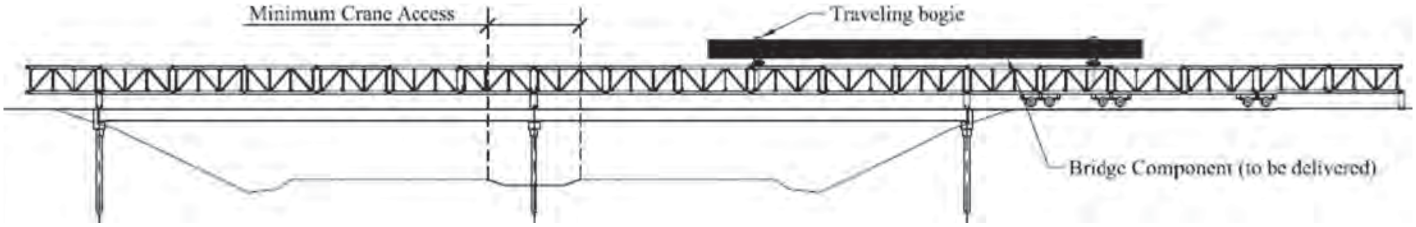

Figure 3.50. Modified LTTB launched across span and delivering a bridge component.

shown in Figure 3.50. Once the modified LTTB has bridged the new span, it would be stabilized and supported at each pier or abutment substructure unit.

This ABC Construction Technology would be applicable whenever new bridge structures are to be erected; it could also be applicable when an existing bridge or set of twin bridges are planned to be widened, as illustrated in Figure 3.51.

Although the modified LTTB concept is intended for use on more-typical ABC bridge components, a heavier, more specialized version of the concept could be used for those cases in which the component lengths or weights exceed the limits and capacities of conventional equipment.

Advantages of the LTTBs include the following:

- Provides construction options when launching demands on the permanent structure add extra cost to the steel, concrete, or posttensioning.
- Minimizes disruption to traffic and the environment at lower level of a bridge project.
- Can be used where conventional crane access is limited by site constraints (launched option only).
- Component delivery occurs at the end of the existing bridge, which minimizes disruptions at the lower level of the project site.
- Reduces need to work around existing traffic and reduces need to reduce lanes, shift lanes, and detour lanes, which in turn improves safety for both the workers and the traveling public.
- Increases the possibility of erecting longer spans without significantly increasing the cost of bridge spans because the components of the spans can be delivered without additional temporary erection stresses.
- Where access to the bridge site is limited due to approach roadways that are difficult to traverse, shorter girder

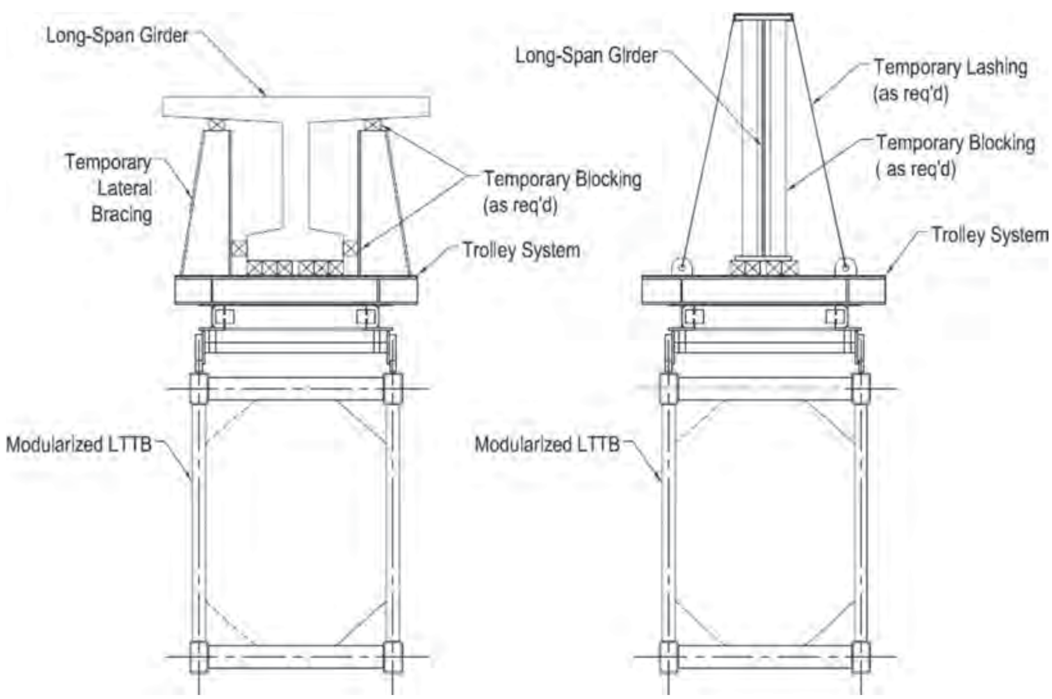

Figure 3.51. Modified LTTB concept used for erecting a new bridge structure or possibly for widening an existing bridge structure.

Figure 3.52. Example of an SPMT.

segments can be delivered and preassembled at the site and then delivered as one unit over the span.

- Allows work to proceed on multiple fronts (i.e., where multiple-span LTTBs are used, girders can be set while the next girder is delivered).
- Temporary loads are introduced directly into piers, minimizing the need for falsework bents.
- Can be used to deliver prefabricated, modularized components of ABC substructures and superstructures.

SPMTs or Other Wheeled Carriers

Self-propelled modular transporters (SPMTs) and other wheeled carriers are used to remove entire spans or full-length strips of existing bridges and to replace them with new preassembled structures in a quick and efficient manner.

SPMTs are multi-axle mobile lifting devices that have been standardized by the heavy lift industry, as shown in Figure 3.52. They are easily transportable and can be mobilized with minimal assembly time. With the use of computer synchronization, SPMTs can “crab crawl” in numerous directions, perform 360° carousel-type turns, and traverse uneven terrain. With the steel cribbing and timber dunnage mounted to its lift table, an SPMT unit can be used to raise, transport, and set any type of heavy preassembled structure. Other types of wheeled carriers have been developed on a more specific project-by-project basis, but their purpose is similar to that of the SPMTs—to raise, transport, and set preassembled structures.

The U.S. bridge industry is growing to accept SPMTs and other wheeled carriers as plausible ABC Construction Technologies. Success stories involving the use of SPMTs are well documented:

- The Utah DOT: 4500S over I-215E (Figures 3.53 and 3.54).
- The Florida DOT: Graves Avenue over I-4.
- The Louisiana Department of Transportation and Development: I-10 over LA-35.

Source: Utah DOT.

Figure 3.53. 4500S over I-215E—removing the existing bridge.

Source: Utah DOT.

Figure 3.54. 4500S over I-215E—installing a new bridge.

- The Chicago Transit Authority, Illinois: Main Street Viaduct Replacement.
- The City of Toledo, Ohio: MLK Replacement Bridge.

AASHTO has produced a *Manual on Use of Self-Propelled Modular Transporters to Remove and Replace Bridges* (FHWA, 2007c), which was developed through cooperation of the FHWA, AASHTO, NCHRP, and the Florida DOT. The manual provides information about the use of SPMTs as well as guidance toward working with and specifying the use of SPMTs for bridge projects including cost benefits, planning criteria, design considerations, contracting considerations, and case studies.

The Utah DOT has developed its own *Manual for the Moving of Utah Bridges Using Self Propelled Modular Transporters*

(SPMTs) (Utah DOT, 2008e) which includes a general overview of the responsibilities of design, owner, and contractor teams and a comprehensive checklist for the engineer(s) of record, the contractor, the heavy lift team, the resident engineer, and the specialty bridge engineer.

Heavy lift contractors in the United States that provide services that use SPMTs include the following:

- Mammoet in Houston, Texas;
- Fagioli in Pearland, Texas;
- Berard Transportation in New Iberia, Louisiana;
- Barnhart Crane and Rigging in Memphis, Tennessee; and
- Bigge Crane and Rigging in San Leandro, California.

This ABC Construction Technology has proven to be very applicable when removal of complete existing bridges and installation of preassembled new bridge structures must be achieved in a very short time. SPMTs may also be applicable to bridge widening when the new widened portion of the structure is to be installed as a preassembled unit.

Advantages of the SPMTs and other wheeled carriers include the following:

- The standard SPMT unit is available in four- and six-axle units and can be grouped in any number of longitudinal and transverse combinations.
- The standard SPMT units and power packs have been developed with quick-connect pin and hydraulic connection points, making assembly and reconfiguration relatively easy.
- By pre-erecting the bridge in its entirety on a support system mimicking the permanent condition, the deflections and reactions can be monitored to confirm design assumptions.
- Traffic disruptions are minimized and safety for workers and the traveling public is improved by moving critical demolition and erection work tasks off-line from the traffic.

Figure 3.55. Example of bridge structure that has been launched or installed by sliding or lateral shifting technologies.

- The quick removal of existing structures and quick installation of new preassembled bridge structures are provided for.
- Safety of the traveling public is improved by minimizing exposure to construction zones.
- By pre-erecting the new structures in a controlled environment, improved quality control can be achieved.

Launching, Sliding, and Lateral Shifting

The incremental launching construction method was developed in Europe in the 1960s and is now typically used for construction of prestressed concrete, steel, and steel/composite bridges. The method involves building a bridge at a single construction location in sections and launching the bridge incrementally as each section is completed. Examples are shown in Figure 3.55 and Figure 3.56.

Similarly, lateral shifting is a method in which a structure is constructed adjacent to an active bridge that is to be replaced. On a given closure period, the existing bridge is lifted and slid out of the way of the new bridge. The new bridge is then slid into position on the existing alignment.

Figure 3.56. Examples of bridge structures that have been launched or installed by sliding or lateral shifting technologies.

As illustrated by the examples below, both launching and lateral shifting technologies involve the use of specialized equipment. To launch a bridge structure, a contractor would potentially require a launching nose, a method of movement (jacks, tuggers, push/pull rams), guides, and rollers. To shift a bridge laterally, a contractor would require falsework to support the new bridge while it is under construction, falsework to support the track system, a method of movement (jacks, tuggers, push/pull rams), guides, and rollers.

To launch a bridge requires the use of a staging area behind one or both of the abutments where a partial or complete structure is constructed. For concrete structures, the entire bridge section is launched integrally. For steel structures, the entire steel system (girders, diaphragms, and floorbeams) is launched and then on completing the launch, the deck is cast in place and made composite with the steel girders. Typically a launching nose is provided to reduce the weight of the lead cantilevered section. Depending on the available length in the staging area behind the abutment, the structure may be constructed full length and launched in one sequence or can be erected and launched in a series of partially completed stages. Depending on the available support mechanisms, the launched structure can either be pushed from a deadman at the back end of the staging area, be pushed or pulled by using rams that anchor to a travel track, or be pulled with winches anchored to the abutment or pier ahead of the launched structure. Launching uphill against gravity is preferred to provide control of the launch movement; however, it is not mandatory as long as the braking and locking mechanisms are properly engaged.

To slide or laterally shift a bridge requires the use of an available staging area adjacent to the existing bridge structure to be replaced. The falsework, track, and movement mechanisms are similar whether the structures are cast-in-place concrete slabs or boxes, precast girders with concrete decks, or steel girders with concrete decks. The size of the falsework, track, and movement mechanisms will be based on the weights of the different bridge types. By using the sliding or lateral shift technology, an existing structure can moved independently of a new structure or coincidently with the new structure. Examples are shown in Figures 3.57 and 3.58.

A number of reference manuals and research papers discussing incremental launching techniques exist; however, most of these focus on the launching of concrete structures. Although many aspects of the technology are similar, there are no specific reference manuals that focus specifically on the launch of steel structures. Neither are there specific guides addressing minimum design criteria for lateral shifting technology. For reference purposes, readers may consult the following publications in this report's reference list: AASHTO, 2003; American Segmental Bridge Institute, 2008; Podolny and Muller, 1982; Rosignoli, 2002, and Hewson, 2003.

Figure 3.57. Example of a horizontally curved launched steel girder.

This ABC Construction Technology would be applicable when the construction of a new structure is challenged with difficult terrain below the bridge, when crane access below the bridge is limited or prohibited, or when traffic volumes require minimal disruption. Although not common, this ABC Construction Technology may also be applicable to bridge widening.

Advantages of launching, sliding, or lateral shifting include the following:

- Minimizes traffic disruptions and improves safety for workers and the traveling public by reducing or eliminating detours, lane reductions, or traffic shifts.
- Improves quality control by pre-erecting the new structures in a controlled environment.
- Minimizes the need for temporary works between spans (specific to launching).
- Allows substructure and superstructure construction activities to occur concurrently.

Figure 3.58. Example of skewed bridge installed by lateral shifting technology.

- Minimizes disruption of environmentally sensitive areas under the bridge.
- Reduces crane size requirements by pre-erecting the new structure behind the abutment.
- Reduces overall construction time.

Development of the ABC Construction Technology Selection Matrix

The role of owners and engineers is to define the goals of a bridge construction project, survey the limits and constraints that could affect the design and construction of the bridge project, evaluate the impact of those limits and constraints, and develop a set list of criteria that can be used to prepare plans and specifications for construction.

To assist owners and engineers in this process, the first goal of this part of the Phase II report on ABC Construction Technologies was to "develop a matrix of questions for owners and designers that will guide them toward the proper selection of the construction technology that best fits a project's needs."

The proposed tool for their assistance is the ABC Construction Technology Selection Matrix. This matrix contains a logical progression of questions; after answering these questions, the user will be able to ascertain whether a bridge renewal project warrants further consideration of the use of specialized ABC Construction Technologies or whether the site and project limits are more suitable for the use of conventional equipment and technologies.

Although this SHRP report is focused on ABC Designs for bridge renewal and widening, we have included new construction as part of the matrix criteria to give a complete picture of the total range of the selection process.

The ABC Construction Technology Selection Matrix compels owners and consultants to consider the following variables:

- Bridge project type
  a. New construction;
  b. Bridge widening; and
  c. Bridge replacement.
- Site and traffic constraints
  a. Volume of traffic on the mainline;
  b. Volume of traffic on the secondary road;
  c. Detour possibilities or restrictions; and
  d. Feasibility or possibility of lane reductions or traffic shifts.
- Available space (if any, where and condition) for construction staging areas
  a. Off site, away from project site;
  b. Adjacent to existing bridge;
  c. Behind one or both abutments; and
  d. Physical limitations or potentials of the surrounding grades and structures.
- Environment surrounding the project site
  a. Effects on residential neighborhoods;
  b. Effects on business districts;
  c. Degradation of sensitive areas of the natural environment; and
  d. Critical access to existing roads.
- Project construction time period
  a. Total project time;
  b. Limited closure times; and
  c. Early milestones.

The questions and evaluation criteria within the ABC Construction Technology Selection Matrix have been selected to allow the evaluation of the proper construction technology to be independent of the structural system, potential costs, span configurations, and span lengths. It is the research team's opinion that including these variables in the evaluation criteria at this stage would distract from critical design issues behind the reasoning for classifying the renewal project as a potential ABC project.

The structural system, cost, and span data may eventually play a role in the final selection of the construction technology; however, a similar selection matrix has already been developed in this report for the independent evaluation of the proper ABC substructure and superstructure designs that best fit a renewal project. Once the independent evaluations of the two matrices are complete, the results can be reviewed and the best design and construction technologies can be selected.

The ABC Construction Technology Selection Matrix recognizes that all bridge projects fall into one of four basic categories of design and construction types:

- ABC bridge designs built with ABC Construction Technologies;
- ABC bridge designs built with conventional construction;
- Conventional bridge designs built with ABC Construction Technologies; or
- Conventional bridge designs built with conventional construction.

Furthermore, the ABC Construction Technology Selection Matrix recognizes that all bridge sites can be categorized as one of the following four typical cross sections (see Figure 3.59 through Figure 3.62).

As a result of completing the matrix, owners and engineers will have a list of possible construction technologies that are applicable to their renewal project. These can be summarized into one of the following groupings:

- ABC Construction Technologies are not practical given project restraints.
- Launching or use of LTTBs.

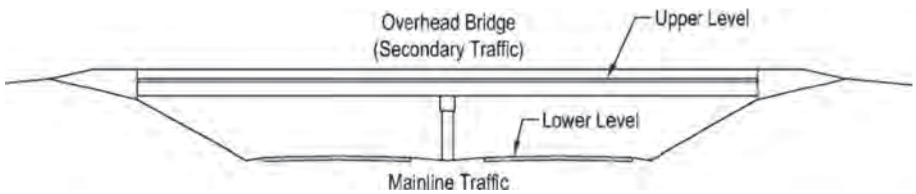

Figure 3.59. Secondary road over mainline–rural (single or twin).

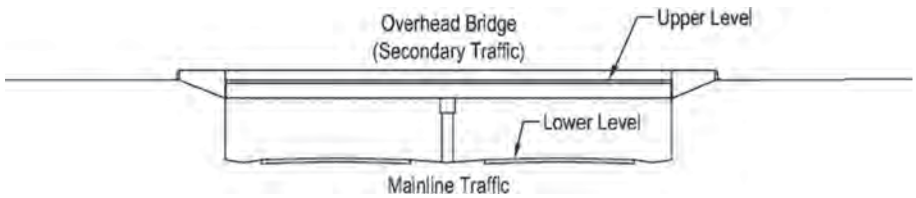

Figure 3.60. Secondary road over mainline—urban (single or twin).

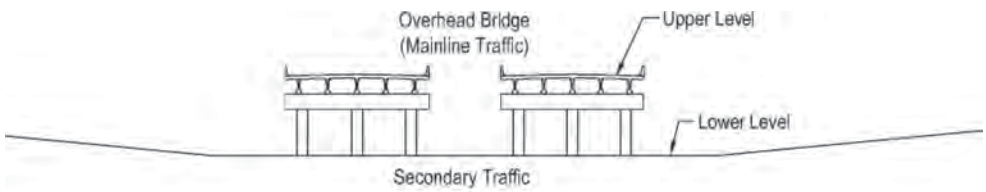

Figure 3.61. Mainline road over secondary road (single or twin).

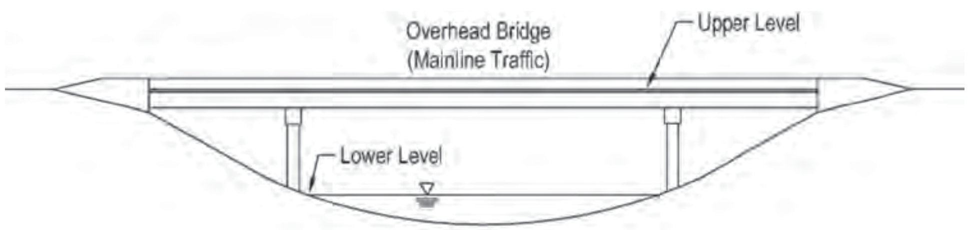

Figure 3.62. Mainline crossing road (single or twin).

- Use of only ADDCs.
- Use of only SPMTs.
- Launching or use of LTTBs or ADDCs.
- Launching or use of LTTBs or SPMTs.
- Use of ADDCs or SPMTs.
- Launching or use of LTTBs, ADDCs, or SPMTs.
- Sliding and lateral shifting.
- Launching, sliding, and lateral shifting or use of LTTBs, ADDCs, or SPMTs.

Step 1. Matrix Questionnaire A: Bridge Project Type

The first of the selection matrices was developed to sort projects quickly by project type. Once the bridge project type is selected, three subsequent questions sort and establish the worthiness of the project to warrant further consideration for ABC Construction Technologies.

- Can service be disrupted with full closures during construction of the bridge project?
  a. If full closures are allowed, the matrix directs the user to a follow-up question to test the worthiness for ABC consideration.
  b. If full closures are not possible, this becomes a key criterion to establish possible worthiness for ABC Construction Technologies and the matrix directs the user to a new matrix.
- Is a short construction duration critical to the project completion?
  a. If a shortened construction period is required, this becomes a key criterion to establish possible worthiness

for ABC Construction Technologies and the matrix directs the user to a new matrix.

b. If a shortened construction period is not mandatory, the matrix directs the user to a follow-up question to test the worthiness for ABC consideration.

- Is there full equipment access to the lower level of the project site?
  a. If equipment can be used on the lower level, the project at this point in the matrix has not demonstrated a mandatory reason to consider ABC Construction Technologies. Therefore, the matrix suggests that the user consider more conventional construction equipment and techniques.
  b. If equipment cannot access the lower level, this becomes a key criterion to establish possible worthiness for ABC Construction Technologies and the matrix directs the user to a new matrix.

Step 2a. Matrix Questionnaire AB: New Construction

Once the user has been directed to the New Construction matrix questionnaire, the probability of the project being worthy for consideration of an ABC Construction Technology is very good. A second series of questions specific to the project site will help the user further evaluate whether ABC Construction Technologies might be applicable and, if so, which technologies best suits the project criteria. Note, however, that the answers may also demonstrate that due to physical site constraints, ABC Construction Technologies may not be applicable for the project.

The New Construction matrix focuses on the physical aspects of the project site.

- Is there room or access nearby for a staging area on the lower elevation where complete or partial assembly of the new bridge can occur?
  a. The use of SPMTs requires the ability to preassemble the new bridge structure in an off-line staging area, away from the project site. If there are no open areas nearby, SPMTs may not be applicable; however, the matrix directs the user to other questions that may open the possibility for other ABC Construction Technologies.
- Is the staging area accessible to the bridge site without overhead, width, or grade restrictions?
  a. If an off-line staging area is available, the SPMTs and their cargo must be able to travel without interference from the staging area to the project site.
  b. If there are physical restraints preventing a clear travel path between the staging area and the project site, the matrix will direct the user to other questions that may open the possibility for other ABC Construction Technologies.
- Is there room behind one or both abutments for a staging area?
  a. If there is room behind one or both of the abutments for a staging area, launching or use of the LTTBs may be applicable.
  b. If there is no room behind the abutments for a staging area, at this point in the matrix, the physical constraints of the project site have severely limited the possibility of using ABC Construction Technologies. Therefore, although some of the design criteria indicate the need for ABC consideration, the matrix suggests the user consider more conventional construction equipment and techniques.
- Is a short duration closure or detour (1 to 2 days) allowed?
  a. If short closures are allowed, then SPMTs may be applicable.
  b. If short closures are not allowed, the work may have to be performed from above with launching techniques or by use of the LTTBs.
- Is there space adjacent to the proposed bridge location to preassemble the new structure?
  a. If there is a staging area adjacent to the proposed bridge location, the new bridge superstructure could be erected as a parallel activity to the substructure construction and the bridge eventually moved into position by using sliding or lateral shifting technologies.

Depending on how the questions are answered, the New Construction matrix questionnaire either will lead the user to the conclusion that the site constraints restrict the applicability of ABC Construction Technologies or will lead the user to one of the following set of ABC Construction Technology opportunities:

- Launching, sliding, or lateral shifting, or the use of SPMTs or LTTBs.
- Launching or use of SPMTs or LTTBs.
- Sliding or lateral shifting, or the use of SPMTs.
- Use of only SPMTs.
- Launching or the use of LTTBs.

Step 2b. Matrix Questionnaire AC: Bridge Widening

Once the user has been directed to the Bridge Widening matrix questionnaire, a series of questions focusing on criteria developed for traffic control, the number of bridges, and the type of widening proposed will help the user further evaluate the applicability of ABC Construction Technologies and, if applicable, which technologies best suit the renewal project criteria. Note, however, that the answers may also demonstrate that due to the ability to adjust the traffic flows, ABC Construction Technologies may not be required for the project.

The Bridge Widening matrix focuses on the traffic control, the number of bridges, and the type of widening proposed for the project.

- Can traffic on the lower level be partially disrupted by short-duration closures, shifting, or detours, or is the lower level devoid of traffic?
  a. If the traffic below the bridge has high volumes and has been determined to be unsafe for traffic flow adjustment, this becomes a key criterion to establish possible worthiness for ABC Construction Technologies.
  b. If the traffic below the bridge can be disrupted or adjusted or if there is no traffic below the bridge, the matrix points out to the user that ABC Construction Technologies may not be required for the project. This does not preclude ABC Construction Technologies from consideration but does demonstrate that conventional construction equipment and technologies can be considered.
- Is access for crane equipment limited on the project site?
  a. If crane access will be limited, this becomes a key criterion to establish possible worthiness for ABC Construction Technologies.
  b. If the site allows for easy crane access, the matrix points out to the user that ABC Construction Technologies may not be required for the project. This does not preclude ABC Construction Technologies from consideration but does demonstrate that conventional construction equipment and technologies can be considered.
- Is the project a twinning of a parallel structure or a widening of an existing structure?
  If the project is a twinning, the matrix directs the user back to the New Construction matrix questionnaire; otherwise, the user is directed to additional questions related to the widening of an existing structure. Note that if the user had declared that the twin bridge was new construction in the opening matrix, the user would have been directed to the same New Construction matrix.
- Is the existing structure a single bridge or twin bridges?
  a. If the bridge project consists of widening a single structure, the matrix directs the user to a new matrix specific to the widening of a single bridge.
  b. If the bridge project involves widening two parallel bridges, the matrix will ask additional questions related to traffic control design criteria.
- Can traffic from one direction be shifted to the other bridge?
  a. If the project involves twin structures and traffic from one direction cannot be shifted to the other bridge with a crossover, this becomes a key criterion to establish possible worthiness for ABC Construction Technologies.
  b. If the project involves twin structures and a temporary crossover can be constructed to move traffic completely off one bridge, the matrix points out to the user that ABC Construction Technologies may not be required for the project. This does not preclude ABC Construction Technologies from consideration but does demonstrate that conventional construction equipment and technologies can be considered.
- What type of widening is proposed?
  a. The user will be directed to the next matrix level based on the type of widening proposed for the twin set of bridges.
     1. Widen each bridge to the outside.
     2. Widen each bridge to the inside.
     3. Widen each bridge along both edges.

Depending on how the questions are answered, the Bridge Widening matrix questionnaire either will lead the user to the conclusion that the flexibilities related to adjusting traffic patterns do not mandate that ABC Construction Technologies be considered or will lead the user to one of the following matrix questionnaires:

- Widen Exterior Edges of a Single Bridge;
- Widen Both Interior and Exterior Edges of Twin Bridges; or
- Widen Interior Edges of Twin Bridges.

Step 2c. Matrix Questionnaire AD: Bridge Replacement

Once the user has been directed to the Bridge Replacement matrix questionnaire, a series of progressive questions will help the user evaluate the applicability of ABC Construction Technologies and, if applicable, which technologies best suit the renewal project criteria. Note, however, that the answers may also demonstrate that due to physical constraints of the project site, ABC Construction Technologies may not be applicable for the project.

The Bridge Replacement matrix will focus on availability of staging areas at the project site.

- Is a short-duration closure or detour (1 to 2 days) allowed?
  a. If short closures are allowed, SPMTs may be applicable.
  b. If short closures are not allowed, the work may have to be performed from above by using launching techniques or by use of the LTTBs.
- Is there room or access nearby for a staging area on the lower level, where complete or partial assembly of the new bridge can occur?
  a. The use of SPMTs requires the ability to preassemble the new bridge structure in an off-line staging area, away from the project site. If there are no open areas nearby, SPMTs may not be applicable; however, the matrix directs the user to other questions that may open the possibility for other ABC Construction Technologies.

- Is there space adjacent to the proposed bridge location to preassemble the new structure?
  a. If there is a staging area adjacent to the proposed bridge location, the new bridge superstructure could be erected as a parallel activity to the substructure construction, and the bridge eventually moved into position using sliding or lateral shifting technologies.
- Is the staging area accessible to the bridge site without overhead, width, or grade restrictions?
  a. If an off-line staging area is available, the SPMTs and their cargo must be able to travel (without interference) from the staging area to the project site.
  b. If there are physical restraints preventing a clear travel path between the staging area and the project site, the matrix will direct the user to other questions that may open the possibility for other ABC Construction Technologies.
- Is there room behind one or both abutments for a staging area?
  a. If there is room behind one or both of the abutments for a staging area, launching or use of the LTTBs may be applicable.
  b. If there is no room behind the abutments for a staging area, at this point in the matrix, the physical constraints of the project site have severely limited the possibility of using ABC Construction Technologies. Therefore, although some of the design criteria indicate the need for ABC consideration, the matrix suggests that the user consider more conventional construction equipment and techniques.

Step 3a. Matrix Questionnaire AC3: Widen Exterior Edges of a Single Bridge

If the user has been directed to the Widen Exterior Edges of a Single Bridge matrix questionnaire, the series of question will further focus on criteria developed for traffic control, availability of staging areas, and the proposed width of the widening, and will help the user evaluate the applicability of ABC Construction Technologies and, if applicable, which technologies best suit the renewal project criteria. Note, however, that the answers may also demonstrate that due to the ability to adjust the traffic flows, ABC Construction Technologies may not be required for the project.

The Widen Exterior Edges of a Single Bridge matrix focuses on the traffic control, the availability of staging areas, and the width of the widening proposed for the project.

- Can traffic on the bridge be reduced to allow partial closures along each edge of the bridge?
  a. This question will help establish the boundaries for possible erection equipment.
  b. If the ability to safely constrict the traffic lanes to create space along the bridge edges for the contractor to run equipment exists, ADDCs may be applicable.
  c. If the space along the bridge edges must be kept to a minimum, there may not be sufficient room for construction equipment to be staged on the bridge.
- Is there room behind one or both abutments for staging area(s)?
  a. If there is room behind one or both of the abutments for a staging area, launching or the use of LTTBs may be applicable.
  b. If there is no room behind the abutments for a staging area, at this point in the matrix, the physical constraints of the project site have severely limited the possibility of using ABC Construction Technologies. Therefore, although some of the design criteria indicate the need for ABC consideration, the matrix suggests that the user consider more conventional construction equipment and techniques.
- Is the widening a single girder or multiple girders?
  a. If the widening involves multiple girders, there may be an option to preassemble major, if not all, components of the widened bridge structure. This would open opportunities for launching or the use of SPMTs.

As a result of completing the matrix, owners and engineers will know whether the site and traffic constraints minimize, restrict, or promote the need for ABC Construction Technologies. If ABC Construction Technologies are promoted, they will be categorized into the following set of opportunities:

- Launching or the use of ADDCs or LTTBs, or explore the use of SPMTs.
- Launching or the use of ADDCs or LTTBs.
- Use of ADDCs or explore the use of SPMTs.
- Use of only ADDCs.
- Launching or the use of LTTBs, or explore the use of SPMTs.
- Launching or the use of LTTBs.

Step 3b. Matrix Questionnaire AC4: Widen Both Interior and Exterior Edges of Twin Bridges

The logic behind the questions in this matrix match those discussed for Matrix Questionnaire AC3. If the widening on twin bridges is to only exterior edges, the results of the question set will exactly match the results of the question set of the AC3 matrix. Note, however, that due to the physical restrictions of widening the interior edges of twin bridges, some of the potential ABC Construction Technologies available for widening exterior edges will not be applicable to widening the inside edges.

As a result of completing the matrix, owners and engineers will know whether the site and traffic constraints minimize, restrict, or promote the need for ABC Construction Technologies. If ABC Construction Technologies are promoted, they will be categorized in the following set of opportunities:

Exterior Edges
- Launching or the use of ADDCs or LTTBs, or explore the use of SPMTs.
- Launching or the use of ADDCs or LTTBs.
- Use of ADDCs or explore the use of SPMTs.
- Use of only ADDCs.
- Launching or the use of LTTBs, or explore the use of SPMTs.
- Launching or the use of LTTBs.

Interior Edges
- Launching or the use of ADDCs or LTTBs.
- Use of only ADDCs.
- Launching or the use of LTTBs.

Step 3c. Matrix Questionnaire AC5: Widen Interior Edges of Twin Bridges

The logic behind the questions in this matrix match those discussed for Matrix Questionnaire AC3 with the exception that, due to the physical restrictions of widening interior edges, some of the potential ABC Construction Technologies available for widening exterior edges will not be applicable to widening the interior edges.

As a result of completing the matrix, owners and engineers will know whether the site and traffic constraints minimize, restrict, or promote the need for ABC Construction Technologies. If ABC Construction Technologies are promoted, they will be categorized in the following set of opportunities for interior edges.

- Launching or the use of ADDCs or LTTBs.
- Use of only ADDCs.
- Launching or the use of LTTBs.

Depending on how the questions are answered, the matrix either will lead the user to the conclusion that the site constraints restrict the applicability of ABC Construction Technologies or will lead the user to one of the following set of ABC Construction Technology opportunities:

Demolition of the Existing Bridge
- Demolition with short-duration closures.
- Complete removal with the use of SPMTs, followed by demolition off-site.
- Removal by sliding or lateral shifting, followed by demolition with short-duration closures.

Installation of the New Bridge
- Launching, sliding, or lateral shifting, or the use of SPMTs or LTTBs.
- Launching or the use of SPMTs or LTTBs.
- Sliding or lateral shifting, or the use of SPMTs.
- Use of only SPMTs.
- Launching or the use of LTTBs.

To test the logic of the series of ABC Construction Technology Selection Matrix questionnaires, a random sample of past projects was evaluated by using the progression of questions presented in the various matrix questionnaires. The results of the evaluations were then compared with the actual construction technology used to complete the project.

ABC Construction Technology Design Checklists

It is assumed that once owners or engineers have completed the series of ABC Construction Technology matrix questionnaires, they will have been directed to a variety of nonconventional ABC Construction Technology options for their renewal project. It is also assumed that owners and engineers used the evaluation matrices presented in previous parts of this report to perform a concurrent, yet independent, evaluation of the ABC substructure and superstructure systems to determine structure types for their renewal project.

Once the evaluations of the two matrices are complete, the results can be reviewed and the best ABC Design and ABC Construction Technology can be selected. Once an ABC Construction Technology has been selected, owners and engineers must integrate this technology into the bridge design and must consider a new set of technical issues and challenges specific to that technology.

To assist owners and engineers in this design coordination process, the second goal of this part of the Phase II report on ABC Construction Technologies was to develop checklists of items that must be addressed during the design phase of a project. To be useful to owners and engineers, these lists were tailored to the specific needs of each ABC Construction Technology.

Development of Standard Concept Details for ABC Construction Technologies

To assist owners and engineers with the selection of an ABC Construction Technology, the third goal of this part of the Phase II report on ABC Construction Technologies was to develop a set of standard conceptual details defining terminology and demonstrating the possibilities and limits of each ABC Construction Technology.

Recommendations for Further Development of ABC Construction Technologies

This section focused on the following:

- Developing a matrix of questions for owners and their consultants to assist with the proper selection of a construction technology;
- Developing a checklist of items that owners and their consultants must address during the design and construction phases of a project; and
- Developing of a set of standard conceptual details that owners and their consultants could use as a guide for visualizing the possibilities and defining the limits of the four specified ABC Construction Technologies.

Future development of ABC Construction Technologies could evolve around the demonstration of which technologies work best with the ABC Designs (both substructure and superstructure) proposed in this report. Additional exhibits could be developed and attached with the specific ABC Designs.

Automation of the selection matrix would also be a viable future development for the ABC Construction Technologies. Automation could evolve with the use of a Microsoft Visual Basic program embedded within an Excel file. The program would include a page of questions matching those listed in the matrix questionnaires and would use the selected responses to make recommendations for a construction technology (whether it be conventional or one or more of the ABC Technologies).

Recommended Design and Construction Concepts

The role of Phase II was to provide a critical evaluation of the recommended concepts and technologies from Phase I so that an incremental shortlist of technologies could be created and standardization of details and field demonstrations could be recommended. Standardizing ABC systems will bring about greater familiarity about ABC technologies and concepts and will also foster greater regional cooperation to achieve region-specific customization that will accommodate regional practices and industry needs. Pre-engineered standards to be developed in this project will emulate cast-in-place construction but will be optimized for modular construction and ABC. These standards can be inserted into project plans with minimal additional design effort to adapt to project needs. Using these standardized designs will serve as a training tool to increase familiarity about ABC among engineers.

Modular Bridge Systems for ABC

The modular superstructure and substructure systems will be developed in this project to achieve cost and risk reductions through the adoption of the guiding philosophy for all ABC concepts advanced in this project. This philosophy is stated as follows:

- As light as possible
  - Simplify transportation and erection of bridge components.
- As simple as possible
  - Fewer girders, splices, or bracings.
- As simple to erect as possible
  - Fewer workers on site;
  - Fewer fresh concrete operations;
  - No falsework structures required; and
  - Simpler geometry.

The successful use of prefabricated elements to accelerate construction requires a careful evaluation of the requirements for the bridge and an unbiased review of the total costs and benefits. This part provides a review of the engineering and constructability evaluations, pinpoints implementation challenges, and provides suggestions to overcome those challenges. The recommended concepts from this evaluation have been further screened to provide a shortlist of the most promising technologies for standardization in Phase III and for use in the field demonstrations. The recommended technologies meet minimum standards of readiness for execution, suitability for ABC, and the promise of durability, economy, and value to the owner.

Minimizing road closures and traffic disruptions is a key objective of ABC. For ABC systems to be viable and see greater acceptance, the savings in construction time should be clearly demonstrated. As outlined in Phase I, ABC Design concepts can be classified into three categories:

- Tier 1: ABC concepts that can be completed over a weekend closure;
- Tier 2: ABC concepts that can be completed in a few weeks; or
- Tier 3: ABC concepts that accelerate larger projects and save weeks or months on the overall schedule.

Modular bridge systems are particularly suited to be used as Tier 1 concept for weekend bridge replacements or as Tier 2 concept when the entire bridge may be scheduled to be replaced within a month with a detour to maintain traffic. Complete prefabricated modular systems and construction technologies recommended to be advanced to standard plans and conceptual details in Phase III are presented under the following headings:

- Precast modular abutment systems
  - Integral abutments;
  - Semi-integral abutments; and
  - Precast approach slabs.
- Precast complete pier systems
  - Conventional pier bents; and
  - Straddle pier bents.
- Modular superstructure systems
  - Concrete deck bulb tees;
  - Concrete deck double tees; and
  - Decked steel stringer system.
- ABC bridge erection systems
  - Above-deck driven carriers; and
  - Launched temporary truss bridges.

These design standards aim to overcome known obstacles to widespread adoption of ABC as a preferred method of bridge replacement. Standardizing ABC systems has many benefits that could encourage greater use of these systems in bridge renewals. Overcoming obstacles to implementing the modular superstructure and substructure systems will depend on how effective these systems are in addressing owner and contractor concerns on ABC expressed in the focus group meetings and enumerated in the Task 6 report. ABC and precast construction are viewed by local contractors as projects requiring large amounts of outsourcing of work. Standardized structural systems that general contractors can fabricate as much as possible themselves without having to subcontract out to specialty precasters or fabricators will allow contractors to keep their crews employed and maximize profits.

Perhaps the greatest impediment to increased use of ABC appears to be the higher initial costs. ABC is perceived as raising the level of risk associated with a project. Standardized modular superstructure systems for ABC are aimed at increasing their availability through local or regional fabricators, thereby reducing costs and lead times. Standardizing also makes these systems more familiar to engineers, owners, and contractors, which will reduce complexity and the level of risk associated with the project. Contractors will be more willing to offer competitive bids on a system they have experience with and that they perceive to be proven and easily constructible. Repeated use of a standardized ABC Design also allows owners and engineers to iron out the kinks in the system through continuous improvement, leading to a more superior design than a onetime customized solution. Repeated use of these systems will also encourage contractors to provide their own suggestions about how constructability could be further improved, which will lead to further reductions in overall risks and cost.

Concerns have also persisted about the durability of joints and connections in precast elements. Many earlier ABC systems used grouted joints that did not prove to be very durable, especially under heavy traffic conditions. Seismic performance of precast elements and connections in seismic regions has also been an issue. With these concerns in mind, the modular superstructure systems advanced in this project have placed maximum emphasis on developing durable connection details between the prefabricated elements. Full moment connections using ultra-high-performance concrete (UHPC) or high-performance concrete (HPC) are being recommended as the preferred connection details because they are strong, durable, and seismically sound. Standardizing these connections with proven, easy-to-construct details will go a long way in overcoming the past concerns with performance of joints and connections.

Lack of access for equipment and the need for large staging areas unavailable in urban locations have been a hindrance to large scale prefabrication. Moving complete bridges using specialized wheeled carriers requires large staging areas, which may be in short supply in congested urban areas. Modular systems allow the superstructure to be built in place using smaller components, thus overcoming mobility issues. In short, modular systems allow a more versatile option to ABC not limited by the space available at the bridge site. Standardized designs geared for erection using conventional erection equipment will allow repetitive use of modular superstructure systems, which will make contractors more willing to invest in equipment based on certain methods of erection to speed assembly. Repetitive use will allow contractors to amortize equipment costs over several projects, which is an important component in bringing overall costs in line with conventional construction.

The use of pre-engineered systems in bridge engineering is commonplace. Many states have decided to make best use of their program dollars by greatly standardizing design through the development of pre-engineered systems, plans, and so forth that encompass entire bridge systems including the quantity take-off for various standard configurations. In this environment, engineering need be done only once, to create the standard, so that quality assurance and quality are satisfied in the creation of an accurate standard. The greater portion of program funds can be used for construction instead of design. The use of pre-engineered bridge systems has led to low in-place constructed costs.

The research team believes that a transition from pre-engineered but stick-built systems to pre-engineered and prefabricated systems is a worthy objective of this project. A number of state DOTs, such as those in Utah, Idaho, and

Washington State, have already developed extensive sets of pre-engineered bridges for precast construction or rapid renewal. These serve as the team's starting point.

It is not the intent of the proposed work to redo all of this work and publish many additional sets of pre-engineered bridge plans like those mentioned. This is far beyond the resources this contract provides. Rather, to the extent possible, the research team will collect, synthesize, and build upon the most useful sets of existing pre-engineered bridge plans from a variety of states and recommend modifications of their details (connections, erection methods, etc.) to make them sufficiently suitable for accelerated construction and national use. The research findings from this project, including the survey and focus group findings, lab test results, and lessons learned from the demonstration project, will provide the basis for making significant enhancements to existing ABC systems and for developing new ABC Designs that address these impediments to greater ABC use.

Lab Testing of UHPC Joints

The durability of full-depth deck joints between prefabricated panels has been a major concern for many years. Unless posttensioned, these joints may allow penetration of water and chemicals leading to corrosion. Posttensioning of bridge decks to induce compression in joints, however, has traditionally been a time-consuming field operation not compatible with ABC. UHPC was investigated as a possible solution to this problem because its high bond strength to reinforcing bars allows narrow joints, its relatively high bond strength to precast concrete may negate the need for posttensioning, and its low permeability enhances long-term durability.

UHPC has been tested and implemented as a deck joint material in several joint details, including two instances in the United States—in the positive bending moment region transverse deck joints of Route 23 Bridge in Oneonta, New York, and in the longitudinal deck joints of the Route 31 Bridge in Lyons, New York. This work focuses on a UHPC joint application previously untested, namely in a transverse joint located in the primary negative bending region over the piers of a continuous bridge. This detail was developed for use in a three-span demonstration bridge over Keg Creek on U.S. Hwy 6 in Pottawattamie County, Iowa. The joint investigated in this study is of high significance because if successful, it would enable this prefabricated deck construction strategy to be applied not only to single-span bridges but also to multiple-span, continuous-under-live-load bridges, which are very common throughout the United States and the world. Unlike other UHPC deck joints implemented in the past, this joint will be subjected to significantly higher levels of tensile stresses that will be oriented to be perpendicular to the joint itself.

Three suites of laboratory tests were conducted to evaluate the UHPC deck joints used in the demonstration bridge. The lab tests conducted for this study include abrasion testing of the UHPC closure joint material, constructability testing of the intersecting deck joints, and strength and serviceability testing of the transverse deck joint at the pier. Posttensioning details were added to the Keg Creek transverse joints on the basis of findings of these lab tests.

The testing of UHPC joints was performed by Iowa State University (ISU) and covered three primary areas of interest that were considered critical to the use of UHPC in ABC applications and in the Keg Creek demonstration project. The lab tests that were conducted and their objectives follow.

- Abrasion Testing (Test 1): Grinding of the UHPC closure joint material for the longitudinal and transverse joints
  - Evaluate the grindability of the cast-in-place UHPC in relation to the accelerated construction schedule.
- Constructability Testing (Test 2): Placement, handling, and quality of the UHPC material at the intersecting closure joints
  - Evaluate the constructability of intersecting cast-in-place UHPC joints with respect to the flow characteristics and properties of the material.
  - Qualitatively assess the feasibility of the UHPC joint placement procedure.
- Serviceability and Strength Testing (Test 3): Evaluation of the strength and serviceability of the transverse bridge deck joint at the pier
  - Evaluate the negative bending performance of the module-to-module transverse connection detail at the piers.
  - Determine the cracking moment at this location.
  - Determine the ultimate moment capacity at this location.

The ISU report on the UHPC testing is included in Appendix C and provides a detailed explanation on the testing program and findings. A summary is provided in this chapter.

Abrasion Testing

Abrasion testing of cast-in-place UHPC was conducted to determine the early age grindability of the material when used in the demonstration bridge. Specifically, identifying a period of time in which the contractor is able to grind the joint material without causing damage to the joints or equipment had to be identified. For the demonstration bridge it was specified that the UHPC closure joint attain 10,000 psi of compressive strength before it should be ground. This test helped determine the relative ease of grinding for this material after the 10,000 psi threshold has been reached. Experimental variables for this test included the maturity of the UHPC and the specimen surface finish. Testing of the UHPC material for abrasion resistance was completed at Iowa State University in February and March 2011.

Three surface finish types were tested for grindability during the abrasion testing: a rough top surface, a diamond cut surface, and a smooth formed surface, as shown in Figure 3.63. The UHPC specimens were cured at 40°F, 70°F, and 100°F, and tested 2, 4, 7, and 28 days after casting. To evaluate the UHPC material for grindability, testing was completed following ASTM C944, Standard Test Method for Abrasion Resistance of Concrete or Mortar Surfaces by the Rotating-Cutter Method. Figure 3.63, a plot of the percentage of mass loss versus compressive strength for the three surface finish conditions, presents the results of the abrasion testing.

Based on the compressive strength test results for the demonstration bridge UHPC mix design, the UHPC will reach the 10,000 psi compressive strength required for grinding in the project specifications for the demonstration bridge at approximately 2 days if cured at 70°F. The 14,000 psi compressive strength threshold, required in the demonstration bridge project specifications for opening the bridge to traffic, will likely be reached 4 days after placement. Thus, the contractor will have roughly 2 days to perform grinding of the joints: from the time the 10,000 psi threshold is reached prior to the opening of the bridge to traffic at 14,000 psi compressive strength. The percentage mass loss for both formed and top finishes at the 10,000 psi compressive strength threshold is approximately 0.12%. At 14,000 psi compressive strength of the UHPC mix, the percentage mass loss is approximately 0.07%. Over that 2-day duration of time, the UHPC's resistance to abrasion increases by approximately 40%. That would be a significant factor for the contractor in terms of grinding time and accelerated scheduling.

Figure 3.63 illustrates that the formed surface and rough surface finishes displayed the lowest abrasion resistance. Specimens with formed surface finishes exhibited lower

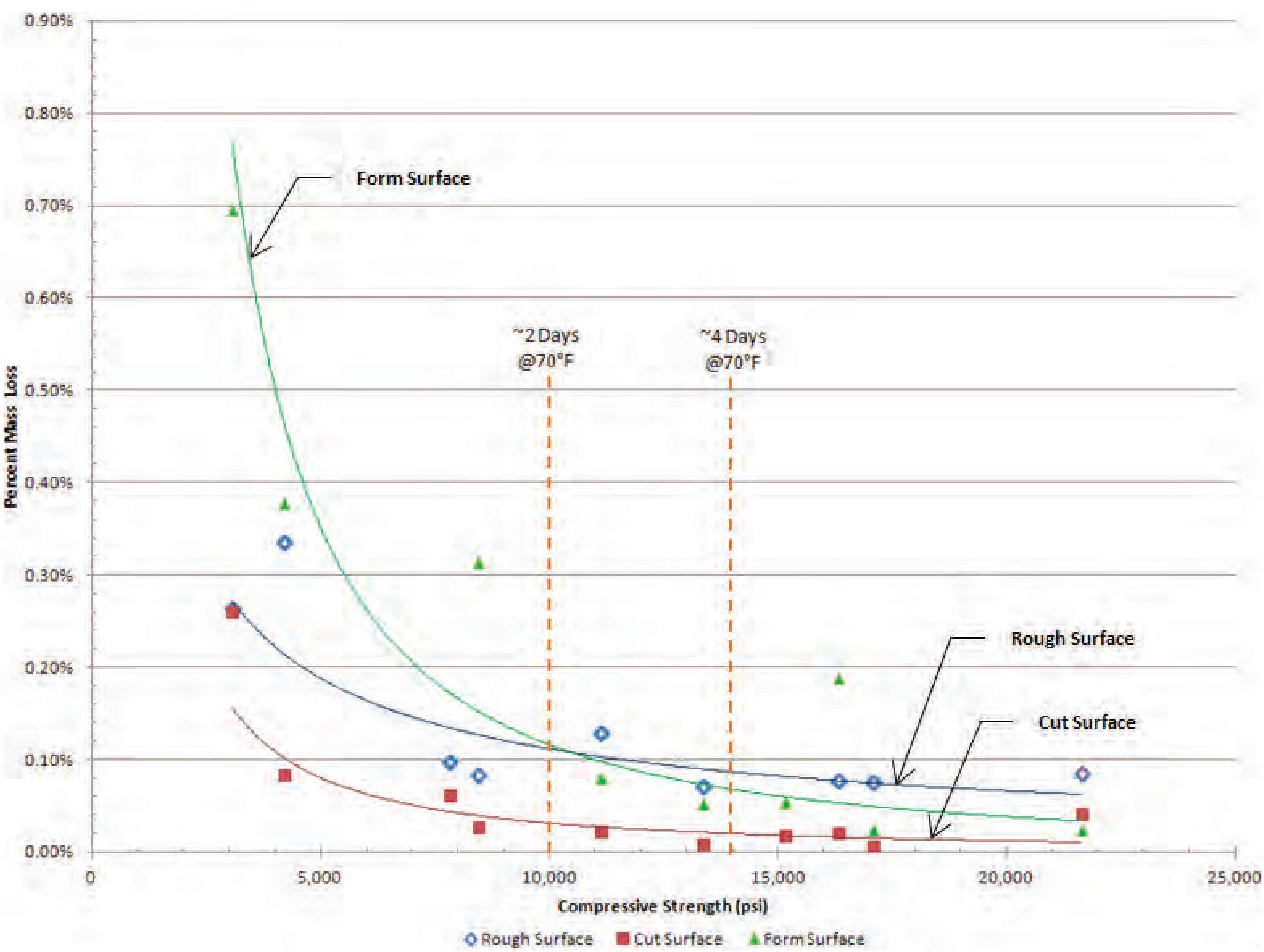

Figure 3.63. Abrasion testing: Percentage of mass loss versus compressive strength.

abrasion resistance than did cut surfaces because of the steel fibers present in the UHPC. At the formed surface, the steel fibers were aligned preferentially, parallel with the surface. Thus, the fibers tended to pull off easily. The fibers lay parallel with the form surface because as the UHPC flowed along the bottom of the form, the fibers tended to align and lie flat. The rough surface finish generally also included small entrapped air bubbles that allowed for easier removal of the UHPC material. As was expected, the cut surface finish had the highest abrasion resistance. Because the cast-in-place UHPC joints in the Project R04 demonstration bridge are to have a formed surface, the abrasion resistance in the field is expected to most nearly resemble that of the formed surface finish observed in the abrasion tests.

Constructability Testing

Joint constructability testing was completed to qualitatively evaluate the intersecting, cast-in-place UHPC deck joints to be used in the demonstration bridge. Specifically, a full-scale mock-up of the intersection between one longitudinal and one transverse UHPC deck joint was constructed to investigate issues relating to casting sequence, material mixing and placement rates, effects of ambient temperature on construction, flow characteristics of the UHPC, and consolidation of material at congested locations. Testing of the UHPC joints for constructability was completed at Iowa State University in April 2011.

Casting Sequence

The original proposal for the construction sequence of the demonstration bridge outlined continuous placement of the entire grid of UHPC deck joints (longitudinal and transverse). Through discussions with the engineer, contractor, and material supplier, several logistical issues arose that challenged the feasibility of full deck continuous placement. Typical mixers used by Lafarge Canada for UHPC placement mix 5.11 ft^3 per batch. On the job site, the mixers are used in pairs in order to provide a continuous supply of UHPC. Each batch is then discharged into buggies and transported onto the bridge to the placement location.

With the large volume of UHPC necessary in the bridge deck joints, continuous placement could be achieved only by using a large number of mixers and laborers. Without employing many mixers and laborers, cold joints could potentially form in the UHPC deck joints. As an alternative, stay-in-place acrylic vertical bulkheads were proposed by Lafarge to control the location of potential cold joints. A prototype of the stay-in-place acrylic vertical bulkheads was fabricated and used during the joint constructability testing, so its performance could be evaluated.

Ambient Temperature Effects on UHPC

The extent of the susceptibility to variations in temperature for the workability and flow characteristics of the UHPC mix design was observed during batching of the joint constructability test specimen and the transverse joint strength and serviceability test specimen that followed. Ambient air temperatures were steady at around 65°F at the time of batching for the intersecting joint specimen. However, during the batching for the transverse joint strength and serviceability specimen, ambient temperatures were 75.5°F. Without compensating for the change in ambient air temperature, the flow characteristics of the mixes were much different.

When ambient temperatures were 65°F, the temperature of the UHPC on discharge from the mixer ranged from 82°F to 85°F for the intersecting joint specimen's three batches. Within this range, the UHPC had acceptable flow characteristics for placement. The temperature of the UHPC on discharge from the mixer for ambient temperatures around 75.5°F was over 100°F. At this ambient temperature, the UHPC never reached its anticipated flow characteristics in the mixer, thus the batch was rejected. To correct the issue, water in the mix design was replaced by mass with ice and the UHPC material temperature was reduced. This modification, the replacement of water by mass with ice, enabled extended working time and improved the flow relative to the previous batch.

Flow Characteristics and Consolidation of UHPC

Evaluating the flow of the UHPC around the corners at the intersection of the longitudinal and transverse deck joints was a critical aspect for this test. Adequate consolidation of the UHPC in the joint cross section around steel reinforcement is important to the deck joint performance. During UHPC placement, when the final mix temperature was limited to a maximum of 85°F, the UHPC material appeared to have adequate flow characteristics to achieve good consolidation and flow around corners at the intersections of longitudinal and transverse joints.

After the specimen was cured and removed from the forms, it was cut into several sections to examine consolidation and potential cold joints. Upon investigation of the cut specimen, no significant voids around steel reinforcing bars were observed. The test also validated the use of top forms and chimneys at the high end of the 2% cross slope at transverse joints. The top forms were applied sequentially as the joint was filled from the lowest elevation to the highest. The chimneys provide additional hydrostatic head in the freshly placed UHPC to aid in consolidation within the joint. It was suggested that top forms and chimneys be used in the demonstration bridge.

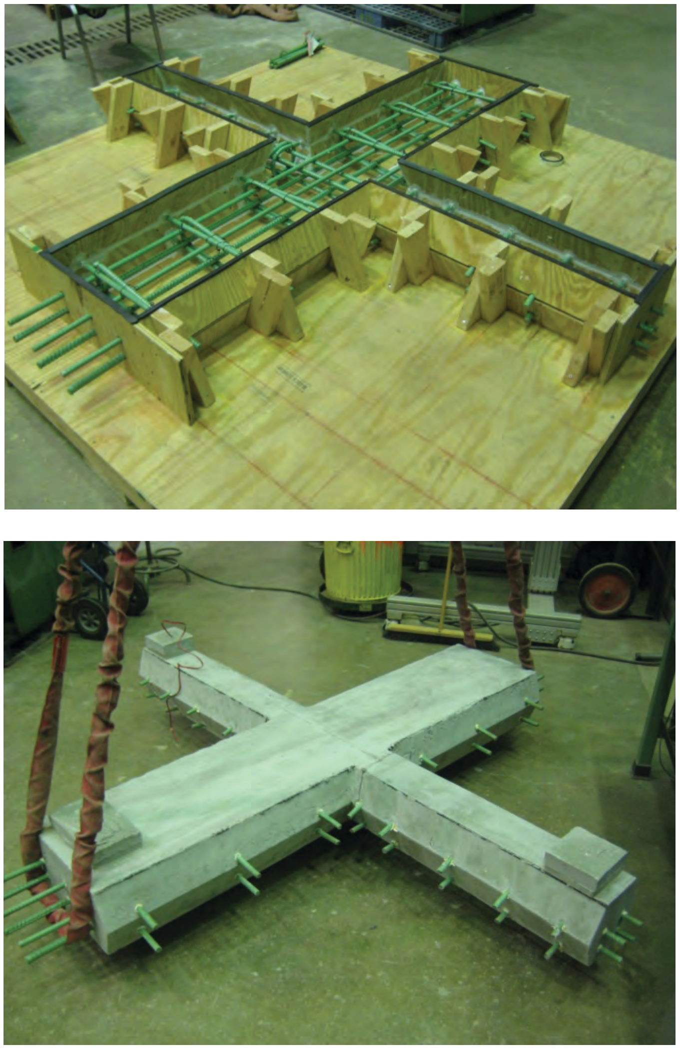

Figure 3.64. Joint intersection form (top) and joint intersection specimen (bottom).

Joint Intersection Detail Recommendations

Final inspection of the specimen upon removal from the forms allowed for additional observations and recommendations. The proposed stay-in-place acrylic bulkhead successfully allowed for sequential placement of the UHPC, but also created a possible infiltration plane where water and chemicals could access the embedded steel joint reinforcement. The reinforcement cage and the finished product are shown in Figure 3.64.

To maintain sequential placement of UHPC in the deck joint grid and avoid possible infiltration planes, a detail for a partial-height, removable acrylic bulkhead was developed and suggested for use in the demonstration bridge, as shown in Figure 3.65.

Removable acrylic bulkheads should be used in the longitudinal joint in compression zones where possible. Placing the bulkheads at these locations will provide better continuity at the interface between the hardened and freshly placed UHPC, which will help prevent the ingress of water and other chemicals.

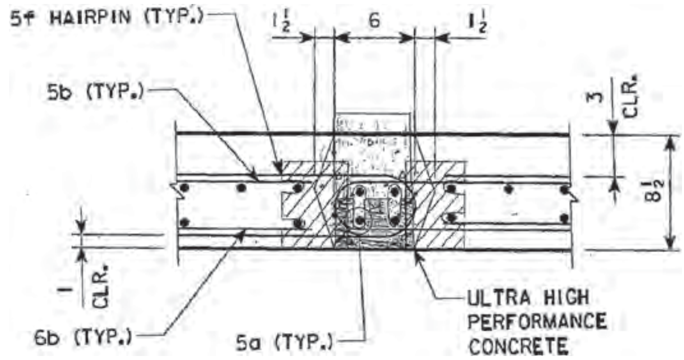

Figure 3.65. Removable acrylic bulkhead.

Transverse Joint Strength and Serviceability Testing

The module-to-module transverse connection used in the SHRP 2 ABC demonstration bridge was a unique and critical detail that had never been implemented in a bridge nor tested to quantify structural performance, as shown in Figure 3.66. Strength and serviceability testing of the module-to-module transverse connection was performed to evaluate the negative bending performance of this detail over the piers, determine its cracking moment, and verify the ultimate moment capacity. Testing of the module-to-module transverse connection was completed at Iowa State University from July to October 2011.

Service-Level Static Test

Load testing through live-load Service Level II moment was completed on the specimen. Loading was completed at 5,000-lb increments in order to complete visual inspection of the specimen and to check for the appearance of cracks and accrual of damage. Strain levels were monitored with the embedded and surface-mounted strain gauges located throughout the specimen. Strain levels for surface-mounted strain gauges at locations that spanned the HPC/UHPC interface exceeded 110με, the HPC cracking strain, at approximately halfway to Service Level I moment, as shown in Figure 3.67. The disparity between immediately adjacent gauges and the strains registering in excess of the HPC cracking strain across the interface suggested debonding and an opening at the interface between the precast HPC deck and the UHPC joint. This raised concern and called the bond strength of the UHPC to the precast concrete into question.

Visual inspection of the joint interface at Service Level II confirmed the debonding and substantial opening of the

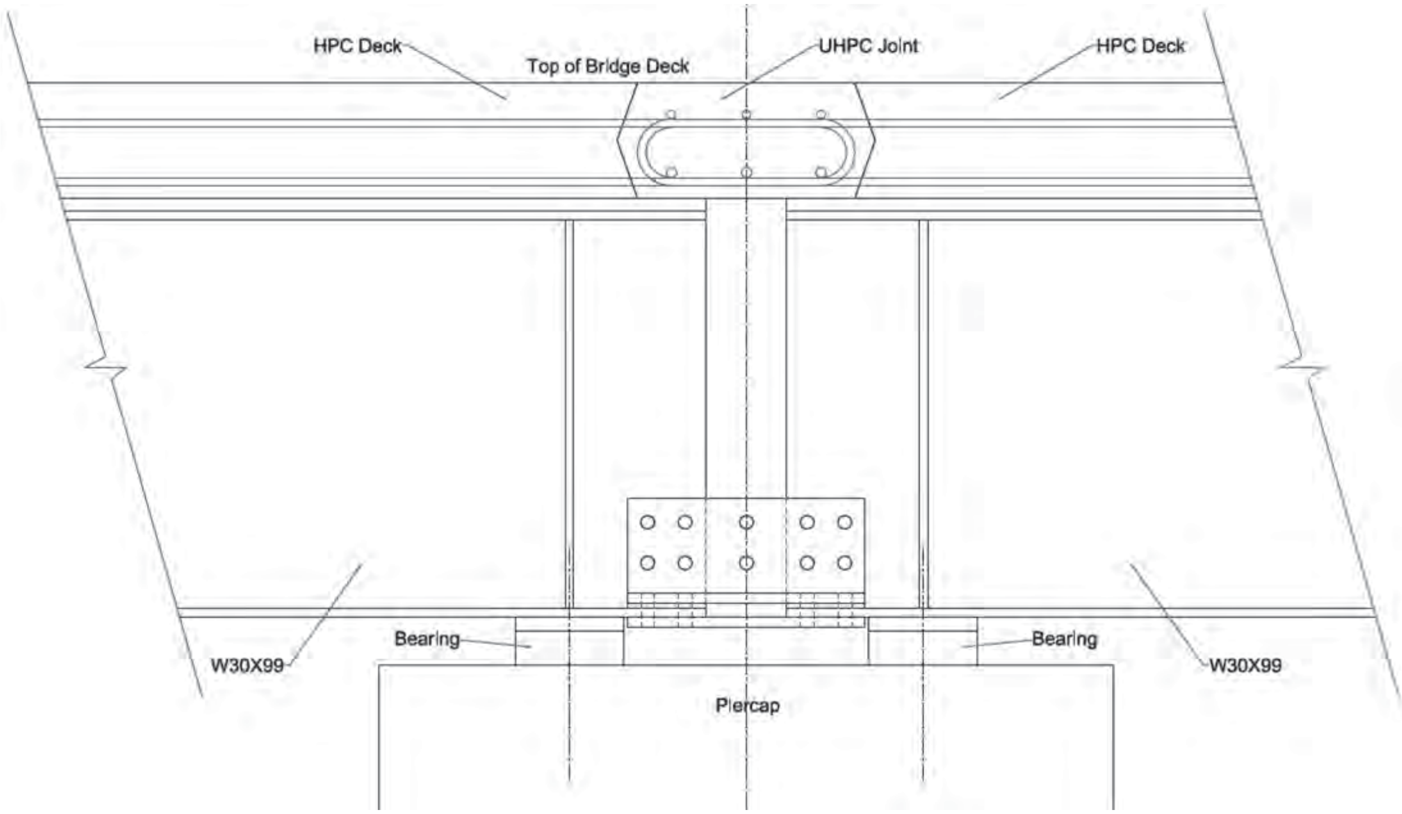

Figure 3.66. Module-to-module transverse connection detail.

interface that was suggested in the strain gauge data, as shown in Figure 3.68. Later, inspection during fatigue testing further confirmed the interfacial debonding and opening occurring below service-level conditions.

In addition to joint interface debonding and substantial opening, strain levels in the embedded strain gauges also registered above the expected HPC cracking strain prior to reaching Service Level I moment conditions, as shown in Figure 3.69. In the top-of-deck reinforcement, maximum strains of 540με, 550με, and 475με were recorded in gauges S1-1-1T, S2-2-2T, and S2-3-2T, respectively. Strains in the UHPC joint were relatively lower in the top-of-deck, not exceeding 160με, which is below the expected UHPC cracking strain level of 250με. Nearly all gauges located at the termination of the joint hairpin bar registered strain levels exceeding 110με prior to the Service Level II conditions. These data suggested cracking in the prefabricated HPC deck modules under service-level loading. Cracking was not visually confirmed near the joint in the HPC deck during the incremental static loading, but opening and closing of the cracks during cyclic loading made cracking in the HPC clearly visible.

The prevalence of the high strains at the termination of the hairpin reinforcement in the top-of-deck suggests cracking of the HPC would be expected for the demonstration bridge, indicating that the transverse connection detail might not satisfy the original project aim to avoid cracking in the deck over the pier.

After the static tests were completed, fatigue testing commenced. Fatigue tests consisted of loading the specimen through the full service-level moment range for 1,000,000 cycles. The embedded strain results for the fatigue testing generally resembled those from the static testing. Similarly, the gauges near the interface consistently exhibited the highest strains while the gauges within the UHPC registered the lowest in each of the instrumentation rows. Some higher strain levels at 1,000,000 cycles when compared to the static testing results suggested propagation of cracking and damage accrual within the specimen. On inspection at 250,000 cycles, cracks were identified in the precast deck. At 500,000, 750,000, and 1,000,000 cycles, further visual inspection confirmed propagation of the existing cracks and formation of new full-depth cracks in the precast deck panels up to 10 ft away from the joint.

To mitigate the serious durability concerns that arose with respect to the early debonding of the UHPC to precast interface and cracking in the HPC deck panels, a modified detail,

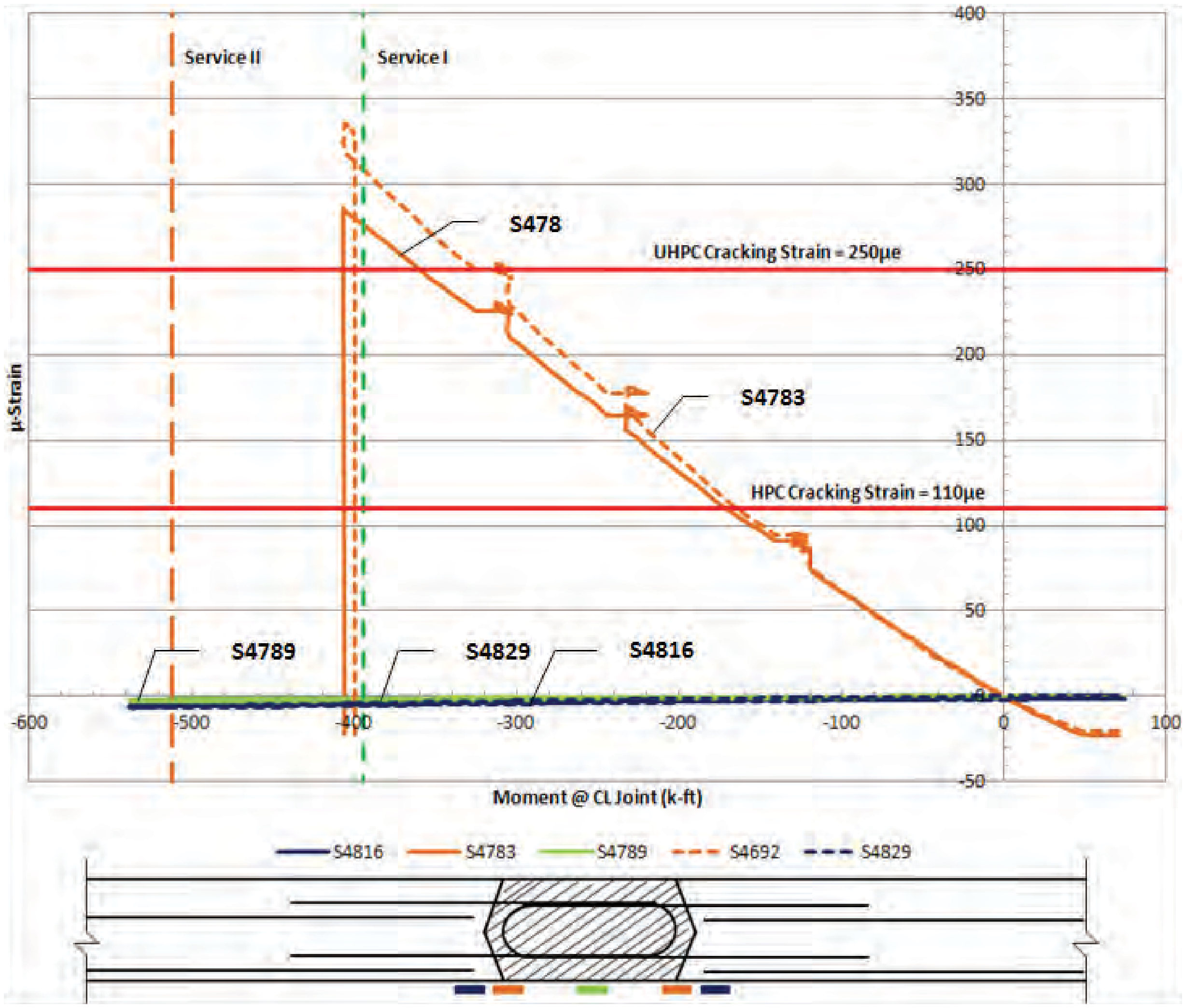

Figure 3.67. Selected surface-mounted strain gauges adjacent to the joint interface.

as shown in Figure 3.70, was devised to posttension the deck in this region and minimize tensile stresses in the concrete through Service Level II moments without compromising the accelerated construction aspect of the project.

The transverse module-to-module connection detail was modified to include high-strength steel rods mounted just under the deck surface to posttension the entire joint region. The retrofit detail was tested through the full range of service-level moments with a 60-kip posttensioning force per rod and again with a 70-kip posttensioning force per rod.

The 60-kip posttensioning force in each of the rods reduced tensile strain across the joint interface such that the HPC cracking strain was not reached until Service Level I conditions. However, strains did exceed the HPC cracking strain in the top-of-deck embedded gauges before reaching Service Level II.

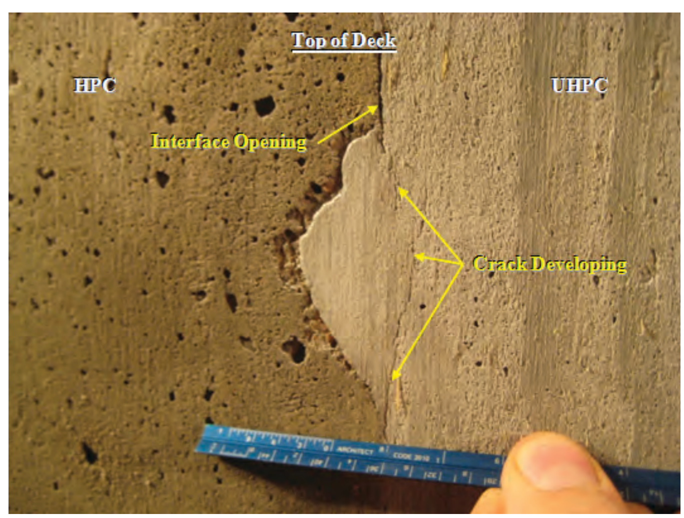

Figure 3.68. Joint interface opening.

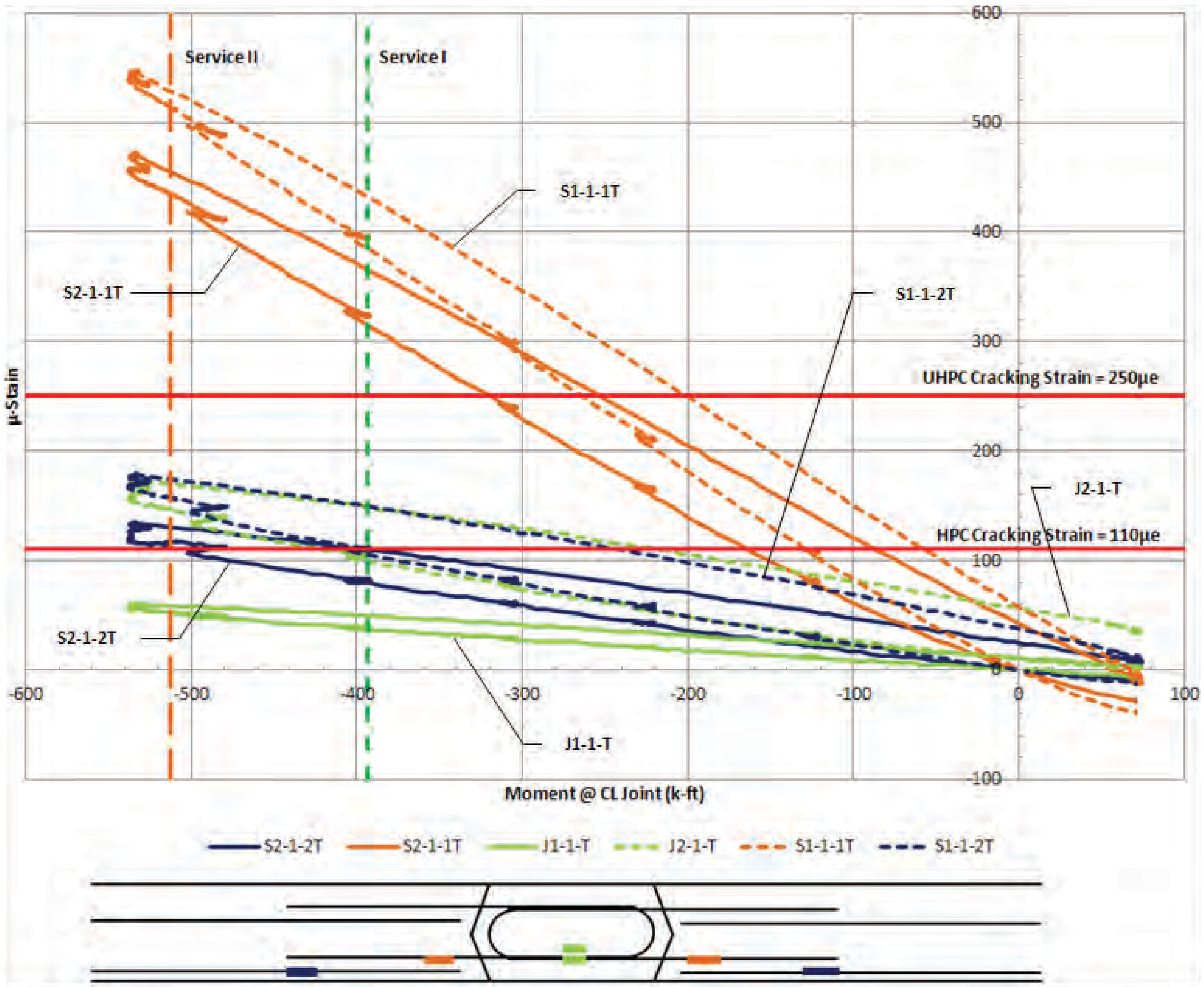

Figure 3.69. Row 1, top-of-deck embedded strain gauges (static).

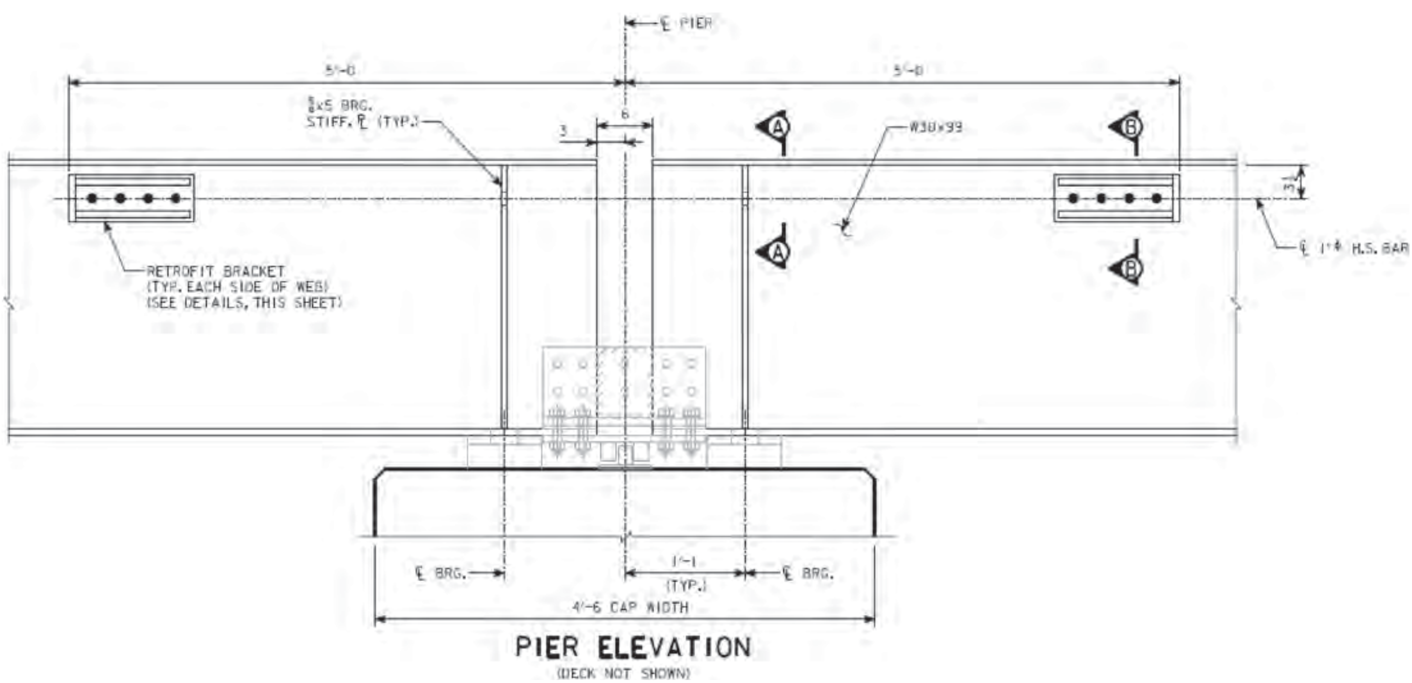

Figure 3.70. Connection retrofit detail.

By contrast, applying 70 kips posttensioning force in each of the rods minimized or negated the tensile strain across the interface entirely when loaded to Service Level I. All surface-mounted strain gauges spanning the interface registered below the HPC cracking strain until after the Service Level I conditions were exceeded, as shown in Figure 3.71. Tensile strain data across the interface revealed a maximum 29με at Service Level I moment. All embedded strain gauges, top- and bottom-of-deck, did not exceed 110με until Service Level II conditions were applied. The 70 kip per rod posttensioning force was recommended for application in the SHRP 2 Project R04 demonstration bridge.

Ultimate Capacity Test

Upon completion of static testing for the modified detail, the posttensioning rods were removed and the transverse module-to-module connection detail was tested to ultimate moment capacity. Strain data for the embedded gauges were analyzed in combination with qualitative observations to determine the failure mechanism for the transverse module-to-module connection detail.

As loading incrementally increased, the opening at the interface between the HPC deck and the UHPC joint widened. In addition, cracks from service-level testing propagated and widened throughout the precast deck, as shown in Figure 3.72. As the specimen was pushed well beyond service-level moments, reinforcement in the HPC deck near the UHPC interface began to yield. Eventually, the moment-displacement curve entered into the nonlinear region, and reinforcement near the joint began to deform plastically. The two large cracks parallel to the joint interface continued to widen, and eventually the UHPC suffered tensile rupture near the shear studs located in the joint, as shown in Figure 3.73.

Spalling at the edges of the precast deck exposed the outermost reinforcement hairpins that entered into the joint, allowing for pullout. Load application continued, and the specimen reached a peak moment of 2,239 kip-ft before successive

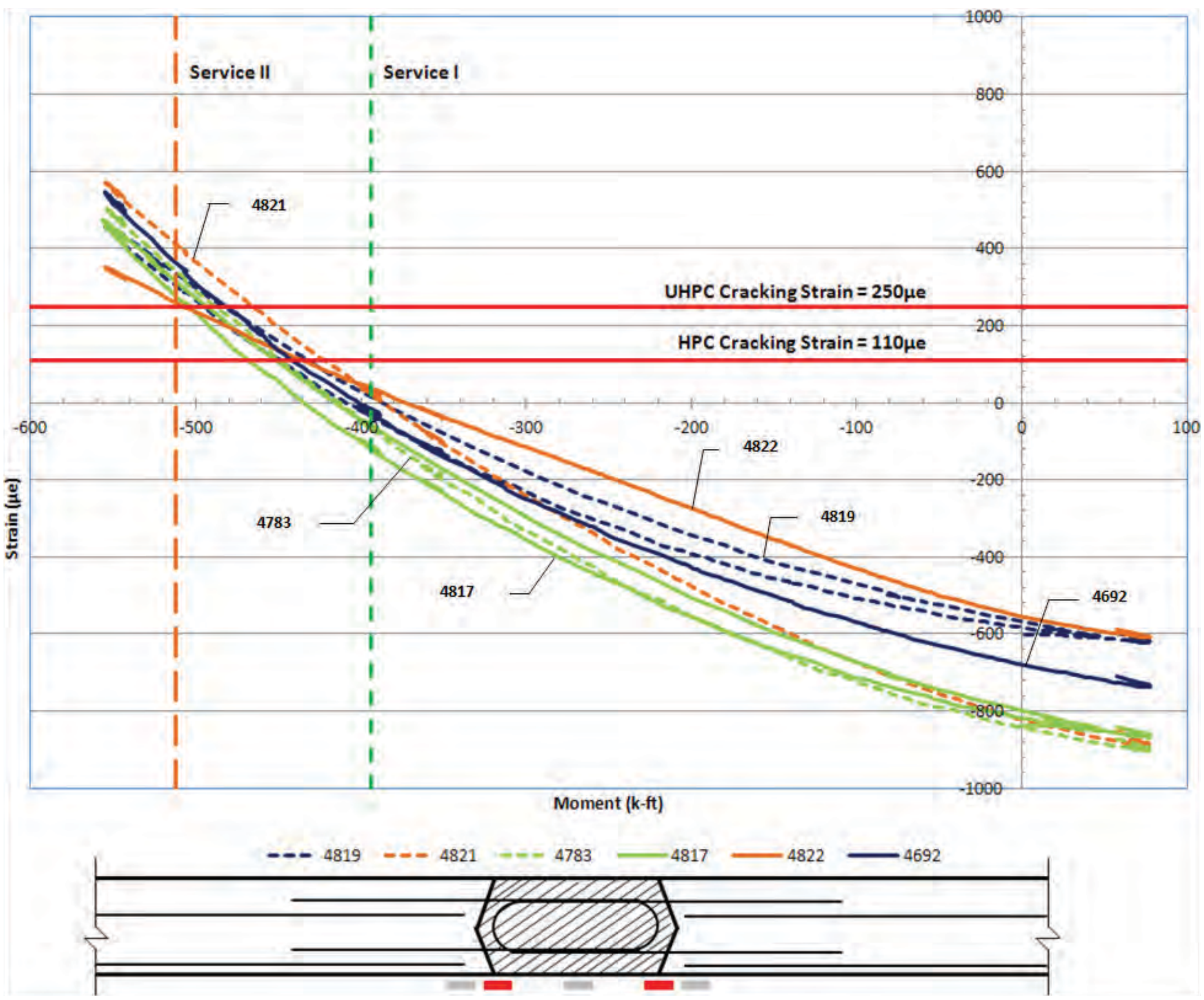

Figure 3.71. Top-of-deck surface-mounted strain gauges over interface (70-kip retrofit).

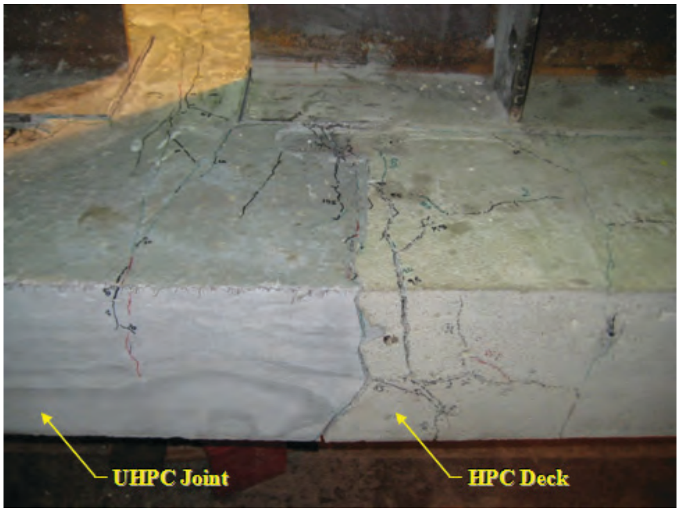

Figure 3.72. Interface opening and crack propagation.

fractures of multiple hairpin reinforcement bars acted as the ultimate mode of failure for the transverse connection.

UHPC Testing Conclusions

The durability of full-depth deck joints between prefabricated panels has been a major concern for many years. Unless post-tensioned, these joints may allow penetration of water and chemicals, leading to corrosion. Posttensioning of bridge decks, however, has traditionally been an additional field operation requiring a specialty subcontractor. UHPC was investigated as possible solution to this problem because its high bond strength to reinforcing bars allows narrow joints, its relatively high bond strength to precast concrete may negate the need for posttensioning, and its low permeability enhances long-term durability.

Through the project's three suites of laboratory tests, the UHPC deck joints were evaluated for use in the ABC demonstration bridge. Abrasion testing was completed to assess the abrasion resistance of the cast-in-place deck joints with respect to anticipated grinding operations, a constructability test was carried out to assess the placement procedure and feasibility of the longitudinal and transverse UHPC joint intersection detail, and strength and serviceability testing were completed to quantify the cracking moment and ultimate moment capacity of the transverse module-to-module connection detail over the bridge pier.

Abrasion and maturity testing of the UHPC material indicated that when cured at 70°F, the compressive strength thresholds required for grinding (10 ksi) and opening the bridge for traffic (14 ksi) were reached at 2 and 4 days after placement of the UHPC, respectively. Thus, the contractor would have 2 days before grinding could commence and 2 days to complete grinding prior to reopening the bridge to traffic. Abrasion resistance increased by roughly 40% for the UHPC material from 2 to 4 days, emphasizing the advantages in time and equipment to grinding the joints as early as possible.

A UHPC placement procedure for the demonstration bridge based on the findings of the constructability testing for the longitudinal and transverse joint intersection detail was discussed with the contractor and material supplier. Partial-height removable bulkheads were recommended in order to control the placement of the UHPC in the deck joints. In addition, the sensitivity of the UHPC mix design to ambient air temperatures was identified while batching for the laboratory tests. Provided that the UHPC's sensitivity to ambient temperature effects were accounted for, the UHPC exhibited excellent flow characteristics and consolidation during placement of the intersecting deck joints. In addition, the accelerated rate of compressive strength gain and higher cracking

Figure 3.73. UHPC rupture (top- and bottom-of-deck).

strain level relative to regular concrete proved useful for application in this ABC project.

While the UHPC displayed several superior material characteristics with respect to the strength of the deck joints themselves, the direct tensile bond strength between the UHPC and the precast HPC deck observed during the strength and serviceability testing raised a durability concern. Testing revealed that the interface between the transverse UHPC joint and the HPC deck underwent early debonding and significant opening well below service-level moment conditions. This raised concerns as to the durability of the module-to-module transverse joint connection for the demonstration bridge. Consequently, a posttensioned retrofit detail was developed and tested in an effort to eliminate opening at the interface and cracking in the HPC deck immediately adjacent to the transverse joint over the pier. With adequate posttensioning force, the retrofit detail successfully limited strains levels to below the cracking threshold of the HPC. For this design using steel girders, the posttensioning retrofit was a successful solution even within the constraints of the accelerated construction schedule. However, if prestressed concrete girders had been used, posttensioning might be more difficult to apply within an accelerated schedule.

The strength of the transverse module-to-module connection detail was found to be more than adequate. However, due to the interfacial bond issues observed over the course of this testing, further investigation into the direct tensile bond strength between the UHPC and HPC is recommended. This testing would help to better evaluate the durability of the longitudinal and transverse UHPC deck joints present in the ABC demonstration bridge and help to determine the long-term viability of this UHPC deck joint detail as a solution in future ABC projects. (See Appendix C for the UHPC Testing Report.)

Field Demonstration Project

Phase III of the project required the construction of a demonstration bridge that used the most-promising bridge details identified earlier in the research and modular systems being incorporated into ABC standards. The US-6 bridge, which crosses Keg Creek near Council Bluffs, Iowa, is representative in size and length to a large majority of bridges across the United States. This bridge was replaced as a demonstration bridge that incorporates proven ABC bridge construction details with the innovative use of ultra-high-performance concrete (UHPC) to shorten the normal bridge replacement period of 6 months to only 2 weeks of traffic disruption. The improvements consist of replacing the bridge located on US-6 over Keg Creek in Pottawattamie County, Iowa. The existing 180-ft by 28-ft continuous concrete girder bridge (with spans of 81 ft, 48 ft, and 81 ft) was constructed in 1953 and was classified as structurally deficient with a sufficiency rating of 33. The replacement structure is a three-span (67 ft, 3 in.; 70 ft, 0 in.; 67 ft, 3 in.) composite steel modular bridge, 210 ft, 2 in. by 47 ft, 2 in., with precast substructures and precast bridge approaches. The bridge replacement is intended to increase the structural capacity of the bridge, improve roadway conditions, and enhance safety by providing a wider roadway. The original and new Keg Creek bridges are shown in Figures 3.74 and 3.75, respectively.

This application provided a unique opportunity to effectively promote ABC for rapid renewal of the bridge infrastructure and to demonstrate various ABC technologies being advanced in the R04 project. The steel modular option was chosen as the most cost-effective on the basis of early discussions with local contractors and fabricators. Although it will not be fully detailed on the design plans, the contractor was allowed to propose a precast concrete modular alternative under a value engineering option if it could be constructed within the same ABC schedule and at a lower cost—none was

Figure 3.74. Original Keg Creek Bridge.

Figure 3.75. New Keg Creek Bridge.

proposed. The bridge was originally designed in-house to be constructed with a planned 13-mi detour (ADT = 4,000) with an estimated construction duration of 6 months. HNTB redesigned this bridge by using ABC techniques and standard designs so that the replacement could be completed in a 14-day period. The ABC period of 14 days pertains only to the time that traffic was disrupted. Although the total duration for the project, including time for prefabrication, was about 7 months, the traveling public was affected for only just over 2 weeks. A daylong workshop, including a site visit, provided an ideal opportunity to disseminate information to bridge owners from around the country.

The demonstration bridge features precast concrete semi-integral abutments, precast columns and pier caps connected with high-strength grouted couplers, and an innovative modular superstructure constructed using prefabricated concrete decked steel stringer units and field-cast UHPC joints. The enhanced durability provided by the elimination of all open deck joints is seen as a major advance in long-life ABC projects and the assembly of precast units without the need for any posttensioned connections avoids the need for specialized contractors.

The project was the first in the United States to use ultra-high-performance concrete (UHPC) to provide a full, moment-resisting transverse joint at the piers. This detail allowed the prefabricated superstructure elements to be erected as a simple span and, once the UHPC joints were constructed, to perform as continuous joints. The project team performed full-scale laboratory testing of the critical field-cast UHPC continuity joints to ensure their long-term reliability and ultimate load capacity. These UHPC joints provided simple construction, additional load-carrying capacity, and a durable joint that prevents moisture intrusion and long-term maintenance problems.

Demonstration Project Innovative Features

This demonstration project implemented a series of innovations. It incorporated details drawn from diverse locations and applied them in a single demonstration project that was visited by DOT and FHWA personnel from numerous states. Project innovations included the following:

- Overall, a complete bridge system was designed and constructed using superstructure and substructure systems composed of prefabricated elements. The bridge approach slab also consisted of precast elements.
- Superstructure units that incorporate precast suspended backwall elements created a semi-integral abutment.
- Ultra-high-performance concrete was used in the joints between the modular superstructure units and between the approach slab panels. UHPC was used for longitudinal joints and transverse joints over the piers. This project was the first in the United States to use UHPC to provide a full, moment-resisting transverse joint at the piers. The elimination of open deck joints provides for a more durable, low-maintenance structure in the final condition.
- Self-consolidating concrete (SCC) was used to improve consolidation and increase the speed of construction for abutment piles (fill pockets) and abutment-to-wingwall connections. Abutments consisted of prismatic precast concrete elements that feature a series of open holes that accommodated driven steel H-piles.
- Use of fully contained flooded backfill at abutments: This proven construction method ideally suited for ABC involves placing a granular wedge behind the abutment backwall that is flooded to achieve early consolidation and significantly reduce the potential for formation of voids beneath the approach pavement.
- A structural health monitoring system (HMS): A monitoring plan was implemented to evaluate and document the innovative aspects of accelerated construction. The monitoring plan included health monitoring instruments to assess the integrity of the structure and deck panel system during and after construction.
- ABC entails prefabricating as many of the bridge components as feasible considering site and transportation constraints. This project takes the approach that for ABC to be successful, ABC Designs should allow maximum opportunities for general contractors to do their own precasting at a staging area adjacent to the project site or in their yards with their own crews. The components were designed such that a local contractor could self-perform all the precasting work without outsourcing much work to precasters. The winning bidder chose to do that by leasing a temporary casting yard next to the bridge site.
- The technologies incorporated into this bridge project have been successfully used in constructed projects drawn from around the United States. The fact that several diverse structural systems have been assembled and incorporated into a single project reinforces the concept that innovation does not necessarily mean creating something completely new, but rather facilitating incremental improvements in a number of specific bridge details to fully leverage previously successful work.

This demonstration project can affect the future practices of the industry and the DOTs. New technologies that are implemented successfully on this project will accelerate the adoption of the innovations in the United States. This will be accomplished by educating and creating awareness related to the innovative features, which will increase confidence in recommending their use on other projects.

Demonstration Project Construction

The construction letting for the project was held on February 15, 2011. A total of seven fully responsive bids were received on the Keg Creek Bridge project, with a low bid of $2.65 million submitted by Godbersen–Smith Construction (G-S) of Ida Grove, Iowa. During the 14-day ABC period, the contractor would be subject to liquidated damages at a rate of $22,000 per day. The project was re-opened to traffic on November 1, 2011. The 14-day ABC period occurred between October 17, 2011, and November 1, 2011.

A detailed account of the construction process for the bridge, including the prebid meeting, contractor bids, the site preparation and prefabrication, the construction process for the bridge including work prior to the ABC period, and the post-construction review meeting, is included in Appendix D.

Demonstration Project Lessons Learned

Overall, the Keg Creek Bridge project was a tremendous success. The bridge was completely replaced in 16 days by using only conventional equipment and labor and without significant problems. All parties (owner, designer, and contractor) worked closely together to resolve challenges as they arose during the ABC period. A SHRP 2 R04 representative was on site during the ABC period to make immediate decisions when questions arose. This was a critical component to the overall success. Following the post-construction review meeting, a number of lessons learned were identified. They include the following:

- On-site prefabrication of bridge components can be performed by contractors and result in a high-quality product. On-site inspection staff should be prepared for work that is not exactly like their normal projects.
- On-site mixing and placement of trial batches of UHPC should be considered to help eliminate fiber balling issues. Early and proactive communication with the UHPC provider is critical to the success of on-site placement operations.
- Special provisions for projects should be carefully written to provide for both on-site and more-traditional precast concrete operations. The special provisions should describe casting, quality assurance, and inspection.
- The bond between UHPC and conventional precast concrete is critical. Surface preparation prior to placement of UHPC should be performed per the manufacturer's recommendations. Future direct-tension testing of bond specimens at ISU will be beneficial in understanding this condition.
- Field placement of UHPC in large quantities can be challenging to manage. For future projects, cold joint bulkheads should be strategically placed to manage the UHPC pours efficiently. It might also be beneficial to separate the UHPC pour in the suspended backwall from the slab joint pour.
- Joint reinforcement using hairpin bars should be carefully evaluated for future projects. It may be possible to simplify this joint construction with reinforcement details that would allow these joints to be more easily constructed. Bars should be staggered, and projecting bars potentially shortened if possible.
- Joint reinforcement congestion should be carefully evaluated for future projects. It may be possible to reduce the number of longitudinal bars that would allow these joints to be more easily constructed. Bars crossing at the joint intersections create congestion and time-consuming placement methods.
- Surveying is a critical element of fast-track bridge replacement projects. To avoid critical and time-consuming errors, two sets of independent surveys should be used to verify accurate pile driving and foundation placement during the ABC period.
- The importance of following project special provisions and supplier-directed UHPC on-site placement protocols to ensure good performance and long-term durability should be stressed in contract documents and contractor communications. All concrete surfaces in contact with the UHPC shall be cleaned and coated with an approved epoxy bonding agent. Both longitudinal and transverse joints are to be coated.
- Consider providing additional isometric views to plans to allow contractor and inspection personnel to better understand how the bridge components fit together.
- Although one was not needed on this project, the contractor should have a backup plan in the event that a bridge component is damaged during the ABC period. At the very least, a repair plan should be agreed on in advance.
- Ideally, the designer should be present on site during the ABC period for quick decision making.

On the basis of lessons learned from the Keg Creek Bridge demonstration project, several adjustments were incorporated into the proposed ABC standards presented elsewhere in this report. These adjustments were intended to improve constructability and reliability, and to provide improved opportunities for future plant-cast and site-cast ABC projects.

Highways for LIFE Workshop

To more widely disseminate the construction process and lessons learned from the Keg Creek Bridge demonstration project, a national Highways for LIFE showcase was held in Council Bluffs, Iowa, on October 28, 2011. The showcase was attended by nearly 80 people from 14 states. These participants represented state DOTs, the FHWA, designers, and contractors who shared an interest in accelerated bridge construction.

The showcase agenda included presentations from a variety of viewpoints. Topics discussed included an overview of the Highways for LIFE program, both national and Iowa perspectives on accelerated bridge construction, and a detailed presentation on the design and construction of the Keg Creek Bridge. Showcase attendees were encouraged to visit the project site during the afternoon. Bus transportation was provided to the project site, and attendees were allowed to freely observe all aspects of the construction progress. Weather was cool and windy, with temperatures in the low 50s in the afternoon. On the day of the showcase, the contractor was placing the UHPC material for all of the superstructure deck joints, an ideal presentation for the attendees. Visitors were able to observe the mixing, transporting, placing, finishing, and curing operations.

Video

A time-lapse video of the ABC period for Keg Creek Bridge and a longer, narrated video that captures each stage of the project and tells the story of how innovative technologies can be integrated into the bridge replacement process can be viewed at http://www.trb.org/StrategicHighwayResearchProgram2SHRP2/Pages/Keg_Creek_Bridge_Project_619.aspx.

CHAPTER 4

Conclusions and Suggested Research

Overview of *ABC Toolkit*

The *ABC Toolkit* developed for prefabricated elements and modular systems in the SHRP 2 R04 project consists of the following:

- ABC Design standards;
- Detailed design examples;
- Recommended LRFD design specifications; and
- Recommended LRFD construction specifications.

The research team developed pre-engineered standards optimized for modular construction and accelerated bridge construction (ABC). Standardizing ABC systems will bring about greater familiarity with ABC technologies and concepts and will also foster more widespread use of ABC. Standard plans have been developed for the most-useful technologies that can be deployed on a large scale in bridge replacement applications. They include complete prefabricated modular systems, as outlined here:

- Precast modular abutment systems
  - Integral abutments;
  - Semi-integral abutments; and
  - Precast approach slabs.
- Precast complete pier systems
  - Conventional pier bents; and
  - Straddle pier bents.
- Modular superstructure systems
  - Concrete deck bulb tees;
  - Concrete deck double tees; and
  - Decked steel stringer system.
- ABC bridge erection systems
  - Above-deck driven carriers; and
  - Launched temporary truss bridges.

The development of detailed design examples for use by future designers provides step-by-step guidance for the overall structural design of the prefabricated bridge elements and systems. The design examples pertain to the same standard bridge configurations for steel and concrete used in the ABC Design standards. The intent was to create design examples that could be used in conjunction with the ABC Design standards so that the practitioner will get a comprehensive look at how ABC Designs are performed and translated into design drawings and details.

The LRFD design specifications do not explicitly deal with the unique aspects of large-scale prefabrication, including issues such as element interconnection, system strength, and behavior of rapid deployment systems during construction. The work in this project entailed identifying any shortcomings in the current LRFD Bridge Design Specifications that may limit their use for ABC Designs and making recommendations to address these limitations. Recommended LRFD design specifications for ABC bridge design are included in this chapter.

Recommended LRFD construction specifications for prefabricated elements and modular systems were compiled by the research team with the intent that they would be used in conjunction with the standard plans for steel and concrete modular systems. As such, these specifications for rapid replacement focus heavily on the means and methods required for rapid construction using prefabricated modular systems. A review of various innovative construction contracting and delivery methods that may be used to enhance the implementation and delivery of ABC construction projects is also included.

Design Considerations for ABC Standards for Modular Systems

Specific design considerations for standardized ABC modular systems developed in Phase III include the following:

- Standardized designs for superstructure systems that cover span ranges from 40 ft to 130 ft and that can be transported and erected in one piece.

- Substructure modules that have dimensions and weights suitable for highway transportation and erection using conventional equipment.
- Designs and specifications that allow the contractor to self-perform the precasting of non-prestressed components.
- Prefabricated modules designed to be quickly assembled in the field with full moment connections.
- Designs for routine bridges that can be used for most sites with minimal bridge-specific adjustments.
- Modules that can be used in simple spans and in continuous spans (simple for dead load and continuous for live load). Details to eliminate deck joints at piers and abutments.
- Use of high-performance materials: HPC/UHPC concrete, HPS, or A588 weathering steel.
- Modular systems with integral wearing surface so that an overlay is not required. Use of overlay is considered optional as part of a long-term preservation strategy. Use of an integral wearing surface in lieu of an overlay is recommended to expedite construction. Typically an extra 1½-in. monolithic concrete slab thickness above the top layer of rebars serves as an integral wearing surface. This approach has been increasingly common in segmental bridge design. At the end of construction, milling ½ in. ensures an excellent final riding surface. The design permits replacement with an overlay in the future, along with removal of the integral wearing surface.
- Prefabricated components can be the most cost-effective solution for any alignment. However, straight alignments without skew allow multiple identical components, which tend to be the most economical. Preference should be given, if possible, to straightening the roadway alignment along the bridge length and eliminating skew for lower initial and life-cycle costs.
- Lightweight high-performance concrete (LWHPC) can be an option for precast systems to achieve reduced weights that could be beneficial for shipping and erection.

These design considerations and concepts are discussed further in the following sections.

Modular Substructure Systems

Substructure construction takes up a significant portion of the total on-site construction time. Reducing the time it takes to complete substructure work is critical for all rapid renewal projects. With this goal in mind, ABC standards are provided for abutments, wingwalls, and piers that are commonly used in routine bridge replacements. These standards include the following:

- Precast modular abutment systems
  - Integral abutments; and
  - Semi-Integral abutments.
- Precast complete pier systems
  - Conventional pier bents; and
  - Straddle pier bents.

Integral and Semi-Integral Bridges for Rapid Renewal

One of the most important aspects of design, which can affect the speed of erection, structure life, and lifetime maintenance costs, is the reduction or elimination of roadway expansion joints and associated expansion bearings. Besides providing a more maintenance-free durable structure, continuity and elimination of joints can lead the way to more-innovative and aesthetically pleasing solutions to bridge design. Providing a joint- and maintenance-free bridge should be an important goal of rapid renewals. The use of integral or semi-integral abutments allows the joints to be moved beyond the bridge. Integral abutment bridges have proven themselves to be less expensive to construct, easier to maintain, and more economical to own over their life span. Integral and semi-integral abutments have become the preferred type for most departments of transportation (DOTs).

When deck joints are not provided, the thermal movements induced in bridge superstructures by temperature changes, creep, and shrinkage must be accommodated by other means. Typically, provisions are made for movement at the ends of the bridge by one of two methods: integral or semi-integral abutments, along with a joint in the pavement or at the end of a reinforced concrete approach slab. The terms "integral bridges" and "integral abutment bridges (IAB)" are generally used to refer to continuous jointless bridges with single and multiple spans and capped-pile stub-type abutments. The most-desirable end conditions for an integral abutment are the stub or propped-pile cap type, which provide the greatest flexibility and hence, offer the least resistance to cyclic thermal movements. Piles driven vertically and in only one row are highly recommended. In this manner, the greatest amount of flexibility is achieved to accommodate cyclic thermal movements.

A semi-integral abutment bridge (SIAB) is a variant of the integral abutment design. It is defined as a structure in which only the backwall portion of the substructure is directly connected with the superstructure. The beams rest on bearings that rest on a stationary abutment stem. The superstructure and backwall move together into and away from the backfill during thermal expansion and contraction. There are no expansion joints within the bridge.

Reasons for Jointless Construction for ABC

ABC seeks to reduce on-site construction time and to mitigate long traffic delays through innovative design and construction practices. Integral bridges and semi-integral bridges incorporate many innovative features that are well suited to rapid construction. Only one row of vertical piles is used, which means fewer piles. The backwall can be cast simultaneously with the superstructure. The normal delays and the costs associated with bearings and joints installation, adjustment, and anchorages are eliminated. Some of the advantages to jointless construction for ABC projects are summarized as follows:

- Tolerance problems are reduced. The close tolerances required when using expansion bearings and joints are eliminated with the use of integral abutments. Bridge seats need not conform exactly to girder flange slope and camber corrections, since the girder loads are ultimately carried by the concrete comprising the end diaphragm. Minor mislocation of the abutments creates no fit-up problems.
- Rapid construction. With integral abutments, only one row of vertical (not battered) piles is used and fewer piles are needed. The entire end diaphragm/backwall can be cast simultaneously and with less forming. Fewer parts are required. Scheduling problems with suppliers and manufacturers are avoided.

  Integral abutment bridges are more quickly erected than jointed bridges, thereby decreasing construction costs. The construction time for IABs is commonly shorter because connections are simple to form and expansion joints are not required. In addition, it is common for IABs to use only one row of vertical piles, meaning a smaller number of piles are typically used than for many jointed bridges, and cofferdams are not required for constructing the intermediate piers. Installing fewer piles and not constructing cofferdams results in decreased construction time and lower bridge construction cost.
- Reduced removal of existing elements. Integral abutment bridges can be built around the existing foundations without requiring the complete removal of existing substructures. Reduced removal of existing substructures will greatly reduce the overall construction durations of bridge replacements.
- No cofferdams. Integral abutments are generally built with capped pile piers or drilled shaft piers that do not require cofferdams.
- Improved ride quality. Smooth jointless construction improves vehicular riding quality and diminishes vehicular impact stress levels. It provides for lower-impact loads and reduced snowplow damage to decks.
- Added redundancy and capacity for catastrophic events. Integral abutments provide added redundancy and capacity for all types of catastrophic events. In designing for seismic events, considerable material reductions can be achieved through the use of integral abutments by negating the need for enlarged seat widths and restrainers. Further, the use of integral abutments eliminates loss of girder support, the most common cause of damage to bridges in seismic events. Joints introduce a potential collapse mechanism into the overall bridge structure. Integral abutments have consistently performed well in actual seismic events and have significantly reduced or avoided problems of backwall and bearing damage that are associated with seat-type jointed abutments.

Precast Abutments and Wingwalls

Precast modular abutments are composed of separate components fabricated off site, shipped, and then assembled in the field into a complete bridge abutment. Precast modular abutments have been constructed in several states. Integral connection of the superstructure to the substructure will be preferred for ABC construction. Since not all states employ the use of integral abutments, standards have been created for both integral and non-integral abutments. The individual precast components should be designed to be shipped over roadways and erected using typical construction equipment. To this point, the precast components are made as light as practicable. Voids can be used in the wall section, which will reduce shipping weights and allow for larger elements to be used. Voids are also used to attach drilled shafts or piles to the cap for stub-type abutments. Once the components are erected into place, the voids and shear keys are filled with self-consolidating concrete. Wingwalls are also precast with a formed pocket to slide over wingwall piles or drilled shaft reinforcing. Once in place over the wingwall piles or drilled shaft, the wingwall pocket is filled with high early strength concrete or self-consolidating concrete.

Connections

Full moment connections between modular substructure components are used to emulate cast-in-place construction. The closure pours are constructed using self-consolidating concrete that can be completed quickly and result in the highest-quality durable connection. Self-consolidating concrete, also known as self-compacting concrete or SCC, is a highly flowable, nonsegregating concrete that spreads into place, fills formwork, and encapsulates even the most congested reinforcement, all without any mechanical vibration. SCC is also an ideal material to fill pile pockets in substructure components. It is defined as a concrete mix that can be placed purely by means of its own weight, with little or no vibration. SCC allows easier pumping, flows into complex shapes, transitions through inaccessible spots, and minimizes voids around embedded items to produce a high degree of homogeneity and uniformity. As a high-performance concrete, SCC delivers these benefits while maintaining all of concrete's customary mechanical and durability characteristics.

Precast Complete Piers

Precast complete piers are also composed of separate components fabricated off site, shipped, and assembled in the field into a complete bridge pier. Piers with single-column and multiple-column configurations are common. Foundations can be drilled shafts, which can be extended to form the pier columns. Driven piles may be used with precast pile caps, or precast spread footings may suffice where soil conditions permit. Pier columns are attached to the foundation by grouted splice sleeve connectors. Precast columns can be square or octagonal, the tops of which are connected by grouted splice sleeves to the precast cap. Pier bents can have a single column or multiple columns. The precast cap is typically rectangular in shape.

Some states, specifically those in high seismic regions, employ the use of integral pier caps. However, the standards in this project were developed only for non-integral piers, which are the most common and most suited for rapid construction. In many cases, the integral pier cap connections are constructed with cast-in-place concrete; however, the connection can also be made with precast concrete. This connection is often quite complicated and congested. There are also tight controls over tolerances and grades. For these reasons, the most common form of connection is a cast-in-place concrete closure pour. In a non-integral pier cap, the superstructure and deck will be continuous and jointless over the piers. Also, non-integral piers would be easier to reuse.

Like the precast modular abutment, the components of the precast complete pier have been designed to be shipped over roadways and erected using typical construction equipment. To this point, the precast components are made as light as practical. Precast spread footings can be partial precast or complete precast components. A grout-filled void beneath the footing is used to transfer the load to the soil, avoiding unexpected localized point loads. Column heights and cap lengths will be limited by transportation regulations and erection equipment. Alternatively, the cap length limitation can be avoided by utilizing multiple short caps combined to function as a single pier cap. Precast bearing seats can also be used.

Modular Superstructure Systems

Modular superstructure systems composed of both steel and concrete girders have been included in the pre-engineered standards. In the Task 6 evaluations, deck bulb tees and decked steel stringer systems received the highest scores, as these are proven systems for rapid renewal. Each modular system is expected to see a 75- to 100-year service life due to the quality of its prefabricated superstructure, the use of high-performance concrete, and the attention given to connection details. Standards for modular superstructures will include the following concrete and steel systems:

- Decked steel stringer system;
- Concrete deck bulb tees; and
- Deck double tees.

Precast Concrete Deck Bulb Tee and Double Tee

Conventional precast concrete girders have been well established for bridge construction in the United States for more than 50 years. There is wide acceptance for their use among owners and contractors because they are easy and economical to build and to maintain. In most cases, the girders are used with a cast-in-place (CIP) deck built on site. For ABC applications the key difference lies in that the girders have an integral deck, thus eliminating the need for a CIP deck. The use of decked precast girders has become increasingly popular in several states, though they are not available in every state. Additionally, an integral wearing surface, typically 1½ in. to 2 in., can be built monolithically with the deck slab. In the future, the wearing surface concrete can be removed and replaced while preserving the structural deck slab. The precast deck bulb tee girders and double tee girders combine all the positive attributes of conventional precast girder construction with the advantage of eliminating the time-consuming step of CIP deck construction. Contractors familiar with conventional precast girder construction should have no difficulty in adapting to these newer deck girders installed by using an ABC approach. Deck bulb tee and double tee girders are proven systems that have been standardized for use by several Western states, including Utah, Washington, and Idaho. The northeast extreme tee (NEXT) beam, a variation of the double tee, was developed by the PCI Northeast to serve the ABC market. The research team expects the deck girder bids to be very competitive when compared with the girder and CIP deck systems and that they may come in even lower for sites in which constraints to deck casting operations may exist. Cast-in-place closure pours are typically used to connect girders in the field. The girder flanges can be made to different widths to fit site and transportation requirements.

Decked Steel Stringer System

Similar to the concrete deck girder system, the decked steel stringer system is also a proven concept shown to be quite economical and rapidly constructed. Prefabricated decked steel stringer systems have been a very popular option for accelerated construction of bridges in the United States. Their light weight, easy constructability, low cost, and easy availability were seen as advantages over other systems. The length and weight of each module can be designed to suit transportation of components and erection methods. Erection can generally be accomplished with conventional equipment. Cast-in-place closure pours are typically used to connect adjacent units in the field. The modules can be made to different widths to fit site and transportation requirements.

Many states are familiar with the Inverset system or a variation of it. The patent for the Inverset system has expired. Standardizing generic designs for commonly encountered spans will provide a big boost to gaining quick acceptance and more widespread use for modular concepts. As for the precast deck girders, the recommended connection will be the full moment connection for the same reasons previously discussed.

An integral wearing surface, typically 1½ in. to 2 in., can be built monolithically with the deck slab. In the future, the wearing surface concrete can be removed and replaced, while preserving the structural deck slab.

Connections Between Modules

The ease and speed of construction of a prefabricated bridge system in the field is paramount to its acceptance as a viable system for rapid renewal. In this regard, the speed with which the connections between modules can be completed has a significant influence on the overall ABC construction period. Additionally, connections between the modular segments can affect the live-load distribution characteristics, seismic performance of the superstructure system, and the superstructure redundancy. Designers need to develop a structure type and prefabrication approach that can be executed within the time constraints of the project site and also achieve the desired structural performance. Connections play a critical role in this approach. Connections of the modular units are important elements for ABC, as they determine how easily the elements can be assembled and connected together to form the bridge system. Often the time to develop a structural connection is a function of cure times for the closure pour.

The number of joints and the type of joint detail are crucial to both the speed of construction and to the overall durability and long-term maintenance of the final structure. The use of cast-in-place concrete closure joints should be kept to a minimum for accelerated construction methods due to placement, finishing, and curing time. Durability of the joint should be achieved through proper design, detailing, joint material selection, and construction procedures.

Posttensioned joints use induced compression to close shrinkage cracks at the joint interface, prevent cracking under live load, and enhance load transfer. The posttensioned joints can be a female-female shear key arrangement infilled with grout or match-cast with epoxied joints if precise tolerances can be maintained. While providing some assurance of good long-term performance, posttensioning requires an additional step and complexity during on-site construction. Its use may not be entirely compatible with ABC goals of rapid field assembly and long-term durability of joints.

Design Considerations for Connections

Design considerations for connections between deck segments include the following:

- Full moment connections that are practical to build quickly.
- Achieving durability at least equal to that of a precast deck.
- Joint details suitable for heavy, moderate, and light truck-traffic sites.
- Achieving acceptable ride quality (similar to CIP decks).
- Not requiring the use of overlays for durability. An integral wearing surface consisting of an extra thickness of monolithic concrete slab may be provided.
- Posttensioned connections can be an alternative for ABC construction.
- Details can accommodate slight differential camber between adjacent modules.
- Rapid strength gain, so that the bridge can be opened to traffic in a matter of hours or days.

Full Moment Connections for Modular Superstructure Systems

Investigations of joint types and material options performed in the previous tasks have identified full moment connection using ultra-high-performance concrete (UHPC) joints as the preferred connection type for modular superstructure systems to satisfy the criteria for constructability, structural behavior, and durability as noted above. UHPC refers to a class of advanced cementitious materials that displays significantly enhanced material properties considered very beneficial to ABC. When implemented in precast construction, these concretes exhibit properties including compressive strength above 21.7 ksi, sustained tensile strength through internal fiber reinforcement, and exceptional durability when compared with conventional concretes. Conventional materials and construction practices for connection details can result in reduced long-term connection performance as compared with the joined components. UHPC presents new opportunities for the design of modular component connections due to its exceptional durability, bonding performance, and strength. The properties of UHPC make it possible to create small-width, full-depth closure pour connections between modular components. These connections may be significantly reduced in size when compared with conventional concrete construction practice, and could include greatly simplified reinforcement designs. A lab testing program was carried out to further evaluate the performance of UHPC in ABC applications in Task 10C of this project.

The UHPC joint detail used had a 6-in. joint width with #5 U bars. UHPC has a strength gain of 10 ksi in 48 hours, which is when deck grinding can begin. It is suitable for Tier 2 projects that use modular systems. (The research team has been informed that new UHPC mixes are available for bridges that would require only overnight closures.) The narrow joint width reduces shrinkage and the quantity of UHPC required, while providing a full moment transfer connection. Tests done at FHWA showed that a 6-in. joint width would be adequate to fully develop #5 bars even when straight bars are used. The New York State DOT has built a few bridges with this detail by using straight bars. The use of straight bars is planned for the second ABC demonstration project under R04.

One of the challenges with using U bars is that to satisfy the minimum bend diameter, a deck thickness greater than 6 in. is required. This is not a problem for the decked steel girder bridges, but it requires a thickening of the flanges for deck bulb tee (DBT) girders from 6 in. to 9 in. The use of straight bars in the joints would be preferable for DBT bridges to minimize the flange thickness and shipping weights. Headed rebars were not considered because tests under NCHRP Project 10-71 have shown that the size of the heads causes constructability issues, making the rebars not well suited for ABC.

Self-Performance of Prefabrication by the Contractor

This project takes the approach that for ABC to be successful and for costs to come down, ABC designs should allow maximum opportunities for the general contractors to do their own precasting at a staging area adjacent to the project site or in their yards with their own crews. This is particularly true for substructure components that have traditionally been constructed by contractor crews. Substructure components are made of conventional reinforced concrete and can be precast by the general contractor. Components are designed to allow the contractor to self-perform the precasting by paying special consideration to

- Components that use non-prestressed reinforcing;
- Components that are simple enough to fabricate;
- Components that can be assembled quickly in the field;
- Components that allow reasonable tolerance for erection;
- Maximum repetition of components to reduce formwork cost; and
- Components that are suitable for highway shipping and erection with conventional equipment.

ABC Standard Plans and Details

Bridge designs for workhorse bridges can be standardized to allow for repetition and prefabrication. The goal would not be to design each bridge individually, but to use repetitive design standards and adapt the conditions (alignment, span length, width) to the standard. The use of modular systems with standard designs is a proven method of accelerating bridge construction. It should also be noted that with regard to the design of new structures that facilitate rapid reconstruction, it is unrealistic to think that one or a few technologies will become dominant in the future. There will need to be an array of solutions for different site constraints, soil conditions, bridge characteristics, traffic volumes, and so forth. Contractors have also developed various proprietary systems and concepts to accelerate bridge construction, and ABC Designs should be open to such innovations as well.

In Phase III of this project, ABC details for superstructure and substructure systems that are suitable for a range of spans were developed. The details presented in the plans included with this report are intended to serve as general guidance to practitioners in the development of site-specific designs suitable for accelerated bridge construction. Bridge designers are well versed with sizing beams and designing reinforcing steel for conventional construction for a specific site, and it would be appropriate for the engineer of record to perform these functions for ABC projects as well. A single set of ABC Designs for national use would not be practical since there are state-specific modifications to LRFD bridge design criteria, including permit load design requirements for Strength II. Individual states may want to modify the details presented to fit their local needs and market conditions.

The designer, guided by the standard plans and details and the accompanying set of ABC Design examples will be able to easily complete an ABC Design for a routine bridge replacement project. These standard plans will need to be customized to fit the specific site in terms of the bridge geometry, member sizes, and reinforcement details. The overall configurations of the modules, their assembly, and connection details, tolerances, and finishing will remain unchanged from site to site. Repeated use of the same system will allow the continuous refinement of the concept, thereby reducing risks and lowering costs. The research team anticipates that the standard plans would be about 60% to 70% complete in their overall coverage, while providing substantially complete details of the ABC aspects of the project. Much of the remaining work is not as much ABC related as it is customization for the specific bridge site. The standard plans used in conjunction with the ABC Design examples will provide training wheels for designers until they get comfortable with ABC. The following sections provide more information on these ABC standards.

Overview of ABC Design Standards

Typical designs for superstructure and substructure modules have been grouped into the following span ranges:

- 40 ft ≤ span ≤ 70 ft;
- 70 ft ≤ span ≤ 100 ft; and
- 100 ft ≤ span ≤ 130 ft.

The superstructure cross section and module widths are shown for a typical two-lane bridge with shoulders having an out-to-out width of 47 ft, 2 in., as shown in Figure 4.1. While the bridge cross section was chosen to represent a routine bridge structure (as was the demonstration bridge), the design concepts, details, fabrication, and assembly are equally applicable to other bridge widths.

Posttensioned Spliced Girders for Spans over 130 Feet

Standardized designs for superstructure systems cover spans to 130 ft as, at many sites, these are spans that can be transported and erected in one piece. In the span range up to 130 ft, the precast designs use pretensioning without the need for on-site posttensioning. Posttensioning can be used to extend the span length of a precast girder to 200 ft and beyond. Posttensioned splice girders can be used to simplify girder shipping because the girder can be fabricated in two or three pieces and can be spliced together in the field. Many details included on the standards can be used for longer-span bridges with additional detailing. The girders are spliced with reinforced concrete closure pours at the site (off-line) and then erected. The posttensioning strand crosses these closure pours and provides the moment capacity at the splice. One useful reference for posttensioned spliced girder design would be the *Precast Bulb Tee Girder Manual* published by the Utah DOT (UtahDOT, 2010b).

General Information Sheets

The sheets containing general information and instructions on the use of ABC standard plans have been included at the beginning of the set to guide users. The general information sheets contain specific instructions to designers so that all the key design and construction issues in ABC projects are adequately addressed during the final design and customization processes.

The general information sheets introduce the intent and scope of the standard plans. They note that the intent of the design standards is to provide information that applies to the design, detailing, fabrication, handling, and assembly of prefabricated components used in accelerated bridge construction and designed in accordance with the AASHTO LRFD bridge design specifications.

The details presented in the plans are intended to serve as general guidance in the development of designs suitable for ABC. The details shall not be perceived as standards that are ready to be inserted into contract plans. Their implementation shall warrant a complete design by the engineer of record (EOR) in accordance with requirements for the project site and DOT standards and specifications. The standards were developed to comply with AASHTO *LRFD Bridge Design Specifications,* 5th ed. The designer shall verify that all requirements included in the latest edition of the AASHTO *LRFD Bridge Design Specifications,* including interim provisions, are satisfied and properly detailed in any documents intended or provided for construction. All formwork for the deck shall be supported from the longitudinal girders similar to conventional construction methods. Shored construction shall not be assumed.

The systems presented in the superstructure design standards consist of prestressed concrete girders with integrally cast decks and a composite decked steel stringer module. Both systems include a full-depth deck as the flange that serves as the riding surface to eliminate the need for a cast-in-place deck. The prefabricated superstructure modules presented in the plans may be used with the prefabricated substructure systems that are a part of these design standards, or they may be used with other new or existing substructures that have been adapted to conform to the bearing requirements for these superstructure modules.

Substructures are the portions of the bridge located between the superstructure and the foundation (supporting soil, piles, or drilled shafts). Geotechnical design, pile design, and detailing are not considered substructures and are not covered in these design standards. Foundation design is driven by site soil conditions. The substructure details depicted can be adapted to fit other foundation types. The prefabricated substructure systems presented in the plans for precast abutments, wingwalls, and piers are intended to be used with the prefabricated superstructure systems that are part of the design standards, but may be adapted to other superstructures as well. The reinforcing details and connection details shown are suitable for use in non-seismic or low-seismic areas.

The general information sheets also provide guidance on key considerations specific to ABC design and construction of prefabricated modular systems, including

- Lifting and handling stresses;
- Shop drawings and assembly plan;
- Fabrication tolerances;
- Site casting requirements;
- Geometry control;

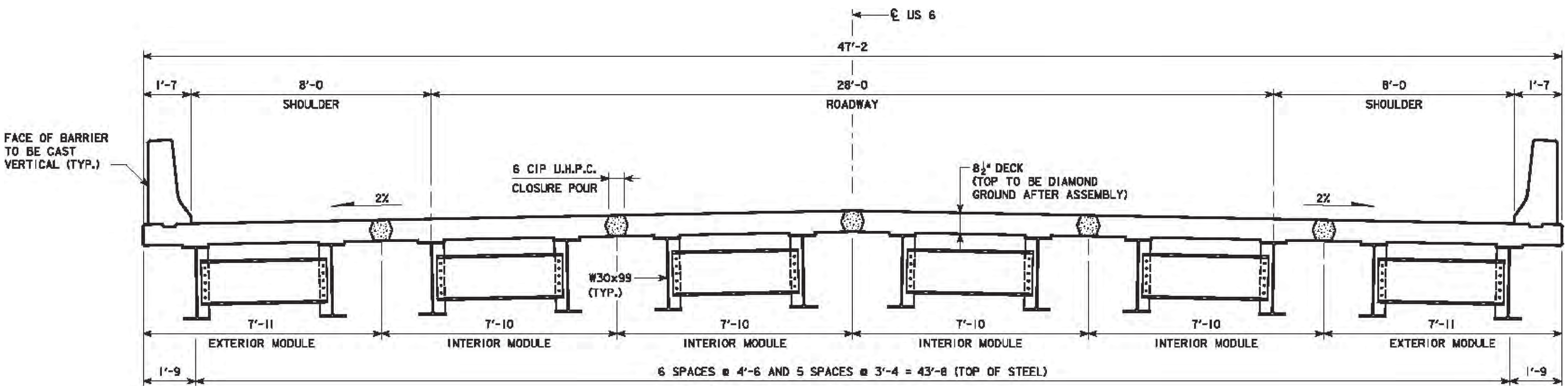

Figure 4.1. Channel Bridge, Carpenter Road, New York State.

- Mechanical grouted splices;
- Element sizes; and
- General procedure for installation of modules.

Organization of ABC Design Standards

The systems presented in these ABC Design standards consist of the following items, which are listed in Tables 4.1 and 4.2:

- Sheets A1 through A12
  - Semi-integral abutments.
  - Integral abutments.
  - Wingwalls.
  - Pile foundations and spread footings.
- Sheets P1 through P9
  - Precast conventional pier.
  - Precast straddle bent.
  - Drilled shaft and spread footing option.
- Sheets S1 through S8
  - Decked steel girder interior module.
  - Decked steel girder exterior module.
  - Bearing and connection details.
- Sheets C1 through C11
  - Prestressed deck bulb tee interior module.
  - Prestressed deck bulb tee exterior module.
  - Prestressed double tee module.
  - Bearing and connection details.

Table 4.1. Substructure ABC Standards Sheets (Abutments and Piers)

Abutment	
Sheet No.	**Description**
A1	General Notes and Index of Drawings
A2	Semi-Integral Abutment Plan and Elevation
A3	Abutment Reinforcement Details
A4	Wingwall Reinforcement Details 1
A5	Wingwall Reinforcement Details 2
A6	Semi-Integral Abutment Section
A7	Integral Plan and Elevation
A8	Integral Abutment Section
A9	Approach Slab 1
A10	Approach Slab 2
A11	Semi-Integral Abutment Spread Footing Option Plan and Elevation
A12	Spread Footing Option Selection
Pier	
Sheet No.	**Description**
P1	General Notes
P2	Precast Pier Elevation and Details (Conventional Pier)
P3	Precast Pier Cap Details (Conventional Pier)
P4	Precast Column Details (Conventional Pier)
P5	Precast Pier Elevation and Details (Straddle Bent)
P6	Precast Pier Cap Details (Straddle Bent)
P7	Precast Column Details (Straddle Bent)
P8	Foundation Details (Drilled Shaft)
P9	Foundation Details (Precast Footing)

Standard Conceptual Details for ABC Construction Technologies

The modular systems discussed in the previous sections may be erected with conventional construction techniques when site conditions permit. Given the proper project criteria,

Table 4.2. Superstructure ABC Standards Sheets (Steel and Concrete Girders)

Steel Girder Superstructure	
Sheet No.	**Description**
S1	General Notes and Index of Drawings
S2	Typical Section Details
S3	Interior Module
S4	Interior Module Reinforcement
S5	Exterior Module
S6	Exterior Module Reinforcement
S7	Bearing Details
S8	Miscellaneous Details
Concrete Girder Superstructure	
Sheet No.	**Description**
C1	General Notes and Index of Drawings
C2	Typical Section
C3	Girder Details 1
C4	Girder Details 2
C5	Bearing Details
C6	Abutment Details
C7	Pier Continuity Details
C8	Camber and Placement Notes
C9	Miscellaneous Details
C10	Alternate Typical Section
C11	Alternate Girder Details

use of conventional equipment would be the first choice for constructing a bridge designed with ABC modularized components. Unlike conventional stick-built bridges, the appropriate construction technology for rapid renewal projects built with ABC modular systems should be selected after careful consideration of project and site constraints and the choice of technologies available. Advances in ABC Construction Technologies have introduced innovative techniques for erecting highway structures using adaptations of proven long-span technologies.

These ABC Construction Technologies can be grouped for use into the following two categories:

- Bridge movement systems. Technologies in which the erection equipment is designed specifically to lift and transport large complete or partial segments of preassembled structures.
- Bridge erection systems. Technologies in which the erection equipment is designed to deliver individual components of a proposed structure in a span-by-span process.

Self-propelled modular transporters (SPMTs), lateral sliding, and launching would be good examples of bridge movement systems. If the best option for a site is to preassemble the structure completely and then move it to its final position, there are several excellent published references on bridge movement technologies, such as the Utah DOT SPMT manual and the FHWA SPMT manual, that can guide designers and owners. Movement of preassembled complete structures is a well-developed technology in the United States; several specialty firms provide this service nationally. Phase IV of this project is designing a bridge replacement by using a lateral slide and will develop design standards for such systems.

Bridge erection system technologies are intended to be easily transportable, lightweight, modular systems. The use of this type of equipment to deliver fully preassembled structures is not practical, although it is possible on a very small scale.

Because the ABC Design standards developed in Phase III are for modular superstructure and substructure systems, the conceptual details for ABC Construction Technologies will focus on bridge erection systems specifically intended to deliver and assemble modular systems. Rapid bridge renewal projects that use modular systems can be categorized into one of the following project types:

- ABC Bridge Designs built with conventional construction; or
- ABC Bridge Designs built with ABC Construction Technologies.

The designer should ascertain whether the bridge renewal project warrants further consideration of specialized ABC Construction Technologies or whether the site and project limits are more suited for conventional equipment and technologies. The use of ABC Construction Technology compels owners and consultants to consider the following variables:

- Bridge project type;
- Site and traffic constraints;
- Available space for construction staging areas (if any exists, where located and what are conditions);
- Environment surrounding the project site; and
- Project construction time period.

The development of ABC Construction Technologies could evolve around the demonstration of which technologies work best with the ABC Designs (both substructure and superstructure) developed in this project. A matrix of questions, shown in Figure 4.2, was created for owners and designers to guide them toward the proper selection of the ABC Construction Technology that best fits a project's needs. Erection technology selection is a complex process that depends on a number of factors including the number of bridges to be built, the convenience of crane support on the ground or by other means, the span lengths, the condition of the existing bridge to support crane loads, and site restrictions.

For rapid renewal applications, the existing bridge must be demolished in a rapid process to allow the erection of the replacement structure. Because the demolition operations require roadway closures and other traffic operations, completing the demolition process quickly and efficiently is often as critical as the replacement bridge erection operations. Typically, the most effective use of field resources is to use the same equipment for the demolition operations and for the replacement structure erection operations. Reuse of the equipment avoids duplication of temporary support conditions such as crane mats, causeways, or trestle bridges.

Overview of ABC Construction Technologies

To assist owners and engineers with implementation of an ABC Construction Technology, a set of standard conceptual details defining terminology and demonstrating the possibilities and limits of each ABC Construction Technology was created. Guidelines are also provided for conventional erection of ABC systems by using cranes. These sheets are intended to be used in conjunction with the design standards for modular systems to achieve closer integration of design and construction starting in the design phase. Such an integrated design approach is critical to convey the designer's intended assembly approach to the contractor and to foster more constructible designs. Once a construction technology

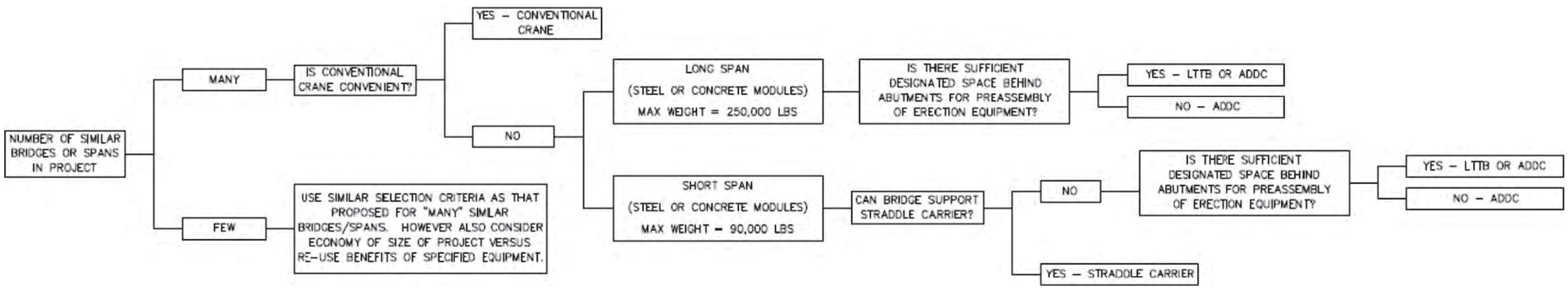

Figure 4.2. Selection flowchart for ABC Construction Technologies.

has been selected, the designer must integrate this technology into the bridge design.

ABC Designs Built with Conventional Erection

This is the typical construction method employed in most construction with prefabricated systems. Most contractors have cranes in their field resources or can easily acquire them. Bridge component erection can be done using land-based cranes (rubber-tire or crawler) or barge-supported cranes. Cranes can also be supported on a causeway, a sand island, or a trestle bridge for river crossings. The benefits of a causeway include cost savings by using native materials instead of building a crane trestle. Culvert pipes are used to allow water flow. Risks include high water flow that could wash away the causeway or sand island. The contractor and the contractor's engineer both plan and design specific temporary structures and specific contractor operations. Anticipating the construction operations early in the design phase can have significant benefits.

Sections that can be transported and erected in one piece are optimal for ABC. Lengths up to 140 ft may be feasible in certain cases. The weights of prefabricated components should be within the lifting capacities of commonly used cranes. Mobility and crane placement constraints for a site could dictate the largest weights that can be safely handled with conventional erection. Keeping the maximum weight under 50 tons will generally allow greater ease of erection. Components up to 125 tons may be used when needed for longer spans or for wider bridge widths after careful consideration of site conditions. Substructure units constitute some of the heaviest elements in a prefabricated bridge. The use of multiple large vertical cavities within the wall elements that are later filled with high early strength concrete allows for larger precast elements and leads to lighter shipping and lifting weights.

ABC Bridge Designs Built with ABC Construction Technologies

Above-deck carriers and launched temporary truss bridges are technologies that allow rapid replacement of structures when ground access for cranes below the bridge may be limited. These technologies could be applied to a river crossing or a bridge over another highway or railway so that traffic disruptions might be minimized both on and under the new bridge.

Above-Deck Driven Carriers

Above-deck driven carriers (ADDCs) are designed to deliver individual components of a proposed structure in a span-by-span process with minimal disruption to activities and the environment below the structure.

Current ADDCs exist in two forms, and both perform a similar function. An ADDC rides over an existing bridge structures and delivers components of a new bridge span by using hoists mounted to overhead gantries with traveling bogies. As shown in the examples in this section, the ADDC equipment can be quite specialized, as in the case of the RCrane Truss system used by railroads to replace existing short bridge spans. Some equipment, like the Mi-Jack Travelift overhead gantry, require specific site adaptations to align their wheelset with the centerlines of the existing girders that support the heavy moving loads. Some examples illustrating the concept are shown in Figures 3.55 and 3.56.

One modified ADDC concept is a combination of the RCrane Truss and the Mi-Jack Travelift to create pairs of lightweight steel trusses supporting an overhead gantry system. This lightweight equipment could then be used on structures in which the existing bridge deck or girders are insufficient to support the heavier wheel loads of current ADDC equipment. This construction technology would be multifunctional, would be easily transportable both on urban and rural road systems, and would be mobilized with minimal erection and de-erection time.

The trusses of the modified ADDC would be modularized into lengths that are easily trucked over both primary and secondary roads (either shipped on flatbed trucks or towed with the mountable rubber-tired bogies). Once assembled at the project site, the system would be equipped with several rubber-tired bogies that would be spaced to reduce and more evenly distribute the localized equipment dead load. Once the modified ADDC is rolled out across the bridge span(s), temporary jack stands would be lowered at the piers and abutments and would bear on the deck where blocking had been added below, from the pier up to the underside of the bridge deck. By bearing at the piers and abutments, the modified ADDC prevents overloading the existing bridge structure during delivery of the bridge components.

This ABC Construction Technology would be applicable when an existing bridge or set of twin bridges is planned to be widened and when portions of the existing bridge are to be replaced. With several movements, the ABC Construction Technology could be used to replace an entire bridge.

Advantages of ADDCs include the following:

- Minimizes disruption to traffic and the environment at lower level of bridge project.
- Can be used when conventional crane access is limited by site constraints.
- Allows for faster rates of erection due to simplified delivery approach of components.

- Component delivery occurs at the end of the existing bridge, which minimizes disruptions at the lower level of the project site.
- Decreases the need to work around existing traffic and lessens the need to reduce lanes, shift lanes, or detour lanes, which in turn improves safety for both workers and the traveling public.
- Can be used to deliver prefabricated, modular components of ABC substructures and superstructures.

Launched Temporary Truss Bridge

Launched temporary truss bridges (LTTBs) are designed to deliver individual components of a proposed structure in a span-by-span process with minimal disruption to activities and the environment below the structure.

Currently, LTTBs exist in many forms; however, the basic principle of the technology is the same for each. LTTBs are launched across or lifted over a span or set of spans and then act as temporary bridges to deliver the heavier components of a span without inducing large temporary stresses into those components. As shown in the examples here, the pieces of LTTB equipment are designed and modified on the basis of varying sets of criteria from project to project. The equipment can be quite specialized in response to the needs of the project and could require extensive modifications from project to project in response to changes in span lengths and component weights.

The idea behind a modified LTTB is to create a set of standardized lightweight steel trusses that would be assembled to a specific length that suits a given project. The truss design and details would follow the quick connect concepts used in crane boom technology and would allow site modifications with relatively minimal effort. The lightweight equipment could then be used to bridge new spans to deliver components for a new bridge structure. This construction technology would be multifunctional, would be easily transportable both on urban and rural road systems, and could be mobilized with minimal erection and de-erection time.

The trusses of the modified LTTB would be modularized into lengths that are easily trucked over both primary and secondary roads (either shipped on flatbed trucks or towed with mountable rubber-tired bogies). Once assembled at the project site, the lightweight equipment could be launched from span to span or could be lifted into position with cranes. Once the modified LTTB has bridged the new span, it would be stabilized and supported at each pier or abutment substructure unit.

This ABC Construction Technology would be applicable when new bridge structures are to be erected and when an existing bridge or set of twin bridges is planned to be widened.

Advantages of LTTBs include the following:

- Minimizes disruption to traffic and the environment at lower level of bridge project.
- Can be used when conventional crane access is limited by site constraints.
- Component delivery occurs at the end of the existing bridge, which minimizes disruptions at the lower level of the project site.
- Decreases the need to work around existing traffic and lessens the need to reduce lanes, shift lanes, or detour lanes, which in turn improves safety for both workers and the traveling public.
- Increases the possibility of erecting longer spans without significantly increasing the cost of bridge spans because the components of the spans can be delivered without additional temporary erection stresses.
- Allows work to proceed on multiple fronts (i.e., when multi-span LTTBs are used, girders can be set while the next girder is delivered).
- Temporary loads are introduced directly into piers minimizing the need for falsework.
- Can be used to deliver prefabricated, modularized components of ABC substructures and superstructures.

Organization of Conceptual Details for ABC Construction Technologies

The erection concepts presented in the drawings are intended to assist owners, designers, and contractors in selecting suitable erection equipment for the handling and assembly of prefabricated modular systems. Examples for the organization of ABC Construction Technologies sheets are provided in Tables 4.3 and 4.4.

Erection concepts presented in the drawings group the bridges into short- and long-span categories by using the following criteria:

- Short-span bridges
  - Bridges with span lengths up to 70 ft.
  - Maximum prefabricated bridge module weight is 90,000 lb.
- Long-span bridges
  - Bridges with span lengths from 70 ft to 130 ft.
  - Maximum prefabricated bridge module weight is 250,000 lb.

ABC Design Examples

The design examples will be instructive in highlighting the differences between CIP construction and modular prefabricated construction and the advantages of modular systems.

Table 4.3. Overview of Drawings for ABC Construction Technologies

Drawing No.	Description
CC3	Short-span bridge replacement using cranes. Single span over waterway. Crane at roadway level at one end.
CC4 and CC5	Short-span bridge widening using cranes. Two-span bridge over roadway. Due to critical pick radius, crane on one side on roadway below.
CC6 and CC7	Short-span bridge replacement using cranes. Two-span bridge over roadway. Due to critical pick radius, crane on one side on roadway below.
CC8 and CC9	Short-span bridge replacement using cranes. Two-span bridge over waterway. Due to critical pick radius, crane on one side on causeway below.
CC10 and CC11	Short-span bridge replacement using cranes. Two-span bridge over waterway. Due to critical pick radius, crane on one side on temporary trestle bridge.
CC12, CC13, and CC14	Long-span bridge widening using cranes. Three-span bridge over roadway. Due to critical pick radius, two cranes on one side on roadway below.
CC15, CC16, and CC17	Long-span bridge replacement using cranes. Three-span bridge over roadway. Due to critical pick radius, two cranes on one side on roadway below.
CC18, CC19, and CC20	Short-span bridge replacement using straddle carriers. Two-span bridge over waterway or roadway. Straddle carriers on permanent bridge.
CC21, CC22, and CC23	Short-span bridge replacement using straddle carriers. Two-span bridge over waterway or roadway. Straddle carriers on launch beams.
CC24, CC25, and CC26	Long-span bridge replacement using above-deck driven carrier. Three-span bridge over waterway or roadway.
CC27, CC28, CC29, CC30, and CC31	Long-span bridge replacement using launched temporary truss bridge. Three-span bridge over waterway or roadway.
CC32	Erection of prefabricated concrete substructure elements.

Currently, economical design that uses CIP construction requires simplified fabrication with less emphasis on weight reduction. However, for ABC, shipping weights have to be minimized for economy and constructability. Shop labor is generally less expensive and easier to control in terms of quality than is field labor. Use of shop-fabricated modular elements also increases the speed of construction. In CIP construction, overall stability needs to be ensured for all stages of construction, with or without a roadway deck. Per LRFD, stability of the shape must be ensured. Girder stability during construction is not an issue for modular construction as it is for CIP construction. This will allow more efficient designs of steel modular systems, which will minimize material and fabrication expenses while ensuring adequate strength, stiffness, and stability.

Once the material of choice, structural steel has been mostly eclipsed by reinforced and prestressed concrete for short-to-medium spans built with conventional construction. Prefabricated modular steel bridges compare favorably with other materials when considering the greater use of shop labor versus field labor, the speed at which bridges can be installed, and the significant reduction in time required to close a given roadway to the public. The light weight of steel modular systems could reverse this trend in ABC designs.

Often designers concentrate on optimizing individual spans by minimizing the number of lines of girders and, in so doing, will generally reduce superstructure weights by 5% to 10%. While that is important, it is the careful determination of span arrangement and module dimensions for shipping and erection that can add significant savings in ABC design. In fact, the cost of the substructure, particularly the intermediate piers, for each design usually determines the most economical span arrangement. It may be more economical to reduce the shipping weight of pier components by adding more piers to reduce the superstructure dead loads on each pier.

The design examples developed in this task serve as training tools to increase familiarity about ABC among engineers. Three design examples are provided in Appendix F to illustrate the ABC Design process for the following prefabricated modular systems:

- Decked steel girder;
- Decked precast prestressed girder; and
- Precast pier.

The design examples pertain to the same standard bridge configurations for steel and concrete used in the ABC standards. The intent was to create design examples that could be used in conjunction with the ABC Design standards developed in Task 10 so that practitioners would get a

Table 4.4. ABC Construction Technologies Sheets

Sheet No.	Description
CC1	General Notes
CC2	General Notes
CC3	Conventional Erection Replacement Single Short-Span Bridge
CC4	Conventional Erection Widen Short-Span Bridge over Roadway
CC5	Conventional Erection Widen Short-Span Bridge over Roadway
CC6	Conventional Erection Replacement Short-Span Bridge over Roadway
CC7	Conventional Erection Replacement Short-Span Bridge over Roadway
CC8	Conventional Erection Replacement Short-Span Bridge over Waterway (Opt. 1)
CC9	Conventional Erection Replacement Short-Span Bridge over Waterway (Opt. 1)
CC10	Conventional Erection Replacement Short-Span Bridge over Waterway (Opt. 2)
CC11	Conventional Erection Replacement Short-Span Bridge over Waterway (Opt. 2)
CC12	Conventional Erection Widen Long-Span Bridge over Roadway
CC13	Conventional Erection Widen Long-Span Bridge over Roadway
CC14	Conventional Erection Widen Long-Span Bridge over Roadway
CC15	Conventional Erection Replacement Long-Span Bridge over Roadway
CC16	Conventional Erection Replacement Long-Span Bridge over Roadway
CC17	Conventional Erection Replacement Long-Span Bridge over Roadway
CC18	Straddle Carriers on Permanent Bridge: Short-Span Bridge
CC19	Straddle Carriers on Permanent Bridge: Short-Span Bridge
CC20	Straddle Carriers on Permanent Bridge: Staged Construction
CC21	Straddle Carriers on Launch Beams: Short-Span Bridge
CC22	Straddle Carriers on Launch Beams: Short-Span Bridge
CC23	Straddle Carriers on Launch Beams: Staged Construction
CC24	ADDC Concept: Plan and Elevation
CC25	ADDC Concept: Typical Cross Section
CC26	ADDC Concept: Staged Construction
CC27	LTTB Concept: Plan and Elevation
CC28	LTTB Concept: Typical Cross Section
CC29	LTTB Concept: Staged Construction
CC30	Typical Erection Truss Module
CC31	Typical Rolling Gantry Concepts
CC32	Erection Of Prefabricated Concrete Substructure Elements

comprehensive view of how ABC Designs are performed and translated to design drawings and details. The design examples focus on the design of the modules and the connection details. Additional features of the design examples include demonstration of any special LRFD loadings during construction and in the final condition, load combinations, stress and strength checks, deformations, and lifting and handling stresses. The design examples have extensive documentation describing the design criteria, the design steps executed, the design philosophy adopted, and the design specifications checks performed. All design examples are based on the *LRFD Bridge Design Specifications,* 5th ed. AASHTO specification references are presented in a dedicated column in the right margin of each page, immediately adjacent to the corresponding design procedure. The examples are organized in a logical sequence to make them easy to follow. Each example has a table of contents at the beginning to guide the reader and allow easier navigation. The design examples may also be found in Appendix B of the *ABC Toolkit.*

Recommended ABC Design Specifications for LRFD

The challenge to the future deployment of ABC systems lies partly in the ability to codify the design and construction of these prefabricated modular systems so that they are not so unique from a design and construction perspective. The LRFD design philosophy should explicitly deal with the unique aspects of large-scale prefabrication, including issues such as element interconnection, system strength, and behavior of rapid deployment systems during construction. For rapid replacement, it is possible that the stages of construction may in fact provide the critical load combinations for some structural elements or entire systems. Ongoing developments in material technology and increasing steel and concrete strengths have allowed designers to extend the useful span lengths of bridges ever farther. In some cases, the most extreme load case that these ever-longer and more-slender beams will ever experience will occur during shipping and handling prior to final erection.

At the current time, under a design–bid–build delivery method, the engineering and design services for the design of a large-scale prefabricated bridge system are performed by different entities. The engineer of record is responsible only for the bridge in its final support condition. It is the contractor who typically proposes some innovative method of construction and thus carries the burden to hire a construction engineering firm to provide the engineering services required to prove that an innovative erection technique can be used. When design–build procurement is used, greater alignment between design and construction could facilitate greater innovation in rapid renewal projects. Closing some of these gaps or inconsistencies in the specifications as related to the engineering and construction of rapid replacement bridges is a worthwhile goal for this project and other ongoing projects related to rapid renewal. Guidance should be developed for alerting engineers to an increased obligation for strength, stability, and adequate service performance prior to final construction.

Maintaining individual module stability and limiting the erection stresses induced through the choice of pick points (crane lifting points) would be a critical consideration for modular construction. The location of the pick points should be calculated so that the unit is picked straight without roll or stability problems and with erection stresses within allowable limits. The plans should indicate the lifting locations based on the design of the element. The engineer is responsible for checking the handling stresses in the element for the lifting locations shown on the plans. The contractor may choose alternate lifting locations with approval from the engineer. In order to ensure proper lifting locations are identified in the plans, the design community needs guidance or minimum analysis requirements for various erection methods for modular construction.

Design criteria proposed for the ABC standards are in accordance with the AASHTO LRFD Bridge Design Specifications. The design life, or period of time on which the statistical derivation of transient loads is based, is 75 years for these specifications. Therefore, the completed structure will need to satisfy the same design requirements as any conventionally built bridge. Any new bridge system should meet this minimum design life requirement for wide acceptance and implementation. However, it is not necessary or economically feasible for prefabricated systems during construction to be bound by the same criteria as the completed structure. The design of bridges that use large-scale prefabrication is not specifically covered in the LRFD design specifications.

The work in this task entailed identifying any shortcomings in the current LRFD Bridge Design Specifications that may limit their use for ABC Designs and making recommendations for addressing these limitations. The primary deliverable was to develop recommended specification language for ABC systems suitable for future inclusion in the *AASHTO LRFD Bridge Design Specifications.* Design issues specific to ABC include the following:

- Construction loads. What kinds of loads are unique to rapid construction? For example, loads associated with support conditions during fabrication that may be different than the permanent supports, loads associated with member orientation during prefabrication, loads associated with suggested lift points, loads associated with various erection methods, impact considerations for shipping and handling of components, loads associated with camber leveling, and so forth.
- Limit states and load factors during construction. What are the applicable limit states during construction? Mandatory strength limit states should be checked, such as STRENGTH I. What is the appropriate load factor for STRENGTH I loads? Limit state for checking of construction vehicles. Check of critical stability or serviceability effects as the component is moved, assembled, and erected. Depending on construction sequencing, abutments may be backfilled and subjected to the full earth pressure during construction prior to placement of the superstructure. Requirements for extreme events during construction.
- Constructability checks. Erection analysis to evaluate the lifting and erection stresses in prefabricated components.

To what extent is cracking allowed in prefabricated systems during transportation and erection? What are the limiting stresses, deflections, and distortion during construction for steel and concrete components? Requirements for SERVICE III checks in prestressed members. What are the bracing requirements for transportation and erection of elements and systems? Deck thickness allowances for rideability in the original design.

- Cross frames and diaphragms. What are the requirements for modular construction with regard to these bracing elements during construction? In modular construction, the girder stability is greatly enhanced by the precast deck, which could allow opportunities to ease the requirements for intermediate cross frames and diaphragms and could achieve savings in weight and cost. Additional bracings for temporary support points during construction.
- Analysis methods. What are the minimum recommended levels of analysis or stages of analysis required for bridges erected by various unique methods? Consideration of sequence of loading during construction. Are there any unique changes to structural load distribution that must be addressed for certain prefabricated bridge types and connection configurations?
- Connections. What are the requirements for closure pour design for strength and durability? Development of reinforcing steel and lapped splices in closure pours. Requirements for grouted splice couplers. Provisions for UHPC joints.

Recommended LRFD Design Specifications for ABC

Implementing the recommended ABC Design provisions into the existing sections of the *LRFD Bridge Design Specifications* would be difficult as ABC Design incorporates components from several sections of the code. As such, the specifications are written as if they were to be added as a new LRFD subsection (5.14.6) under Section 5, Concrete Structures, in the *LRFD Bridge Design Specifications.* See Appendix G for the recommended LRFD design specifications for ABC.

Recommended ABC Construction Specifications for LRFD

These ABC construction specifications pertain specifically to prefabricated elements and modular systems (Tier 2) and are intended to be used in conjunction with the standard plans for steel-and-concrete modular systems developed in SHRP 2 R04. As such, these specifications for rapid replacement focus heavily on the means and methods required for rapid construction with prefabricated modular systems.

Implementing ABC concepts into the existing sections of the LRFD construction specifications would be difficult since these ABC concepts include elements from several sections. As such, the following is written as if it were to be added as a stand-alone section in the *LRFD Bridge Construction Specifications.* See Appendix H for the recommended LRFD construction specifications for ABC.

Innovative Project Delivery and Contracting Provisions for ABC

Introduction

Specifications used by DOTs generally attempt to describe how a construction contractor should conduct certain operations by using minimum standards of equipment and materials. These are referred to as prescriptive specifications. However, rapid renewal projects often require more creativity and innovation. Alternative specification language is needed that is less prescriptive and concentrates on the measurement of factors critical to the performance of the final product, while ensuring that accelerated timelines are met and quality is maintained. SHRP 2 Project R04, Performance Specifications for Rapid Renewal, is currently developing different performance specifications that can be used effectively in various contracting scenarios.

This section contains fundamental information on various innovative construction contracting and delivery methods that may be used to enhance the implementation and delivery of ABC construction projects. Innovative construction contracting methods are typically used to expedite construction progress and project delivery and to minimize user delays. These methods range from design–build delivery to contracting provisions such as incentive/disincentive clauses and early purchasing of materials. Delivery methods are primarily focused on shortening the time needed to develop and deliver a construction project. Contracting provisions are targeted to minimize user delays and expedite construction progress.

Regardless of the contracting method chosen, there are several contracting provisions that are commonly used on ABC projects. For successful ABC implementation, the contract must be structured in such a way that there are meaningful incentives for on-time or accelerated completion and meaningful disincentives that are applied for delays in delivery, quality, or other contract terms. Additionally, acceleration techniques that require long lead times, such as early purchase of materials, are used in ABC contracts to expedite construction progress and minimize delays.

This section provides an overview of these methods and processes. More information can be found at the FHWA website, www.fhwa.dot.gov, and at DOT websites listed in the following references: Caltrans, 2007; Colorado DOT, 2006; FHWA, 2010b; Florida DOT, 2007; Michigan DOT, 2007; Minnesota DOT, 2008; Montana DOT, 2009; and Ohio DOT, 2006.

Innovative ABC Project Delivery Methods

Innovative project delivery is a natural fit for accelerated bridge construction. The two most common innovative project delivery methods that are in use on ABC projects are design–build and construction manager/general contractor.

Design–Build

Design–build is the project delivery technique that has garnered the most attention in recent years. In its simplest form, design–build places a designer and contractor under the same contract to the owner. Typically, the engineer works for the contractor in these arrangements. The design–build concept is predicated on using a performance expectation rather than prescriptive requirements so that an engineer and contractor team can provide solutions uniquely tailored to their strengths that provide the fastest and most economical solution to an owner.

The primary motivation for design–build contracts is time compression. With only a concept of the project, the engineering and contractor are completely responsible for project engineering and construction. Although the goal is primarily time savings, the method has other benefits, such as providing a single point of contact for the owner, reducing owner claims for errors and omissions, allowing maximum contractor flexibility, teaming design and construction expertise in a single team, allowing for innovation in design and construction methods and in contracts with warranty provisions, and allowing for some owner certainty following completion. Critical elements of design–build contracting include selecting projects appropriate for this method of delivery, developing prequalification procedures, establishing ranking criteria and proposal selection, controlling quality, and considering warranty or future operations.

Typically, design–build contracts are awarded after the owner has completed some preliminary design, the environmental process is complete (or nearly complete), and right-of-way is secured. The level of preliminary design is typically 10% to 30% and depends greatly on the risks associated with the project. From the contracting agency's perspective, the potential time savings is a significant benefit. Since the design and construction are performed through a single procurement, construction can begin before all design details are finalized. Design–build projects are typically tailored to large construction projects but can be used on smaller projects.

The design–build concept allows the contractor maximum flexibility for innovation in the selection of design, materials, and construction methods. With design–build procurement, the contracting agency identifies the end result parameters and establishes the design criteria. The prospective bidders then develop design proposals that optimize their construction abilities. The submitted proposals may be rated by the contracting agency on factors such as design quality, timeliness, management capability, and cost, and these factors may be used to adjust the bids for the purpose of awarding the contract.

Advantages of the design–build process include the following:

- Project delivery can be accelerated.
- The design can be tailored to the contractor's expertise and equipment.
- The team can take advantage of innovative construction processes.
- Design–build teams can modify the preliminary designs to save money.
- Owners can obligate monies very quickly on projects.

Disadvantages of the design–build process include the following:

- The owner needs to clearly identify and communicate the desired project outcomes.
- The design–build team needs to complete a relatively detailed design process in order to bid the project. Stipends are sometimes used to defer the cost of this process.
- The owner does not have complete control over the final design.
- Some agencies have reported higher costs with design–build.

Construction Manager/General Contractor (CM/GC)

The construction manager/general contractor (CM/GC) project delivery method allows an owner to engage a construction manager during the design process to provide constructability input. In a CM/GC project, the owner has a direct contract with an engineering firm and a separate contract with a construction company. The construction company is the construction manager (CM) for the project.

The team approach provides for input from all team members throughout the design and construction phases. The ability of the CM to input constructability reviews, construction phasing, erection methods, and cost estimating throughout the design process results in a more constructible project and reduces project construction delays and project costs.

The engineer and construction manager are generally selected on the basis of qualifications, past experience, or a determination of best value. At approximately 60% to 90% design completion, the owner and the construction manager negotiate a "guaranteed maximum price" for the construction of the project that is based on the defined scope and schedule. If this price is acceptable to both parties, they execute a contract for construction services, and the construction manager becomes the general contractor. The CM/GC delivery method is also called the construction manager at-risk method. CM/GC is particularly suited for ABC projects. This method has been successfully used by the Utah DOT on several ABC projects.

This method of project delivery is similar to design–build in that the designer and contractor work together to complete a design; however, in CM/GC, both the designer and the contractor have contracts with the owner. In CM/GC, the owner does not relinquish control or risk as in design–build. By having control over the entire design process, the owner can stipulate the construction method(s) that will best suit the traveling public.

CM/GC includes the following key aspects and advantages:

- A preliminary design does not need to be completed prior to selection of the designer or the contractor.
- Time savings are possible by fast-tracking design and construction activities.
- The design engineer is selected by the agency by using a qualifications approach.
- The design engineer works under the direction of the agency, not the contractor.
- The contractor is selected by the agency by using a qualifications approach.
- The selected firms form a design team. The team, working with the owner, develops the design.
- This method allows for innovation in ABC Designs through constructability recommendations from the CM.
- Since a guaranteed maximum price is established, the CM invests more during the design phase and cost estimating.
- This method fixes project cost and completion dates.

Disadvantages to the CM/GC method include the following:

- Price is negotiated with a CM and not competitively bid.
- The department retains design liability.
- The guaranteed maximum price approach may lead to a large contingency to cover uncertainties and incomplete design elements.
- Use of a guaranteed maximum price may lead to disputes over the completeness of the design and what constitutes a change to the contract.
- This method has had limited use and experience nationally on transportation infrastructure projects.

Innovative Contracting Provisions for ABC

Cost-Plus-Time Bidding

Reduction in construction time and specifically the time during which traffic is disrupted is the desired objective of ABC projects. Cost-plus-time bidding, more commonly referred to as the A+B method, involves time, with an associated cost, in the low-bid determination. The A+B method can be an effective technique for ABC projects to significantly reduce high road-use-delay impacts.

Under the A+B method, each bid submitted consists of two components:

- The A component is the traditional bid for the contract items and is the dollar amount for all work to be performed under the contract.
- The B component is a bid of the total number of calendar days required to complete the project, as estimated by the bidder.

Calendar days are used for the B component to avoid any potential for controversy that may arise if work days were used. The bid for award consideration is based on a combination of the bid for the contract items and the associated cost of the time, according to the formula:

$$(A) + (B \times \text{road user cost per day})$$

This formula is only used to determine the lowest bid for award and is not used to determine payment to the contractor.

A disincentive provision that assesses road-user costs is incorporated into the contract to discourage the contractor from overrunning the time bid for the project. Liquidated damages may also apply with the disincentive. In addition, an incentive provision should be included to reward the contractor if the work is completed earlier than the time bid. The maximum amount of incentive is usually limited to a certain percentage of the estimated construction costs.

Cost-plus-time bidding is an effective technique for projects that have critical completion dates. A+B+C bidding is similar to A+B bidding except that a third component is added to the equation. The C component is normally used for specific milestone time frames or critical completion dates. For example, a contract may have a B component that is tied to final project completion, and a C component that is tied to the completion of a phase of construction. The dollar values assigned to the time components are somewhat subjective, difficult to calculate, and sometimes hard to justify in an age of shrinking transportation funding. Agencies need to develop a standardized approach to identify these costs and make rational decisions based on the needs of the project and the effects on the traveling public. The user costs need to be consistently applied on

A+B projects to build confidence and acceptance of this method of procurement.

Incentive/Disincentive Clauses

Standard incentive/disincentive clauses (I/D clauses) have a long history of use as a method to motivate the contractor to complete work or to open a portion of the work to traffic on or ahead of schedule. I/D clauses provide a bonus for early completion or early opening to traffic. They can also be used as penalties for late project completion or for lanes not open to traffic. This method continues to be an effective option on ABC projects. The bonus or penalty is based on road-user-delay costs, and the maximum bonus is usually limited to a percentage of the project costs. Progress clauses may list any additional liquidated damages linked to agency costs that apply to late completion. The use of I/D clauses will inevitably bring about a need for careful time tracking. Delays in agency approval of submittals may be grounds for time extensions, which will greatly affect the I/D values.

An alternate form of I/D clause, known as the no-excuse incentive, is a method used to motivate the contractor to complete work or open a portion of the work to traffic on or ahead of schedule by providing a bonus for early completion or opening. The owner will give the contractor a drop-dead date for completion of a phase or project. If the work is completed in advance of this date, the contractor will receive a bonus. There are no excuses for any reason, such as weather delays, for not meeting the early completion or open-to-traffic date. Conversely, there are no disincentives (other than normal liquidated damages) for not meeting the early completion or open-to-traffic date. This technique is applicable to projects that must be open by a critical date, such as a major sporting event.

Lane Rental

Like cost-plus-time bidding, the goal of the lane-rental concept is to encourage a contractor to minimize the amount of time that through lanes are closed, and therefore limit the associated road-user-delay effects. Under the lane-rental concept, a provision for a rental fee assessment is included in the contract. The rental fee rates are stated in the bidding proposal in dollars per lane per time period, which could be daily, hourly, or in fractions of an hour. The rental fee rates are dependent on the number and type of lanes closed and can vary for different hours of the day. The contractor estimates the amount of time for which the rental assessment will apply and must bid a positive lump sum amount for the lane rental. Neither the contractor nor the contracting agency gives an indication as to the anticipated amount of time for which the assessment will apply, and the low bid is determined solely on the lowest amount bid for the contract items. The tally of cumulative lane-rental assessments are then deducted from the original lane-rental lump sum bid on a biweekly or monthly basis until the contract work is completed.

The intent of lane rental is to encourage contractors to schedule their work to keep traffic restrictions to a minimum, both in terms of duration and number of lane closures. The lane-rental concept has merit for use on projects that significantly affect the traveling public; major urban projects are prime candidates for this approach. Lane rental should not be used to reduce overall contract time but to focus on the time that roadway users are affected by construction traffic

Early Purchase of Materials

Early purchasing of materials is used to expedite the delivery of critical materials for a project. These contracts are let prior to larger contracts to ensure critical materials are on site and ready for installation on or before a specified date so that the larger contracts can remain on schedule. This method has been used on prefabricated elements and steel beams for bridge construction projects and could be particularly beneficial on projects subject to accelerated project delivery or critical completion dates. It reduces the risk of project delays from materials requiring long lead times or from potential supply shortages. It could also save cost by removing the risk of price escalation. (Separate payment under a materials-on-hand provision is allowed under FHWA guidelines.) This method requires special provisions be included in both the early purchasing and larger contracts that clearly and logically specify the contractual requirements for each contractor and their obligations for the fabrication, delivery, storage, testing, and acceptance of the materials.

Suggested Future Research

On the basis of experiences and observations, the R04 team has assembled a number of items that are recommended for future research. Research on UHPC behavior is among the most popular topics being considered. It is suggested that additional testing of UHPC bond strength with adjacent conventional concrete be performed. Iowa State University has started some tests with uniaxial pull-off specimens, and the early results indicate the bond strength is significantly less than anticipated during design. Also, a study of UHPC mixes for early strength gain that can be used in overnight closure applications could prove to be very worthwhile.

Regarding UHPC, it suggested that long-term monitoring of UHPC joints in ABC systems be put into place in order to gain more insight to joint behavior. Tests could be done for strength and serviceability of the UHPC joint and the adjacent deck concrete. Testing of transverse UHPC joints that have smaller joint widths or modified reinforcement details different than those used in the Iowa project could also be studied. Research investigating how to improve economy is critical. Can acceptable joint performance be achieved while reducing the number of hairpin bars, replacing them with straight bars, or potentially eliminating the transverse bars inserted through the hairpins? Connections that use grouted coupler details, especially in high-seismic areas, could be investigated as another alternative joint design. Perhaps looking beyond the idea of UHPC and experimenting in the testing of transverse moment connections that use high early strength conventional concrete could provide useful research data that might possibly influence future designs. Can these joints be used with the posttensioning retrofit to save the cost of UHPC for future bridges? Replacing steel fibers with carbon fiber reinforcement in UHPC mixes is another technology that could be investigated. This will eliminate the need for Buy America waivers for the steel fibers.

It is suggested that research also be directed to broader-scope project tasks. Such tasks may include research of standardized user cost models for use in ABC projects; guidelines on how to determine incentive/disincentive costs for the ABC period; performance measures for ABC projects; guidelines for the design and construction of deep foundation systems for ABC projects; and recommended tolerances, specific to ABC construction, for prefabricated elements and systems. Further research into the erection process would provide a greater understanding of potential risks and considerations that could develop during construction that are specific to ABC projects. Such research topics regarding erection may include measurement of erection stresses in modular systems so that more accurate impact values may be specified for design. Also, measurement of concrete tensile stresses from the leveling of beam differential camber and validating design approaches to account for these stresses is suggested.

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APPENDIX A

ABC Case Studies

Task 1 of this project included a comprehensive review of published and unpublished literature. Using various online databases to access published sources and relying on various outreach efforts to solicit unpublished information, the team gathered and collected the literature data throughout Phase I. Representative examples of various types of projects and ABC solutions are presented in this appendix to illustrate the broad range of solutions available to owners, designers, and contractors.

This appendix focuses heavily on case studies of prior projects, with specific focus on the accelerated bridge construction (ABC) aspects of the projects; provides information on ongoing research that has not yet resulted in field applications; and provides some information on contractual and planning aspects of ABC projects. For the case studies, examples of different types of projects are presented to highlight the variety of ABC solutions available. They are generally grouped into projects that executed ABC through prefabrication and those that used innovative construction techniques or structure movement as the core of the solution. Many projects increasingly involve both of these components, so a strict division was not always possible.

The following discussion provides a general overview of the material to be presented and then, by using a series of illustrative projects, describes various innovations. Although this section presents the material as a literature review, the team has in many instances written a short synopsis on how a particular concept, technology, method, etc., can be applied to future ABC projects. Many of these reviews should thus be considered as concepts for implementation. The literature review clearly demonstrates that through repetition of and improvement on existing ideas, dramatic progress can be made. Owners and designers should consider the following material a starting point for future improvements and greater implementation.

Standard Bridge Systems

This section addresses the development of standard bridge systems. By standard bridge systems, the report refers to work that attempts to look at the problem or opportunity of ABC from the standpoint of integrating various components into a cohesive model. Several agencies have completed work in this area, and their reports are summarized here. However, one synthesis study (Shahawy, 2003) is particularly noteworthy; it provides a wealth of pertinent references and information related to prefabricated bridge elements and systems for ABC applications.

Notably, one effort is not specifically discussed because it is a work in progress, and that is the development of a series of ABC standards by the State of Utah. (Note that only the creation of standards in the State of Utah is a work in progress; the overall ABC initiative in Utah is significant, as evidenced by the number of case studies from Utah contained throughout this appendix.) Utah DOT, through a series of workshops involving local and national experts, contractors, consultants, and state and FHWA personnel, has embarked on a substantial effort to make ABC standard practice. That goal will partly be accomplished through the development of standards and components that work together in a systems approach to the problem. Users and owners should look to the future completion of this work as one of the starting points for ABC deployment.

Other states can and likely will transition their standard practices to ABC (or at least allow it as an option) through modifications to their standard drawings. States such as Iowa, Texas, Washington, and others have extensive sets of standard drawings in presentation or near presentation quality, but these sheets are typically detailed for conventional construction. Re-detailing a number of these standards for ABC implementation is a fairly simple matter. For instance, the Iowa DOT has completely designed and detailed piers and abutments for various standard roadway widths and span lengths as part of its standard bridge plans. Piers consisting of hammerhead and multi-column bents are fully detailed and designed—but for cast-in-place construction. Simple alternative detailing of the column/footing joint and column/cap joint would make these plans immediately useful for precast ABC substructure construction. Similarly, the Iowa DOT has fully detailed slab designs based on the standard cross sections for common roadway widths. Simply replacing the cast-in-place deck detailing with a standard full-depth precast deck and adding the required connection pocket and posttensioning details (if used) could convert these plans for ABC superstructure use. Certainly, other states could do the same.

Several illustrative project summaries are presented on the following pages.

Project Title: Guidelines for Accelerated Bridge Construction Using Precast/Prestressed Concrete Components
Citation: Precast/Prestressed Concrete Institute Northeast Technical Committee, 2006
ABC Design Features: Complete construction of bridges by using precast concrete components
ABC Construction Features: Rapid construction of typical small and medium-size bridges that use common materials and equipment

Project Description

The PCI Northeast Bridge Technical Committee worked collectively with engineering, producer, and DOT members to develop a design and construction guidance document for providing completely precast/prestressed/prefabricated bridges for rapid erection. This appears to be the first and, as yet, only comprehensive document that addresses bridges as a complete product.

As Figure A.1. indicates, the manual provides guidance on all elements of traditional bridge construction, including details for prefabricated superstructure elements such as full-depth deck panels, various details for common bridge piers such as multi-column bents, and more uniquely, details for prefabricated abutments. Connections between the various subsystem components are provided. Most innovative are the abutment details including various grouted void details for attaching the abutment shells to a pile group.

A portion of the manual is related to connection design, specifically the promoted use of grouted sleeve splice connectors to join abutting elements. The use of grouted splice sleeves allows for what is known as "emulative design," in which the member is designed as if the connection is not present. These connections are easily executed in the field and develop, as a minimum, 125% of the yield strength of the rebar. Some commercial products can resist a force of 150% of the strength of the rebar.

ABC Opportunity

Agencies looking to develop ABC standards for typical bridges should examine the PCINE recommendations in detail. They provide simple and tested connections to bond various precast concrete elements. They also provide case studies that demonstrate the use of these products in several bridge reconstruction projects.

Project Title: Alabama Precast Bridge System
Citation: Fouad, Rizk, and Stafford, 2006
ABC Design Features: Creation of complete integrated precast bridge system
ABC Construction Features: Rapid construction of complete bridge systems by using precast elements and conventional equipment

Project Description

In a study by the University of Alabama, Fouad et al. (2006) discuss the development of an entirely precast bridge system for the State of Alabama. The study focused on all elements of a traditional bridge, specifically deck panels, superstructure elements, bent caps, columns, solid piers, and abutment configurations. The intent was to create an integrated precast bridge system that could be constructed rapidly, presumably by local forces and contractors. The final result consists of bulb tee beams, a rectangular bent cap, hollow column sections, and precast abutment sections.

In developing the final products, the team was careful to use only commonly available materials and design strengths and to specify that all connections between precast elements be made with the commonly available NMB splice sleeve grouted coupler. [Note that this same coupler has been used for decades in the precast building and bridge market and is currently being advocated by others in the transportation industry as a standard method for connecting precast elements through a process known as emulative design.] The following limits were adopted for design of the standard bridge system:

- Girder weight, not to exceed 160 kips nor a length of 130 ft;
- Bent cap weight, not to exceed 150 kips; for example, a cap 5 ft (h) by 4.5 ft (w) by 45 ft (l);
- Maximum column weight, 100 kips, and maximum slenderness, $kl/r = 100$ (to preclude second order analysis); and
- Standard roadway widths of 40 ft and 44 ft.

The girder section chosen is a deck bulb tee beam, patterned after the section in the PCI *Bridge Design Manual.* The top flange varies; either 85- or 93-in.-wide flanges are used. The section depth varies; 36-, 50-, or 64-in.-deep sections are

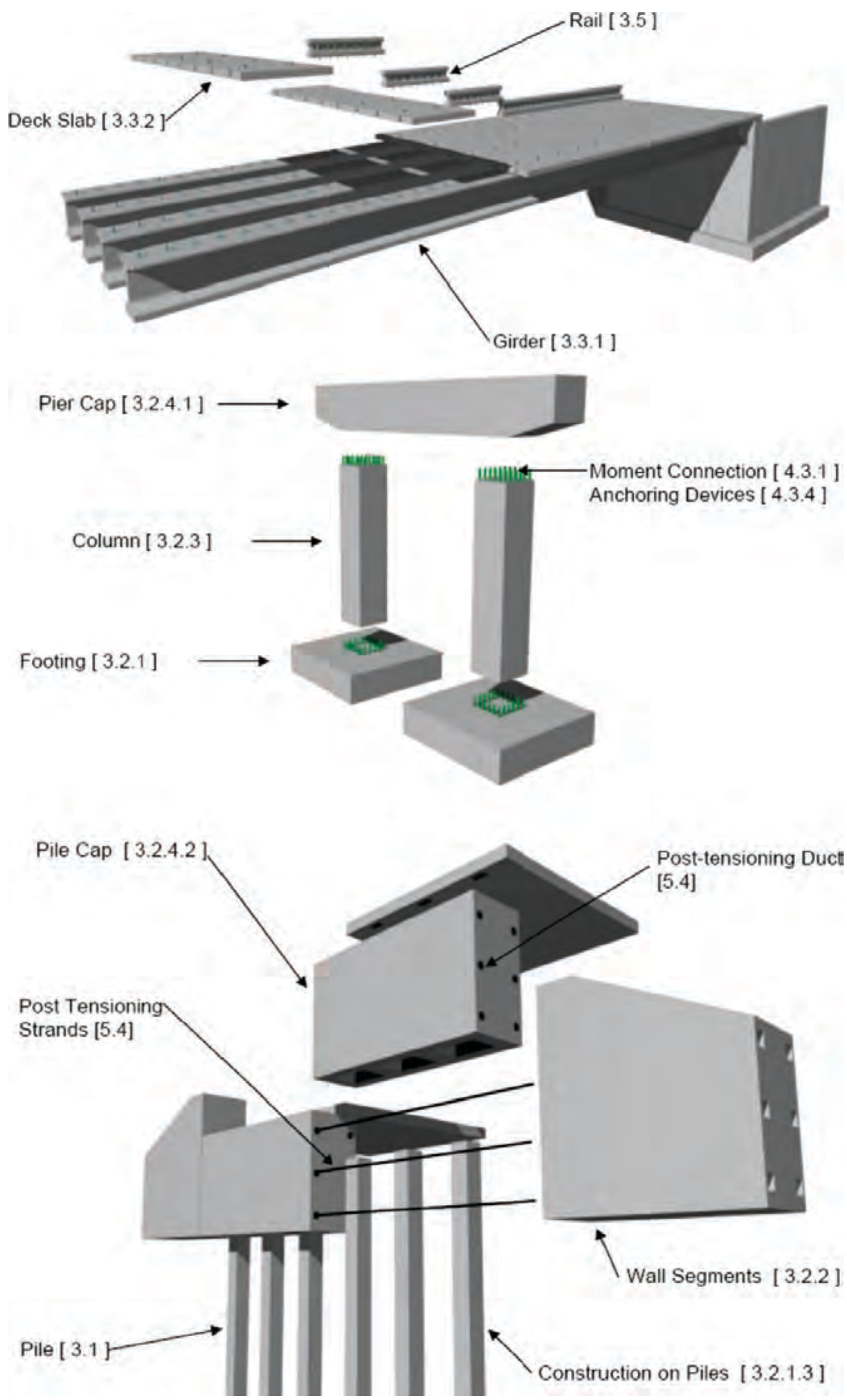


Figure A.1. PCINE modular bridge system.

proposed. This gives a span range capability of 40 ft to 130 ft with three sections. The beams are connected in the field with welded flange connections and covered with a concrete topping to obtain the required ride profile and additional strength.

For the bridge columns, rectangular sections with an inner void were chosen to limit weight. The end sections near the connections are solid. This design is similar in many regards to pretensioned box beams with solid end sections. Based on both the weight and kl/r requirements for the column, limiting maximum lengths were established for columns, including 36-, 42-, 48-, and 54-in.-square sections. These columns could typically have heights of 45 ft to 55 ft, generally tall enough for a large variety of bridges.

The precast caps were designed to be used with a two-column bent and the roadway widths described previously. The caps are connected to the pier columns with the grouted sleeve couplers also described previously. They are made of conventionally reinforced concrete. Details are provided to modify the bent cap for attachment to multiple piles as would commonly be used for a pile bent.

Abutment details allowing for the use of spread footings, driven pile, or drilled shaft foundations were also developed in concept. In addition to the design details, shipping and handling details were provided for the various elements. Borrowing from the traditional pick point diagrams commonly used in the prestressed concrete industry and found in sources such as PCI design manuals, stripping, lifting, and bracing details for various elements were provided. Several example structures were designed and partly detailed in the Alabama study to illustrate application of the design and detailing recommendations.

ABC Opportunity

This project and its findings can be immediately implemented at the state level or modified by others to suit their local practices. The project illustrates the development of a cohesive system concept of entirely precast elements with weights, dimensions, and levels of innovation that will not inhibit their use by local contractors. Some of the components can be assembled ahead of time for rapid movement; but even in the basic configuration, erection of a completed bridge is possible in days once the foundation construction is completed or if portions of the existing foundations can be reused.

Project Title: Washington State DOT Prestressed Bridges for High Seismic Regions

Citation: Hieber et al., 2005a; Wacker et al., 2005; Hieber et al., 2005b; Khaleghi, 2005; Khaleghi, 2008

ABC Design Features: Systematic development and evaluation of precast systems for high-seismic ABC environments

ABC Construction Features: Use of fully precast pier and superstructure systems erected with conventional equipment

Project Description

The Washington State DOT has been sponsoring research in the area of ABC applications for precast/prestressed structural concrete for more than a decade. With a large amount of research in the area of precast/prestressed elements for ABC application, Washington State DOT was looking to evaluate the viability of these systems for use in the high-seismic regions of western Washington State. A series of studies evaluated existing structural systems, commented on their merits and disadvantages, and then developed new concepts suitable for high-seismic use and rapid erection for ABC applications.

In Hieber et al. (2005a), which appears to be the first of a series of incremental reports in the use of precast elements for ABC construction, the authors evaluate various ABC concepts, including the use of full-depth deck panels, partial-depth stay-in-place form panels, prestressed concrete multi-beam superstructures (i.e., double-T systems), prefabricated composite units, and precast concrete pier units. For each type of system evaluated, the authors identified various key issues that represented long-term durability concern or possible constructability issues, or certain aspects that were troubling or unique from a seismic design perspective. A brief description of their findings follows for some of the systems evaluated.

Full-depth deck panels. The full-depth deck panel was studied and determined to be an attractive system for rapid deck installation (or redecking of existing bridges) even though the authors acknowledge that such systems have an inconsistent performance history. The study examined connection details between the panels and girders, the joints between panels, required prestressing, and wearing surface issues. The panels were recommended for seismic area use despite the lack of complete confidence in the water tightness and durability of the transverse joints and some question about the diaphragm action capabilities of the deck in resisting seismic loading.

Multi-beam superstructure. These structures are composed of systems such as double-T, box beam, inverted channel, and deck bulb tee sections placed adjacent to one another and attached via grouted joints, shear keys, mechanical fasteners, or transverse posttensioning—or sometimes a combination of methods. The evaluations concluded that all such systems are viable and have performed well. Concern centered on the durability of the longitudinal joints (grouted or mechanically connected) and establishing a durable connection for higher average daily traffic locations. Other concerns pertained to constructability, such as camber match issues between adjacent beams and attaining a smooth riding surface as a consequence. None of these is a substantial impediment to the use of the system. Again, adequate diaphragm resistance must be achieved in spite of the many longitudinal joints in the finished bridge.

Precast pier systems. The most extensive portion of the study by Hieber et al. (2005a) focuses on the potential use of precast substructure units for bridge pier construction in high-seismic regions. Although bridge superstructures traditionally have some degree of vulnerability, the nature of seismic loading is that the controlling demands are concentrated in the substructures, more specifically in the bridge piers. This study focuses on the development by others of various precast pier system concepts and the implications of seismic loading on the various concepts.

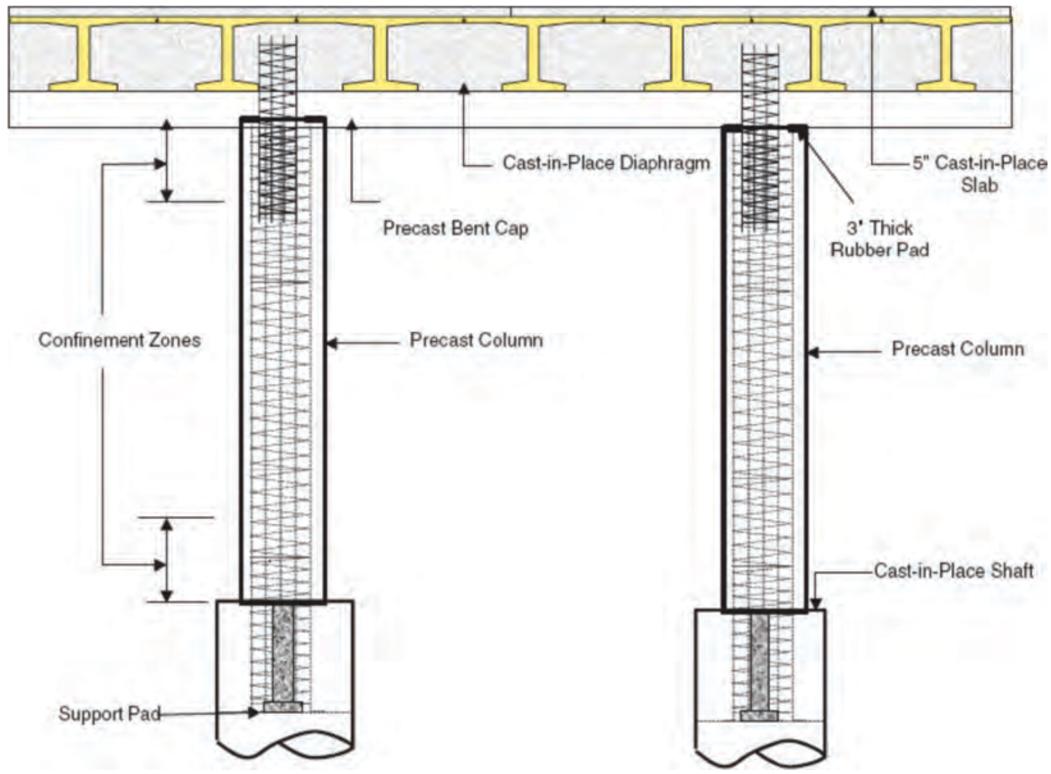


Figure A.2. Washington State DOT precast bridge system.

Given the possibility of high-demand seismic loading, the authors identified various critical issues that would need special attention in the use of prefabricated columns. These issues include the development of a connection between the footing and precast segmental column segments, connection between segments, connection between segments and the cap beam, connection of cap beams together for a wide frame, and weight/handling concerns.

The report discusses details for connecting the base segment to the footing, including the options to use multiple staged pours to anchor the footing and base segment and variations of dowel bar connections between the two elements. Regarding joints between column segments, the authors advocate the use of match cast segments to provide the proper dimensional tolerance and fit for the columns. These segments must be prestressed enough that the transverse joints do not open under lateral loading. A high degree of prestress introduces a high compression in the concrete and a high initial strain in the tendons, both of which lead to limited ductility. The possible use of large tendons at a lower stress is discussed as a solution. For the joint between the column and cap, details such as individual dowels in preformed holes, posttensioned connections, and the use of headed bars to develop the connection strength in a shallow cap depth are all considered and discussed briefly. Weight limitations were discussed with local fabricators and contractors in Washington State, resulting in a range of acceptable weights for precast elements listed as 120 kips to 180 kips. On the basis of only those weights, the authors computed the length and diameter of columns that would be achievable in these conditions. When kl/r considerations are also used to control column slenderness, the range in heights for a prefabricated piece meeting a limit k/r = 100 and the 120-kip or 180-kip limit is such that a 3-ft-diameter column could have a maximum unsupported length of 41 ft, and a 6-ft column could have an unsupported length of as much as 75 ft.

The study recommends that precast segmental column concepts be considered for column design in high-seismic regions. The authors suggest that unbonded posttensioning in conjunction with mild steel bridging the joints between segments would be the most effective total solution. The unbonded posttensioning allows for a ductile behavior during seismic loading and creates a restoring force to recenter the column.

In summary, precast/prestressed elements could have many uses in a high-seismic region such as Washington State. Many other such areas exist around the United States, the obvious being California; but locations such as St. Louis, Missouri; Memphis, Tennessee; Charleston, South Carolina; Salt Lake City, Nevada, and others are challenged to provide a valid seismic design for all projects as well. The precast solution has the promise of being standardized.

Hieber et al. (2005b) conducted a detailed analytical study. The researchers used nonlinear pushover analyses and simulated earthquake loading to evaluate the performance of precast pier systems constructed with reinforced concrete connections designed by using an emulation design concept and a second pier system connecting the pier cap to the column by using a posttensioning system. The results of the parametric study indicated that acceptable performance could be expected for both a 10% in 50 years and a 2% in 50 years earthquake event. No experimental work was conducted—only a series of analyses was done. The researchers recommend that additional physical testing be conducted, that further analysis of the joint regions be done, that constructability again be evaluated, and that some consideration of post-earthquake repair be part of further studies.

Wacker et al. (2005) focused on developing displacement and force-based design procedures that do not require nonlinear analysis and that could be used in a design office. The focus again is on the emulation design and hybrid posttensioned connections studied first in the Hieber report (2005b).

The study first looked at developing an equivalent lateral force design approach to pier design—analogous to the design approaches currently used in the AASHTO specifications and based on the use of "R" factors to scale the design forces. The downside of the R factor approach is that no description of expected behavior is possible.

In a second effort, a displacement target design method was developed that allows the user to choose acceptable levels of pier displacement during a design event and, using elastic dynamic analysis methods, determine the pier forces. Both methods led to similar and acceptable results. Both the reinforced emulation details and the hybrid posttensioning connections were found to provide acceptable designs.

Khaleghi (2005) presents the Washington State DOT implementation of this research as well as other work in the area of precast elements used for structures in high-seismic demand regions. The reports details the state DOT philosophy on connection design between superstructure elements in prestressed concrete bridges made continuous for live load, and also details the design criteria for the connection between superstructures and substructures. Khaleghi also discusses the use of precast/prestressed concrete elements for rapid pier construction. He briefly details the design requirements for the main member design, including geometric considerations for ease of constructability. And he provides a suggested design procedure for successfully using precast pier elements in high-seismic regions with a specific application to ABC projects. Similar information is presented in Khaleghi (2008), including the precast seismic resistant bridge with decked girders presented in the Figure A.2.

ABC Opportunity

The use of precast pier elements has obvious advantages over the slower process associated with cast-in-place construction. A hesitancy to adopt a structural design concept with discrete elements connected mechanically at the joints has led to slow adoption of precast substructure systems in areas of high-seismic demand. The studies by the Washington State DOT, with support from the University of Washington, have evaluated most critical aspects of precast element use in ABC applications in high-seismic demand areas and provided adequate design and detailing guidance.

Bridge Superstructure Concepts

This section focuses on components that can be used to accelerate bridge superstructure construction. Obvious solutions such as conventional prestressed concrete beams are not explored. Those are common practice, and no additional review is required. The projects that follow involve various newly researched systems and details that are unique to U.S. practice.

The concept of developing a bridge system from precast concrete components for accelerated construction is not a new idea. A pair of bridges along I-87 between the Westchester Expressway and Armonk, New York, was constructed completely of precast and epoxy connected concrete components in 1965 (*Engineering News Record,* 1965). The bridge superstructures consist of precast concrete box girders supported on precast concrete piles. The substructure consists of precast pile cap beams with cast-in-place abutment backwalls.

Bridge superstructure erection can be advanced in many additional ways other than those presented here. Concepts such as the Nebraska Inverted T beam system, the Washington State DOT's use of deck bulb tee girders, and others have been used to accelerate bridge superstructure erection. There are numerous references for these projects, and they are reasonably well known. The use of deck bulb tees is the focus of ongoing research as part of the NCHRP 12-69 project, and recommendations on improved joint details and design considerations should be available as the project progresses. Data will be incorporated as they become available in subsequent phases. Similarly, some useful and unique information related to precast segmental construction of major bridges internationally is available, including a report on the Sutong Bridge in China which is the world's longest cable-stayed bridge (Liu et al., 2007).

The experimental Roize Bridge, located near Grenoble, France, uses an innovative composite space truss and unique modular construction methods specifically developed for efficiency and economy. The Roize bridge, completed in 1990, was the first structure to use a composite space truss combining concrete, structural steel, and prestressing to provide a stiff, yet lightweight structural system (Mueller, 1993).

The Texas DOT (Cox, 2008) has developed a prestressed concrete decked slab beam (with accompanying standard drawings). These decked slab beams are cited by William Cox as being "a good selection when rapid construction is desirable and when minimal superstructure section depth is necessary for bridges on low-volume roadways." With a standard width of only 24 ft, these decked slab beams are not likely to be viable for most locations where rapid construction is desired. However, the promotion of these types of standard sections represents a significant step forward.

The I-95 James River Bridge in Richmond, Virginia, completed in 1958, is six lanes wide and 4,185 ft long (Kozel, 2009). In 1979, a latex concrete overlay was used to rehabilitate the bridge deck. Approximately 20 years later at a cost of $49 million, the superstructure on the bridge was replaced. By using an innovative reconstruction technique, rehabilitation of the bridge was managed with minimal disruptions to the traveling public. Before work began on the superstructure of the bridge, dozens of preconstructed composite units consisting of an 8¾-in. deck and steel plate girders were fabricated approximately 1 mi away; those units varied in size and weight, with the heaviest weighing close to 120,000 lb. Minimal disruption was achieved by working on the bridge only Sunday through Thursday between 7:00 p.m. and 6:00 a.m. During that time period, the six lanes were reduced to one lane in each direction. Using concrete saws, construction crews cut free a segment of the existing superstructure. That segment was then removed by using two high-capacity cranes and hauled away. Next, a new preconstructed segment was transported to the bridge and set in place with the same cranes. Each segment carried three lanes of traffic; two segments were needed to form one span of a three-lane roadway. At the end of the 11-hr construction period, the six lanes of the bridge were reopened for rush-hour traffic with the new superstructure segments in place. This procedure was repeated until the entire superstructure of the bridge was replaced.

By using precast bridge abutments, wingwalls, and deck girders, the replacement bridge at Mitchell Gulch was completed in 37 hr of actual construction time over a 46-hr period over one weekend in August 2002 (Culmo, 2009). Before this period, the contractor had driven H-piles to support the precast substructure; and those H-piles were positioned to avoid the existing bridge structure, which was removed in just over 5 hr. Each abutment (consisting of lower and upper back wall units) was 44 ft wide; wingwalls were 23 ft long. Superstructure elements, which also provide the deck of the bridge, were 5 ft, 4 in. wide, 18 in. deep, and 38 ft, 4 in. long; eight of those units were required to obtain the desired bridge width. To save time, the outside deck girders were constructed with integrated bridge railings. Since precast concrete elements made up 90% of the structure, a significant amount of welding was required to connect them. More than 1,200 linear feet of field welding was required in this short construction time, which led to one observation that other methods are needed for connecting precast units in ABC projects.

By replacing the deck on the Lewis and Clark Bridge on SR-433 over the Columbia River between the states of Washington and Oregon, the two state DOTs extended the life of the bridge by an estimated 25 years (FHWA, 2009d). More than 70% of the deck (3,900 ft) was replaced using 103 prefabricated full-width deck panels that were 36 ft wide and varied in length from 20 ft to 45 ft. The panels, which were 7 in. thick (6 in. of lightweight concrete plus a 1-in. modified concrete overlay), comprised two longitudinal steel stringers with several intermediate transverse stringers. In addition to the full closures, which were allowed for only 120 nights between 9:30 p.m. and 5:30 a.m., four weekend closures were allowed. Rather than use the Washington State DOT–designed placement procedure, the contractor developed a placement procedure by using self-propelled modular transporters (SPMTs) with specially designed steel truss frames for lifting, transporting, and installing the new panels. The modified SPMTs were able to transport a new panel onto the bridge, remove the old panel that had just been cut out, and then lower the new panel into place before taking the old panel off the bridge. One panel was replaced each night in approximately 6 hr. One indication of the success of this system is that the contractor received a bonus for early completion.

In all likelihood many additional ideas have been used successfully by owners, both domestic and international. (That an idea is not discussed in this report should not be interpreted as having any connotations.) This report is not intended to catalog every possible idea but to highlight and draw attention to the concepts this team reviewed that appear to have promise and have already performed successfully in limited or broader application. Both steel and concrete bridge solutions are provided in the following illustrative projects.

Project Title: I-95 James River Bridge Replacement, Virginia
Citation: Kozel, 2009
ABC Design Features: Nighttime replacement of modular sections of existing superstructure
ABC Construction Features: Preassembled segments lifted into position during overnight closures

Project Description

The Interstate 95 James River Bridge in Richmond, Virginia, is approximately 4,200 ft in total length and carries six lanes of traffic with a total volume of 110,000 vehicles per day. The original structure was built in 1958. The renewal project consisted of substructure rehabilitation and superstructure and deck replacement. The renewal project was started in 1999 and completed in late summer 2002. The Virginia DOT decided to use an innovative construction method allowing partial closure (leaving one lane open in each direction), Sunday through Thursday nights, with the bridge returned to nominal full capacity each morning by 6:00 a.m. to maintain traffic during peak travel times. While substructure rehabilitation was under way, large sections of the replacement superstructure were constructed at an assembly site within a mile of the bridge. At closing each night, existing deck and superstructure spans were cut at the bents and removed. Bearing pockets were prepared, and preassembled replacement superstructure and deck segments were set in place by using high-capacity flatbeds for transport and two ground-mounted cranes for hoisting. Preassembled segments were typically less than 100-ft long and weighed an average of 60 tons. Each segment carried three lanes of traffic, and the contractor was able to set one segment per night. A full span (six lanes) required two preassembled segments. A total of 102 preassembled segments were replaced. For the main span of 800 ft, the existing steel truss superstructure was left in place. The existing concrete deck was removed and replaced with precast deck panels, also during nighttime partial closures. The contractor was allowed 179 nights of closure for segment removal and replacement but needed less than that.

ABC Opportunity

The James River Bridge in Virginia was a good candidate for accelerated construction for a variety of reasons. The bridge carries a high volume of rush-hour traffic through a major metropolitan area. Also, the Virginia DOT and the design/consulting team carried out a thorough community dialogue before deciding on a final design and construction method. That process, along with a thorough public relations and communication plan during construction, created greater acceptance of the use of innovation and accelerated construction. Removal and replacement of the existing bridge spans over the James River was accomplished with minimal disruption of traffic through the use of full-span, partial-width, steel-girder-with-concrete-slab prefabricated segments erected during off-peak hours with high-capacity cranes and conventional flatbed trailers. Each span was replaced before reopening the bridge at 6:00 a.m. The contractor was able to set up a temporary fabrication yard close to the bridge site, and the span weights and geometries (with the exception of the main span) were such that they could be designed to accommodate off-site assembly and transported into their final position.

Project Title: Lewis and Clark Bridge Replacement over Columbia River, Washington and Oregon
Citation: FHWA, 2009d
ABC Design Features: Full-depth precast deck replacement
ABC Construction Features: Preassembled segments lifted into position during overnight closures

Project Description

The Lewis and Clark Bridge across the Columbia River carries SR-433 between Oregon and Washington State near the Port of Longview, Washington. The bridge is 5,478 ft in total length, with 34 spans carrying 21,000 vehicles per day (13% trucks). The main center span is 1,200 ft long. The original structure was built in 1929 and appears on the National Register of Historic Places because, at the time of construction, it was the longest and highest cantilever steel truss bridge in the United States.

After considering complete replacement with a new cable stayed bridge, the Washington DOT opted to extend the life of the existing bridge by 25 years by using full-depth precast deck replacement design. An innovative approach to deck panel replacement involved the contractor's use of large specialized transport equipment that could both remove the old deck panels and transport new panels to the site. Full-width, full-depth, precast replacement deck panels were hoisted into location with a specially designed steel truss framed gantry system. This use of innovative construction equipment and prefabricated design elements allowed the contractor to meet scheduling constraints.

The precast concrete deck panels were constructed of lightweight concrete and received a concrete overlay after placement. The panels and overlay were supported by a frame of longitudinal and transverse steel stringers. The bridge was also widened by incorporating prefabricated sections supported by a single longitudinal steel girder. Construction was further accelerated by using precast approach slabs. The project was completed in August 2004.

ABC Opportunity

The Washington DOT was committed to accelerated construction to minimize the impact on the traveling public and maintain efficient operations within the Port of Longview adjacent to the bridge. To accomplish the accelerated schedule, the contract allowed for full closure nightly, from 9:30 p.m. to 5:30 a.m., for 120 nights (124 were actually used), plus four weekend closures (three were actually used). Traditional designs and construction methods would have required either a 4-year construction schedule if performed under traffic (single-lane closures) or full closure for several months. In addition to accelerating the construction schedule, use of prefabricated elements allowed for material inspection before installation, which eliminates the need for specialized testing equipment and promotes worker safety by reducing exposure to traffic during construction. Furthermore, the use of full-depth, full-width precast replacement deck panels, precast widening sections, and precast approach slabs improved the constructability of the bridge.

Project Title: Steel Box Girder Bridges Made Continuous for Live Load

Citation: Azizinamini, Yakel, and Niroumand, 2008
ABC Design Features: Design of a pre-topped steel box for accelerated construction
ABC Construction Features: Use of prefabricated steel box girder modular superstructure units

Project Description

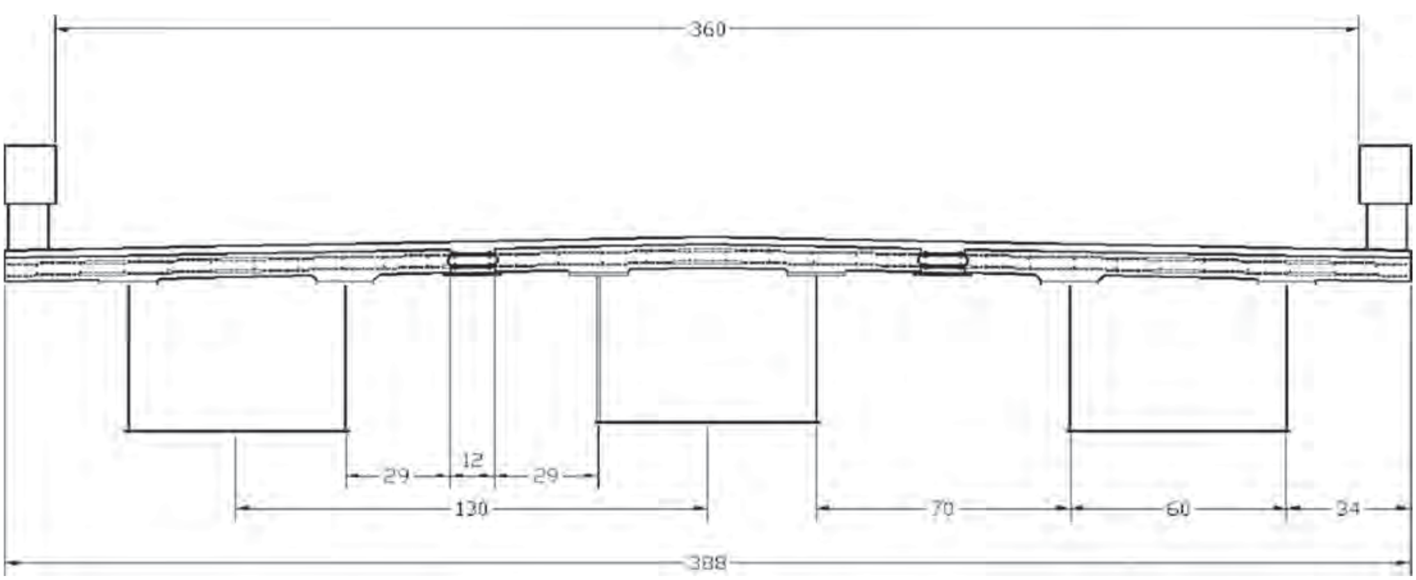
Azizinamini et al. (2008) focuses on the use of pre-topped steel box girder units as an ABC technology. The team of researchers had previously conducted research in the area of steel bridges constructed as simple span for dead loads and continuous for live loads. This is analogous to the traditional approach used to construct prestressed concrete beam bridges.

In the conventional approach for prestressed concrete, beams are erected in a span-by-span situation, simply supported and not connected to each other in any way at the substructures. Because of their own weight and the slab weight, the girders resist loads in a simple-span condition. Only after the deck hardens can the structure behave in a continuous fashion through special detailing of the closure pour at the pier locations. This technology has traditionally allowed for rapid erection of concrete bridges. Steel bridges, on the other hand, have been designed and constructed as continuous structures, largely because we can connect them in the field. That requires a timely operation to complete bolted field splices in the air, many times over active traffic lanes. Azizinamini provides a design approach to use the simple-made-continuous approach for steel bridges as well. The slight loss of economy in the positive moment region—due to lack of continuity for dead loads—is counteracted by a reduction in the negative moment at the piers—since continuity is present for only a portion of the loads. These two tend to cancel each other in terms of steel required to resist the loads. Additionally, substantial benefits are attained in stability and safety by using simple-made-continuous systems.

Earlier development involved the use of simple-span steel I-beams and a cast-in-place concrete deck. The new proposal by the University of Nebraska researchers is to construct steel box girders in lieu of I-beams and to add a concrete deck to the box, with the completed section lifted into place on a bridge. A simple longitudinal closure pour is required to complete the bridge. The researchers propose that, in lieu of traditional lap splices, the closure pour contain headed reinforcing bars, shown to result in shorter lap lengths and thus a smaller closure pour.

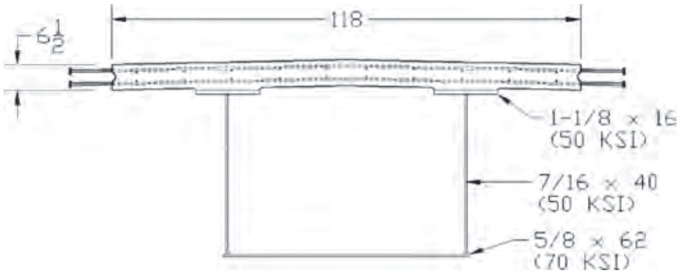
In Azizinamini, various end details are proposed and tested for the steel beams. To achieve continuity, a concrete pier diaphragm is cast that envelops the ends of the beams. In negative bending, a compression block forms near the bottom flange and can be transferred by the flanges bearing against each other or—at greater expense and with greater difficulty—by welding in the field. That was deemed too expensive and time-consuming. A modified detail was developed that eliminates the contact condition for the flange and uses end plates to provide the bearing surface. For the tension component, the connection again is as in a concrete bridge; supplemental longitudinal slab reinforcing is used to make the connection. The report provides several details.

Figures A.3 and A.4 illustrate a typical section of a bridge; dimensions are given in inches. The slab width for a typical prefabricated section is 118 in. (about 10 ft), and the slab measures 6.5 in. deep with a subsequent overlay. A hybrid girder with Grade 50 steel for the top flange and webs and Grade HPS70 steel for the bottom flange is proposed. A typical prefabricated section is shown.



Source: A. Azizinamini.

Figure A.3. Typical section of modular steel box beam bridge.



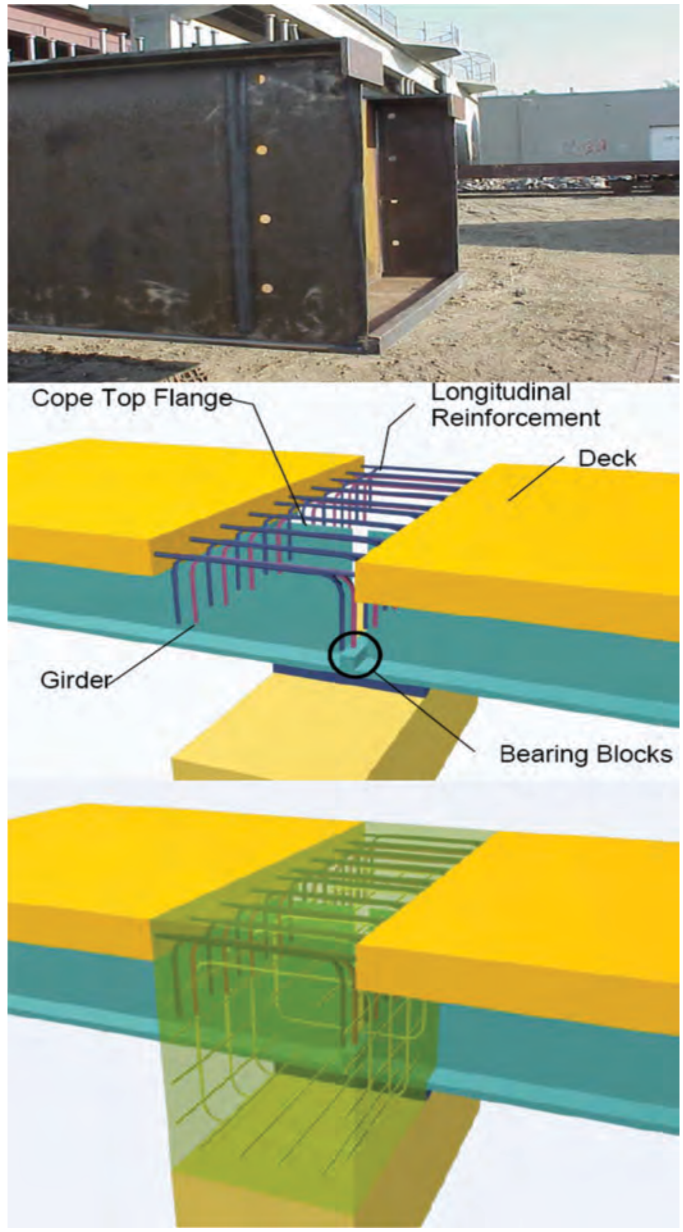
Source: A. Azizinamini.

Figure A.4. Modular steel box beam bridge, typical unit.

Once the composite beams are prefabricated, they are erected in simple spans and adjacent to one another. The longitudinal closure pours are completed, as is the pier diaphragm connection. Completed pre-topped sections can be constructed with either lightweight concrete (120 pcf) or normal-weight concrete and again with either the 6.5-inch subdeck or a conventional 8-in.-deep complete deck. Azizinamini provided data to the study team as a supplement to the published paper, indicating that for a series of study bridges, the completed box sections would range in weight from roughly 70 tons to 100 tons. That is comparable to the weight of heavy prestressed concrete bulb tee or tub girder bridges, so erection with cranes is possible with this solution. It is promoted specifically as an ABC solution involving the use of conventional technology. However, it is conducive to ABC construction techniques as well. The study cites the case of a planned bridge replacement of 262nd Street over I-80 near Lincoln, Nebraska. Because a convenient detour is available, the bridge can be closed and demolished. However, the new bridge will be constructed on the existing approach paving, at grade, and then launched forward by using an SPMT at the leading end to move the bridge.

ABC Opportunity

The ABC opportunity provided by this technology is the possibility of eliminating the costly, time-consuming, and sometimes dangerous operation of field splicing steel girders while in the air. Analogous to concrete bridges, the girders are erected in simple-span conditions under noncomposite dead load; following the pier joint closure pour, the bridge is made continuous for composite loads such as barrier rails, live load, and any future wearing surface or overlay. The girders can be erected in a pre-topped condition with a full-depth deck or subdeck, and yet the pick weights still remain within the capacity of commonly available cranes. Thus, an opportunity exists to erect a superstructure in not much more time than it could be installed with movement techniques such as SPMT or lateral skidding. The only extra time is the required field closure pour and curing time.



Source: A. Azizinamini.

Figure A.5. Pier continuity details.

Project Title: Pre-Topped U-Beam Structures
Citation: Griffin, Wolf, Freeby, Hyzak, Hohmann, and Cox, 2008
ABC Design Features:
ABC Construction Features: Rapid construction of highway overpass structures with traditional materials

Project Description

In response to the development of a pre-topped steel tub girder system by the Texas DOT, the Precast/Prestressed Concrete Manufacturers Association of Texas hired Project R04 team member Structural Engineering Associates (SEA) to develop a similar pre-topped U-beam concept using precast girders for the structural system. Following review of the system, the Texas DOT elected to institute a pilot project of four bridges in the Waco District.

The pre-topped U beams developed for these bridges have an overall structure depth of 3 ft, 2 in. and a span length of 115 ft. The beams' construction process is geared toward minimizing the time of on-site construction. To minimize fabrication cost, the pre-topped U beam has the same soffit width, strand layout, and web face slope as standard Texas DOT U beams. The pre-topped U beam is composed of a 10.5-in.-thick bottom flange, 6-in.-thick webs, and a 7-in.-thick top flange. The top right and left corners of the top flange are dapped (9 in. wide by 4.5 in. deep).

After fabrication, the beams are transported to the contractor's yard (near the job site), where they are erected on temporary supports that simulate the permanent supports—providing the same cross-slope, longitudinal slope, and relative elevation between beams. By using a screed machine and blockouts for the closure pour strips, a 4-in.-thick topping is placed on each beam. The metal portion of the exterior rail is then installed on the exterior beams. Once the 4-in. topping has cured, the beams are placed side by side onto the permanent substructure. The required reinforcement is then placed in the longitudinal and transverse closure areas, and the closure strips are then poured. After the strips are cured, the bridge is ready for traffic. The erected weight can be as much as 150 tons per beam line. Even so, production rates of two spans of beams per night have been achieved.

An additional seven bridges have since been designed. These last seven bridges use a modified pre-topped U-beam system. The modified system uses the same pre-topped U beams. The difference is that, in this modified system, the beams are transported from the fabricator directly to the site and erected onto the permanent substructure. The entire topping and beam joints together are then cast. The beams themselves provide a majority of the required formwork. Some steel plates are needed to span the gaps between the beams. Although this method may increase on-site construction time slightly, the quality of the riding surface can be better controlled. To reduce the on-site construction time, the slab and beam joint reinforcement may be pre-tied near the construction site, so that immediately after beam erection, the reinforcement can be placed. Welded wire fabric can also be used.

Figure A.6. Typical construction details.

ABC Opportunity

The proposed system offers a precast concrete beam option with span–depth ratio and total span capabilities typically reserved for shallow steel tub or I-beam bridges. The use of butted flanges and a pre-topped section minimizes the amount of field forming and thus expedites slab construction. Providing a completely cast-in-place topping provides the optimum ride quality and control over final deck geometry. Large-capacity cranes are able to erect several spans of girders in a single night given the simple procedure of span-by-span and butted construction.

Project Title: Blackhawk County Prefabricated Bridge Construction
Citation: Wipf, Klaiber, Phares, and Fagen, 2000
ABC Design Features:
ABC Construction Features: Rapid construction and erection of prefabricated bridges by county personnel

Project Description

A steel-beam precast unit bridge was developed through laboratory testing at Iowa State University. The steel-beam precast (S-BP) units consist of two steel beams connected by a reinforced-concrete deck of limited thickness. The thin deck reduces the weight of the units and makes it possible to construct the units off site and to transport them to the bridge site for rapid assembly. Once the S-BP units are connected, a cast-in-place reinforced-concrete topping is cast over the S-BP units, and the bridge railing is attached. This system can be economically used in bridges in the 30-ft to 80-ft range.

A demonstration bridge consisting of four S-BP units (each 7 ft, 6 in. wide and 65 ft long, weighing 23 tons) was constructed. The units were fabricated in the Black Hawk County Maintenance Facility in Waterloo, Iowa, after which they were transported to the bridge site and assembled. A cast-in-place reinforced-concrete deck was placed over the S-BP units to provide the required total deck thickness. The demonstration bridge was instrumented and load tested at various states of construction. On the basis of those tests, the S-BP unit bridge was shown to be a viable and economical alternative that can be rapidly constructed.

The same county completed an additional ABC project. The substructure portion of the project is highlighted. To expedite foundation construction, a cantilevered wall was built for the abutment and wings. Interlocking sheet piles were used for the wing walls. The abutment front face consists of vertical steel H piles with a concrete panel infill. Once the piles were driven, forms were attached, and the space between the piles was filled. It is possible to use precast wall panels with this concept as well, though tolerance of the pile driving must be controlled.

ABC Opportunity

A simple and cost-effective abutment and superstructure concept were developed and tested at Iowa State University and constructed by local agency forces for use in ABC projects. The project demonstrates that simple and durable systems can be deployed with limited equipment and resources and with construction times measured in days.

Project Title: Minnesota DOT/Poutre Dalle Inverted T-Slab System
Citation: Hagen, Ellis, Fishbein, Molnau, Wolhowe, and Dorgan, 2005; Bell, French, and Shield, 2006a and b
ABC Design Features: Development of an inverted T-beam precast deck system
ABC Construction Features: Rapid construction of short-span bridges with conventional equipment

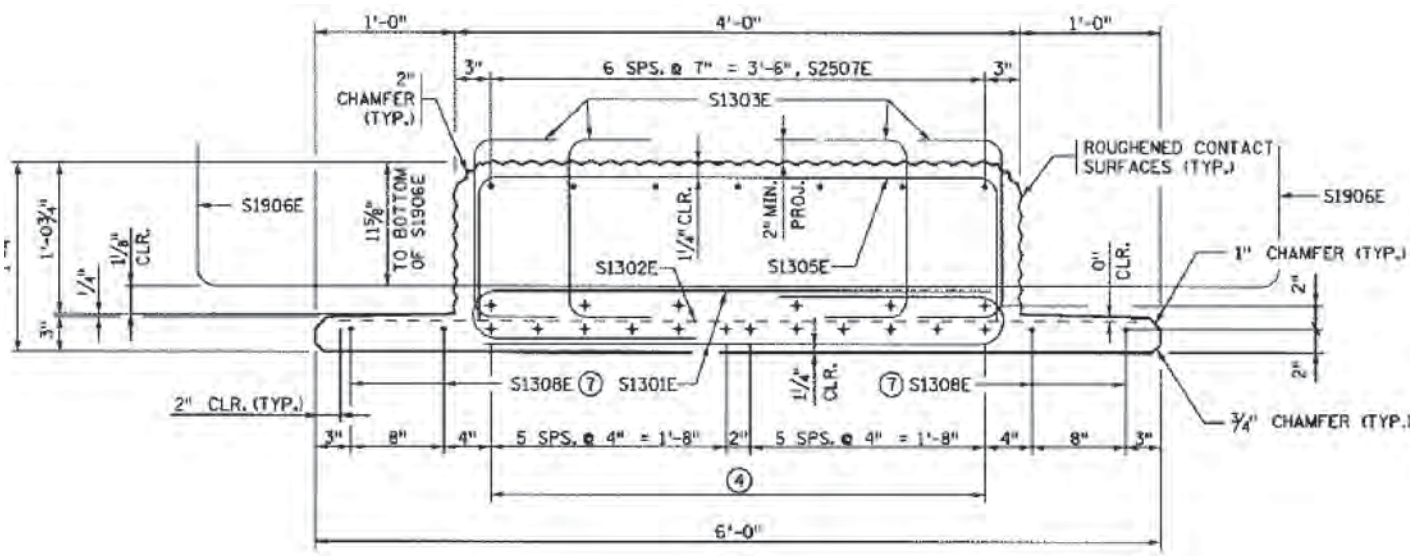
Project Description

One of the objectives of the 2004 prefabricated bridge elements (PFBE) scan tour was to identify technologies that could be used for rapid bridge construction in the United States. The Minnesota DOT sponsored research and a series of research projects to develop a local variation of the French Poutre Dalle system, a system of inverted precast slabs used for short- and medium-span bridge replacements. The specific application of the Minnesota DOT precast slab system (PSS) is to replace bridges that were commonly built as cast-in-place slabs requiring extensive falsework and long construction times.

The bridge uses a series of precast/pretensioned sections, with deep reinforced closure pours between sections and a cast topping providing composite action and the riding surface. The section was designed with input from fabricators and contractors. It makes use of existing stressing beds and forming systems for maximum economy. The section is designed with the AASHTO load and resistance factor design (LRFD) specifications and can be used for simple or continuous spans. Secondary effects from creep and shrinkage restraint were a significant part of the testing and analysis of the system. The system has a span–depth ratio of as much as 30 and can achieve maximum spans of 65 ft.

In the first of two demonstration projects, a three-span bridge with equal spans of 45 ft over the Tamarac River was reconstructed. The existing substructures were retained, and the bridge superstructure was replaced with PSS components with typical segment weights of approximately 25 tons. The beam costs for the project were \$43 per ft^2, and the total project cost for the staged reconstruction was \$83 per ft^2.

In a second project, a three-span bridge with spans of 22 ft, 27 ft, and 22 ft was completely replaced. Piece weights on the order of 10 tons were installed with a light crane. Total construction time was 20 days to construct abutments, pile bents, and the PSS units. The PSS units cost \$48 per ft^2, and the total bridge cost was \$157 per ft^2. This is much higher than traditional cast-in-place concrete slab projects, which in Minnesota range from \$80 to \$85 per ft^2.



Source: Minnesota DOT.

Figure A.7. Inverted T-slab installation.

Subsequent to the demonstration projects, the Minnesota DOT has continued to advertise projects that use the PSS system, and the cast-in-place prices have continued to decline. The latest data indicate total installed costs of \$95 to \$100 per ft^2, with other projects in the pipeline. The department continues to work with researchers and fabricators to economize the system further by reducing the amount of reinforcing and through wider use of the products.

Source: Minnesota DOT.

Figure A.8. The Minnesota DOT inverted T-slab bridge.

The Minnesota DOT has observed some longitudinal and transverse cracking on the deck surface on several of the PSS bridges and has begun an investigation to determine the cause. Several changes have been included in the latest generation of PSS designs and will be closely monitored after initial construction.

ABC Opportunity

The proposed system offers a rapidly constructed, shallow, mostly prefabricated solution for structures that conventionally would be constructed as cast-in-place slab systems. With light elements easily erected by local contractors and even by county or department personnel with access to light-duty equipment, the system provides another option to owners in the short- to medium-span precast bridge market.

Project Title: PCINE NEXT Beam Bridge
Citation: Precast/Prestressed Concrete Institute Northeast, 2008
ABC Design Features: Development of new double-T precast bridge superstructure modules
ABC Construction Features: Rapid construction of completed superstructure by using standard modules

Project Description

The Precast/Prestressed Concrete Institute Northeast regional association has made multiple advancements in the area of accelerated construction and in standardizing the practice among the New England states and New York State. The association's latest effort is to develop a short- and medium-span shallow replacement structure specifically for standard application and for use in accelerated bridge construction (PCINE, 2008). The use of precast double-Ts is not a new advancement. They have been popular bridge choices in many states throughout the country. The northeast extreme tee beam (NEXT beam) does offer some structural and construction advantages over the other systems, as will be described.

The proposed beam section is presented, and as indicated by standard notes, it comes in an 8-ft or 12-ft nominal width. When stacked side by side, the top flange is considered to be nonstructural; it simply replaces stay-in-place formwork. A full-depth cast-in-place concrete slab is then applied to create a fully composite section. The section is also designed for an additional 3-in. bituminous overlay. The standard 8-ft section has a span capability of approximately 85 ft, and the 12-ft section has a span capability of 75 ft. The section depth is 36 in. Charts and graphs are included to provide the span capability of each variation of the NEXT beam, listing section width, number of strands, and concrete strength. The sections do not need to be connected along the longitudinal joints with any welded connection as is common with other double-T systems, nor do they require transverse posttensioning—another cost and complication avoided with this refinement.

ABC Opportunity

This simple bridge shape—requiring a cast-in-place topping as the only field-cast concrete—accommodates a wide range of spans consistent with bridge sizes in the existing inventory. The sections are shallow (maximum depth of 36 in.) and are easily erected; the heaviest section weighs less than 60 tons and erected easily with commonly available cranes. These sections could just as easily be combined ahead of time (including the slab) and moved into place with one of various heavy move techniques.

Project Title: CBDG Concrete Modular Bridges
Citation: Benaim, 2006
ABC Design Features: Innovative design of box girder system for short- and medium-span bridge replacement
ABC Construction Features: Innovative box system built by crane, gantry, or incremental launching

Project Description

The Concrete Bridge Development Group (CBDG) is a concrete bridge industry association in the United Kingdom. The group sponsored development of standardized modular concrete bridges for the construction of routine overpass bridges as a marketing initiative to establish prestressed concrete as the preferred solution for spans up to 50 m (Benaim, 2006). The result of those efforts was development of a shell system that is precast off site and shipped to the project; permanent prestressing is applied and then the shell is filled, forming a solid section. Details are provided. The development and capital costs of new equipment can be capitalized over an estimated 5 to 10 bridges, indicating that an initial investment would likely be paid off within several years for any contractor with a steady stream of work.

What makes the system unique is, first, the lack of a top slab. That makes forming more cost-effective than traditional inner-void box girder forming. Additionally, unlike U.S. practice, even for external prestressing the entire box is filled with concrete as protection for the prestressing. That step adds substantial weight to the final section and limits its efficiency in some ways. However, it is included, at least in part, because of U.K. limitations and past problems with hollow box sections and external prestressing.

The advantages of the system include the use of light sections, adjustability in length and in width, usability for various bridge widths and lengths, and suitability for multiple erection methods including gantries, cranes, incremental launching, and full-span erection. Once the shells are erected, they are prestressed together, the infill concrete is placed, and the full span prestressing is applied. Shell depths of 1.0 m to 1.5 m can be used for spans up to 25 m; shell depths of 1.5 m to 2.0 m can span up to 35 m; and shell depths of 2.0 m to 3.0 m deep are required for spans of 35 m to 50 m. The depth–span ratio is 1:17 for a typical installation.

ABC Opportunity

The issue of standardizing designs is again proposed as an ABC solution for bridges of the routine overpass type. The CBDG system advocates the use of segmental shell systems to create bridges of various lengths and widths by simple variations in segment geometry. The bridge is designed to be erected by both conventional and innovative methods, mostly innovative due to the segmental nature of the structure. Like most segmental bridges, one method of erection involves the suspension/support by a truss/gantry until a span is complete. Unlike other segmental bridges, the shells are light enough to be posttensioned together on the ground and lifted like a single-span box girder into place, at which point the infill and final prestressing are applied. Other installation techniques include the use of incremental launching or SPMT installation.

Project Title: Flexi-Arch Construction
Citation: Macrete, 2008
ABC Design Features:
ABC Construction Features: Rapid installation of arch bridge structures for short and medium spans

Project Description

The construction of an innovative short- to medium-span bridge can be possible through the use of an innovative precast arch system. In a system developed in the United Kingdom, precast arch bridges are constructed by using an articulated block system that ships flat but rotates into the desired arch shape when lifted and placed against simple footings. Once the footings are in place, typical superstructure erection is completed in a day for a typical bridge.

Construction consists of flat stacking the various arch segments and shipping them to the site on a flatbed truck. This system has some advantages over shipping precast arch segments in that fewer trips are likely needed and the rise of the arch is no longer a shipping consideration.

Assembly of the bridge consists of several simple steps: placing the footings (which could be precast), erecting the adjacent Flexi-Arch segments, erecting spandrel walls, and typically filling the section with lightweight flowable fill. The lightweight flowable fill ties the arch sections together, ties the spandrel walls to the arch, and provides a nonsettling and waterproofed fill to the arch system. Additionally, given the reduced weight of the flowable fill, the arches can be designed for lesser load than if conventional granular backfill was used. Once the flowable fill is allowed to cure, the final riding surface (asphalt or concrete) can be applied.

ABC Opportunity

This system offers an alternative structural system for short- to medium-span bridges. It competes against other structural systems such as precast arches, large culvert sections, arched metal pipe systems, and short-span prestressed beams such as planks or voided slabs. It is inherently faster than all of those systems except the precast arch or culvert, which probably have similar construction durations and simplicity.

Project Title: FEHRL New Road Construction Concepts
Citation: Forum of European Highway Research Laboratories, 2008/Tanis, Nicolas, Cardin, Keller, Schaumann, Toutlemonde, and Godart, 2007
ABC Design Features:
ABC Construction Features: Development of multiple bridge replacement concepts that use high-performance materials

Project Description

The Forum of European National Highway Research Laboratories (FEHRL) is sponsoring a program analogous to the SHRP 2 initiatives to investigate changes in the highway design, construction, and maintenance practices that are likely to influence the profession for many decades. The challenge for European road owners is the same as in the United States: a multiplicity of road owners, slow innovation, lack of consistent design guidance, and incomplete cost-benefit ratio information. The New Road Construction Concepts (NR2C) program aims to address these deficiencies by investigating new concepts for the road of the future and specifically investigating the enabling technologies and concepts. A series of reports and working documents have been published describing the evolving process of bridge concept development in Europe. These reports are described here to reflect the progress of a similar research effort and the findings.

The *State of the Art Review—A New Vision for Bridges* (Tanis et al., 2007) provides an overview of the needs and expectations for future European bridges, describes various new structural materials that might be used in future bridge concepts, and provides a vision for what the bridge of the future might be.

Though not the same as the U.S. bridge population, European owners face a similar problem of aging infrastructure, deficiencies on ever-larger portions of the road network, and inefficient programs for rehabilitation and replacement. The expectation of owners moving forward is for bridges with robustness, long life, safety, economy, little maintenance, and some interest in improving construction methods. They are not particularly concerned about the ability to widen or lengthen a bridge in the future. That is understood to be a necessity of a future road improvement project, and the public is expected to understand the need for such changes.

Considering new materials, various applications for new materials are proposed. High-performance steel is advocated as a way of making composite girders (steel-and-concrete) more effective in low-clearance environments (shallower structures are possible), for use in moveable bridges, and for situations where lessening the structure's weight can result in savings in shipping, erection, or foundation costs. This can be useful in many bridge types and situations. Very-high-performance concrete and ultra-high-performance fiber-reinforced concrete (UHPFRC) are both discussed for their ability to provide high strength and durable solutions in future bridges. Among the uses of UHPFRC, in particular, are highly efficient beam shapes, reinforcing-free concrete decks, shallow or thin arch sections, box beam sections with perforated webs, and also in bridges where shock loading (blast, earthquake) are of concern. In all of these applications, the materials' high strength, easy workability, long-term durability, and reinforcing-free nature (except for internal fibers and prestressing) are the reasons the proposed innovation is possible or contemplated.

The use of fiber reinforced polymer (FRP) materials continues to receive attention as a material of the bridge of the future though we are well into our second decade of research and development of trying to use these materials in civil engineering applications. Because of their light weight, high strength, and long durability, these materials are proposed for various structural applications. In addition to use as external reinforcing and retrofitting (a relatively mature and established market already), their use is proposed for all-FRP bridges as well as in composite applications that blend other materials (e.g., UHPFRC) with FRP in a hybrid structure.

An interesting aspect of the NR2C project is its focus on timber for short-span bridges. Viewed positively as a renewable resource, timber has environmental attributes that, from a holistic project viewpoint, make it an attractive solution when deployed appropriately. Other materials such as aluminum and titanium are also discussed as having desirable properties.

The report also presents the prospect of enhancing existing structural concepts for use in future bridges. The greatest possibility for innovation seems to be in using new materials to provide new opportunities for economy and efficiency built on existing design concepts. The maximum opportunity is in the area of beams and slabs. For bridge slabs, the authors advocate the use of FRP reinforcing in lieu of steel reinforcing as the first step toward improving slab durability. As a second level of enhancement, they propose integrating an FRP formwork system that serves to reinforce the bridge deck at the same time, thus providing two functions in one construction operation. Steel-free slabs of UHPFRC are also suggested as a concept for the future. With no reinforcing in the slab, the structural strength derives from the strength of the fiber reinforced concrete alone. Intermittent tension ties are required between beams to anchor the shallow arching behavior that forms at failure. A commercially available precast product known as ArchPanel is also recommended for use in steel-free decks.

For new beam concepts with innovative materials, several FRP superstructure concepts are proposed. In the hybrid tube system, an FRP box girder is constructed. At the bearing locations, the full depth of the box is filled with concrete (for strength, stiffness, and stability) while otherwise remaining hollow. A concrete deck is cast on top. The concrete-filled carbon-filled shell system was a spin-off of filament-wound pipes used for noncivil applications or for the fabrication of shells for column jacketing. Once filled with concrete, the shell is restrained from buckling and the strength of the confined concrete is enhanced. Given the challenges of using round sections for bridge girders, advancements have focused on using concrete-filled square and rectangular sections instead.

The NR2C bridge report also focuses on new bridge concepts. It states the following:

> Developing a completely new concept is the most difficult way to innovate, but also the most challenging and rewarding of the methods discussed. Therefore, combining existing concepts may be useful for coming up with unusual structural solutions. The construction process may become more complicated and thus more expensive. Aesthetics are of prime importance in bridge design and a mixture of structural elements will not necessarily satisfy the observer, since the structural simplicity and visual attractiveness of the bridge may be interfered with.

In its study of new bridge concepts, even these are not so new. The recommended new bridge types to be studied in the future include extradosed structures (a hybrid between post-tensioned internal tendons and a cable-stayed bridge); fin-back bridges, which employ a solid concrete variable depth fin or rib extending above the deck in negative moment regions; and the stress ribbon bridge, possibly one of the oldest bridge types in existence, using draped tensile elements and decking to span two supports.

NR2C's general perspective on innovation is that it is expected to be a slow and continuing process of sustained development. Rarely do remarkable leaps of technology appear in the market (prestressed concrete is cited as one recent example of a dramatic change in materials technology). The researchers comment that radical changes in technology and capability (using bridge spanning capability as an example) have only come about in the presence of new materials. Materials drive innovation. New concepts must continue to meet current functional, safety, schedule, and cost demands. Future innovation should start with a better understanding of existing bridges and existing innovative technologies. Those include making use of better surveying tools, materials testing procedures, management systems, structural monitoring, and innovative repair techniques. Once these existing enhancements are mastered and common in practice, the industry can look to future concepts. The greatest opportunity for innovation is perceived to be in the area of "ordinary" bridges.

Following the preliminary study, the NR2C team went about developing several new bridge concepts for the short- to medium-span traditional bridge market. Several solutions are proposed for a 10-m-span bridge and for a 25-m-span bridge, using various materials.

The first solution proposed using glulam timber beams with a combination of high-strength materials for the top and bottom flanges, UHPFRC for the top slab, and FRP for the bottom flange. The section is hollow. In the simple-span condition, the FRP bottom flange will always be in tension. The UHPFRC slab will be used to increase the bending strength of the section and to form a durable wearing surface. In a second scenario, the glulam timbers are replaced, and the FRP and UHPFRC plates separated by a low-strength material, with foam as the spacer. Available in standard widths and easily cut to size, this solution can be easily customized for various depths, and lengths. For both of these solutions, the proposed module width is 1.25 m and the height is expected to be about 0.8 m for the total depth. A third solution that uses both timber webs and foam infill was also proposed but not shown because it simply married the first two concepts. The presence of the internal foam stiffens the system and also eliminates the need for any internal diaphragms between the vertical glulam beams.

Testing of these three systems indicated that the best solution is the first approach. In testing the second, it was found that the system had much greater deflection due to the low stiffness of the foam infill. That resulted in high tensile stresses in the slab due to local longitudinal shearing distortion of the foam in high-shear regions. The authors compared the total weight of the first solution with other types of bridges that could be used for a 10-m span. Comparisons were made between a conventional reinforced-concrete slab, a composite-steel bridge, prefabricated concrete girders, and Solution 1. The usual weights of these structures are as follows: concrete slab, 125 kN/m; prefabricated girders and composite girders, 90 kN/m; Solution 1, 31 kN/m. Thus, weight savings are substantial.

A UHPFRC waffle slab system was also studied. The system consists of 0.6-m by 0.6-m panels, with a top slab 50 mm thick. The system has a total depth of 475 mm, the longitudinal ribs being somewhat deeper than the transverse ribs. The system is prestressed in the longitudinal and transverse directions for strength and stiffness and to join the segments in the field. When shipped full width, with transverse joints, each section is the width of the bridge by 2.5 m long. When shipped full length with longitudinal joints, segments 1.8 m or 2.3 m wide are available. The volume of material in the section is equivalent to assuming a uniform slab thickness of 164 mm for the entire width and length. A traditional concrete slab bridge would have a thickness of 500 mm for this same span. The weight is thus about one-third of a traditional slab bridge. One nuance of this design may change the results for U.S. application: The service limit state deflection allowed for this system is L/350. Traditional U.S. practice has limited live-load deflections to L/800 or L/1,000, depending on whether the bridge is designed for vehicular loads only or also carries sidewalks.

Finally, the authors provide solutions for a 25-m span. The first system, not depicted, is a waffle slab concept similar to those described above but made deeper for the additional span length. This solution increases the rib spacing to 1 m on center both ways, and the top slab is thus thickened to 80 mm. The total structural depth is 1.05 m for a span–depth ratio of about 25. A standard module is full length, and the sections are 2 m wide. They are prestressed longitudinally, with external tendons passing through deviator diaphragms and transversely adjacent to the ribs. The resulting bridge has an equivalent quantity of material analogous to a 230-mm-thick slab, a dramatic reduction in materials from that used in a conventional bridge.

Other more exotic solutions are also proposed, including a notched slab system. In the notched slab system, used for a 25-m span, portions of the longitudinal ribs are removed to save weight. In additional modifications, the longitudinal posttensioning is eliminated, and additional tensile capacity is provided to the system by bonding FRP plates to the bottom of the notched ribs. The plates measure a total of 50 mm thick. This system is about 50% heavier than the traditional waffle slab, mainly because it requires much wider longitudinal webs to bond with the FRP plates. The high volume of FRP added to this system brings into question whether this is truly an innovation above and beyond the traditional post-tensioned slab system.

A hybrid deck system is also proposed, meeting the objectives of combining a stay-in-place forming system with the advantages of FRP reinforcing. The total slab depth of 200 mm matches the conventional 8-in. bridge decks in use for common beam spacings. However, the comparison ends there. The proposed system uses an FRP grid to span in the transverse direction and provide forming and strength to the system. Lightweight concrete is used for a majority of the deck depth to reduce weight, and a normal-weight UHPFRC topping is added. Schaumann and Keller (2007) provide experimental test results of short- and long-span specimens fabricated as indicated. Their results indicate that full composite action between the grid and concrete is possible. Different types of lightweight concretes were studied as fill. Only simple-span conditions were studied, so the behavior of this system in negative moment regions (such as over beam lines) is untested.

ABC Opportunity

Many possible innovations are discussed. The basic waffle slab system appears to have the most immediate promise. In the 10-m-span configuration, small bridges are easily replaced with a very lightweight system that is quickly erected. The team also envisions the 10-m-span waffle slab system as a potential concept for a deck system with widely spaced girders. Traditional concrete slabs become excessively thick and heavy with wide beam spacing (e.g., two girder systems). The waffle slab can serve as an effective spanning element in that scenario. Another option for the thin waffle slab is to use it to span through-girder edge beams, a possible concept for future bridges in low-clearance environments. The deeper waffle slab for the 25-m-span condition is the same or lesser depth than bridges used today for such a span, yet is much lighter and easier to install. Finally, the concept of stay-in-place FRP decking that also reinforces the tension face of the slab is a time and money saving opportunity.

The performance of this deck in negative bending requires further validation.

Bridge Deck Concepts

This section focuses primarily on components that can be used to accelerate bridge deck construction. By far the most widely available information for ABC is related to precast bridge decks, and the information dates back to the 1970s and earlier. The main focus is on concrete because, in the short term, that will likely continue to be the predominant material used for bridge deck construction. This expectation reflects concrete's long-standing position as the "familiar" material, its ready supply, ability to be widened and repaired when needed, and other characteristics. A variety of other deck types exist. These include glued-laminated and stress-laminated timber decks; open-grid steel decking; partially filled systems such as half-filled grid decks and exodermic bridge decks; and lastly, various forms of fiber reinforced polymer (FRP) bridge decks. FRP decks deserve special, but brief, mention because of FHWA's level of research investment in the recent past and because many state DOTs surveyed during this project consider them still to be in a demonstration phase of implementation.

More than a decade of published history exists concerning FRP bridge deck systems, and hundreds of installations in the United States and around the world have been performed (many of the installations were done with demonstration project funding). In spite of the generally good performance of these bridge decks (save several overlay failures), the number of commercial manufacturers is down to one or two firms in the United States, and fabricated prices continue to hover in the range of \$75 to \$100 per ft^2, a high cost for a deck system (higher in fact than some states' total in-place bridge costs on a square-foot basis). The promise of FRP is its light weight and long life. From an innovation standpoint, FRP bridge decks may still hold promise over the long term as a viable deck system for rapid construction. However, recent innovations in concrete decks may begin to chip away at some of the inherent advantages of FRP.

Several useful sources of information related to FRP deck construction are available on FHWA websites, including lists of completed FRP deck projects (FHWA, 2009b) and a library containing pertinent FRP-related publications (FHWA, 2009c). Another noteworthy reference describing innovative and prefabricated bridge deck types for minimizing traffic disruptions during construction is the National Cooperative Highway Research Program (NCHRP) Synthesis study (Shahawy, 2003). In addition, Bakis et al. (2002) is a state-of-the-art review that contains useful bridge deck information. From an international standpoint, FRP bridge deck applications began in the early 1980s and continue to be constructed. Two recent studies of FRP bridge decks implemented in Asia (Prachasaree and Shekar, 2008; Kim et al., 2009) validate the continued interest in better understanding the potential application of FRP decks.

Several representative summaries related to both partial- and full-depth deck systems are presented below.

A partial-depth prefabricated deck is a deck that combines a layer of precast concrete and a layer of cast-in-place concrete. This system can be developed with either steel or concrete girders. The deck is connected to the steel I-beam through shear studs on the beam. The deck is placed in the field in panel segments; once all of the segments are placed, a cast-in-place concrete deck is poured on the precast deck. The precast deck acts as a stay-in-place form, eliminating the need for temporary formwork. This system can also be installed by using a precast girder and partial deck system. In that case, concrete girders are used instead of steel I-beams. The installation procedure is identical regardless of the superstructure material (Russell et al., 2005).

The inverted-tee system is another partial-depth prefabricated deck developed to accelerate the construction of bridges. This system is designed to accommodate longer span lengths. The inverted-tee portion of the deck is precast and placed at the construction site. A cast-in-place concrete deck is constructed later. Researchers have shown construction of this system to be faster and less expensive than conventional methods of construction. The inverted-tee deck system can be customized for different bridge projects and used in spans up to 26 m (Kamel and Derrick, 1997).

A full-depth prefabricated deck, which has a few features that make it unlike other deck systems, is one that includes a completely precast concrete deck. Deck panels are designed with shear keys that are filled with a high-strength grout for a strong connection between panels. Composite action between the deck and the girders is created by using shear studs or high-strength bolts with steel beams and precast (PC) beams. Posttensioning strands are generally used in the longitudinal direction to keep the deck panels in compression. Full-depth prefabricated decks are effective because they can be installed quickly and constructed in stages to allow traffic to flow during the process (Issa, Yousif, and Issa, 1995).

Durability, ease of construction, and reduced maintenance are only a few of the advantages of using a precast deck system for the rehabilitation of deteriorated bridge decks. The systems can be adapted to any bridge, even those with horizontal and vertical curves. Segmental bridges were first constructed in Europe in the late 1940s as a response to the demolition of many bridges during World War II and the need to replace those bridges and decks quickly. Segmental construction came to the United States in the early 1970s and has improved dramatically since then. One application that has gained popularity is the precast deck system. Within this category are two types: stay-in-place precast deck panels and integrally constructed full-depth decks. The stay-in-place panels are used as a working platform for workers and machinery and as the forms for the final cast-in-place deck pour. The full-depth precast concrete decks are just that, no cast-in-place concrete is needed to finish the deck. With these two deck applications, as with other precast concrete applications, careful attention must be paid to the joints between each panel, as rapid deterioration may occur.

Biswas (1986) provides more than 20 case studies of applications of precast bridge deck systems. Each case study emphasizes the system construction and the economy of using precast concrete. The case studies also include detailed descriptions and drawings of joints and joint materials used. The author discusses the advantages and difficulties during the design and construction of the precast deck system in each of the case studies.

Case studies of concrete deck options are highlighted here along with an innovative steel plate deck system.

Project Title: Lake Pomme de Terre Precast Deck Replacement
Citation: Desai and Blakemore, 2008; Wenzlick, 2005
ABC Design Features:
ABC Construction Features: Precast full-depth deck panels, match cast adjacent to the site, night work

Project Description

The Missouri DOT bridge carrying Route 64 over Lake Pomme de Terre is a 1,684-ft-long steel stringer bridge that, because of deck deterioration and a desire for a slightly wider deck, was reconstructed with full-depth deck panels. With one lane of traffic in each direction, half-width construction with traffic signals at each end to control the flow was not feasible due to long cycle times and large queues at each end. Full closure was also not an option because of prohibitive detour lengths. The only viable option was nighttime closures. This decision dictated the use of prefabricated deck panels. A typical panel is shown in the figure.

The contractor was able to arrange for a strip of local farmland to be used for an at-grade long line, match casting bed for the fabrication of the deck panels. Thus, all construction was completed near the final bridge location and the bed could be graded and panels match cast for proper fit in the field.

In the first night of construction, the contractor was able to demolish several panels of existing bridge deck. Time was not sufficient to install the new concrete panels, so a set of steel, open-grid deck panels (already prepared) was used to span the gap. In subsequent nights, the steel panels were removed, several more sections of existing deck were removed, precast panels were placed and grouted, and the steel panels set again to cover the remaining gaps. This process occurred every evening from 7:00 p.m. to 7:00 a.m., with a daily production rate of several 10-ft-long panels. Steel posts were attached to the temporary and permanent deck panels for a temporary railing to be attached as construction progressed. At the completion of the project, the temporary steel rails were removed, the permanent concrete railings were cast, and the deck was overlaid. Placing and curing the overlay required two 1-week patterns where only a single lane was in operation.

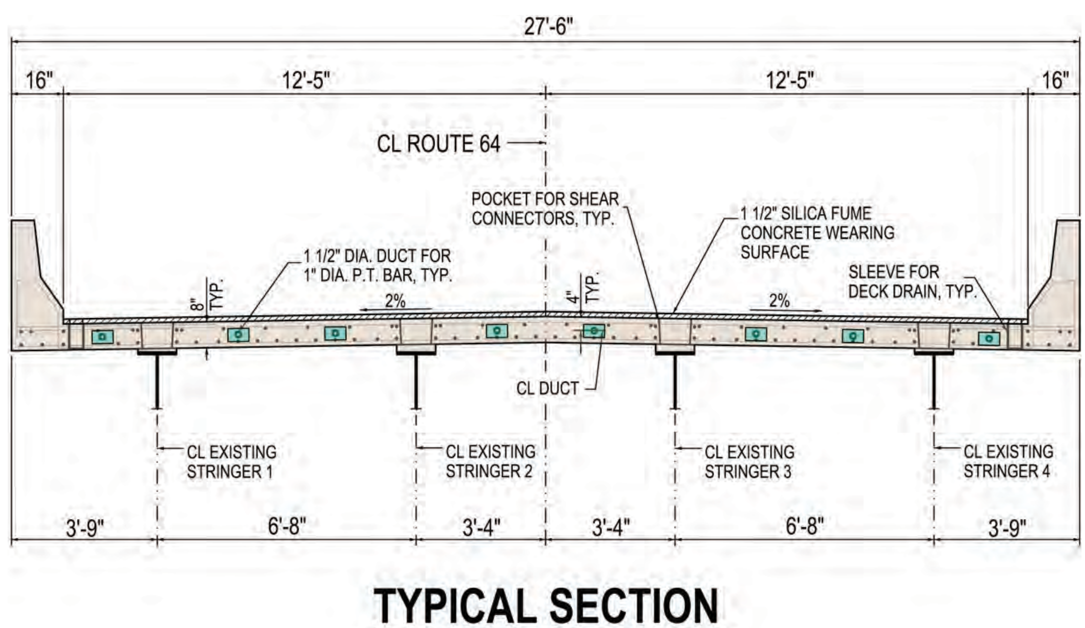


Figure A.9. Lake Pomme de Terre bridge deck panels, typical section.

Figure A.10. Deck panel fabrication, installation, and haunch-forming method.

An innovative screw jack system was used to level the panels and set the haunch. With clamps that would hold the bottom flange of the steel stringers and mating steel tubes for the vertical with a threaded rod screw jack installed in the center, the panel heights could be easily adjusted in the field. That alleviated any complications arising from final girder geometry once the existing deck was removed. The screw jacks raised and lowered steel angles that supported the panels and also provided the haunch forming.

The cost of the deck replacement portion of the project was about \$56 per ft^2; the Missouri DOT's statewide average for deck replacements is generally in the range of \$40 per ft^2. The agency estimates that for this bridge—a long bridge over water—the statewide average would not be a realistic estimate anyway. According to Wenzlick (2005), the costs of a cast-in-place solution would be close to the precast solution but would have required an additional construction season, would have required half-width operations at all times, and would have incurred substantial additional traffic control costs.

ABC Opportunity

The challenges of bridge deck replacement on long, narrow structures is particularly troublesome. Replacing a short bridge, by using a complete closure, might take only a week of construction to remove an existing deck and install new deck panels. That might be tolerable. Alternately, for wider bridges, staging might be undesirable but tolerable and at least possible. For this bridge, neither closure nor staging was realistic. An innovative approach that uses match cast precast deck panels, infill open-grid steel deck panels, and nighttime closures effectively demonstrated a low-impact reconstruction project completed with conventional forces and equipment. An innovative haunch forming/panel leveling system is presented.

Project Title: Rapid Precast Deck Construction
Citation: Ronald and Theobald, 2008
ABC Design Features:
ABC Construction Features: Innovative construction method to place precast deck panels

Project Description

Ronald and Theobald (2008) describe the development of a new semiautomated method for placing full-depth deck panels. By using a series of small carts placed between the bottom flanges of prestressed concrete girders, full-depth precast deck panels are incrementally placed and launched down the length of a span.

Beginning at the supply end of the span, the first set of carts—carrying the first deck panel—is placed between the beams. The carts are winched toward the far end, and the next panel is placed in the same way. The second set is tied to the first so that the train of panels is moved forward incrementally. The carts can be equipped with jacking plates so that screw jacks installed in the panels have a surface to bear on while the panel grade is adjusted. By using a portable computer control system, jacks can be adjusted individually or in a group to set the proper geometry. Posttensioning is completed beforehand, then panels are lowered to reduce any frictional effects between the panels and girder tops.

ABC Opportunity

For long viaduct-type structures in particular, the system has various advantages. Most important, it allows cranes to continue to be dedicated to superstructure beam erection and freed from erecting deck panels. The latter can be done separately with the mobile cart system. However, even for long spans where a long boom or heavy crane would be required to place heavy panels at the far end, this cart system can greatly simplify deck panel placement.

Project Title: UHPFRC Bridge Deck Studies
Citation: Garcia, 2007; Toutlemonde et al., 2005, 2007a and b
ABC Design Features:
ABC Construction Features: Development of lightweight UHPFRC deck panels for composite bridge construction

Project Description

Ultra-high-performance fiber-reinforced concrete (UHPFRC) bridge decks appear to offer promise in many areas, including replacement of traditional bridge decks in multi-beam bridges and as an interesting long-span slab alternative used in two-girder systems (in lieu of more-traditional orthotropic decks). In deeper configurations the material may be used in "slab bridge" types of applications. The use of UHPFRC as deck slab material for girder bridges has been investigated and has potential to radically change concrete bridge deck design in the future.

Beginning with a discussion on UHPFRC bridge decks for traditional deck slab replacement, Garcia (2007) describes the design of a precast deck system for traditional multi-girder bridge construction. The system is depicted in plan and typical section. The slab itself is 2.5 in. deep and is bounded by ribs of the waffle that are an additional 5.5 in. deep, for the total depth of 8 in. The ribs are spaced 24 in. on center. The weight of this system is approximately 55 psf, compared with 100 psf for a traditional 8-in.-thick solid slab. This has significant implications for reducing structural weight (and thus beam section requirements) as well as foundation loadings and structural mass (for reduced-seismic demand). Additionally, for long spans, as dead loads become dominant, the system provides the opportunity to use lighter beams, wider spacings, or combinations of these to gain extra efficiencies.

Garcia compares the design and resulting product with a series of examples produced by Michael Baker and Modjeski and Masters as part of LFRD design examples and transition design examples. Both of these consultants use a traditional I-beam bridge with conventional beam spacing as the basis of their sample designs. Garcia includes the prior designs to provide a baseline for the predicted strength of conventional bridge decks. The UHPFRC system should be designed with these capacities as a minimum requirement.

Because of the unique strength and stress–strain characteristics of UHPFRC prestressed concrete, the traditional AASHTO strength models cannot be used directly. An alternate method is shown, using strain compatibility to establish the equilibrium conditions and resulting strength. The fundamental difference between the prestressed UHPFRC and conventional prestressed concrete is the concrete tensile strength contribution to the force balance equation; this contribution is negligible for normal concrete and is traditionally

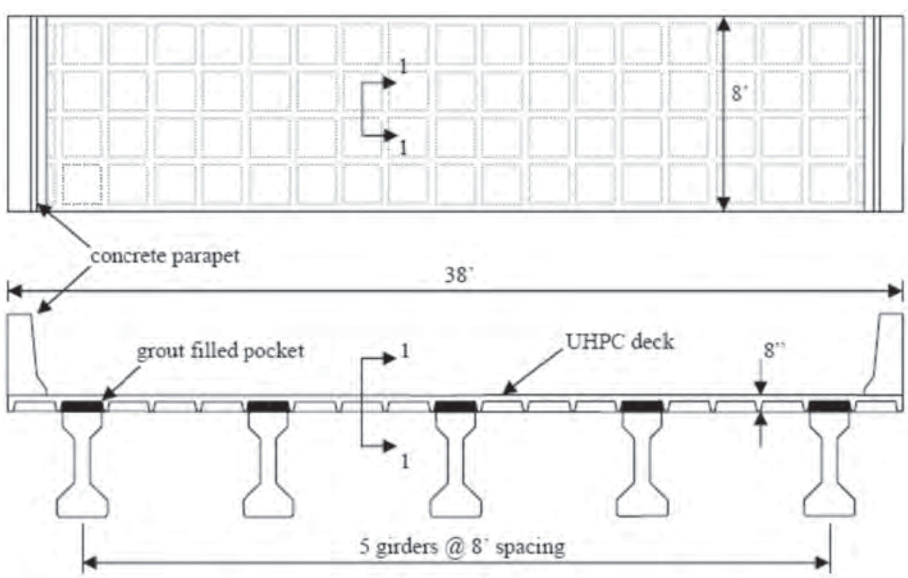


Figure A.11. UHPFRC slab-on-girder concept.

neglected. The sustained tensile strength of the UHPFRC assumed for this design was 1.125 ksi, a significant strength. Compression strength is 23.8 ksi.

Design of the deck was computed in accordance with AASHTO LRFD specifications for vertical loading and collision loading effects on the cantilever slab. In the transverse direction, T-beam behavior was assumed based on the regularly spaced ribs in the waffle slab (24-in. centers). Results of the design development exercise, which is completely detailed and instructive for future designers using UHPFRC, indicate that the prestressed UHPFRC system has somewhat higher strength than conventionally reinforced concrete decks. This is considered prudent until additional validation is performed. The author recommends that additional physical testing be conducted to validate the expected performance.

Waffle slab systems have been tested in Europe, and the use of waffle slabs for slab on girder construction is promoted as a viable new technology for bridge systems.

In a study by Toutlemonde et al., the flexural and punching shear capacity of UHPFRC slabs constructed with different commercially available cements is assessed. Both Ductal and Ceracem cements were used in the study. The motivation for the deck testing was the development of a two-girder bridge concept that uses a waffle slab for long-span bridge applications as proposed in recent French research. The typical section is provided along with section details and several photographs of the section during fabrication. Though not shown, the prototype bridge has two 11.5-ft-wide lanes, two 3-ft shoulders, and two 3-ft-wide sidewalks. The main girder spacing is 19.7 ft, and the prototype bridge has spans of 295 ft, 426 ft, and 295 ft. In this span range, weight reduction has dramatic benefits in overall economy. This slab system weighs roughly 80 psf. As a frame of reference, a conventional 8-in. slab, used for beam spacings up to 10 ft (half of the provided spacing) weighs 20 psf more. This is a highly efficient system from a weight standpoint.

The study describes various details of the testing. The objective was to determine the flexural strength and punching shear strength. Load was applied to the slab through contact patches set to simulate the Eurocode design loading of 16 in.^{2} on top of an overlaid slab; the effective bearing area is 2 ft^2 at the interface of the overlay and deck. Other tests were conducted on a slab without overlay (direct loading of the slab) and with a reduced contact areas as small as 8 in. by 10 in.

Only in the deck tests with direct loading on the base deck was failure achieved. Before the punching shear tests, the waffle slabs had already been subjected to substantial static flexural and fatigue load testing. Those tests were not able to degrade the performance of the slab in any discernable way. The punching shear tests on the reduced contact area had the worst performance factor: 2.5 times the level required by the Eurocode for design. When more realistic contact areas and strength along the four-sided shear perimeter are considered, the load factors are 4.6 to 6.0 times that required by the Eurocode.

As mentioned, the punching shear tests were conducted on specimens with prior fatigue testing. These slabs were subjected to about 20 weeks of constant cycling in the lab under loads simulating the Eurocode axle spacing but with magnitudes that for some cycles greatly exceeded normal fatigue-loading magnitudes. The initial loading consisted of 10,000 cycles of 2-kip to 34-kip wheel loads per axle. Those are described as very high and represent the extremes of loads at the service limit state. An additional 2 million cycles were applied with 2-kip to 22-kip loads; that represents a damage equivalence of 100 years of routine fatigue loading. That test was followed by 100,000 cycles of 27-kip and 34-kip axle loads. The deflection and strain behavior was stable throughout this testing.

The researchers examined the fatigue behavior of the system as well as the local performance of the slab and ribs in the most heavily loaded waffle cells. They concluded that no fatigue damage is expected in the slab with unlimited applications of 22-kip wheel loads and periodic overloading with

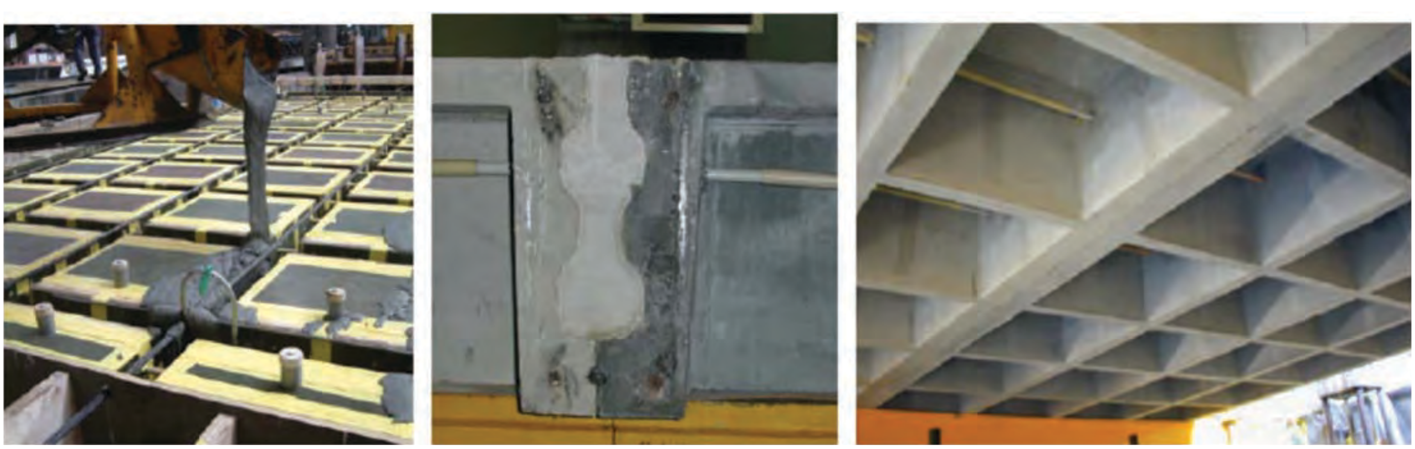


Figure A.12. Fabrication details and completed waffle slab.

34-kip wheel loads. The results of the testing indicate that the safety factor for fatigue with 100 years of standard loads is about 1.25 and for elevated loads is about 1.15. Thus, the system is adequate for fatigue for a long design life.

ABC Opportunity

Several research projects demonstrate the viability of UHPFRC bridge decks. With this new material, significant weight savings and a promise of long-term durability are shown to be possible because of the excellent environmental performance of the UHPFRC material. The long spanning capability of the material opens the possibility of wide girder spacing (including the viability of two-girder systems), lighter beams, longer spans, etc., as byproducts of the weight savings. These decks can easily be made composite with steel or concrete beams simply by filling one of the cells over the beam lines.

Project Title: FRP Grid Reinforced Concrete Decks
Citation: Bank et al., 2006; Matta et al., 2006; Matta et al., 2008
ABC Design Features: Design of a steel-free concrete bridge deck
ABC Construction Features: Rapid construction through use of FRP grids as internal reinforcing

Project Description

Multiple research projects focus on the use of bidirectional, double-mat FRP grids to replace the internal reinforcing of reinforced concrete deck slabs. The work of two such projects is described here. The first project, using bidirectional grids coupled with a stay-in-place forming system, is described in two articles: Matta et al. (2006) is a summary article highlighting the construction process; Matta et al. (2008) is the full research report containing recommended construction specifications and detailed design calculations.

The FRP grid consists of I-beam-shaped bars running in the direction perpendicular to traffic (for main bending strength), while the I-sections are drilled and round rods are passed through in the longitudinal direction. Spacers are used to keep the mats 100 mm apart. Attached to the bottom mat of the grid is a continuous FRP faceplate that also serves as the stay-in-place forming system. A four-span bridge in Green County, Missouri, with a total length of 144 ft and a width of 24 ft, was decked by using this system of prefabricated panels. The panels are 24 ft wide and 8 ft long yet weigh only 900 lb—or less than 5 psf for a system that includes the formwork and all required reinforcing.

The bridge deck construction time for a typical cast-in-place operation is estimated as 3 weeks for this structure; that includes the time required to place forms, place and tie the four mats of steel rebar, and pour the deck. The Greene County project was completed in 5 days. Placement of all 18 FRP panels was completed in 1 day. On Day 2, prefabricated FRP railing cages were placed and tied, and the finishing machine was set up. Deck casting was done on Day 3, and the railings were formed and cast on Days 4 and 5. The cost estimate for this system, in-place, is about \$45 per ft^2, including \$26 per ft^2 for the fabrication and delivery of the prototype FRP panels.

Similar research was conducted by Bank et al. (2006) for a project in Wisconsin. This project also included laboratory testing and field validation of the concept. A bridge similar in length to the Greene County project was constructed with the bidirectional grid system. However, the De Neveu Creek project in Wisconsin was a much wider structure. The total deck size for the De Neveu Creek project was 130 ft by 45 ft.

For the De Neveu project, the deck panels were again fabricated in the full width of the bridge and in panels 8 ft long. Construction involved placement of all 18 deck panels in a single day. Panels were typically lifted to the deck in stacks of four and then maneuvered manually by four workers. Time for placement of a panel averaged 11 min per piece. The bridge was built next to a companion bridge that was also redecked but with conventional steel-reinforced deck. The data indicate the following:

- Construction time: FRP—111 work-hours, steel deck—239 work-hours;
- Equipment hours: FRP—21 hours, steel—32 hours;
- Bridge deck cost: FRP—\$31.33 per ft^2, steel—\$11.54 per ft^2; and
- Reinforcing cost: FRP—\$22.50 per ft^2, steel—\$3.05 per sq ft^2.

The authors comment that the bid cost is not a fair reflection of the true costs of the system. The contractor's bid is believed to have substantially overestimated the construction difficulty and did not reflect the rapid installation of the grids. Also, a substantial part of the grid cost was in developing a spacer system to separate the decks in this prototype. The connectors were a small material component but comprised 30% of the installed grid costs. That cost will come down for future projects with improved details and given that the R&D costs have already been incurred.

ABC Opportunity

Both of these projects demonstrate a rapid method of reconstructing bridge decks in the field with schedule savings of two-thirds compared with a traditional project. Although costs appear high, the same is true of all demonstration projects, and capitalization of some of those costs plus familiarity in the construction community will bring the price down. Whether this grid system will equal steel-reinforced decks in cost is unknown. However, it involves a simpler and much faster installation and provides a higher degree of durability. Most likely, these grid systems could also be used as internal reinforcing for precast deck panels—if such a system were desired. That would likely be the most rapid method of installation but would involve lifting much heavier pieces as well. A combination of FRP grids, precast panels, and high-performance concrete (or regular concrete with HPC/UHPFRC overlays) would result in a dramatic advancement in bridge deck technology yet not require any real changes in construction practices or equipment.

Project Title: FRP Reinforced Concrete UHPFRC Decks
Citation: Perry, Scalzo, and Weiss, 2007
ABC Design Features: Design of an FRP reinforced concrete bridge deck with UHPFRC closure pours
ABC Construction Features: Rapid construction through use of FRP grids as internal reinforcing

Project Description

In an innovative use of ABC systems (precast decks) and new materials (UHPFRC and FRP rebar), Perry et al. (2007) describe the staged reconstruction of a bridge deck carrying Ontario Highway 11 over the Canadian National Railway at Rainy Lake, Ontario. The existing bridge, roughly 24 m long and 11 m wide; construction was staged in half widths. To accelerate construction, precast full-depth concrete panels were selected for this project.

Several innovations were implemented for this project. First, the use of precast full-depth deck panels, though growing in popularity, is still considered an innovative method of construction. Second, because of concerns about durability in the deck panels, the owner, Ontario Ministry of Transportation (MTO), chose to provide a top mat reinforced with bidirectional glass fiber reinforced polymer (GFRP) reinforcing bars; the bottom mat was reinforced with steel reinforcing bars. The panels were constructed of conventional 35-MPa concrete (7 ksi). An additional feature, which will be described in greater detail, is the use of UHPFRC materials for the shear stud blockouts, transverse joints, and new longitudinal joint configuration.

The nuances of the innovative details are best understood through a comparison with traditional details. The Utah DOT recently prepared standard precast deck panel plans, and those plans represent current thinking on the design and detailing of closure pour details. The closure pour must provide flexural and shear continuity of the deck. The issue is the required development length of deck reinforcing, traditionally epoxy reinforced, 100% of it spliced at a single plane, and typically designed to near its maximum capacity. That combination of factors causes long lap slice lengths; and that requires a correspondingly large closure pour in the field.

A similar situation exists for the transverse joints between panels. Except for single-span bridges where states have allowed simple unreinforced female-to-female joints to exist between panels, development of continuous reinforcing steel in these joints has also required wide joints to lap-splice the longitudinal reinforcement. This is particularly critical for continuous structures in negative bending, although prestressed concrete structures under long-term creep and shrinkage camber growth can place a deck slab in tension even in simple-span conditions. At its best, the traditional system requires the use of female-to-female joints without reinforcement (simple spans only) or, more likely, with wide transverse joints (conventionally reinforced) or tight joints with the need for longitudinal posttensioning to control cracking and provide durability.

The work of the NCHRP 12-65 project, resulting in the publication of NCHRP Report 584 (Badie and Tadros, 2008), addresses some of these details with new "non-prestressed" continuity details between panels, which are quite elaborate. The Rainy Lake project takes advantage of the unique material qualities of the GFRP rebar and UHPFRC joint to create a simple detail for both the traditional transverse joints between panels and the sometimes required longitudinal closure pour for staged construction.

On the combined basis of Canadian and Swedish research, it was determined that the full strength of the reinforcing bars could be developed with as little as 190 mm (about 8 in.) embedded in the UHPFRC material used for these joints. The joint material, Ductal made by LaFarge, has a 48-hour compressive strength of 100 MPa and 28-day strength of 140 MPa. Equally impressive is the flexural strength of 30 MPa. This material is highly durable and has equally impressive permeability and carbonation characteristics. Thus, it is well suited for critical applications such as closure pours, long known to be a weak link in structural performance.

The longitudinal joint was placed at a critical stress location, a maximum negative moment region on top of a beam flange. Yet the small size of the joint is large enough to fully develop the strength of the lapped connection, and the flexural strength of the material is far greater than any traditional reinforced concrete that would be used in a similar application. Use of the top-mat GFRP rebar was simply another level of corrosion protection and long-term durability desired by the owner. It does not appear to be a fundamental requirement of the system, though pull-out strength tests of steel bars would need to be conducted to substantiate any changes in the joint design.

Construction of the deck was otherwise similar to other precast concrete full-depth deck panel projects. The panels were leveled, and the Evazote-filled joints were tested for watertightness. Following that inspection, the joints were grouted, shear pockets were filled, and the haunch constructed—all with the Ductal material. The material was completely batched on site with a 0.2-m^3 portable mixer and delivered to the various locations along the bridge with a small powered buggy. The entire operation took approximately 8 hr to complete. The deck was left to cure for 48 hr, any high spots were removed by grinding, and the bridge traffic moved to the deck for completion of the other half of the bridge.

ABC Opportunity

The opportunity to use this system of deck construction in ABC applications is fairly obvious. The system builds incrementally on past successful projects and research in the area of full-depth deck panel design and construction. In traditional applications, full-depth deck panels either were provided with large cast-in-place joints (filled with traditional concretes) or were limited to simple-span applications or posttensioned decks for proper durability. The Canadian deck project clearly demonstrates that existing concepts can be enhanced with the addition of new materials. The specific enhancement in this project is the use of durable and high-strength UHPFRC materials for all closure pours and shear pocket grouting for the deck system. The addition of the GFRP rebar as an upper mat provides an even greater overall protection to the deck system.

Project Title: Sandwich Plate Steel Decks
Citation: Kennedy, Dorton, and Alexander, 2002; Kennedy, Ferro, and Dorton, 2005; Knoblauch, 2004
ABC Design Features:
ABC Construction Features: Rapidly installed lightweight deck system

Project Description

The sandwich plate system is a novel deck system concept comprising a sandwich of steel faceplates and an elastomer core. The deck system was developed for more traditional, stiffened plate applications such as in the shipbuilding industry. It is a product of collaboration between a subsidiary of the BASF Group and Intelligent Engineering.

Kennedy et al. (2005) detail the design of a sandwich plate system to meet the intent of the Canadian Highway Bridge Design Code. Design rules are proposed to cover missing aspects of the code that are unique to this configuration.

The elastomer core's role is to separate the steel faceplates and thus provide a high bending stiffness to the system. It provides no strength since it is a weak material. But given its intimate contact with the plates, it prevents local buckling of the plates and provides damping.

The panels are simply fabricated. Two steel plates are spaced the required distance apart with containment plates on all four sides; the void is injected with an expanding elastomer. The panels attain composite action with the main stringers by bolting longitudinal angles installed under the panels to the main stringers. This process can take some time, but the time is not much different from what is needed to grout shear pockets and wait for those to cure for several days. Another field process is to groove weld the seams between adjacent panels. The decks are thin. For the Shenley Bridge, each faceplate measured 6 mm (<.25 in.) and the core was 38 mm (1.5 in.) for a total thickness of 50 mm (about 2 in.). The deck plates are attached to cold-formed angles that are 250 mm tall (10 in.). The weight of the system is 35 psf, or about 35% of a typical concrete deck and comparable to other innovative deck solutions such as FRP, grid decks, exodermic decks, etc. for this span range.

ABC Opportunity

This product adds another solution to the list of deck options for lightweight deck construction. Its simple construction of spaced faceplates should lead to economical fabrication. The expense of the system appears to be in the connection to the beams. In a deck replacement in particular, extensive field drilling would be required. It may be possible to modify this system to use shear pocket blockouts and conventional shear stud technology for retrofit projects.

Bridge Substructure Concepts

This section focuses on components that can be used to accelerate bridge substructure construction. Bridge substructure and foundation construction are always on the critical path in bridge construction and reconstruction project. Whereas superstructure components can be made in advance and readied for installation when needed, the construction of foundations and substructure units is a linear process. All time savings in this area immediately translate into total project time savings. A number of techniques will be presented that allow combinations of foundation construction to occur without hindering existing operations and for bridge piers and abutments to be built quickly once the site becomes available.

The most-common solutions involve the use of prefabricated substructure elements. Cast-in-place construction must be eliminated to the maximum extent possible to expedite project delivery. Several concepts have been proven in the past and should be encouraged. One is the continued use of pile bents for bridge piers or spill-through abutments. Constructed rapidly of driven piles, pile bents represent a rapid construction technique that no longer requires cast-in-place construction. Traditionally built of driven piles and cast-in-place cap beams, advancements in precasting and connection design now allow for precast caps with grouted connections to the piles. The pockets are made somewhat larger than the piles to allow for some driving tolerance. Another solution (not specifically described here but effective for short abutments) is a soldier-pile or sheet-pile abutment. With short projecting lengths from the ground, the soldier or sheet piles are outfitted with a cap for beam support. In the case of the soldier piles, a concrete panel lagging system is used to retain the earth; for an interlocking sheet-pile wall, no additional elements are required. For taller walls, tiebacks can be used, but that affects the economics of the system, so other systems (e.g., short cantilevered concrete abutments) may be more cost competitive. Additional representative substructure systems are described below.

One substructure design used on accelerated bridge projects is a precast abutment and pier cap component supported by driven piles. This design worked well for a bridge built in Boone County, Iowa. On that project, steel H-piles were driven within a given positional tolerance. Precast abutment and pier cap elements were then placed on the piles. Individual blockouts were formed for each pile, and each pier cap section was installed in less than 30 min. A grouted connection was then used to bond the cap segments to the piers. A high early strength concrete was used in the blockouts (Bowers et al., 2007).

Another precast pier cap was used on a bridge in Black Hawk County, Iowa, combining concrete and steel. In that design, a W-section was laid on its side with the flanges set vertically. Concrete, along with reinforcing steel, was then cast around the steel from the web up. That created a concrete pier cap with the properties of steel in the bottom half. Before casting, holes were torched into the steel, creating composite action between the steel and concrete. The flanges protruding from the bottom of the pier cap can be used to set it onto an abutment wall (Wineland et al., 2007).

Pier and abutment caps are not the only precast components used in a substructure. Abutment walls can also be prefabricated and brought to the site to reduce construction time. In this arrangement, a precast, posttensioned abutment wall is placed on a concrete footing. The footing can be installed by using either cast-in-place or precast methods. The precast abutment segment is then lowered onto the footing. Dowels protruding from the footing are typically used as the connections between the footing and abutment wall. In some cases the dowels are secured to posttensioning strand through the abutment wall. On one project, cast-in-place footings were chosen because of the flat bearing surface they provide (Scanlon et al., 2002).

All of the substructure designs previously described can be combined to produce a completely precast substructure system. One such design was developed by the New Hampshire DOT. The foundation of that design is composed of a precast footing. A flowable grout bed surface was placed below the footing to provide a flat bearing surface. Leveling screws were installed at the corners to ensure the footing was level. A precast cantilevered abutment was then placed on top of the footing. All vertical joints in the system were sealed with grouted shear keys. Grouted splicers connected the abutment wall to the footing. This system has decreased substructure construction time from 1 month to less than 2 days (Stamnas and Whittemore, 2005).

An innovative pier construction method used in Japan has also been shown to accelerate substructure construction. This method is called the Sumitomo precast form for resisting earthquakes and for rapid construction, or SPER for short. The SPER system uses stay-in-place concrete forms to construct piers. The forms are stacked on top of each other at the site and sealed with epoxy joints. Cast-in-place concrete is used to fill the forms. Shorter piers are composed of solid forms, and taller piers are made with hollow forms. The SPER system has proven to be 60% to 70% faster than conventional cast-in-place substructure systems (Russell et al., 2005). For shorter piers, the segments are stacked on top of each other, epoxied together, and then filled with cast-in-place concrete, creating a solid pier. For taller piers, inner and outer panels are used to create a hollow pier. For both types of piers, cross ties and couplers are used to provide transverse reinforcement. High-strength bars are typically used for the transverse reinforcement to reduce congestion between the panels. Cast-in-place concrete is typically used to connect the piers to the superstructure (Russell et al., 2005).

Precast bridge substructures can be made earthquake resistant with a variety of innovative designs. The University of Nevada at Reno successfully tested an unbonded segmental precast column under high-seismic conditions. The column consisted of a footing, three hollow column segments, and a column head. The segments were constructed separately and joined together with an epoxy adhesive. Prestressing forces were then applied throughout the column cross section by using 12 prestressing strands. The column was tested by using motion comparable to the Kobe earthquake and performed successfully, exhibiting very small residual displacements and limited damage. Throughout testing, the joints between column segments remained secure, but the joint at the footing experienced some disconnecting action. It was determined that any repairs necessary for the column could be completed in a short amount of time (Sanders et al., 2006).

Additional work was done at the University of California to determine the effects of various materials to improve the self-centering capability of columns. Residual displacements experienced during seismic loading can cause significant damage to or even failure of concrete columns. Researchers analyzed several design elements, including conventional reinforced concrete, partially prestressed concrete, longitudinal posttensioning, bonded and unbonded longitudinal mild reinforcing, and the use of a steel jacket. Experimentally, partially prestressed concrete performed much better than conventional reinforced concrete. Reinforced concrete and prestressed concrete experienced drift indexes of 1.0% and 0.1%, respectively. The introduction of longitudinal post-tensioning strand greatly increased the self-centering capability of the column. Mahin et al. (2006) recommend using debonded longitudinal mild reinforcing in columns. Debonded reinforcing carries less strain because it is debonded from the concrete, thus decreasing the chance of fracture. Unfortunately, using debonded reinforcement slightly increases residual displacement in the columns. The use of a steel jacket was also advantageous because it prevented spalling of concrete at the base of the column.

Additional research was conducted by Amjad Aref to find the optimum steel ratio in bridge columns in seismic areas. Aref used analytical, finite element, and experimental methods of testing. Different ratios of mild steel reinforcement were added at the segment connections. It was found that, as the mild steel reinforcement ratio increases, the energy dissipation ability increased. Unfortunately, the residual displacement of the column also increased with increasing ratios of steel. Aref concluded that a steel ratio between 0.38% and 0.7% was most favorable range for seismic conditions (Aref, 2006).

Several pertinent prefabricated bridge projects are summarized in an article by Ralls et al. (2004). For example, during construction of the Newark, New Jersey, monorail system, a unique prefabricated steel bent cap and column system was used. In that application the unique construction/fabrication methods were used to meet the rather significant design constraints at the site. The authors also describe the Loop 340 bridges over I-35 near Waco, Texas, which were constructed with precast, pre-topped U beams. The U beams were cast near the site with the top slab and outside curbs cast in place. To speed construction of the substructure, precast column shells were cast nearby and quickly erected on site. The combined system (which cost approximately 40% more than conventional construction) minimized the impacts to I-35 while also minimizing the associated environmental impact of construction. Ralls et al. also reports that the precast construction improved the aesthetics of the project as well.

The possibility of seismic forces or terrorist attacks has increased the need for blast-resistant bridges. The Multidisciplinary Center for Earthquake Engineering Research completed a study on an innovative column design subjected to blast-type loading. The column consisted of a concrete column poured within a steel tube. The concrete-filled steel tubes (CFSTs) were framed by fiber-reinforced concrete pier caps at the top and bottom. The CFST columns proved to be resistant to a range of experimental blast forces. They were both resistant and ductile, preventing any damage to the pier caps. In addition to being a seismic-resistant design, CFST columns can be constructed in an accelerated manner (Bruneau et al., 2006).

A review of current practice related to precast bent cap systems for seismic regions was conducted as part of NCHRP Project 12-74. Although the overall project is expected to be completed at the end of 2009, an interim publication (Toboloski et al., 2006) summarized the state-of-the-practice and the work needed to provide designers with proper tools. The investigators found that precast bent caps have been used by almost half of U.S. states and in other countries and continents. Further, more than 60 unique details could be classified as being used as part of either an integral or non-integral connection, with the greatest number of details being of the non-integral type. Most use of precast bent caps has occurred in low- to nonseismic regions. Most non-integral bent caps use some type of cap pocket. Generally, the investigators found that these types of connections are not suitable for seismic activity resistance. Of principal interest, the investigators noted the inherent difficulty in actually achieving an integral connection when using precast construction. Realizing that the ultimate goal of Project 12-74 is to develop tools that are useful to designers, the investigators will use a combination of analytical and experimental work. The project deliverables are planned to include design and construction specifications (LRFD format), design examples, standard details, and an implementation plan.

Yen and Aref (2007) provide a summary of ongoing work related to the development of construction details for seismic zones. Their paper specifically discusses the development of a model for predicting the behavior of segmental piers. The authors describe the developed model as a two-stage approach. The first stage consists of "pre-decompression" behavior during which a bridge column behaves like a conventional column with a fixed based as there is no opening between segments. During the post-decompression stage, the joints begin to open and are no longer in full contact. This behavior creates analysis difficulties because of the lack of strain compatibility. Further, a finite element model was developed to investigate the seismic behavior in detail. The researchers found that the simplified model is consistent with the one used in conventional bridge columns in that the model can be used with conventional techniques. The finite element model was found to help the engineer understand the behavior at a very detailed level.

A unique technique which employs a steel–concrete composite socket joint has been used to connect the foundation steel piling directly to the steel pier elements quickly without the need of a concrete foundation (Yoshida and Horiguchi, 2005; Takashima et al., 2005). To the authors' knowledge, this procedure has not been used in highway bridges. However, it has been used in railroad bridges and has been extensively tested in the laboratory. The procedure has also been analyzed by using finite element method (FEM) analyses to develop structural design data. The connection has flexibility to account for misaligned piling. Through testing of this connection, it was determined that shear connectors were not required between the steel shell and the core concrete.

Various substructure and foundation ABC solutions are presented in the following case studies. For a list of additional details and concepts, consult FHWA *Connection Details for Prefabricated Bridge Elements and Systems* (Culmo, 2009).

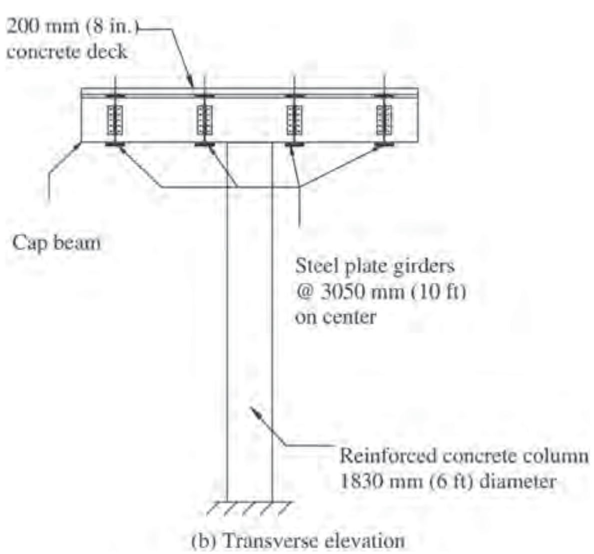
Project Title: Integral Steel Box-Beam Pier Caps
Citation: Wassef, Davis, Sritharan, Vander Werff, Abendroth, Redmond, and Greimann, 2004
ABC Design Features:
ABC Construction Features: Possible adaptation of integral steel boxes for ABC purposes

Project Description

NCHRP 527 focuses on developing design recommendations and approaches and validating the performance of integral steel box girder/concrete column connections. Integral steel box connections are an attractive solution for low-clearance environments in particular—the beam and cap lie in the same vertical plane as opposed to conventional construction where the beams pass over the cap. A schematic of a bridge concept with an integral steel box pier cap from the NCHRP 527 report is provided. In this typical scheme, a multi-span plate girder bridge is spliced directly to an equal depth box girder. A full-moment connection between the box and columns is required for stability. This type of structure promotes beam continuity and—because of the elimination of joints—promotes ride quality, structural efficiency, and redundancy.

Several connection details are provided in the report. The uppermost portion of Figure A.14 provides the beam continuity detail. Given the need to transfer shear and control box distortion under torsion, the box has a rigid interior diaphragm. Top and bottom tie plates transfer the flange forces between spans and partly into the box. Since compatibility between the stringers and box is enforced in this situation, deflection and twist of the girders generate forces in the box girder cap beam.

Figures A.13 and A.14 depict the connection between the concrete column and steel cap. The test connections and prototype bridge designed in the NCHRP 527 report depict a box with a perforated bottom flange, allowing the column steel to penetrate the center void of the pier cap. Bounded by the four walls of the box and two internal diaphragms, this chamber will be filled with concrete to establish the final connection. To promote shear transfer between the column and cap, the top of the column is voided. A group of shear studs welded to the underside of the box eventually rest in this void and, by using a small hole in the bottom of the box, the void is grouted. This provides the horizontal shear connection without relying on the shear friction capacity of the vertical steel.



Source: Wassef et al., 2004.

Figure A.13. Integral bent cap concept.

ABC Opportunity

Though not envisioned as an ABC innovation, this connection–pier concept has many ABC possibilities. Coupled, for instance, with a precast pier column, it is possible to quickly erect the column using grouted splice sleeve couplers and techniques such as SPMT or lateral sliding, to bring a two- or more-span bridge to the site, and to lower it directly onto the columns. Once the grouted connections are made, the structure is nearly complete. Whether or not the deck is installed before or after movement should be left to the conditions of the project and how much weight can be supported reliably during moving. Variations on the column-to-cap connection could also be proposed, the most apparent being the use of headed reinforcing in shallow-cap situations where development length is a potential problem. High-strength self-consolidating concrete would effectively fill this joint with little problem. Another possibility is to eliminate the studs penetrating the top of the column and simply rely on the shear

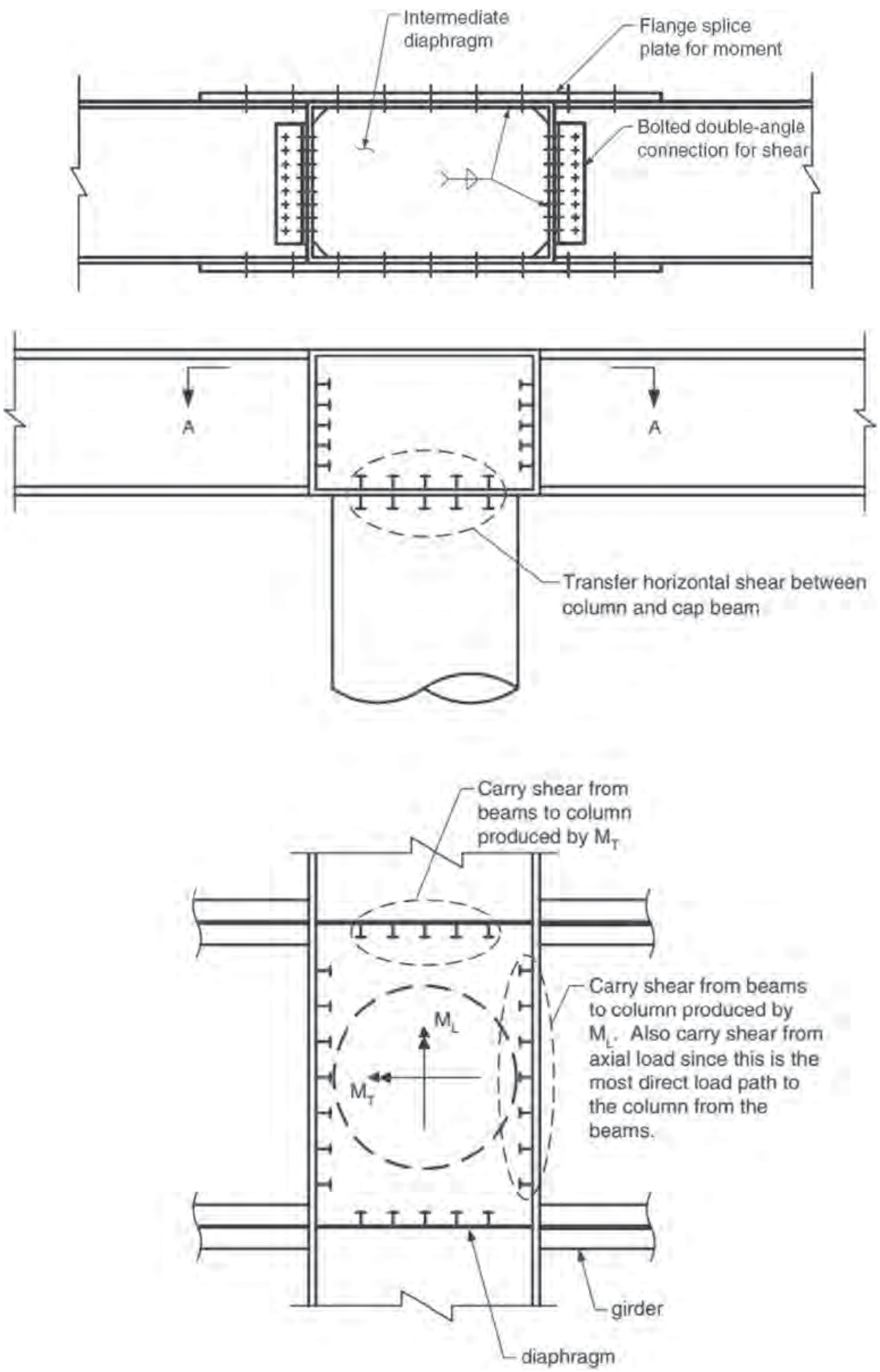
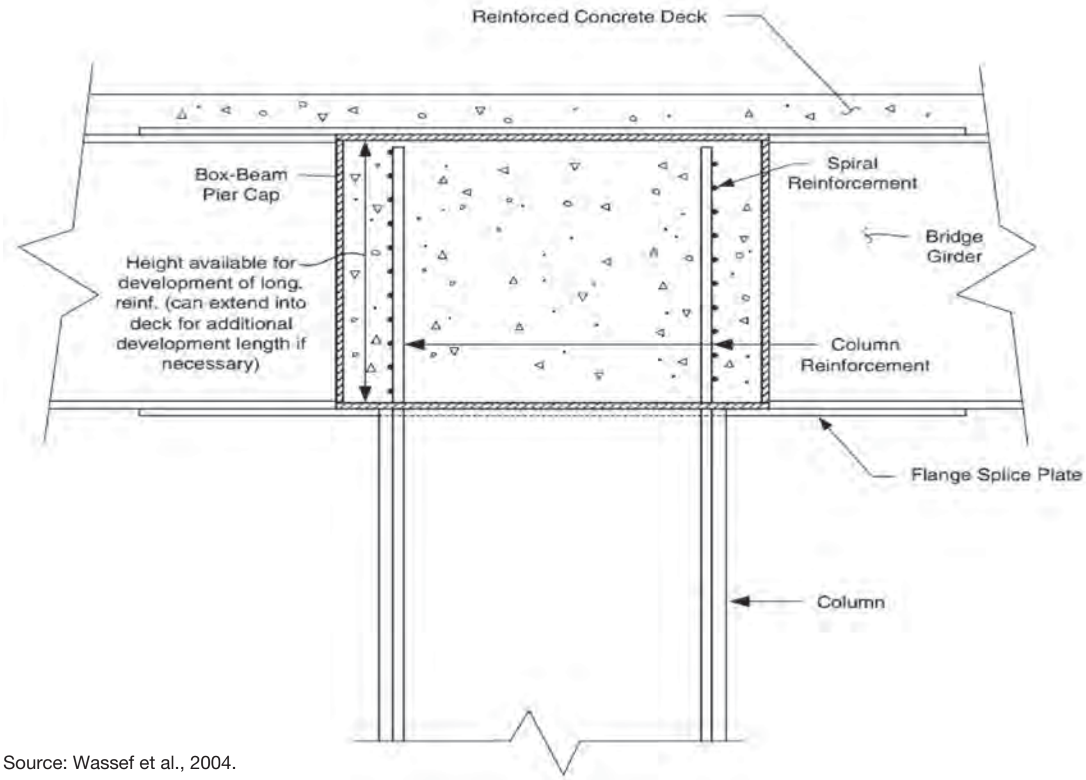


Figure A.14. Bent cap connection details.

(continued on next page)



Source: Wassef et al., 2004.

Figure A.14. Bent cap connection details (continued).

friction of the longitudinal steel. For geometry control, it may make sense to stop the column just short of the required elevation and, by using friction collars and an edge band, form the last inch or two of the section after the cap is lowered and the proper grade and cross slope are established. At that point the short void between the column and cap would be pressure grouted with a high-strength grout.

Project Title: Concrete-Filled Steel Tubes for Bridge Columns
Citation: Roeder and Lehman, 2008
ABC Design Features:
ABC Construction Features: Rapid pier construction with concrete-filled steel tubes for bridge columns

Project Description

Roeder and Lehman (2008) present the results of experimental testing of concrete-filled steel tubes to be used for bridge column construction. Large-diameter, thin-walled steel tubes both stay in place while forming but also serve as the vertical (primary) and lateral (confining/shear) steel simultaneously. The net column diameter is less than that of a traditional reinforced concrete column. Their work also includes development of a simple and strong connection between the concrete-filled tubes (CFT) and a proposed precast concrete pier cap or footing. The proposed system allows for complete piers to be built in a matter of days instead of the weeks or months needed for typical cast-in-place construction.

As part of this study, alternate connection details capable of sustaining large inelastic rotation demands of seismic loading were developed and tested—with excellent results. A simple connection between the CFT and a concrete footing was developed. In that connection, a concrete footing is constructed in two lifts. The first lift is poured to support the steel column. This first lift has a series of hooked anchor bolts in a circle that will attach to an annular steel plate welded to the underside of the steel column. Once the column is erected and bolted to the first lift, the second lift is poured, completely enveloping the bolted connection and base of the steel column in concrete. No variations to the footing are required for the two-stage construction; the footing is conventionally reinforced for shear and flexure. This connection performed well, but further economizing was thought to be possible, specifically in the need for two-stage construction and the expensive full-penetration welded annular plate at the base of the tube.

In the modified connection, a steel annular ring is still provided but only as an erection aid. The footing is cast in a single lift with a corrugated metal pipe blockout to receive the column at some future time. When the column is installed, light bolts are used to stabilize the column; the annular space between the outside of the steel column and the corrugated metal pipe is filled with a high-strength, nonshrink, fiber-reinforced cementitious grout. Simultaneously, the inside of the tube is filled with self-consolidating low-shrinkage concrete. Completion of an entire column is likely to take just hours from start to finish, and smaller columns could be completed in under an hour.

To connect a pier cap, a cast-in-place pier cap can be constructed, but for maximum speed of construction precast cap elements should be used. To connect a precast cap, a friction collar or similar device is installed at the top of the column to support the weight of the cap. The cap contains a void similar to the footing that receives the upper portion of the column. The connection between the column and cap is established by pressure grouting; small vents through the top of the cap ensure complete filling of the grout pocket.

The authors cite test results that indicate the spiral-welded tubes used for the columns were more slender than typically permitted by code, with diameter-over-thickness (D/t) ratios as high as 80 and strengths of 70 ksi. This slenderness did not have a detrimental affect on the CFT performance. Test results of the bolted detail indicate drift ratios as high as 8% were attained with stable hysteretic behavior. This result was attained with a column penetration into the footing of 0.9 D, thus indicating that large footings are not required to anchor this detail. No significant damage of the footing was noted. The authors comment that for similar demands, concrete columns begin to spall at 6% drift, and their performance was not as good as the CFT option. Embedment of as little as 0.6 D is possible and can fully develop the yield strength of the tube. However, significant damage occurred at large displacements.

The authors provide some narrative indicating that corrosion and fatigue do not appear to be issues of any significance for these structures. Details to shed water away from the connection are proposed; and despite the high degree of compression expected in the bridge columns, no evidence suggests that fatigue of the spiral welds will be of any concern.

ABC Opportunity

Roeder and Lehman present an alternative to concrete columns for ABC construction. Because the section is composed of a spiral welded steel shell, erected first and then filled with concrete, the construction can be accomplished with a light crane. The external steel shell serves three purposes at once: external formwork, longitudinal reinforcing, and confinement/shear strength. Several proven connection alternatives are provided that develop the plastic strength of the column section while preventing damage penetration into the footing. Of these, the voided footing with fiber reinforced concrete infill offers the simplest and most cost-effective detail.

Project Title: Concrete Filled FRP Shells for Pier Construction
Citation: Li, Shi, and Mirmiran, 2008
ABC Design Features:
ABC Construction Features: Rapid pier construction with concrete-filled tubes for bridge columns and caps

Project Description

Li et al. (2008) provide details of a concrete-filled FRP forming system to be used for pier construction (columns and caps), with the FRP serving both as formwork and as part of the required reinforcing. As with the steel shell system, external confinement is desired to enhance the ductility and performance of such frames under seismic loading. Four ⅙-scale two-column bridge piers were fabricated and tested.

In the test frames, the columns included internal reinforcing. The reinforcing included longitudinal reinforcing but with a greatly reduced amount of transverse steel—only enough to control the geometry of the bar cage. The columns were constructed with FRP tubes (glass, carbon, and a hybrid), and the FRP formwork for the pier was composed of multiple layers of fabric.

Traditional push–pull cyclic load tests were conducted on the model piers. Whereas a reference reinforced concrete pier attained a drift ratio of about 3% at a lateral load of 20 kips, the specimen using external glass fiber reinforcing attained a lateral strength of nearly 40 kips, doubling the reinforced concrete (RC) frame strength and achieving a drift ratio of 5.4%.

The series of tests was conducted on a frame with monolithic pier-to-column joints; all concrete was cast in place. The external FRP forms were used for all elements. Follow-up was conducted with the principal investigator (PI) for this project to gather additional details that were ABC relevant. The question was whether the intent was to develop this into a system that uses externally FRP-reinforced precast sections that could be assembled in the field. The PI indicated that the intent is to make these piers of precast elements, externally confined. Testing has already been done on frames with precast caps and on pier frames with posttensioning used to connect the cap, column, and footing. Work continues on this project.

The results from the precast pier system testing of these FRP-reinforced pier elements are presented by Zhu et al., (2004). This paper describes the precast variant of the system as well as the details of testing various types of connections, including simple dowel bars, embedment of the FRP tube into the footing, male and female insertion joints, and posttensioning to join precast elements. A schematic of the pier and the various details tested is provided, and the results are briefly summarized. The posttensioned connection used to join the cap beams was the most robust and ductile connection. The male and female joints did not provide adequate performance due to geometric imperfections in the mating of the joints. Embedment of the tube into the footing provided performance benefits (as observed by Roeder and Lehman for steel filled tubes as well), and the embedment is recommended. The study concluded that internal reinforcing could be eliminated outside the connection regions. Further work on this system was suggested. We were unable to find any follow-up testing of this system.

ABC Opportunity

The ABC opportunity for this system is similar to the concrete-filled steel shell system described by Roeder and Lehman (2008) but uses FRP materials as the shell system. The difference with this concept is that the shell forming system is used for the pier cap as well as the columns. The system has been tested with both monolithic joints and precast joints. Only reduced-scale tests appear to have been conducted. Additional testing should be performed to validate this system. Availability of FRP shell materials may affect the use of this concept unless the FRP shells are laid up or spun for the specific application.

Project Title: UHPFRC Bridge Column Shells
Citation: Brühwiler and Denarié, 2008
ABC Design Features:
ABC Construction Features: Rapid pier construction with UHPFRC stay-in-place forms for bridge columns

Project Description

A related application is the use of prefabricated ultra-high-performance fiber-reinforced concrete (UHPFRC) shells for the rehabilitation of deteriorated bridge columns. Brühwiler and Denarié (2008) discuss the use of UHPFRC in various applications as a rehabilitation and restoration material for deteriorated structures or bridges requiring partial restoration. The idea is to use UHPFRC to "harden" particularly vulnerable portions of structures, taking advantage of its high strength but, more important, low permeability and enhanced durability to protect existing structures. The concepts explored include remedial repairs such as thin bonded overlays or barrier rail overlay that uses UHPFRC to provide prolonged life.

In an application that has potential technology transfer to new ABC bridge construction, the authors discuss the retrofit of existing reinforced concrete columns with a UHPFRC overlay/jacket. The existing bridge columns in the case study were 40 years old and had substantial chloride contamination. The rehabilitation strategy was to hydro-demolish the contaminated concrete and then install new UHPFRC shells around the existing column. The shells were 4 cm thick, cast as high as 4 m, and shipped in split halves to the job site. Once the shells were erected around the existing column, the interstitial void was grouted under pressure to both fill the void and bond the shell to the existing column.

ABC Opportunity

This concept was explored as a method of new column construction by the R04 team, including discussions with LaFarge, the North American producer of Ductal, a commercially available form of UHPFRC. The concept explored by the R04 team is to build on the "shell" concept used for retrofit, or explored by others using steel or FRP shells, but for new column construction. In the proposed new column application, extruded or spun cast shells would be used as "lost forms" to construct a traditional reinforced concrete column. The UHPFRC would serve as formwork but also an extremely effective environmental barrier with low permeability and high tensile strength. Those qualities make it desirable to use as forming. The column would be built in traditional fashion; but instead of erecting steel or wood forms, the concrete shells would be used. Alternative concepts include the use of thicker walls for hollow column applications. These precast elements could then be stacked and posttensioned together for an ABC column application. At this point, solid columns composed of UHPFRC do not appear to be a cost-effective use of the material, thus the shell-type applications seem most suited for future development.

MSE Abutments

Though mechanically stabilized earth (MSE) abutments are not a new technology, general discussions with geotechnical engineers over time and more recent conversations with engineers specifically related to the R04 project reveal that the use of MSE abutments is an underused existing technology, proven through many decades, which should be used to accelerate bridge construction. Brabant et al. (2008) present some data to substantiate this statement. Currently, about 300 bridges annually are constructed with MSE abutments, but the opportunity is much larger. When selected, MSE abutments are chosen because of speed of construction, cost savings, and ability to be built over poor native soils.

The true MSE abutment consists of facing panels and anchors placed near the beam seat level. A lift of compacted gravel is placed at this location; on top of that is set a grade beam with pedestals to support the superstructure beams. A backwall is used to restrain the backfill. In this system, the reinforced fill is used to support a spread footing (the grade beam), and the loads are carried thusly. It has been the authors' experience, however, that many owners do not prefer this system; the concern is generally settlement. The second type of

MSE abutment is one in which the MSE facing is simply used to restrain the embankment fill, while the bridge has a deep foundation of either drilled shafts or more traditionally driven piles. In this mixed system, owners expect to get zero settlement in the completed foundation, which thereby supports the bridge in a fashion that is familiar and similar to other types of bridge foundations.

MSE wall abutments offer various advantages to ABC projects. They can be used in phased construction to effectively widen and replace existing bridges. Because of the way they are built from the bottom up with small equipment, they have been used in many instances for construction underneath an existing bridge to within several panels (5 ft to 10 ft) of the existing bridge envelope. Only at that point does demolition need to proceed to advance the completion of the abutments. Production rates for typical MSE walls vary but can equal 2,000 ft^2 per day. With average abutments for small- to medium-span bridges ranging as high as 10,000 ft^2, the total abutment construction time can be measured in just days, dramatically different from the speed of construction of a traditional cantilevered abutment on deep foundations. Once the abutment is completed, the sill and beam seat (precast in advance) can simply be installed.

Geosynthetic Reinforced Soil Abutments

Geosynthetic reinforced soil (GRS) abutments have been constructed in various research and field deployment projects over the past two decades. Documented in various reports—including test results; analytical studies; and design, construction, and specifications information—these reinforced soil substructures have the opportunity to change the way bridge substructures are constructed (Wu et al., 2006; Adams, 2008).

In the GRS substructure concept, what appears to be similar to an MSE structure is in fact a very different structural system. In an MSE substructure, the facing panels serve to anchor the straps and contain the fill and are thus an integral part of the structural system required to support the reinforced soil mass. In the GRS system, the role of the discrete facing elements is more decorative than functional. The primary stability of the system lies in the use of shallow lifts (approximately 8 in.) with geosynthetic reinforcing installed between each lift for strength and stability. The facing blocks are typical segmental block retaining wall units that are used in various forms of residential and commercial construction or split-face cinder blocks. The GRS need not be positively attached to the blocks; often, it is simply laid between courses as a friction anchor. Additionally, no pins or mechanical anchors between block courses are required since the lateral pressures exerted on the facing are minimal. Wu et al. detail various installations of such systems around the world, provide the results of various physical tests of such systems, and present a comprehensive design methodology specifically to be used for GRS abutments with flexible facings (such as dry-stacked block facing units).

Adams (2008) provides a detailed description of 11 bridges built in Defiance County, Ohio. Each bridge used local forces and conventional equipment and had bid cost savings of approximately 25% over a conventional project. The schedule compression was substantial, with a typical bridge requiring only 2 to 3 weeks of normal construction by a five-man team, the fastest bridge being competed in 10 days. Unlike some ABC solutions that we accept as costing a premium to justify their speed, GRS construction has proven to result in a product constructed both faster and cheaper. The in-service experience also indicates excellent ride quality after opening and stable performance even in the event of a flood as each of the new bridges has been nearly completely inundated by recent flooding events.

Each of the 11 bridges constructed by Defiance County was constructed without the use of any cast-in-place concrete, a major advantage to rapid construction. In addition to the use of the GRS abutment technology, the bridges were constructed by using traditional and readily available adjacent concrete box beams. On Bowman Road bridge, the beams sit directly on top of the compacted abutment backfill. Additional GRS fill is placed behind the ends of the beam in the bridge approach area, and continuous asphalt concrete paving is used to bridge the approach roadway and bridge pavement. In-service inspection of the bridges reveals minimal settlement and excellent performance of the bridge deck and approach paving. Performance is considered better than for conventionally constructed bridges with regular abutments and approach pavement sections.

ABC Opportunity

The ABC opportunity for this project is to replace the time-consuming process of cast-in-place abutment construction with a rapidly constructed integrated bridge superstructure, substructure, and approach paving system entirely completed in several weeks with small construction equipment and local forces or small local contractors. It demonstrates a complete systems approach to bridge replacement that could be applied to literally thousands of bridges. The system could just as well be used for abutments for bridges crossing land features as for stream crossings. The system could also be used in bridge widening projects as well as to extend the width of existing abutments in a low-impact way. Potentially, bridge piers could be constructed by using this system. This type of foundation can be used in marginal soils since a relatively low bearing pressure is applied. Additionally, the strength of the confined soils is sufficient to support most common bridges. The bearing capacity of the reinforced soils varies from 1.9 tsf to 2.9 tsf for a 5-ft-wide footing/sill to as high as 4.5 tsf to 6.9 tsf for a 2-ft-wide sill/abutment beam seat. These numbers convert to span capabilities for usual structure self-weights and design live loads that range from a low of 120 ft to nearly 200 ft depending on backfill quality and sill width. Thus, this abutment type is readily adaptable to the vast majority of potential bridge replacements in the United States.

Bridge Movement for ABC Projects

The previous discussion focused primarily on prefabrication solutions for ABC projects. Those concepts make use of existing contractor equipment and are already in deployment around the country to some degree. Acceptance of prefabricated solutions will likely grow substantially, and the team expects that future ABC projects will make significant use of some of those concepts.

Another class of ABC projects consists of those that involve transporting large components or completed bridges by using various movement techniques. These techniques include self-propelled modular transporters (SPMT); bridge sliding, skidding, and rolling with the use of various sliding surface movement methods; and incremental launching. The following discussion focuses on these methods since they are demonstrated techniques with broad applicability. Other methods of erection are presented as well, including the use of gantry systems and several unique structure erection concepts.

Notably, some topics are not covered. These include bridge rotation, which is the construction of a bridge on a rotating foundation, usually over a navigable waterway, and then rotation of the bridge to a closed position when completed. First, many international articles discuss this method, which is not a new technique. Double-leaf swing bridges have been constructed this way since the concept was invented centuries ago. Second, bridge rotation is perceived as a niche application that does not have the wider range of applicability of some other techniques. Another technique that this report omits is the floating of bridges into place using barges. Again, the concept is neither new nor broadly applied. The team discovered one interesting concept worth mentioning briefly: segment floating. A Chinese bridge was recently constructed such that the center-span steel box girder, 100 m long, was fabricated more than 1,000 miles from the bridge site. It was hermetically sealed and floated like a barge itself down river to the bridge site. At that point, strand jacks were used to raise the section into place. Though innovative and unique, potential applications are few and far between. Finally, one recent and unique article worth mentioning describes arch bridge construction that uses an accelerated construction method with chorded arches (Zhou et al., 2008).

Each highlighted movement technology is first presented, then followed by sample cases of its use.

Design Practices

One of the concerns about bridge movement by various end users is overloading or in some way damaging the bridge during the process. The R04 team is aware of two European documents on this subject, CUR 68 and CUR 81 authored by de Boer, but to date they have not been made available for our review and use. (English versions of these documents do not exist.) Having discussed design criteria with multiple agencies, the team has concluded that no consistent set of standards is applied. Contacts at the French agency SETRA (Service d'Etudes Techniques des Routes et Autoroutes) did provide some clarity on this issue—at least from the perspective of their agency's practices. Generally, the rules for strength and stability of structures being moved are the same as for the bridge in its final position. The Eurocode does provide some guidance. Some stability checks are specific (e.g., web buckling due to a patch load between two vertical stiffeners in EN 1993-1-5, or overall stability in the absence of the concrete slab in the case of a composite bridge). Checks are applied at the ultimate and service load conditions, and the checks are required to take into account the erection history. The loads during erection are defined in EN 1991-1-6. Some loads could be different due to their short application period (wind, for example, in the case of a 2-day launching). Special care is taken for geometrical tolerances during erection, uncertainties on some loads (e.g., counterweights), and any special equipment supported off the partial structure.

SPMT Bridge Construction

As an outgrowth of the FHWA scanning tour of prefabricated bridge elements and systems (PBES), subsequent AASHTO Technology Implementation Group (TIG) efforts, FHWA promotion, and the success of several high-profile emergency replacement and demonstration projects, SPMTs continue to gain popularity and use in the United States for bridge movement (Ralls et al., 2005b). The findings from the FHWA PBES scanning tour can likely be considered the first concerted step in accelerating bridge construction through the use of mobile bridge technologies. The most prominently mentioned technology in the PBES scanning tour report is the SPMT. The PBES scanning tour report documents many interesting projects (although the report lacks details because of its overview approach). The report identifies several bridge construction projects, including two multi-span bridge projects that were placed with SPMTs. The findings are significant because these types of moves have not yet been tried in the United States.

SPMTs, though somewhat new to the U.S. bridge building market, are becoming increasingly familiar to owners, engineers, and contractors. Numerous conferences, presentations, and publications in the past few years have drawn considerable attention to unique projects whose construction was made possible (or caused less impact) through use of SPMTs. For example, self-propelled trailers have the ability to lift very heavy loads, turn them to respond to site constraints, and vertically and laterally position them with precision. Several projects are highlighted here, and excellent references provide substantial guidance to end users (owners, engineers, contractors) on the use of SPMTs for bridge construction projects. Additionally, since demand is still relatively low, the SPMT heavy-lift firms continue to provide specialized support and guidance for specific applications.

In 2007, in response to an increasing interest in SPMT use for bridge construction, the FHWA published the *Manual on Use of Self-Propelled Modular Transporters to Remove and Replace Bridges* (FHWA, 2007d). An excellent primer that provides general characteristics of SPMTs (which vary by manufacturer), this document is available in electronic form from the FHWA website. The manual presents important characteristics, such as length (varying from 20 ft to 30 ft for a 4- to 6-axle unit), width (8 ft to 10 ft), height (platform height of 4 ft), vertical travel or adjustment capability (as much as 2 ft), axle capacities of 26 to 33 tons (useful capacity is less because of the weight of shoring, etc.). It provides an excellent definition of the "usual parameters" so that a designer or owner can assess how an SPMT lift might be configured or proposed for a specific project. Given the geometric characteristics of the trailers and their load capacities, an "as-designed" lift could be designed by the engineer of record, understanding that the contractor might still choose another approach. The manual also presents the benefits and costs of SPMT projects. The benefits include items such as reduced traffic disruption and enhanced safety and quality due to off-site fabrication; the costs include those of equipment rental as well as cost savings measured by a reduction in user costs.

In a section on design, the manual highlights several key issues. First among the considerations for SPMT use is the issue of adequate space and conditions in which to make the move. Following those are issues related to soils capacity. Given the high loads that are transported on pneumatic tires, high contact pressures are generated. These can be in the range of 1,500 psf to 2,000 psf. Adequate ground capacity must be available. Otherwise, ground improvements such as the addition of a gravel haul road or steel plating of the traveled way might be considered for softer soils.

A significant portion of the manual relates to contracting considerations and guidance for the owner. Included as a starting point are sample user cost models to assess the worthiness of SPMT use, sample specifications for SPMT projects, and sample project acceleration specifications—such as incentives and disincentives, bonuses, lane rental, value engineering, and partnering. Other sample documents, such as traffic control and structure movement path plans, are provided as well.

Under a contract with Corven Engineers, the Utah DOT became the first state to develop a specific manual for use on state-sponsored SPMT projects. The objective of the manual is to assist the department in its rapid transformation to ABC deployment. Given that this is a new technology, with multiple layers of contracting, the manual was developed to describe the various elements of an SPMT project, the roles and responsibilities of various parties, engineering requirements for certain critical operations, and to provide guidance on geometry control during various movement phases (Utah Department of Transportation, 2008e). The title is *Manual for the Moving of Utah Bridges Using Self Propelled Modular Transporters (SPMTs)*, and a copy is available on the Utah DOT website.

The front matter of the Utah DOT manual primarily provides guidance to agencies; although it focuses on the Utah DOT, it could be considered standard guidance for any agency considering an SPMT project. The type of information provided relates to the interaction of various parties, including the state agency, bridge contractor, engineer of record, specialty moving firm, moving firm's specialty engineer, and other parties that might be involved. The process by which these individuals interact and in what contractual forms are presented.

Structural design considerations are provided primarily for the use of the engineer of record. A challenge to date has been providing analysis and stress and stability requirements for bridges to be moved in place with SPMTs. This section recommends critical stages that should be evaluated. The analysis is based on an assumed lifting location. A method is proposed for evaluating stresses and deflections in the structure as it is lifted from its supports. Relevant sections of the LRFD specifications are referenced. These include specific recommendations for controlling stresses in already-cast concrete—such as might be the case if a bridge with a deck already cast on top is moved. Typical lifting locations are some distance in from the permanent bearings; that places portions of an already-cast deck and railing in tension during the lifting location since they cantilever out past the temporary supports. Guidance is also provided to the contractor's specialty engineer in the area of load recommendations for the design of strong and stable falsework on which to construct the initial bridge. Some guidance is also provided for the specialty mover.

Specialty movers and bridge contractors may need to perform certain activities to support the operations of the other party. Generally, the specialty mover relies on the bridge contractor to provide various support functions. The manual presents information on some of these interactions. For instance, a specialty mover may require the bridge contractor and his specialty engineer to design the temporary support structures. Sometimes that work is done by the SPMT firm. This step should be clarified between the two parties. Additionally, the bridge contractor should ensure that a proper traffic control plan is in place for the time of the move; that a movement path of proper grades, turning radii, and soils bearing capacity is provided; and finally, that a large enough staging area is available to build the bridge, stage the moving equipment, and complete the move.

Geometry control of the bridge during moving is critical in that excessive twisting of the structure (e.g., one of the corners departing from the plane of the deck) can result in deck cracking, excessive girder stresses, cross frame or diaphragm distress, or other undesirable behaviors. A finite element study of structure twist was performed to determine what stresses are induced by varying amounts of settlement or change in planarity of one of the bridge corners. Recommendations for structural twist of W/200 to W/300 are provided, where W is the width between edge beams. The maximum permissible twist should be limited to 2 in. to 3 in. (Note that in conversations between the team and SPMT manufacturers about their ability to meet these tolerances, the manufacturers all said this standard was generous and easy to meet.)

Several appendices to the Corven–Utah DOT report provide useful information, such as contact information for heavy lifters and a sample set of plans for an SPMT movement project—including staging area and movement path drawings.

Project Title: Graves Avenue Bridge, Florida
Citation: Trimbath, 2006
ABC Design Features: Precast concrete girder and deck system
ABC Construction Features: SPMTs used to move out old bridge and move in new bridge

Project Description

The Graves Avenue Bridge is two-span bridge composed of a precast concrete beam and deck system. The two-span bridge extended 286 ft across Interstate 4. Each span is 143-ft long and 59-ft wide. The total weight of each span is 1,300 tons. The existing bridge needed to be replaced with a longer span to accommodate a wider road below. The Florida DOT decided to experiment with the use of self-propelled modular transporters (SPMT) for the removal and installation stages of the project. SPMTs are computer-controlled multi-axle trailers that can lift precast elements into place. They have been used extensively in Europe and have proven to be a reliable method of accelerated construction in the United States as well, with successful implementation in Washington State, Oregon, Utah, and elsewhere. Completed in 2006, the Graves Avenue Bridge replacement project was one of the first interstate overpass projects to employ SPMT. By using SPMTs, traffic disruption was reduced from 32 days for a conventional bridge replacement project to just under 4 days.

ABC Opportunity

The Graves Avenue Bridge in Florida was a good candidate for accelerated construction for a variety of reasons. The bridge is located very near a high school, and the project had to be finished before the start of the school year. Also, the Florida DOT wanted to reduce the effect on traffic as much as possible, thereby reducing road-user costs. Removal and replacement of the existing bridge spans over the interstate was accomplished quickly and with little interruption to traffic. The spans were removed one at a time by using two SPMT trailers. Each span was raised 6 in. from its supports and then removed from the site. The removal process was completed on two separate nights without blocking traffic. Once the deck was fully removed, eight trailers were used to raise the new spans. The contractor constructed a temporary fabrication yard close to the bridge site. The spans were then brought to the site and placed at their final resting locations. The erection stage was completed in less than 4 days.

Project Title: I-10 Louisiana Twin Span Bridges, New Orleans, Louisiana
Citation: Fossier, 2005
ABC Design Features: Original precast units replaced; where original units were lost, Acrow steel truss was used
ABC Construction Features: Staged construction and SPMTs used

Project Description

The infrastructure of New Orleans and the surrounding areas took a significant blow during the Hurricane Katrina disaster of 2005. The I-10 Bridge over Lake Pontchartrain was hit particularly hard by the storm. During the surge, a total of 473 spans were either moved or pushed into the lake. Because this bridge was part of one of the few major routes in and out of New Orleans, it had to be repaired immediately. The Louisiana DOTD used a staged construction schedule and SPMTs to restore traffic across the bridge.

The I-10 Bridge over Lake Pontchartrain is a twin-span bridge crossing a total distance of 5.4 miles in 436 spans. Most of the spans are low-level and are composed of precast, prestressed concrete girders. Three spans at the north side of the lake are high-level spans and provide access for marine traffic. Those spans are longer and are composed of composite-steel plate girders. The substructure is a cast-in-place reinforced concrete cap supported by precast, prestressed cylinder piles. Each bridge carries two lanes of traffic.

Following the storm, 170 eastbound spans and 303 westbound spans were displaced. Of those, 38 eastbound spans and 26 westbound spans were completely submerged in the lake. In addition to the span displacements, nine cylinder piles were damaged by the collapsing spans. The department was unable to reuse the spans that were completely submerged in Lake Pontchartrain. To replace those spans the contractor used temporary "Acrow" steel truss bridges.

The Louisiana DOTD had to move quickly to restore traffic through the area. They made use of accelerated contracting and construction techniques to facilitate a rapid repair. The bridge was partially opened only 47 days after the storm and completely open to traffic a few months later.

ABC Opportunity

Immediately after investigation of the damage, the Louisiana DOTD went to work on the contract. All plans and specifications were completed in 1 week, and the bids were submitted 12 days after the storm.

To minimize traffic disruption, the decision was made to replace the eastbound bridge first because it had fewer spans. That bridge would then be open to traffic, and the westbound bridge would be repaired. The contractor was given 45 days to complete the first phase of construction but finished in 28 days. The second phase of construction was scheduled for 120 days. The spans were repositioned in their original locations by using a combination of a barge and SPMT.

Project Title: Yokogawa Bridge Corporation—YS Quick Bridge
Citation: Yokogawa Bridge Corporation, 2007
ABC Design Features:
ABC Construction Features: Movement of completed bridge systems to create grade-separated intersections at busy road crossings

Project Description

Continued traffic congestion problems in Japan demand innovative construction approaches. As one solution to this problem, at-grade intersections are being partially replaced with elevated through lanes; surface construction is reserved for local lanes and turning movements. The traditional approach has resulted in traffic restrictions for more than 1 to 2 years, causing new traffic jams. Thus, public support for such projects has been difficult to attain. Furthermore, securing temporary access rights to the land needed for construction has also been difficult.

The YS Quick Bridge Method was proposed to solve these problems. In this method, the superstructure is assembled in the median of the approach to the road to be elevated, near the proposed intersection. Construction is completed in an elevated position on falsework, including the new bridge piers. As soon as the structure components are completed, they are assembled and moved into place with SPMT or heavy haul trailer technology. A time lapse image of construction is presented. Numerous innovative aspects to this project include the use of existing lanes on the approach road as a staging area. Though imposing some restrictions, this method has less impact than erecting the bridge over an active intersection.

Additionally, the connection between the bridge and foundation involves a unique "well" connection. By using the same work zone restrictions as for the superstructure, drilled shaft foundations are constructed with the upper portions left open, protected by steel casing. The bridge is then driven to the final position and lowered into the wells—where the final connection is made between the above- and below-grade portions via a cast-in-place closure pour collar. This method of construction is called the precast reinforced concrete (PRC) well method. It is suitable to urban environments since drilled foundations create less noise, are low vibration, and are installed in more-compact situations than is a multi-element driven foundation. The shaft is drilled to within 3 m of the finished ground line. A steel casing is used throughout and extends to the ground line. The superstructure with integrated bridge column is delivered and then lowered into the void provided by the steel casing. In the Quick Bridge system, the bridge pier is also a large-diameter steel column.

This prototype joint type has been in use for a number of years, primarily for railroad bridge construction. Nonetheless, physical testing of the joint was conducted. During reversed loading push–pull lateral load tests, the system attained a behavior in excess of six times the yield displacement of the column with significant energy absorption as evidenced by stable hysteretic response. Displacements as large as 250 mm were recorded.

ABC Opportunity

The opportunity afforded by this system is to move completed integrated bridge superstructures and substructures into place and to rapidly form a connection to the foundation element. Though construction of the bridge will take some time, and traffic restrictions are required for this method, it may prove to be one of few viable ABC techniques for crowded urban environments.

Project Title: Innovate 80 Bridge Replacement Projects
Citation: Utah Department of Transportation, 2008d
ABC Design Features:
ABC Construction Features: Use of SPMTs and skid shoe systems to deliver and launch bridges

Project Description

The use of SPMTs and lateral skid shoes to install bridges was combined for multiple bridges installed by the Utah DOT in the summer of 2008.

A staging area was created several miles from some of the bridges. Dubbed the "bridge farm," the area formed a common location to build multiple bridges. The bridge farm was located adjacent to an interchange. As individual structures were demolished and new substructures readied for the new bridge, the completed structures were driven up and onto the Interstate, sometimes crossing old bridges, sometimes newly installed ones, until the desired location was reached.

When the bridge reached the site, it was transferred from SPMTs to the skid shoe systems. Figure A.15 shows a view looking back at the new structure still supported on SPMTs and being driven toward the abutment. The span cantilevers out; a series of skid shoes with jacks were used to incrementally receive the load from the SPMTs and drive the bridge across the opening. Once specified temporary support locations were reached, the bridge was jacked down and lowered to the permanent bearing condition.

ABC Opportunity

The combination of SPMT movement from a remote location and use of skidding as a variation on incremental launching was effectively used for multiple bridge replacements in Salt Lake City, Utah. The combination of multiple techniques in a project demonstrates that no one particular technology will always work or work alone. Flexibility in the planning process, coordination with contractors, and the use of heavy movers were all essential for project success. The engineer of record would not likely have contemplated this method of installation or been able to develop the correct details. A project such as this can be accomplished only with early and effective contractor interaction that employs contracting methods such as design–build or payment-in-advance for planning studies.

Figure A.16. Skid shoe span launching.

Figure A.15. SPMT span delivery.

Structural Skidding and Sliding

The use of structural skidding and sliding is not a new concept. Bland and Miller (1915) discussed the lateral sliding of new truss spans to replace a series of temporary spans installed after a flood washed away a large portion of a railroad bridge in Ohio. The paper details the movement of a three-span truss weighing 3,500 tons in 10 min, 17 sec—between consecutive trains. That demonstrates the capacity of low-technology equipment to move large loads quickly and stably from a section parallel to the existing bridge into the final alignment location.

Several movement techniques are described in this section. Lateral sliding requires that adjacent space parallel to the existing bridge be available. Sometimes space is required not just to stage the new bridge before rolling it in but also to roll the old bridge out—unless it can be demolished in place quickly and without impact to the roll-in operations.

Two jack-based systems are described. The first uses a Teflon-and-steel sliding surface lubricated with ordinary dish washing detergent for maximum efficiency of the move. The skid shoes have capacities of as much as 600 tonnes (660 tons) each, with vertical stroke capacity of as much as 2 ft. Thus, the vertical alignment and horizontal sliding are all provided in one unit. The vertical adjustability of the jacks allows skid shoe systems to be used even on less than ideal soils. The jack can be extended if any soft spots in the foundation are located under the skid shoe. With a fully automated control system, the jacks self-compensate for any geometry imperfections. Horizontal movement is managed through the use of standard modular steel guide channels, which can be quickly placed at a site and removed afterward. The jack responsible for moving the structure locks into this channel and pushes the bridge a set amount. At the limit of the strokes capacity, the jack retracts, advances itself forward along the track and resets for the next sequence.

In the second jack-based system, sliding occurs via an air pad. Best described as a hovercraft or "air hockey" game, the jack rests on a cushion of pressurized air resulting in very low frictional resistance. A launching jack pushes the structure forward, and this too results in the jack resetting itself for the next series of moves.

Finally, moving structures by using the French Autoripage system is discussed. In this concept, structures slide on a concrete sleeper mat on soils that range from excellent to poor. The new bridge is built on top of this mat. When the bridge has to be moved, a pressure grout layer of bentonite is injected between the bridge foundation system and sleeper system.

Project Title: Lateral Sliding of Oakland Bay Bridge Approach Spans
Citation: Warta, 2007
ABC Design Features:
ABC Construction Features: Lateral skid shoe system for rapid bridge installation

Project Description

Following demolition of the existing approach spans, a new bridge was jacked laterally into place with the Mammoet skid shoe system over the course of a Labor Day weekend closure in 2007. The new bridge measures roughly 350 ft long by 92 ft wide and weighs more than 7,100 tons. Once the road closure was established, demolition of the bridge deck was completed in several hours, followed by demolition of the existing bridge columns and site clean up. Once the site was turned over to the mover, a series of skid rails were installed to extend those already in place under the new bridge section. The new bridge was jacked up off its temporary supports by skid shoes and integrated jacks, with a capacity of 660 tons each. A total of 16 jacks/shoes were used. The skid shoe system incrementally locks itself into the track, extends the jacking cylinder, advances the locking mechanism, and repeats the process. The lateral shift of the span was completed in a little over 2 hr, and the bridge work was completed 11 hr ahead of schedule.

ABC Opportunity

This project demonstrates the unique capabilities of high-capacity skid shoe systems. A bridge of considerable length, width, and height was moved laterally as a complete unit. The skid shoes allow for synchronized hydraulic elevation control of the entire structure during the move, and the incremental ratcheting movement of the jacks provides steady and consistent movement of the bridge. This technique is ideally suited for an installation like the Oakland Bay project where working room is readily available alongside the existing bridge. In that case an entire bridge can be built and, when the road closure is established, simply lifted from its temporary bents and moved laterally. The skid shoe system does not allow for turning structures, so its use is limited to linear moves.

Project Title: Lateral Sliding of Bridge on Cantilever Brackets
Citation: Hebetec Engineering, Ltd., 2008
ABC Design Features:
ABC Construction Features: Lateral movement system for rapid bridge installation

Project Description

This project involves an innovative method for replacing small to medium structures. In this example, a small bridge weighing 240 tonnes is replaced with a similar structure weighing 360 tonnes. The structure is over a small river or canal. A self-reacting framework is built around the existing wall pier.

In the usual situation, the grillage would be erected beneath the pier cap(s) without any interruption to traffic above and minor or no disruptions to traffic likely below the bridge. With nighttime or off-peak closures, materials for the new bridge would be delivered by using the old structure as the delivery platform. Various forms of prefabricated elements could be employed to speed construction of the new bridge. One issue is the asymmetric loading of the pier. For large-wall piers, this type of loading may not pose a problem since their string axis strength is usually many times the demand. For narrower piers, such as typical hammerheads or single round- or square-column narrow piers, an assessment of the pier should be done with greater care. Very likely, the pier could not accommodate the highly eccentric loading. In that case, the tip of the cantilever brackets could be supported directly with a vertical prop or shoring tower. That would relieve any bending moments from the system during erection and lateral sliding. The outboard cantilever should also be reinforced in anticipation of movement of the old bridge onto the falsework.

This system would likely be constructed with common materials that contractors typically have on hand. The efficiency of the materials being used in direct tension or compression allows for shapes such as H-pile (HP) sections and salvaged steel sections to be used, so a cost-effective falsework system is likely available. If needed, commercially available adjustable shoring elements could also be supplied. For additional safety and load carrying capacity, the top tie element could be easily prestressed with high-strength steel rods (Dywidag or similar) to relieve some tension force from the system. However, this would generally not be required and would likely add cost without much benefit.

Once ready for moving, the concept is easily implemented. The existing bridge is lifted several inches on jacks resting on the transverse girders. It is moved laterally with center hole jacks connected to high-strength bars or by strand jacks attached to a temporary multistrand tendon. Movement of the existing bridge and new bridge should be synchronized by linking the two systems, which is easily done.

ABC Opportunity

This system can be used in unique circumstances where removal of the old bridge by SPMT or skidding system is not possible yet a rapid removal is desired. One instance is a project analogous to the example bridge: over water but not deep enough or otherwise conducive for barge access. In the example described here, an internal reaction frame is used to carry the structural load. The existing piers must be checked for their capacity to serve as part of the removal and sliding system. Once the old bridge is relocated, it can be disassembled and removed from the project site in numerous ways.

Project Title: Forges Les Eaux, French Bridge Displacement Using the Air Pad Sliding System
Citation: Freyssinet, 2006
ABC Design Features:
ABC Construction Features: Rapid lateral movement of a three-span bridge by using air pad sliding

Project Description

The Forges Les Eaux bridge in France was moved laterally with the Hebetec Air Pad Sliding (APS) system. Best described as analogous to hovercraft technology, the APS system uses a cushion of trapped air under heavy lift jacks to provide a very low coefficient of friction for moving heavy structures. Originally conceived and prototyped for use in shipyards and heavy load moving for industrial uses, the APS system also has many advantages in civil construction.

With a weight of 1,300 tons, this bridge was a significant load to be moved. The project required that the bridge be shifted laterally a distance of 45 m. Because of the low coefficient of friction of the movement system and quick cycle times of the small hydraulic jacks, it was moved at a rate of 20 m per hour, or a total movement time of just over 2 hours.

The sliding system has modular components that are arranged as necessary to execute the required move. A total of eight APS units were provided for this lift, each with a vertical capacity of 3,850 kN and a jack stroke range of 280 mm. Coupled with these APS sliding jacks are standard push–pull units, each with the ability to extend the jack and then retract, ratcheting themselves forward down the track. A total of four 320-kN capacity jacks were used with a stroke capacity of 1,250 mm. These small jacks are sufficient to move this heavy structure because the low moving friction rate is less than 1%.

ABC Opportunity

This project, using instead an air pad “frictionless” system to move the completed bridge, demonstrates an alternative to conventional skid shoe systems. Hydraulic jacks lift and control the vertical elevation of the bridge, and small jacks push the bridge along the slide tracks. Thus, the system is relatively inexpensive to mobilize and requires only a firm footing for the slide tracks themselves.

Project Title: Autoripage Bridge Erection Methods
Citation: Freyssinet, 2005
ABC Design Features:
ABC Construction Features: Lateral sliding of fully completed bridges into place

Project Description

The use of structural sliding can be applied to complete structural systems including bridges with complete foundations and pier elements. The Freyssinet Autoripage system is one such method of rapidly moving completed structures over large distances and over marginal soils.

An example of this technique includes the replacement of a high-speed railway bridge in Saint Cheron, France. Given the traffic density on the rail line, bridge replacement was not viable unless completed in an accelerated manner. The decision was made to cast the new bridge next to the old one. In the Autoripage system, a concrete sliding slab is first cast under the entire project site. This concrete is not structural; it is there to level the site and to provide a sliding surface. Once the slab has cured, construction of the new bridge can commence. The innovation lies in the method of movement. When the bridge is to be moved, a layer of bentonite is pressure injected between the sleeper slab and the slab that connects the various foundation elements. The two concrete surfaces are lubricated to facilitate sliding; and a reaction block, located at the final bridge location, and strand jacks are used to pull the bridge into place. This bridge weighed 2,400 tons with a total length of 35 m. Three 1,000-ton jacks were used along with a strand jack to move the bridge into place. Speeds up to 13 mph have been attained with this method. For the Saint Cheron bridge, the total time for movement was 5 hours, including a substantial amount of time for excavators to remove the required embankment materials to allow the new bridge to fit in the prior opening. Because of the large sliding surfaces between the bridge and sleeper slab, very low contract pressures are applied. For this bridge the pressures were less than 10 psi or ¾ tsf. This makes the Autoripage system well suited for construction of bridges where poor soils are present.

ABC Opportunity

The Autoripage system is a unique method of sliding heavy loads particularly in instances where the soil conditions are poor and might limit the use of other removal methods with higher contact pressures. The system is unique in its use of a large "sleeper slab" cast under the entire construction site. Strand jacks are synchronized and used to move the load across the slickened surfaces. The process is similar to the use of skid shoes and other lateral sliding techniques but appears best suited for marginal soils locations

Incremental Launching

Bridges have been constructed with the incremental launching method (ILM) for more than 100 years. In this method of construction, the bridge superstructure is assembled on one side of the obstacle to be crossed, then pushed longitudinally—or launched—into its final position. The launching is typically performed in a series of increments so that additional sections can be added to the rear of the superstructure unit before subsequent launches. The launching method has also been applied to tied-arch or truss spans, although those are fully assembled prior to launching.

The incremental launching method will never become the most economical procedure for constructing all bridges, yet it has unique advantages for complex site conditions. The ILM requires considerable analysis and design expertise but not very sophisticated construction equipment. However, the ILM may often be the most reasonable way to construct a bridge over an inaccessible or environmentally protected obstacle.

When used for the appropriate project, the ILM offers significant advantages to both the owner and the contractor. These advantages include the following:

- Minimal disturbance to surroundings, including environmentally sensitive areas;
- Smaller but more concentrated area required for superstructure assembly; and
- Increased worker safety, since all erection work is performed at a lower elevation.

The ILM can be used to construct a bridge over a wide range of challenging sites that feature limited or restricted access, including those with the following characteristics:

- Deep valleys or water crossings;
- Complex urban environments (complex road and railway facilities);
- Steep slopes or poor soil conditions, making equipment access difficult; and
- Environmentally protected species or cultural resources beneath the bridge.

Estimates indicate that more than 1,000 bridges have been constructed with the incremental launching method worldwide over the last 50 years. Despite its advantages, however, this method of construction has seen limited application in the United States. The Po Bridge in Italy was an early precast structural bridge system that used incremental launching (Gentilini and Gentilini, 1974). As noted, the list of bridges that have used this construction technology is too long to provide here, so some selected literature is provided for moderate- and relatively long-span bridges, including multiple precast concrete bridges in Hong Kong (Tung et al., 1988), the Kap Shui Mun Bridge in Hong Kong (Lau and Wong, 1998), the Donghai Bridge in mainland China (Ren et al., 2005), and the Pingsheng Bridge in mainland China (Cheng, 2006).

Applicability and Limitations of Incremental Launching

During the launching of a bridge, the superstructure acts as a continuous beam supported on roller or sliding bearings and is transversely restrained by lateral guides that prevent drifting movement. Any constraint eccentricity (vertical misplacement of launching bearings or transverse misalignment of lateral guides) will cause unintended secondary stresses and may cause launching problems such as excessive wear of bearing devices.

The case studies presented herein highlight the fact that incremental launching is applicable to a wide variety of challenging bridge sites. The recent FHWA PBES scanning tour of Europe and Japan identified a number of bridge launching projects for which launching was considered the most efficient solution to a difficult bridge construction problem.

Ideally, a bridge intended for incremental launching would be designed along a tangent alignment in both horizontal and vertical planes to simplify fabrication and construction. However, bridge sites rarely fit those ideal conditions. Although somewhat more challenging, it is possible to construct a bridge by incremental launching while maintaining a curved alignment in either or both planes. To eliminate the relocation and adjustment of lateral bearings, however, those surfaces must remain perfectly aligned with the superstructure during launching operations, and that can only be guaranteed in the case of a common geometry. Rosignoli (1998b) states that a bridge constructed by launching must be designed with one of the following alignments:

- Tangent in plan and tangent or circular in profile;
- Circular in plan and horizontal in profile (no launch gradient);
- Circular in plan and inclined with respect to the horizontal plane; or
- Curvilinear both in plan and in profile.

The geometry of curved structures and the desire for uniform distribution of launch stresses strongly favor the use of constant-depth superstructures such as a parallel flange I-girder. A variable-depth steel superstructure can be used, with temporary steel plate or trussed extensions of the bottom flange. A variable-depth superstructure is greatly complicated by the higher dead load present during launching operations.

Structural Monitoring During Construction

The use of structural monitoring during construction of an incrementally launched bridge has received considerable attention from both owners and university researchers. Structural performance information through monitoring can supplement visual observations and may provide critical alerts during the launch stages at structure locations during the launch process. It can also provide validation of the design and construction process, which is useful for implementation of subsequent ILM projects.

Project Title: US-20 Iowa River Bridge

Citation: LaViolette and McDonald, 2003; LaViolette, Wipf, Lee, Bigelow, and Phares, 2007

ABC Design Features: Innovative use of incremental launching with steel I-girders

ABC Construction Features: Erection by incremental launching to satisfy multiple environmental and geotechnical constraints

Project Description

The bridge consists of two parallel deck superstructures, each with five equal spans of 302 ft. A 62-ft prestressed concrete jump span is provided on each end of the steel unit. The bridge was constructed by using the incremental launching method because of a number of stringent environmental restrictions near the project. The environmental issues included endangered mussel species residing in the Iowa River, endangered plant species near the site, and Native American artifacts near the site. In addition, a bald eagle roosting area was identified near the site. An extensive environmental monitoring program was established and maintained during construction.

To make the I-girder superstructure act as much like a torsionally rigid box girder as possible during launching, a stiff system of diaphragms and lateral bracing was used. A diaphragm spacing of 23 ft was used for Spans 2 through 5, but that was reduced to 11 ft, 6 in. in the leading span that would be cantilevered during launching. The I-girders were fabricated from ASTM A709 Grade 50W steel; they are 11 ft deep and spaced at 12-ft centers, and the constant ⅞-in. web thickness was designed as unstiffened for steel dead load.

The bridge superstructure was completely erected on steel falsework and custom-made 18-in.-diameter rollers behind the east abutment. A 146-ft-long, tapered steel launching nose was erected at the leading end of the girders and used to reduce the cantilever deflection during each launching operation. After each span was launched forward, additional steel girder sections, including diaphragms and bracing, were pushed forward to land on the subsequent pier. The process

Figure A.17. Incremental launching of US-20 bridge; span at rest pier location.

Figure A.18. Tail section and tugger beam.

was repeated five times for each steel superstructure. After the complete launching of the eastbound girders, the falsework was removed and reinstalled to perform an identical launching of the westbound superstructure.

ABC Opportunity

This project illustrates the ability of a determined owner, designer, and contractor to construct a unique bridge to solve challenging environmental concerns. Although the project did not specifically intend to advance the ABC agenda, it provides evidence that incremental launching can be successfully applied to larger, more complex projects.

Project Title: Tiziano Bridge Launch and Shift
Citation: Rosignoli and Rosignoli, 2007
ABC Design Features: Uses proven bridge system of precast, posttensioned box girders
ABC Construction Features: Rapid construction of complete bridge systems by using innovative combination of proven bridge movement concepts of incremental launching and transverse shifting

Project Description

This project involved the longitudinal launching and transverse shift of a new bridge across the Tanaro River in Alessandria, Italy. Initial plans called for a 656-ft-long, 55.4-ft-wide bridge comprising cast-in-place concrete constructed by using a movable formwork system. The Tanaro River is prone to sudden floods, however; and the girders supporting a movable formwork system would have significantly diminished the available hydraulic section during construction. A value engineering approach was presented that included the following:

- Construction of a 663-ft-long, 26.7-ft-wide box girder by incremental launching;
- Transverse shifting of the first box girder by 28.7 ft until it reached its final location, thus clearing the launch alignment for the construction of a second box girder;
- Incremental-launching construction of the second box girder; and
- Joining of the twin box girders with a cast-in-place central closure curb.

This proposal was accepted, and the bridge was constructed in 2001 using this innovative technique.

Although launching the bridge required some specialized construction equipment, the components were inexpensive and were limited to the following:

- A 184-ft-long foundation beam for the casting yard;
- A 39.4-ft-long steel launching nose;
- Four temporary piers constructed on single foundation shafts; and
- Steel launch saddles for the transverse shifting of the deck.

Notably, although this project was constructed on reinforced-concrete temporary piers, construction of future bridges on smaller, reusable steel falsework towers may be possible.

Compared with the cost of a movable formwork system, the cost savings were significant. Additional savings resulted from avoiding the transportation, assembly, use, and final dismantling of the moveable formwork. Avoiding disruption of the hydraulic section of the river also proved especially important, as two floods reached the 200-year return level during construction. Casting the bridge in a highly repetitive series of 48 segments produced highly efficient construction by allowing iron workers to work independently of the carpenters.

Casting the superstructure on a rigid foundation made setting the form geometry easy because precamber was not required. The rigid foundation also minimized formwork deflections during concrete placement and therefore helped avoid formation of cracks in the setting concrete. Both precamber and cracking would have been expected for a bridge constructed by using a movable formwork system because casting an entire span takes several hours and the progressive deflection of the formwork affects the setting concrete.

The use of temporary launch piers reduced the number of launch tendons and increased the number of continuous service tendons for a more efficient final prestressing scheme. In turn, transverse deck shifting allowed the use of a single set of temporary launch piers and a single foundation beam for

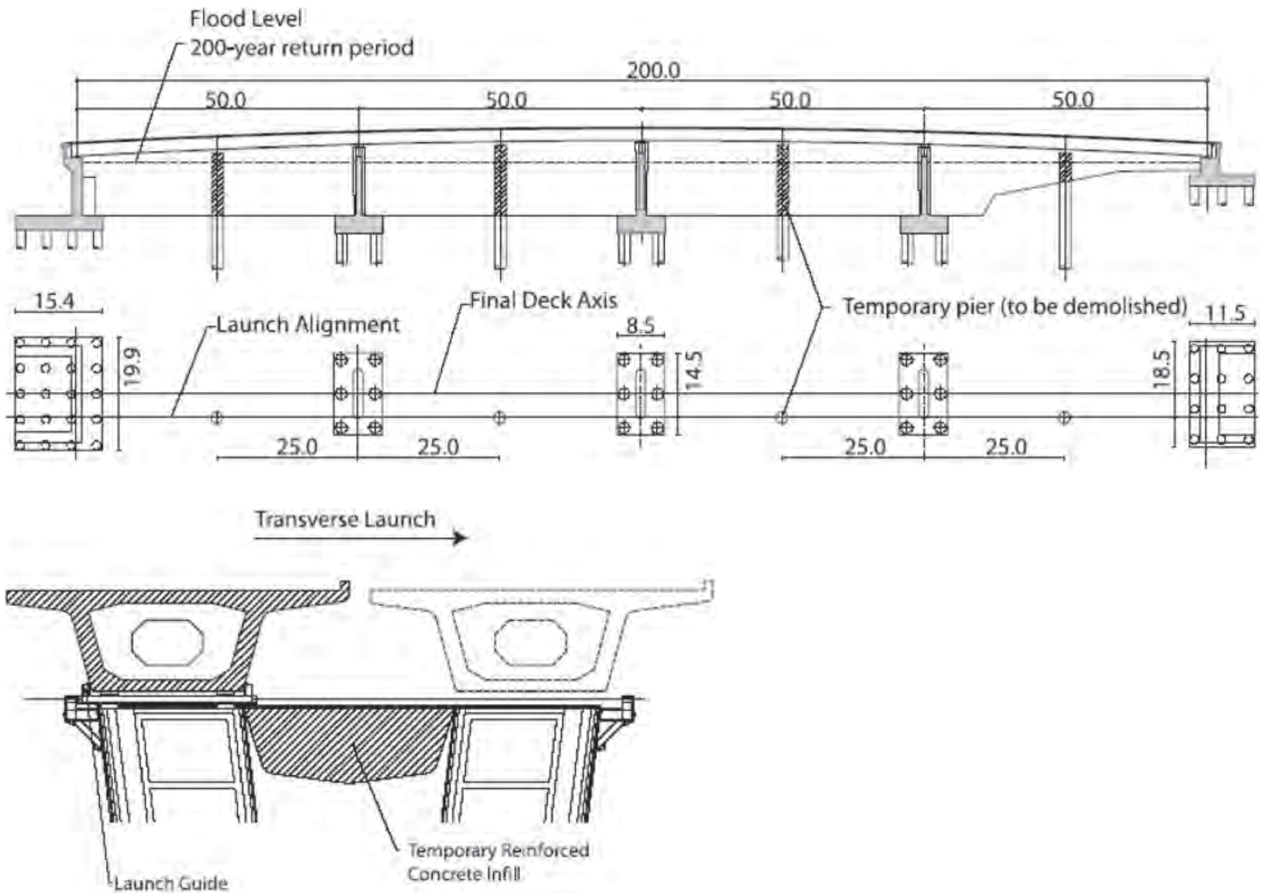


Figure A.19. Tiziano Bridge elevation and plan views.

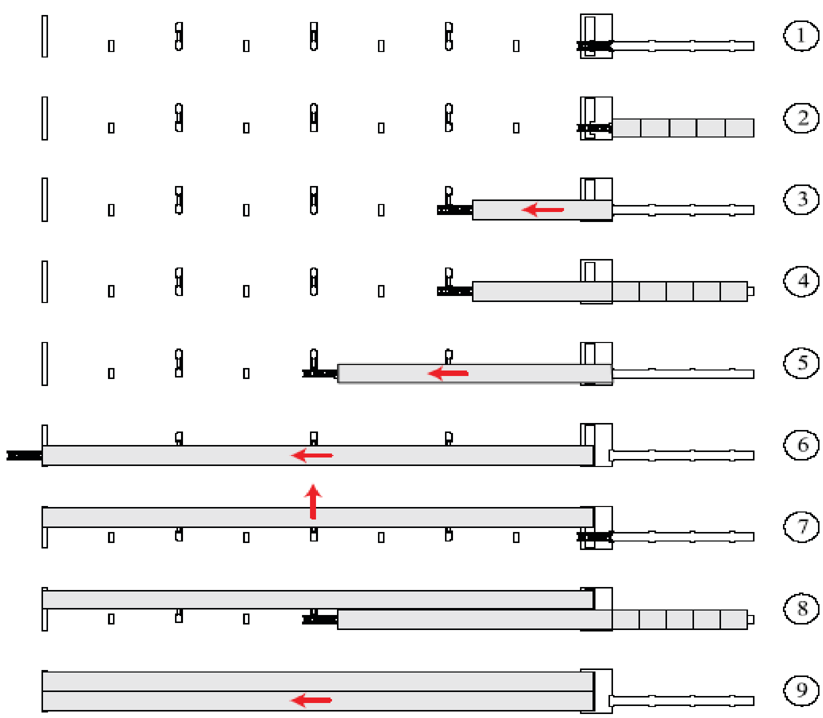


Figure A.20. Sequence of launching and sliding operations.

both box girders. Because of the numerous advantages, the owner quickly approved the value engineering proposal.

ABC Opportunity

This project illustrates how two proven bridge movement techniques—namely, incremental launching and transverse shifting—can be combined in an innovative way to construct a complete bridge superstructure to suit complex site challenges. This concept could be utilized in a much smaller context and constructed with readily available jacking and roller equipment to construct a pair of bridge superstructures on a combined substructure with minimal effect on the traffic below.

Project Title: Reggiolo Bridge Launching
Citation: Rosignoli, 2007
ABC Design Features: Uses bidirectionally prestressed and posttensioned concrete waffle slab
ABC Construction Features: Construction of a very short bridge by using monolithic launching to avoid interference with obstacles below bridge

Project Description

This project involved the launching of a small prestressed concrete deck plate bridge that spans the Verona–Mantua Railway in Reggiolo, Italy. The railway had to remain fully operational during construction, which would have made a more traditional bridge difficult to construct because of limited locations for falsework.

The superstructure consists of a multicellular prestressed concrete plate spanning 85.3 ft. The bottom surface is horizontal in the transverse direction and slightly inclined longitudinally; the top surface is inclined in both directions to shed water. Thus, the total thickness of the structure varies slightly. In the transverse direction, the plate is stiffened by three 15.7-in.-thick internal diaphragms spaced at 21.3 ft and two 3.3-ft-thick end beams at the abutments. The prestressing level in the superstructure was quite high because of the railway authority's request for a fully prestressed section in both the longitudinal and transverse directions under full design loads and also during launching. The prestressing included longitudinal launch tendons in the internal webs and edge beams, longitudinal tendons in the bottom slab,

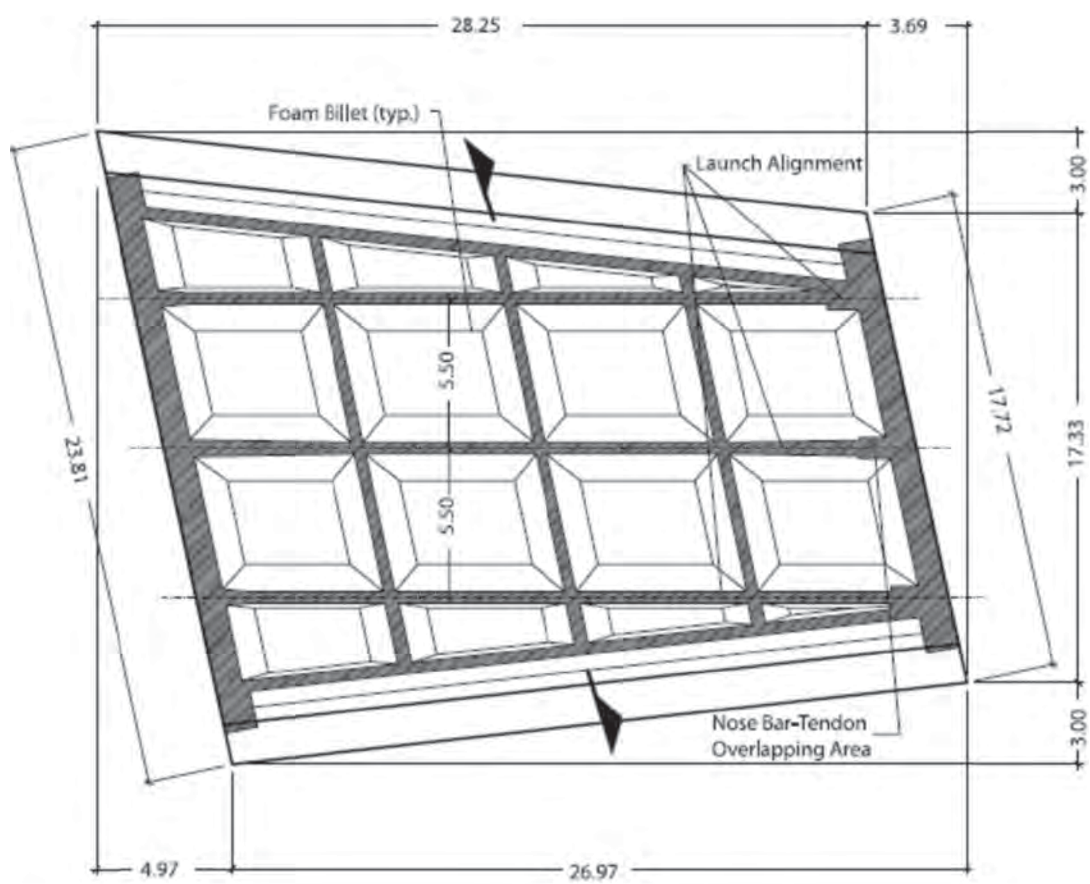


Figure A.21. Plan view, Reggiolo Bridge.

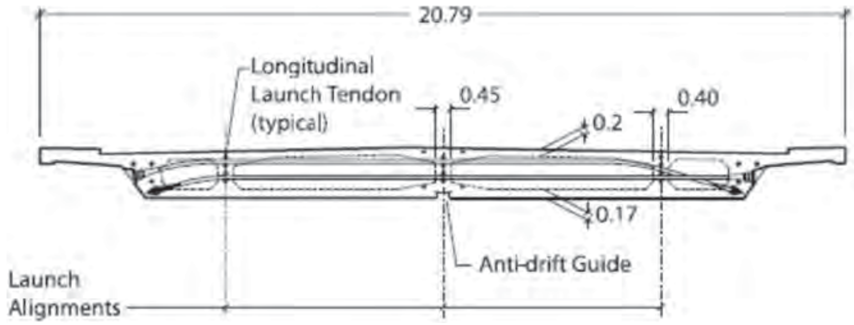


Figure A.22. Reggiolo deck section.

transverse tendons in the end beams, and internal diaphragms and transverse tendons in the deck slab.

Three disposable prestressed concrete launching nose girders were installed at the front of the deck to control negative moments by shortening the cantilevered span and to limit the risk of the deck overturning. High negative moments were alleviated by cutting the launch span in half by using a temporary steel pier placed near the railway.

Because of its varying width, the deck alignment could not be maintained with conventional lateral guides during launching. Instead, guide pins at the rear of the deck and at the launch abutment were used. The pins guided the bridge through a steel-lined recess in the underside of the deck. At the temporary pier, the central guide was used to laterally stabilize the pier through the deck.

The results of launching and service load analyses showed that positive moments in the longitudinal webs were governed by service conditions, while launching governed negative moments (which would not have existed with conventional deck construction on shoring). The use of hydraulic launch supports in the casting yard—instead of conventional continuous low-friction extraction rails—generated substantial savings. And the launch equipment was particularly inexpensive. Because of the bridge's modest weight—about 1,000 tons—a pair of small long-stroke launch pistons generated adequate operational speed at minimal cost. Constructing the bridge in this way—compared with erecting and dismantling the bridge on shoring, plus the higher embankments required for working clearance above the railways—generated about 12% overall savings.

ABC Opportunity

This project illustrates how two proven bridge movement techniques—incremental launching and transverse shifting—can be combined in an innovative way to construct a complete bridge superstructure to suit complex site challenges. This concept could be used in a much smaller context and constructed with readily available jacking and roller equipment to construct a pair of bridge superstructures on a combined substructure with minimal effect on the traffic below.

Project Title: Bridge Launching in Milan, Italy
Citation: Rosignoli, 2008
ABC Design Features: Uses proven bridge system of precast, posttensioned box girders
ABC Construction Features: Incremental launching over heavily traveled rail line without suspending traffic

Figure A.23. Small casting yard serviced by conventional equipment.

Project Description

This project involved the longitudinal launching of three prestressed concrete bridges in downtown Milan, Italy. The location required that the bridges be constructed over six electrified railroads, and the project represented one of the most complex applications of ILM construction in Italy.

Site constraints, including road and tramway junctions on either side of the bridge and difficult maintenance conditions, and the architectural requirements of urban bridges complicated the project. Reduced clearances and the operational requirements of six electrified railroads suggested construction by incremental launching, as this method is particularly suitable for crossing sensitive obstacles.

The road bridges comprised 18.0-m, 49.0-m, and 26.5-m continuous spans and their three-cellular section is 13.5-m wide. The base of the eastern approach embankment was erected, an off-shelf formwork was placed onto it, and the first road bridge was cast and incrementally launched over the railroads (Figure A.24). After diverting traffic onto the first road bridge and demolishing the old Palizzi Bridge, the eastern embankment was widened southward, and the second road bridge was built. The entire project—including utility relocation, the construction sequence for the bridges, and the demolition of the old bridge—was designed to reduce interference with train traffic (>300 trains a day) and heavy road traffic. Parallel activities were strictly coordinated, and the completed structures were immediately used to open new sites for the next activities.

Launching the road bridges over the long central span required the use of a temporary pier in conjunction with a launching nose. The temporary pier was cable-stayed to the rear abutment to diminish the bending moment generated by launch friction in the narrow foundation block. The launch operations for the first road bridge were initially limited to a 2-hr work window each night. Initial launching operations proceeded so smoothly and safely that eventually the railroad authority allowed launching to proceed without any restrictions on the activities and the train traffic. After the first launches, the electric catenaries were kept energized during launching operations.

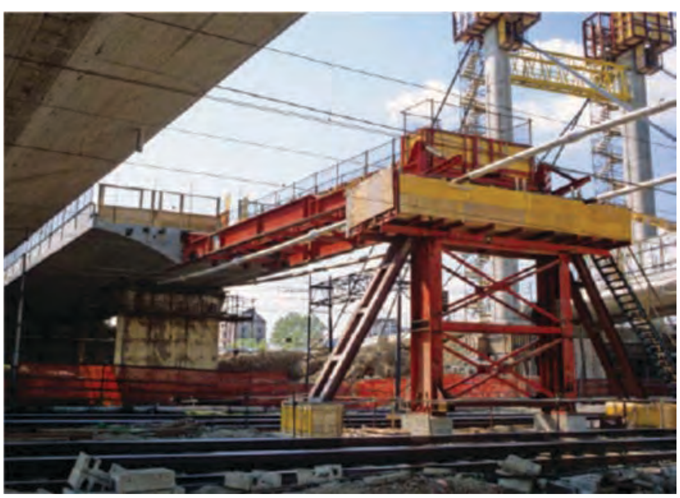


Figure A.24. Incremental launching and use of temporary piers.

ABC Opportunity

The project demonstrates the value of incremental launching for crossing areas that severely limit construction access. In this case study, the railway authority allowed construction to continue without any delay to rail traffic. The applicability of incremental launching to short bridges is also validated.

Other Movement and Construction Techniques

Other techniques are described here. Some are proven techniques used in the past for larger structures (e.g., overhead gantry systems), while others, such as beam launching, are not frequently used in the United States. Several other methods of bridge erection are also discussed, including the combination of gantry and SPMT technology and a few innovative techniques. Several international articles worth noting include the use of land beam transporters to deliver very large precast components for the construction of major bridge crossings (Liu, 2009; Lu et al., 2006).

A significant amount of published literature exists on the use of heavy-lift cranes for moving prefabricated bridges (or components) into place. One example is the replacement of a high-speed rail bridge in Scotland. The project required ABC techniques because the owner specifications required that the existing bridge be removed and a new structure placed during a single 5-day closure. Several sliding options were considered, and the contractor and designer investigated the feasibility of using large mobile cranes (primarily to reduce temporary construction in the river). The replacement structure is a half through plate girder bridge. The main girders were approximately 6 ft deep and spaced at approximately 13 ft. Once on site, the superstructure was assembled into four sections varying in weight from 120 tons to 170 tons. The individual units were placed by using a 1,000-ton mobile crane (Hackney et al., 2002).

Project Title: Beam Erection Using Mobile Overhead Truss
Citation: VSL Singapore, 2007
ABC Design Features:
ABC Construction Features: Rapid erection of superstructure components

Project Description

Rapid erection of superstructure components is made possible with the use of a small gantry system. This type of system is ideal for crowded urban environments where occupying existing ground with construction equipment such as cranes would have a severe impact on traffic flow. This design feature was developed in the 1980s and involved the construction of an elevated freeway in Jakarta, Indonesia. Beams were erected over live traffic; the road below was occupied for only the several minutes needed to deliver the beams.

The gantry rests on two piers at any one time and has a crane bridge with winch lifting system attached to it. The gantry travels to the lateral extents, picks a new beam from the delivery truck, lifts it to the proper setting height, and then traverses laterally to the correct beam line position. Setting rates in excess of one span per day are possible. The cycle time of the gantry controls the speed of erection. Once a span is set, the truss launches forward to the next span, rolling on top of the existing legs. This process repeats.

ABC Opportunity

This construction method could become viable and popular for routine viaduct construction. In a recent project in Tampa, Florida, a reversible high-occupancy-vehicle bridge was built with similar site constraints, down the middle of an existing roadway. However, that bridge was made of box girders. This gantry system would allow for conventional steel and concrete multi-beam bridges to be built with industrialization. The truss would not be nearly as expensive as those used for segmental bridges since the level of sophistication is less. However, a significant investment, likely several million dollars, would be required for such a system. This system could be broken down and reused many times for similar urban viaduct construction or for situations in any environment where work restrictions below the bridge present a complication.

Project Title: Full-Span Launching of Concrete Box Girders
Citation: Beijing Wowjoint Machinery Company, 2008
ABC Design Features:
ABC Construction Features: Highly automated construction of long viaduct structures

Project Description

The use of full-span launching has been applied successfully on a number of recent projects in Asia. Among these are various highway and rail expansion projects in Korea and China. One example is the erection of full-span concrete boxes for the Beijing–Tianjin High Speed Railway. For this project, approximately 75 miles of new track work, largely elevated viaduct, was constructed to support trains operating in excess of 215 mph. The solution was to establish multiple casting yards along the line and, by using multiple gantries, begin construction simultaneously. The system includes precast/posttensioned box girders (35-m spans, 900 tons) that are cast in a yard and then delivered via SPMT down portions of the already completed viaduct to the new span location. The spans are then received by the gantry, lifted forward and down to their final bearing locations, and then the gantry rolls across the newly completed span and establishes its new position. On average, two to three spans were completed in a given work shift for this project.

ABC Opportunity

This equipment affords the opportunity to radically change the construction of longer viaduct structures. According to the manufacturer, these gantry systems cost on the order of US$3 million in 2008, so the project or opportunity must be large to capitalize that cost. However, the system is not limited to a single use and can be easily collapsed and used for other projects. Large contractors might invest in these types of systems for projects such as urban viaduct reconstruction where the equipment could just as easily be used for large-scale demolition as for new-span installation. A similar opportunity would exist for over-water crossings.

The Florida DOT has conceptualized a similar concept to use half-width redecking as a reconstruction solution for urban interstate projects. The department communicated its interest in such a system to the R04 team and has made conceptual estimates of two to three spans per day, or an average of 1 mile a month of completed viaduct. The hypothetical project is the construction of a "managed lanes" facility, such as a tolled high-occupancy-vehicle facility in the median of an existing facility. Using top-down construction would allow for SPMT delivery of new spans and the gantry to place them in halves. A similar new bridge construction project is under way in North Carolina (the Washington Bypass Project) where top-down construction is being used to drive prestressed pile bents, erect bent caps, and set the beam lines; the decks are cast in place. An average of one completed span per week is being attained.

This concept is not relegated to heavy element delivery. For years, individual beam lines have been erected with this type of delivery system in much smaller systems. Beam launchers are used to set individual beam lines or possibly smaller decked beam sections, and capacities of these systems are up to 200 tons. This concept could easily be deployed for rapid erection of new multi-span bridges; in some cases it might even be useful for shorter spans to aid bridge removal by lifting large sections from above and moving them out of the way. A short or medium bridge gantry system is considered viable as a reusable and flexible ABC delivery system.

Project Title: Overhead Gantry Lift Method
Citation: Mi-Jack Products, Inc., 2007
ABC Design Features:
ABC Construction Features: Low-impact construction with highly adaptable and customizable equipment

Project Description

An underutilized technology is the use of travel lift cranes commonly found in fabrication yards but not often used for on-site bridge construction. A common situation is infill construction in the median of an existing roadway. This work can be complicated from below because of crane placement issues or challenges in lifting and placing materials, or because of access constraints such as waterways, environmental constraints, etc. The travel lift can run on rubber tires or rails and be supported on a narrow strip of existing bridge deck, such as the shoulder area. Several example projects are cited to demonstrate the use of this system.

Route 46 over the Overpeck Creek is a busy thoroughfare in northern New Jersey and is a major feeder route to the busy George Washington Bridge. The superstructure required complete reconstruction. The estimated cost of a conventional replacement was $20 million with a 3-year schedule. That was deemed unacceptable. The New Jersey DOT estimated $19,000 per day in user costs for this construction project (Keith, 2006).

One of the first solutions that helped compress the schedule was to insist on prefabricated superstructure units in lieu of cast-in-place concrete. Complete new beams and decking were furnished for the project in sections measuring 9 ft, 4 in. wide by 93 ft long and weighing 70 tons. The contractor determined that a crane large enough to erect those sections would take up too much room and was unsure if the existing bridge, in its deteriorated condition, could support the weight of the cranes plus the lifted loads. The solution proposed by the contractor, Railroad Construction, was to erect a longitudinal runway system and use an overhead gantry crane for erection. In that configuration the gantry could run the entire length of the bridge, and the contractor could work in two closed lanes 24 hr a day while traffic flowed in the two lanes remaining open. The project cost $18 million and was completed in less than a year.

A similar positive example is the reconstruction of I-77 in Virginia over the New River. The project involved building a new superstructure in the median between two existing bridges. Each bridge measures 1,800 ft long and rises as high as 140 ft in the air. With the presence of existing bridges, construction from below was difficult. Even if possible, a 140-ft vertical lift of heavy sections would have required a large crane mounted on barges that would have to have been constantly repositioned to erect the various spans. As a solution, the contractor elected to use a 70-ton capacity Mi-Jack mobile lift crane. The crane was used to erect all the structural steel in the bridge.

ABC Opportunity

The use of overhead gantry cranes provides substantial flexibility for erecting and dismantling structures that would be more difficult to erect with traditional cranes because of operating constraints and space limitations. Using an existing bridge as a work platform (or constructing a runway system independent of the bridge) allows the gantry to run freely throughout long stretches of the job site, delivering materials and completed portions of a bridge. Conversations with gantry suppliers indicate that only a small fraction of bridge contractors use these methods compared with the perceived available opportunity. Gantries of various capacities and sizes are available for lease or purchase, and availability does not appear to be a limiting factor on use.

APPENDIX B

Focus Group and Survey Results

First Owners Survey Results

The owners survey was created to gain insight into the successful practices of bridge owners engaged in accelerated bridge construction and to learn about the challenges faced by bridge owners who have not been successful with ABC. The survey contains 40 questions that focus on ABC goals, practices, experiences, hindrances, and opinions. The advanced tools of the survey allowed questions to be displayed or hidden depending on the respondent's answers, so only the questions deemed applicable were answered.

- 38 completed surveys were submitted by officials from many agencies throughout the United States,
- 68 surveys were abandoned, and
- 24 were partially completed.

The questions in the owners survey were created to provide information on several aspects of an agency's experience and views of ABC. The survey consists of four pages, with the questions on each page geared toward soliciting information about a specific aspect of ABC. The first page serves as an introduction and presents questions designed to gauge the agency's overall view of ABC. If that agency has experience with ABC techniques, a series of questions are presented to determine both the type of ABC elements with which the agency has experience and whether or not the agency would consider using those elements again. The second page contains questions about the agency's goals for ABC, what aspects are most appealing, and whether the agency is engaged in research or looking for opportunities to use ABC. The survey also asks agencies to evaluate their readiness to implement various ABC techniques to determine what might be keeping agencies from using ABC. The third page asks respondents to describe a specific project that they may have completed using ABC techniques. Those responses were used for follow-up purposes: Rather than using them to learn more about an agency's views on ABC, they were used to find previously unknown projects that could provide useful information on innovative techniques and practices. The fourth page provides space for respondents to describe any ABC practices they feel might be important to this study as well as any additional feedback regarding the survey. Respondents were required to provide contact information for follow-up, although the team did ask for their permission to contact them for additional information.

Owner ABC Implementation

The first series of questions pertain to the current level of experience, attitudes, and familiarity with ABC as a project delivery method. Though U.S. deployment of ABC is not nearly as common as it is in Europe, many state DOTs have some experience with certain aspects of ABC, such as prefabricated elements, complete bridge movement, or innovative contracting methods. Thus, 73% of the agencies responded that they had experience with some aspect of ABC delivery.

When asked about their agency's disposition toward ABC, the following results were obtained:

- 82% of respondents indicated an impetus to implement ABC, suggesting that many agencies are looking for ways to enhance the speed with which bridge projects are delivered. Respondents reported that DOT executives, department heads, staff, consultants, and fabricators and suppliers are very supportive or supportive of ABC implementation.
- 95.4% of DOT executives, 88.7% of department heads, and 73.3% of staff are very supportive or supportive of ABC implementation.

Those results suggest that support for ABC is highest at the executive levels but is generally present at all critical levels of the organization. That implies that DOT staff, contractors, consultants, fabricators, and suppliers would benefit most from ABC information sessions, as the lack of information on ABC has been identified as a hindrance for many agencies. When states lacking experience in ABC methods were queried, the top reason cited for unsuccessful implementation of ABC was a lack of funding. One respondent cited a lack of appropriate opportunities for ABC implementation. Given that ABC methods are new to many agencies, respondents were asked if they are currently involved in any aspect of research related to ABC deployment; 43% responded affirmatively.

Owner Project Experiences

The owners were asked a series of questions about various types of ABC solutions they have tried. In particular, the questions focused on prefabricated solutions, accelerated construction techniques, contracting methods, project selection, and selection criteria. Depending on their responses to those questions, owners were also asked whether the ABC applications were successful and whether they would use such solutions again. This portion of the survey included numerous data-laden questions to help the team gain a general understanding of the experiences of various owners. Abbreviated responses follow here.

When asked to select the types of ABC elements with which their agency had experience, respondents gave varied answers since a large number of choices were provided. This indicates that owners do not necessarily agree that only certain types of solutions are suitable for accelerated construction. In general, the strongest conclusion relates to project satisfaction and potential reuse. Owners expressed a high degree of satisfaction with completed ABC projects and a strong likelihood to use the techniques again.

In questions about why they used ABC techniques, respondents were asked to rate the importance of various criteria: traffic disruption mitigation, environmental impact mitigation, public safety, worker safety, enhanced durability, lower life-cycle costs, and public satisfaction. Most of these were cited as very important or important considerations in choosing to implement ABC. Respondents were not asked to rank the various criteria.

- Environmental impact mitigation was least likely to be cited as very important or important but still earned a high response rate of 64%.
- The criterion most likely to be considered very important or important was traffic disruption mitigation at 98%.

Agencies were queried about their project selection process. Roughly three-quarters of the respondents said that they do not have an identification system to select projects that are well suited for ABC implementation. Among the roughly one-quarter of respondents who do have such a system, it is used mostly in the preliminary engineering phases. For the states without a selection process, most respondents (71%) seem to think such an evaluation method would be helpful in making informed decisions about ABC implementation. With regard to factors that influence project selection, of the many factors that agencies were able to choose from, only noise pollution, cost, and shorter construction time did not positively affect an agency's decision to choose a project for ABC. At the top of the list for positively influencing the selection of ABC techniques was traffic congestion. That implies an ABC technique is more likely to be implemented if it reduces traffic congestion than if it shortens construction time, even though lessened traffic congestion can be a direct result of shortened construction time.

The survey asked a series of questions to determine the readiness of the respondent's agency to implement several ABC techniques. The respondents were asked if their agency could immediately implement a specific method (e.g., extensive use of prefabrication, use of SPMTs, and so on.) or if further development was needed. The respondents were allowed to choose from three reasons as to why further development was necessary to implement a certain method: it requires additional research, legislative action, or institutional change. Methods that the respondent's agency considers experimental or methods with which the agency is unfamiliar were categorized as requiring further research. Some methods, such as certain contract types, might only be available following legislative action, such as a disaster declaration. Other methods might face institutional roadblocks, such as unwillingness within the agency to pursue ABC.

A majority of respondents considered their agency ready to implement all of the methods except the extensive prefabrication of substructure.

- 42% of respondents indicated that extensive prefabrication of substructure elements could be implemented immediately.
- 79% of respondents indicated more research was needed.
- 37% said that institutional change was required.

Notably, all the states with significant seismic design requirements felt that extensive substructure prefabrication was not immediately viable and that additional research was needed.

First Owners Survey Observations

While most agencies are aware of ABC, very few practice it on a large scale. According to the survey results, agencies are ready to implement many ABC techniques in theory, yet ABC has yet to gain significant traction. Some techniques are viewed as too new and requiring research and development; in other cases, ABC simply lacks a champion to push for its implementation on bridge projects.

ABC has distinct benefits that are definitely attractive to many agencies. However, despite the gradual lowering of costs and life-cycle cost savings, DOTs are hesitant to use ABC techniques because of their higher initial costs. ABC has been used on emergency replacement projects as well as on typical highway overpass replacement projects. And when an agency has used certain techniques, it has generally been pleased with the results. Nonetheless, few agencies have committed to using those same techniques as part of an overall ABC program.

Owner Personal Interviews

As a supplement to the electronic online survey, multiple owners were contacted individually by team members to gain their insights into various aspects of ABC. (The survey contained questions for owners both with and without experience.) Detailed responses were collected from Arkansas, Massachusetts, Mississippi, Missouri, New Jersey, New Hampshire, New York, North Carolina, Texas, Utah, Washington, and Wisconsin as well as the Federal Highway Administration. Of those, only Arkansas and Massachusetts indicated they did not have experience with ABC projects; the experiences of the other states were highly varied. Little consistency of effort or experience is evident among the states at this point. A brief description of the findings from the interviews and conversations is presented on a state-by-state basis.

Arkansas

Arkansas is an example of a state with no active ABC program. The Arkansas DOT's perspective on project acceleration is that it should be done when it is possible to reduce project costs. Construction time savings are considered secondary and have a limited ability to provide project incentives. The perception is that ABC projects will be more expensive and thus counter to the desire for cost savings. There is some interest in ABC use but no active programs.

California

Caltrans described several recent projects in which various ABC methods were employed. For the I-40 Marble Wash bridge replacements, a series of bridges were replaced by using prefabricated or precast concrete abutments and bulb tee construction. Complete reconstruction was accomplished in 4 weeks and was required because of high average daily traffic (ADT). Caltrans indicated that meaningful incentives and disincentives greatly motivated the contractor. For the Oakland Bay Bridge approach span replacements at Yerba Buena Island, large spans were replaced by using a skidding system with a complete closure of 3 days over a holiday weekend. The agency estimated that the costs greatly exceeded a conventional project; yet a conventional project would not have been possible or would have involved very difficult traffic staging. Again, significant incentives and disincentives were used.

In general, individuals at multiple levels within Caltrans expressed a strong degree of interest in and support for ABC projects, as did local fabricators and trade associations. They generally agreed that the role of the designer should be to develop plans for ABC projects in conjunction with construction staff to maximize the constructability of the project. Several limitations inhibit the greater use of accelerated construction techniques for Caltrans, including seismic concerns limiting use of precast pier elements, long-term durability, and concerns about the ability to balance the increase in construction costs against the user costs savings.

Mississippi

The Mississippi DOT has significant experience in recent Gulf Coast reconstruction (following Hurricane Katrina) with precast concrete cap beams. The agency is reluctant to use precast columns or footings because of concerns about connection durability and would welcome the development of durable connections for those precast elements. It does not use integral abutments because of concerns about approach slab connection details. The Mississippi DOT has heard complaints from contractors about the diminished profitability of projects using large precast elements. That concern is perceived as more of a political issue than a construction concern. The agency expressed the opinion that many bridge construction projects are not on a critical path—roadway elements drive the schedule. In those cases they see a diminished value in acceleration.

In practice, ABC is applied selectively at this point and is reserved for emergency reconstruction or projects with special conditions such as emergency access or site constraints. The Mississippi DOT senior management must be convinced of the advantages of acceleration. In the meantime, managers would appreciate having a catalog of ideas to choose from as opposed to prescriptive standards when deciding how or whether to pursue an ABC project.

As a small state, they feel that a regional consensus is needed for ABC to move forward since contractors and fabricators in their part of the country work in multiple states. They believe that the local fabricators would embrace new shapes and technologies as long as a commitment to a large number of projects was made. Individuals expressed an interest in seeing actual comparisons of conventional versus ABC schedules and cost estimates to justify the advantages to senior management. They are also interested in data on durability of joints and connections between precast elements as this is a hindrance right now in their use of such systems.

Missouri

Individuals at the Missouri DOT cited various examples of recent ABC deployment, including their frequent use of stay-in-place concrete deck panels, the successful use of Inverset systems, and a long bridge redecked with full-depth precast deck panels with nighttime closures. They also cited the use of adjacent box beams with a concrete topping erected as an emergency replacement and the application of latex-modified high-early-strength wearing surfaces in lieu of conventional overlays as a way of expediting highway reopening times. Those projects were substantially more expensive than the agency's conventional approaches but done with ABC techniques because of traffic constraints.

The Missouri DOT does not have a formal process for ABC implementation but takes the position that it must minimize traffic impacts to the extent possible with available project funds. In their experience, ABC significantly increases project expenses and must be used judiciously. The agency encourages the submission of value engineering (VE) proposals from contractors that save time or money over the as-designed solution. Agency management encourages the use of ABC when appropriate, and the local contracting industry has responded favorably.

The Missouri DOT was one of the few agencies that discussed innovations in substructure construction as a means of accelerating construction. It is using mechanically stabilized earth (MSE) walls as abutments to speed construction, shorten bridges, reduce costs, and allow for the use of concrete instead of steel beams—all of which allow for a faster total project. Concerns were expressed about seismic and durability issues, and the department is working with local university partners for ABC assistance.

Missouri has aggressively pursued implementation of alternative technical concepts (ATCs) on several smaller projects, and more recently on a larger scale for the New Mississippi River Bridge Crossing in St. Louis. Essentially, the ATC process consists of an owner preparing a baseline design; anyone can bid on it as-is or can confidentially propose cost- and time-saving measures. Those measures must be approved before bidding, in which case the owner prepares an amendment to the contract drawings to reflect the contractor's ATC for bidding purposes. ATCs are not necessarily an ABC method, but they provide an incentive for contractors to develop confidential cost- and time-saving innovations that will give them a competitive advantage.

New Hampshire

The New Hampshire DOT was interviewed as an agency with experience in ABC projects. The department has used precast elements extensively on several projects; and, specifically, the department received significant publicity for its replacement of the Main Street Bridge in Epping, New Hampshire. That project, in which a new bridge was completely erected of entirely precast elements in essentially a 1-week construction period, was noted as an ABC success. Even so, little follow-up to that project has occurred. This issue was raised in conversations with department staff. The response was that not enough people at the agency are interested in ABC as a project delivery tool. They don't question the effectiveness of the project but have insufficient motivation to do it again. The University of New Hampshire continues to do research in the area, but the New Hampshire DOT does not have the same level of interest. Agency staff indicated that not many opportunities have arisen where acceleration appears justified. They also reported that the Epping project was 2.2 times as expensive as a conventional bridge replacement and that until the cost premium comes down to 25%, they would have difficulty promoting ABC.

The only serious impediment to the design of ABC projects is the lack of ABC champions within the New Hampshire DOT. Among contractors, ABC is generally accepted; but when given the option, they seem reluctant to use it. Thus, when considered feasible and appropriate, projects are engineered to make use of ABC techniques, rather than leaving that decision to contractors. Contractors hesitate because ABC involves the use of new technology, and they want to keep their own employees working rather than subcontracting work to precasters. Very likely, contractors will use ABC when time constraints and disincentives are severe enough.

New Jersey

The New Jersey DOT provided an extensive interview focused mainly on the issue of project and agency impediments. The department has had some successful ABC projects, including the Hyperbuild initiative projects to replace several deteriorated bridges near Trenton and a nearly completed project to accelerate the replacement of the Route 70 bridge over the Manasquan River. That project made extensive use of precast elements and was nearly a year ahead of schedule as a result. However, these are isolated experiences.

The question was why ABC has not taken hold. The underlying answer is that the department's engineers and, in particular, the project managers do not consider it a solution in many situations. That thinking relates to their past practices and impediments established by other units within the agency structure. The agency is generally risk averse. ABC is perceived as raising the level of risk associated with a project. The level of risk needs to be shown to be manageable for the concept to gain traction.

There is currently no impetus for ABC. At the same time, there is no obvious opposition other than the inertia in the organization's practices. When the department has tried to accelerate projects, its own construction engineering department has been reluctant to support the schedule. Schedules are frequently lengthened on the basis of traditional practices. Apparently, the traffic operations staff has allowed only short closure windows, which has the effect of prolonging projects and impeding ABC efforts.

The New Jersey DOT incentive/disincentive opportunity on projects is tied to the computation of roadway user costs. These costs are typically thought to be very low and do not justify acceleration as a strategy. The department recognizes the need to study and update the user cost model and its application. The state has no mechanism to screen or choose projects for ABC. A ranking was done at some point in the past, but it was done by staff from an engineering perspective. No systematic approach is in place.

Designers are reluctant to suggest innovative approaches. The concern is that project managers will not accept such proposals, so they have no incentive to be creative. The state does not procure contracts requiring innovative design and construction solutions. Thus, the same approaches are used again and again. FHWA engineers have provided limited support for New Jersey's own initiatives to use prefabricated technologies or accelerated approaches.

Another area discussed in the interviews pertained to removing obstacles. Department staff expressed the opinion that data on accident rates in work zones would be useful. If it could be shown that before construction, the rate was A, that during construction it rose to B, and that after construction it returned to A, then that would be a compelling case for using ABC methods. Another way to promote ABC would be to provide data on lost-time accidents in the field (working around active traffic) versus construction that occurs in a plant, yard, or similar off-site facility, which would show a benefit to using ABC. It would also be useful if quality differences could be demonstrated between projects constructed in the field and projects built with prefabricated elements. This includes bridges built off alignment and moved to their final location.

New York

The New York State DOT reports having completed approximately 10 projects that could be considered as having used ABC methods. The department also states that though ABC is the exception rather than the rule, more and more ABC techniques are gaining acceptance, especially downstate in the region around New York City. For example, self-propelled modular transporters (SPMTs) were used in a project over the Van Wyck Expressway on Long Island. But the need for large staging areas is a problem in such a heavily developed environment, where ABC is most beneficial. Generally, ABC projects in New York are engineered to use ABC, but sometimes the contractors submit substitute proposals using ABC methods.

Hindrances include construction costs for ABC methods, especially the use of precast or prefabricated elements and off-site construction using roll-in methods. The New York State DOT also has some concerns about the durability of precast component connections and joints. Furthermore, some local contractors resist the use of extensive prefabrication because of the large project share that is subcontracted out to specialists. Standardization would be less effective because the most beneficial applications of standardization tend to be less standard, such as projects in urban areas.

Several pilot projects are under way that make use of ultra-high-performance concrete for joints between precast components, deck bulb tee beams for one bridge, and full-depth precast deck panels for another. The New York State DOT is also involved in ultra-high-performance concrete (UHPC) research investigating fatigue in precast element joints.

North Carolina

The North Carolina DOT has several ABC experiences including the reconstruction of seven bridges on Ocracoke Island, with crews working 24-hr days to replace the bridges in 90 days. Additionally, the Washington Bypass project is ongoing and employs an innovative construction gantry allowing for complete construction of a new viaduct from the top without any intrusion into environmentally sensitive areas.

The department has project selection criteria for ABC projects and spent some time in the interview process discussing the role of the Alternative Project Delivery Unit. The unit was specifically set up to work on design–build projects, VE proposals, and alternative contracting mechanisms. The department typically allows for innovation in three ways: as a proposal from the contractor in design–build contracts, as an as-designed solution for special projects, and as a VE proposal. The department is currently exploring the use of MSE abutments as well as geosynthetic reinforced soil abutments as a means of expediting foundation construction.

Texas

A significant outreach effort in Texas was conducted by project team member Structural Engineering Associates. Various individuals from the Texas DOT Bridge Division and the San Antonio District were interviewed about ABC implementation. Additionally, a major local contractor and precaster were included in the discussions. All of these people were asked about current ABC implementation status, obstacles to moving forward, and concepts for future success. The opinions of the group collectively are expressed as follows.

As funding is currently structured, the owner has no financial incentive to use rapid renewal methods other than staged construction. The Texas DOT districts are limited to using only 5% of the project cost for incentives. Also, no more than 25% of the road-user delay costs may be used for incentives. Although road-user costs are considered, the owner has no way of collecting any such savings. Therefore, if they spend additional funds to reduce road-user costs, they have fewer funds for other projects. That is a disincentive. One suggestion was for the federal government to create a mechanism that would allow owners to capture savings of road-user costs for use on other projects. For example, the mechanism could be a federal grant to the owner based on the value of the savings. That would create the needed incentive to promote rapid construction projects.

In Texas, most bridge projects involve existing bridges that are functionally obsolete but not structurally deficient. The most common reason they are functionally obsolete is that the bridge and roadway are not wide enough. In those cases, the roadway construction and not the bridge construction is on the critical path.

It was noted that low-bid contractors may not have the ability to perform rapid bridge replacement. So, instead of selecting the contractor by low bid, a better method might be selecting the contractor on the basis of best value. Part of the best-value evaluation would be the experience of the contractor on similar projects and the contractor's approach to the construction.

Project size is an important consideration for ABC. Because of the small to medium size of many candidate bridges, contractors will not become efficient with ABC methods during the short time frame of an individual rapid bridge replacement project. Additionally, precast components used for bridge substructures are only practical when lack of access makes the construction of cast-in-place components difficult or when there is sufficient repetition. Sufficient repetition makes the precast components more economical and their construction more efficient and faster. The only way to build overall efficiency is to build this capability over the course of several projects. For contractor efficiency, that means rapid bridge replacements must become commonplace. Bundling several bridge sites with similar requirements into a single construction contract would make those bridges more economical. Finally, the contractor should not be forced to use a particular means or method.

When incentives and disincentives are used, the amount must be large enough to pay for additional construction crews and for additional (or special) construction equipment needed to accelerate construction—and still result in profit. As an alternative, the department could consider milestones with no-excuse bonuses. If contractors are able to complete construction without any excuses, then they are awarded a bonus. Contractors will most likely submit bids assuming that they will not be awarded the bonus.

Utah

The Utah DOT has made extensive use of various ABC techniques for the past decade and has provided a substantial amount of information in various forms to the project team for review. The Utah DOT is unique in the level of support from the agency for the use of user costs as a strong consideration in weighing whether to use conventional or accelerated delivery methods. Though many states indicated some consideration of user costs, the Utah DOT has a project selection criterion that frequently leads to the decision to use ABC and that values user costs as real project impacts. Lack of resources, consideration of user fees, difficult materials supply issues, and other considerations have all pushed the department toward making ABC a standard practice by 2010. The department indicated nearly unanimous support at the senior management level for undertaking projects that give strong consideration to minimizing user costs.

At the present time, the Utah DOT is delivering its ABC program through a combination of design–build contracts and a method known as CM/GC (construction manager/general contractor), both of which have proven successful. At the same time, the department is developing ABC standards for deck panels, precast substructures, new prestressed beam sections, and other details. Those standards will give the agency increased flexibility to let contracts using various mechanisms and to communicate its ABC intentions to the design and construction community. Once ABC standards become available for engineers to use in the creation of as-designed ABC bridge plans, they will explore their use in more-conventional design–bid–build contracts. Agency staff believe that precast elements will offer an additional opportunity for cost savings in substructure construction.

During its initial roll-out, the ABC program met with a fair amount of internal resistance, similar to the experience in New Jersey. Middle management presented the biggest obstacle, particularly the attitude that "we don't do things like that around here." Consultants and designers as well as the contracting industry were more easily convinced of the merits of ABC than were department staff. However, a sufficient core was willing to try new things in all parts of the business (department, consultant, contractor), so that a decision was made to move ahead with trial project implementation.

During early phases of implementation, some additional reluctance emerged in the contracting community. The Utah DOT held a series of workshops and scanning tours to help learn from other agency practices. Some contractors have made successful changes to their business practices to compete in the ABC arena, while others are still holdouts. Successful contractors have demonstrated a willingness to get into the precasting business. The projects let to date have demonstrated a 5:1 to 6:1 ratio of user costs saved to construction costs incurred. With repetition, costs have come down. Recent bridge project lettings indicate that full-depth precast decks are cost competitive and occasionally less expensive than traditional cast-in-place concrete decks. Time and quality savings have come as well.

Although the Utah DOT has moved aggressively toward ABC implementation, some unanswered questions and areas of potential improvement remain. Those are generally technical issues related to existing experiences and are not implementation related. Concerns include issues such as seismic detailing, design consideration for structures to be moved, acceptable deformation limits during movement, a need for better specifications, and some additional concern about connection details and durability. Those issues notwithstanding, the department is aggressively moving forward with ABC as a standard delivery mechanism.

Washington

The Washington State DOT has employed ABC methods in some fashion on various projects. Those projects were completed by using traditional design–bid–build procurement with some redesign of the structures to accommodate the ABC approaches. The projects have included complete bridge prefabrication as well as large-scale prefabrication of superstructure and substructure elements. In general, department staff believe that the use of prefabrication and ABC approaches did not have an effect on project quality but did have a beneficial effect on project safety. The department does not have a specific requirement to consider user impacts as project cost components but has used incentive/disincentive clauses to motivate project completion.

Wisconsin

Wisconsin is beginning to implement ABC practices, having recently redecked a major structure with full-depth precast deck panels. That first foray into accelerated reconstruction was a staged reconstruction of a bridge deck, and even from the first to second phases, the quality and speed of construction demonstrably improved. The Washington State DOT believes this demonstrates that local forces can effectively execute a complex project and learn the proper techniques rapidly. Up-front training for others would be beneficial to better educate the construction community.

Given that this process is new in Wisconsin, the levels of support and opposition are not well established. The support is perceived as lukewarm, with no strong opposition. Contractors have indicated that as long as they can make money, they are willing to participate. The department is exploring other opportunities for demonstration projects and is funding research into precast substructure units.

Second Owners Survey Results

In June and July of 2009, several additional outreach activities were completed, allowing further insight into hindrances to the implementation of ABC methods, their causes, and their solutions. New surveys and interviews focusing on impediments and the acceptance of ABC within the industry were conducted with state departments of transportation, contractors, and fabricators. The owner questions focused on the following:

- Experience with accelerated bridge construction (ABC);
- The general level of acceptance of ABC in the state;
- Whether the state engineers projects to employ ABC, or leaves that to the contractor;
- Current impediments in the design and construction processes to greater use of ABC methods;
- Whether availability of standardized elements and systems suitable for ABC and more durable connection details for prefabricated elements and systems would increase ABC implementation;
- Ongoing or recent projects that implement ABC elements, in particular, unique, new, or innovative solutions; and
- Ongoing or recent research related to the concept of ABC.

Twenty-four state and federal agencies responded to the survey. The results are summarized by state.

Alabama

Alabama has completed one ABC project in the past 5 years. Despite successful ABC applications, the state is not ready to make the practice routine. The bridge projects that employed ABC techniques in the past were designed to do so; however, the contractor has the flexibility to modify the construction methods to ensure a better end product or a faster construction schedule. Some of the impediments to large-scale of implementation within Alabama are a lack of manpower and elevated costs. The success of standardization was described as doubtful for typical bridges but may be successful for long structures that require substantial repetition of elements. Alabama is currently involved in the research and testing of four systems of rapid deck replacement on twin structures in the northern region of the state.

California

Caltrans has completed eight ABC projects within the past 5 years and is making a huge effort to include ABC methods and techniques in its bridge replacement program. In California, ABC is considered to be only one part of accelerated project delivery (APD), which is a broader category intended to include all aspects of rapid project completion and to encourage everyone in the industry to take ownership of the ABC process. Steps have been taken to incorporate APD into Caltrans's advanced planning process so that suitable project candidates can be identified, the ABC process can be streamlined, and the required project funding can be secured. The acceptance of ABC has grown with the APD concept, appealing to roadway engineers as well, and is received well when ABC is the best alternative and when funding is available. Seismicity is a major hindrance to the adoption of precast element technology in California, where cast-in-place dominates. More research on the seismic performance of precast bridge elements would be required before systems such as precast abutment systems could be adopted. As in other states with large urban areas, finding suitable staging areas is a problem, though early identification of project constraints could help with this problem. Cost is also an issue, as well as industry reluctance; but with the help of meaningful incentives, contractors may be enticed to use ABC methods. Standardization may make costs more economical, and that, in turn, would encourage all members of the industry to adopt ABC. Caltrans currently is using precast posttensioned girders with integral bent cap connections on a widening project and is engaged in several research projects involving seismic performance of precast bridge elements and connections.

Delaware

The Delaware DOT reported that it has some experience with ABC methods, having completed fewer than 10 projects in the past 5 years with ABC techniques. All of those projects were engineered to employ ABC. Some of the impediments the department describes include higher initial costs and contractors' issues with extended hours and other associated costs—despite a generally good level of acceptance of ABC. Currently, the Delaware DOT is involved in projects that make use of innovative contracting methods such as A+B, A+B+I/D, and lane rentals. The department is also involved in projects that use precast elements, including the purchase of precast elements in advance. The Delaware DOT is not currently involved in any research pertaining to ABC.

FHWA

According to the Federal Highway Administration, several impediments are hindering the acceptance of ABC techniques among the various state departments of transportation. It reported that states are looking for design manuals and other aids to help them design and implement ABC—specifically, manuals such as the recently published connection details manual and a manual for designing prefabricated elements and systems. FHWA also reported that, even though many state DOTs consider cost a main obstacle to implementation of ABC methods, the costs associated with the Utah DOT's ABC projects have steadily decreased over the past 2 years. The tendency for state DOTs and contractors to use cast-in-place rather than precast construction was also described as a problem; the solution is to introduce the industry to precast technology and demonstrate its profitability. Lastly, the lack of effective seismic connections for precast bridge elements must be addressed and is currently under investigation as an NCHRP project.

Florida

The Florida DOT has had a variety of experiences with ABC, having participated in the Graves Avenue Bridge movement with SPMTs and having completed several other projects within the past 5 years. While ABC methods have been worthwhile for constructing certain projects, the practice has yet to be adopted as a standard, and typically traditional methods are used. ABC is, however, considered in the development of every project.

The impediments hindering ABC in Florida are echoed throughout the results of the survey, as most of the states are experiencing some of the many hindrances listed. For example, limited space in highly urbanized areas is a problem for ABC using SPMTs when right-of-way is needed to construct and move a bridge into place. As this ABC method is most useful in precisely those areas, the lack of space is a huge issue. A possible solution is to purchase or lease property during construction, or require the contractor to do so. Another impediment is contractor inexperience with ABC methods; contractors need exposure to more information and opportunities to familiarize themselves with ABC concepts. Maintenance of traffic and phased construction is another issue as many sites have traffic constraints that need to be considered during the design phase. Public involvement is useful for gaining acceptance for the temporary full closure of a roadway. Uncertainties about a contractor's methods for construction is another issue, as the Florida DOT lets its contractors choose their methods of construction by dictating performance specifications rather than mandating the use of ABC. Common among most states is the issue of balancing user costs with the additional costs of ABC, which is hard to justify when budgets are tight and many projects use standard construction practices. Also common among many states is the tendency for contractors to avoid subcontracting work to precasters, as they make their profits placing steel and concrete.

Georgia

The Georgia DOT has some experience with ABC, having completed one such project in the past 5 years. The use of ABC methods needs to be required, and use of ABC is left up to the contractor in general. As with many other DOTs, the largest impediment to the implementation of ABC methods is cost; but the Georgia DOT does consider the standardization of prefabricated elements a way to lower costs associated with ABC.

Illinois

Illinois has completed a handful of projects with ABC methods in the past 5 years. Bridge projects undergo a "bridge planning state" during which ABC is evaluated on the basis of site needs and cost-benefit analyses. The main hindrance experienced by the Illinois DOT is the expectation that ABC will cost more and that user costs are difficult to quantify. As with other DOTs, the Illinois department believes that standardized elements would be useful in curtailing ABC costs.

Iowa

Iowa has extensive experience with ABC, having completed several ABC projects within the past 5 years. ABC is well accepted in Iowa; the state has completed ABC projects with innovative bridge research and construction/deployment (IBRC/IBRD) program funding and has spent its own money to accelerate projects when conditions require it. In Iowa, ABC projects are designed from the beginning to be accelerated, and contractors tend not to use ABC methods as value engineering proposals. According to the Iowa DOT, several impediments to ABC exist in the state, including low traffic volumes, contractor reluctance to adopt ABC because of the perception that it is less profitable, and contractor belief that ABC is too complex. Also, ABC incentives are low, which fails to attract contractor support. Upper-level management supports using ABC wherever warranted; but in some cases, production-level engineers find designing for ABC to be slow and frustrating. Standard plans and shapes would help by easing the design process and saving money through reuse.

Louisiana

The Louisiana Department of Transportation and Development has extensive experience with ABC methods, specifically the use of precast elements such as span and cap segments, as well as float-out, float-in construction to erect long-span bridges over its many waterways. The agency also has experience with precast flat slab bridges for projects off the federal highway system and reported that, though these bridges do not provide the service life of their cast-in-place counterparts, they are more easily constructed in remote areas. Recently, the agency used SPMTs during two 12-hour closures to replace an overpass damaged by over-height vehicles. The department reported that, while soil conditions within the state preclude them from precasting longer girder spans, standardization would be possible for shorter-span bridges. Contractors often request that cranes and crane mats be used on top of the structure, so a standard element that takes into account crane loads would be ideal. The department would like to continue using ABC techniques in the future and has plans to use SPMTs to remove and replace spans in an upcoming project.

Maine

The Maine DOT has experience with ABC, having completed several projects within the past 5 years. The level of acceptance of ABC within the agency is considered to be high. Cost is the largest impediment to ABC implementation in Maine, as it generally costs more to precast elements than to cast them in place. Standardizing elements could help lower costs of precasting as it might encourage fabricators to invest in standard forms for bridge elements.

Massachusetts

MassHighway has not completed any ABC projects but is interested in implementing pilot projects to build familiarity with ABC techniques. Because no ABC projects have been done in Massachusetts, one of the main impediments to implementation is a lack of familiarity with ABC systems and techniques and how they will affect the construction process. As in many other states, a conservative cast-in-place (CIP) culture exists among contractors. But MassHighway believes that increased exposure through the completion of pilot projects will help overcome the tendency of contractors to use tried-and-true methods and hopes that more experience with ABC methods will diminish concerns about financial risk as well. Availability of standardized elements would be useful as that would reduce the need to develop custom details. In addition, the problem of a lack of familiarity could be offset by learning what standardized elements have been successful in other locations.

Michigan

Michigan has some experience with ABC methods, having completed projects that have been designed to implement ABC techniques and projects that have been accelerated as a result of the contractor's input. The main hindrances to implementation are cost, constructability, and quality and performance issues. Life-cycle cost analyses with accurate accounting of the benefit to the public would be useful for addressing higher costs. Constructability and quality issues will likely be addressed by the experience of completing ABC projects in general. Standardization could make ABC methods more accessible to designers and would help contractors gain meaningful experience; both would help lower costs and improve quality in the long term.

Nebraska

While the Nebraska Department of Roads has not used ABC methods on any complete projects, elements of some bridge projects have been accelerated. There is no perceived need for ABC in Nebraska, so it is not widely accepted, and its applications have been limited. In the cases where ABC was accomplished through bridge elements, the bridge was designed to accommodate them; but contractors have also used discretionary methods to accelerate construction, such as using more man-hours. Primarily, ABC is hindered by higher costs. In addition, contractors are hesitant to use precast elements because of the amount of work that would be subcontracted. Though Nebraska has urban areas that would be associated with higher user-delay costs, the user costs of lower-traffic roads and rural routes do not warrant the use of ABC. Standardization of precast elements is seen as a way both to lower costs and to increase the quality and durability of finished bridge projects. ABC bridge and research projects are currently under way in Nebraska, including the use of precast deck panels and heavy lifting of remotely assembled superstructure.

New Hampshire

ABC is generally accepted, but when given the option, contractors in New Hampshire seem reluctant to use it. When considered feasible and appropriate, projects are engineered to make use of ABC techniques, rather than leaving that decision to the contractor. The only real impediment to the design of ABC projects is the lack of ABC champions within the New Hampshire DOT. Contractors are hesitant, as the practice involves the use of new technology, and they want to keep their own employees working rather than subcontracting work to precasters. Contractors are expected to use ABC when time constraints and disincentives are severe enough.

New Jersey

The New Jersey DOT has completed many ABC projects over the last 5 years, and acceptance of the practice is generally good within the agency. In the past, ABC projects have been engineered to employ ABC methods; contractors rarely recommend innovative ABC techniques to achieve expedited construction. Several impediments were identified as having a negative effect on the success of ABC, including issues of sole sourcing and detours. Federal regulations and the prevailing mindset at the New Jersey DOT make the use of proprietary products difficult, but trying to write a generic specification does not allow for the procurement of the specified item. Detours, on the other hand, are used only as a last resort, even when construction can proceed expeditiously; staged construction is preferred to allow the route to remain open. Contractors in New Jersey, as in other states, are reluctant to give up a portion of their work to subcontractors and fabricators. Unless the contract specifies time constraints, contractors will look for ways to complete jobs by using methods with which they are familiar. A suggested solution would be to allow a portion of a project to be constructed conventionally and have the remainder be precast. Furthermore, if AASHTO and other organization would champion publications such as the FHWA prefabrication details manual (Culmo, 2009), the guidance would help encourage the use of ABC throughout the industry. Standardization would be a boon to the prevalence of ABC; and standardized elements and the combination of conventional and ABC methods for a single project would help more contractors accept ABC.

Pennsylvania

The Pennsylvania DOT has some experience with ABC, having used both precast ABC elements and more innovative approaches such as launching and SPMTs a few times over the last 5 years. ABC is considered on a project-by-project basis, but the additional costs from contract risk as well as life-cycle costs generally outweigh short-term advantages. Innovative ABC is usually at the contractor's discretion unless the project would clearly benefit from ABC. In that case, the project is engineered to use ABC methods. Contractors are considered an impediment; they are generally unwilling to assume the additional risk associated with a technology with which they have no experience or that they have to subcontract, and that translates into inflated bids. Once ABC methods become more mainstream, costs and risk will likely decrease to the point where their continued use will be economical. Standardization would provide only limited improvement, as past efforts at standardization have not translated into profits for contractors, who must subcontract work to fabricators, reducing the work for the contractor's own forces. Currently, the Pennsylvania DOT is not implementing ABC methods in any projects but is involved in research in structural details that could be applied to ABC in the future.

South Dakota

South Dakota most recently completed an ABC project in 2001 where SPMTs were used to move a large steel truss superstructure to its abutments from a remote assembly location. In that instance, ABC was a necessity because the bridge was spanning a railroad yard, where closures and outages had to be kept to a minimum. ABC is viewed favorably in South Dakota if project conditions warrant its use. As in many other states, South Dakota has relatively low traffic volumes, so user costs do not justify the elevated costs of ABC. However, there is significant interest in using ABC methods to construct jointless decks of adequate length for little or no increased cost.

Tennessee

The Tennessee DOT has made limited use of ABC techniques, having completed one project in the last 5 years that incorporated ABC methods. Although ABC is always considered for a bridge project, it is not often used. Concerns that impede ABC implementation include questions about durability and quality of precast members, and connection issues—specifically attaching precast bridge decks to beams. With proven installation and serviceability records, standardized elements are considered useful for ABC within the state.

Vermont

The Vermont DOT has a good general acceptance of ABC and has completed five projects with ABC methods within the last 5 years. Projects are typically engineered to use ABC, but the department is considering projects that would contain performance specifications and leave the design of ABC details to the contractor. Because Vermont does not have high traffic, road-user costs are often too small to create meaningful incentive/disincentive clauses in contracts that would encourage contractors to accelerate the construction schedule. Contractors tend to accept liquidated damages and overrun on time. The department would appreciate a way to apply savings from ABC methods, such as the elimination of the need of temporary bridges, to incentive/disincentive clauses.

West Virginia

Within the last 5 years, five ABC projects have been completed in West Virginia. While the practice is generally accepted within the West Virginia DOT, the state does not have enough traffic volume to make it generally feasible for implementation. The projects that were completed with ABC techniques were designed to use ABC and were the result of I/D clauses that were included to motivate contractors to develop ABC approaches. Impediments to implementation include an underdeveloped contracting industry when it comes to ability to perform ABC, and a lack of a precasting industry in the state. Furthermore, the state has few heavy lift contractors, and contractors like to keep the work local. If standards for ABC existed, contractors would likely make use of them. ABC specifications for construction and sample contracts would be useful. And the department is also interested in methods that minimize environmental disruption.

Wyoming

Wyoming's experience with ABC stems from the completion of several projects involving extensive use of precast elements and decked bulb tees for country road bridges. Wyoming has a good general level of acceptance for ABC, and it is used where appropriate, though Wyoming does not have the high traffic problems that some other states have. With lower traffic counts, the main impediment to the implementation of ABC is the justification of the higher costs associated with ABC methods. There is interest in standardization, specifically seeing design standards that have been used by other states, which would likely lead to greater use of ABC designs within Wyoming.

Owner Focus Groups

To glean additional information about ABC within the environment of the departments of transportation, three 90-minute teleconference focus groups were held with representatives of several states. A total of 24 owner representatives participated in the focus group conference calls. The main topics discussed during the focus group meetings were the following:

- Impediments to using rapid replacement methods;
- Design for rapid bridge replacement;
- Construction methods for rapid bridge replacement; and
- Other issues states have encountered with ABC.

Focus group discussion topics and key points made during the discussions are presented in the form of meeting notes and summaries.

SHRP 2 Innovative Bridge Design Focus Group I

9:00–10:30 a.m. EST, July 20, 2009
Participants: Devin Anderson, Wayne Frankhauser, Maine; George Christian, Arthur Yanotti, New York; Paul Liles, Georgia; Sam Fallaha, Florida

Question/Topic

What are some of the impediments hindering the implementation of ABC?

Discussion Points

New York: There is a higher [construction] cost; user costs and savings are not budgeted; hard money costs, so the additional costs must be worthwhile. Near New York City, a contractor proposed a precast deck system for ABC, but the New York State DOT—trying to save money—declined.

Georgia: There was a Highways for LIFE project that used ABC, but the contractor said it could do it in the same amount of time with standard methods (i.e., cast-in-place) despite time requirements.

Maine: User costs and indirect costs are the main issues. For lots of fast-track construction, detour length is a big factor for deciding to use ABC. ABC may cost more, but it might not cost the overall project as much when factoring in building a temporary bridge and long detours. We have used precast abutments on piles, but the precaster was too slow and the contractor used CIP to make the schedule. We have used box beams with precast abutments to complete a project in 30 days, also gave up the leveling slab, and placed the membrane directly on top with shear keys. Existing bridge with traffic was shut down for 96 hours.

Question/Topic

How is federal funding involved?

Discussion Points

Maine: The costs aren't blown out of proportion, but they are not necessarily outside the parameters of a high bid on a project anyway. There is more risk to the contractor, but good contractors can handle ABC well.

Georgia: We had to stop giving contractor weather days, which would allow contractors to sit and argue about the weather.

Question/Topic

What about A+B bidding?

Discussion Points

Maine and New York have used it. Maine uses it off and on, specifically for interstate projects such as the I-95 deck replacement of the northbound and southbound lanes. The contractor was allowed to choose how many days it bid to do the work. Georgia used it only in emergencies.

Question/Topic

General CIP culture exists for DOTs and contractors. How does that play into ABC?

Discussion Points

New York: It's a factor, according to contractors; they are worried about subcontracting to precasters, because it is not putting their own people to work and lowers profitability. With precast decks there have been problems with unions; the ironworkers union complained because there was no deck to reinforce.

Florida: Contractors feel they're giving up part of their work to the precaster and regularly try to keep the work internal to them; they want to keep it that way.
Similar experiences in Nebraska.

Question/Topic

How can we change that?

Discussion Points

Florida: It will change with time and persistence. This will have to happen, as over the decades, construction has gotten more and more efficient, as a result of gradual changes, not big steps. It just takes time for the culture to change and the contractor to realize that you don't have to have your own forces do everything.

Georgia: The contractors will gripe about changes for ABC, but if it's work, they'll do it. If they can do more ABC projects, they will do them, as there is still plenty for them to do.

Question/Topic

The Utah DOT is changing its culture, and essentially said it wants to build bridges like this [ABC] in the future, and to continue doing so with currently existing technology. It will take an institutional commitment from the states before contractors will fully commit to any new way of doing business.

Discussion Points

Georgia: It is still more expensive, though they (the Utah DOT) have a way of applying user costs and can justify them. Utah does not have as many deficient bridges, and when you have 10 bridges to build at a 10% premium, the money spent on ABC could finance the construction of another bridge. Furthermore, SPMTs are expensive, and other states do not factor in user costs. SPMTs will likely be used with high-profile cases in Georgia.

New York: There have been problems with precast systems, specifically with their joints and connections. On the Gowanus Expressway (New York City), they tried many systems, and what they found to work best is a CIP deck with accelerated cure.

Georgia: CIP offers more flexibility, an example of which is when footings were built a bit higher by a foot, and the delivered precast columns did not fit. They ended up switching them out, but in the process created many other issues. CIP would have been more flexible.

Question/Topic

Do you have any issues pertaining to design for ABC?

Discussion Points

Georgia: The problem of precast connections is a big issue, but the FHWA connection manual is good and engineers at the Georgia DOT are finding it useful.

Maine: ABC requires designers to spend more time on site because when you put a contractor into a time-restricted situation, the designer needs to have quick turnaround for answering questions.

Question/Topic

Do you see a need for standardized elements for ABC? Will this promote greater use of ABC?

Discussion Points

Florida: Currently, there are standards for everything, and once preferred rapid details are developed, the design/construction community will be more willing to use ABC. If a consultant wants to design for ABC, there is reluctance to use something that has not been used in the past. The Florida DOT is currently in the process of developing preferred details.

New York: Standardized connection details. The recent FHWA publication (Culmo, 2009) went a long way in promoting that.

Question/Topic

Would a similar manual for elements and systems be useful?

Discussion Points

Maine: One of the things we found most important is that designers have to work more closely with the construction people and pay more attention to detail on things that people are not used to doing and are not comfortable with. A lot of design people are not field people, so communication between designers and those in the field is critical.

Question/Topic

Do you feel that each state will need to have standard details, or can things be more alike among the states?

Discussion Points

Florida: If nothing else, the culture expects each state to have its own standards. Developing standards region by region may be a good idea. Having additional coordination is not easy but, if it is achievable, would probably prove helpful and productive.

Question/Topic

PCINE, with the NEXT beam, and a design manual (not a standard) for ABC, provides guidelines for using precast concrete systems, and there may be openings for similar regional approaches. Has this been used in Maine or New York?

Discussion Points

Maine: I am not familiar with the particular document, but Maine is a member of the PCINE committee. Massachusetts plays a significant role. The NEXT beam is going to be used on a recently bid project, chosen by contractors over bulb tee. The advantage of the NEXT beam is that it has a larger deck area and is almost wide enough to hold a lane of traffic.

Question/Topic

Substructure accounts for 60% to 70% of construction time, and not a lot has happened through ABC in that regard.

Discussion Points

Maine: Pile substructure and precast abutment pieces are being used. If formwork is done ahead of time, reinforcing steel is expedited, and normal 14-day curing standards are relaxed in favor of a strength-based specification (maturity testing), it is possible to set superstructure on top of substructure in 1 to 2 days. This allows a contractor to cast its own abutments rather than subcontracting them out, but the contractor must be ready to move big pieces quickly. Rapid construction doesn't have to mean precast.

Question/Topic

Among construction issues, what works well, etc.?

Discussion Points

New York: New York is using decked bulb tees and UHPC to close joints and experimenting with it between precast components, with good results so far. They are involved in fatigue testing at Turner–Fairbank (Highway Research Center).

Maine: Maine is using new products, such as the Hillman beam, an FRP shell with a very thin arch inside the shell and high-tension strands in the bottom of the shell. They are extremely lightweight and can be installed with just a boom truck. They weigh 33% of a steel beam after you pour concrete.

The University of Maine in Orono has tested it to failure, and it performed exactly as predicted. Maine also has a "Bridge in a Backpack" concept that involves tubes of FRP filled with concrete set in place, and uses FRP decking filled with concrete. Currently, there is a movable bridge that is to be replaced with FRP decking, built off site, and moved into place.

Florida: There is an NCHRP project being done by Cathy French at the University of Minnesota, investigating the durability of precast connections, and they are looking forward to conclusion of that research. Fast Track is design–build, but I cannot comment on how it's going or what's happening.

Question/Topic

Anything else?

Discussion Points

Self-consolidating concrete has been useful, especially when you can't use a vibrator and it's got to take care of itself. It has been a huge success and has not demonstrated any problems with shrinkage or creep yet. Georgia is currently involved in research.

SHRP 2 Innovative Bridge Design Focus Group II

11:00 a.m.–12:30 p.m. EST, July 20, 2009
Participants: Lloyd Wolf, Texas; Ralph Anderson, Illinois; Anne Rearick, Indiana; Norm McDonald, Iowa; Joe Campbell, Dan Dorgan, Minnesota; Ed Wasserman, Tennessee

Question/Topic

What are some of the impediments hindering the implementation of ABC?

Discussion Points

Minnesota: There are a couple of impediments. Funding is one as most of the jobs have cost some extra money. In Minnesota budgets are set up so that the eight transportation districts have [only] so much to spend on projects, so they must seek additional funding if ABC will cost more. IBRD funds helped provide incentives. Right-of-way becomes an issue; Minnesota has been looking to use SPMTs which require available right-of-way. Generally, it should be used in a high-volume, urban area to be most effective, but these are the same areas without excess right-of-way. So SPMT use is very limited. Planning is a 4-year program, therefore education on how to use ABC processes is important for the planning process.

Texas: So many projects involve a lot of roadway work, usually widening if replacing bridges. Typically a project starts much earlier than just the bridge work, and when using phased construction, ABC is not needed. When a long detour exists, ABC can be used and can offset the costs of repairing a long detour route. Often, phased construction eliminates the need for ABC. The Texas DOT can put time limits on work but can't dictate a contractor's means and methods.

Indiana: There is significant resistance from contractors.

Texas: Some contractors prefer to do their own precasting. Precast prestressed fabrication requires plant certification, but precast bent/abutment caps use conventional reinforcement, so the contractor can precast elements in its yard, and the DOT will inspect.

Tennessee: There are not many impediments, but it is not used much. Sometimes ABC is not going to be as easy as one thinks. They have used precast elements on the interstate near Knoxville, but they didn't use any of the details. The SPMT "thing" has been overblown: it has to be a highway crossing or a major river that allows for barge float-out/float-in, so it cannot be used everywhere.

Indiana: ABC is always thought of as building a bridge someplace else and moving it into place.

Tennessee: It is not feasible to replace many bridges in 24 hours. The concept can work, but the situation has to be perfect. Using precast footings and pile sections, a bridge can be built in weeks if everything goes well.

Texas: Similarly to the work in Tennessee, earthwork and paving might take a bridge off the critical path; and weather is a factor. If earthwork and paving could be accelerated, ABC benefits would be greater.

Question/Topic

How much of a factor is the precaster's ability within a state?

Discussion Points

Tennessee: There is no reason to go to a precaster, as the more the contractor does itself, the more money it makes. Unless precast is posttensioned or prestressed, there's no need for a precaster.

Minnesota: Contractors have precasted caps off to the side of the project. When dealing with fabricators, some issues are lifting capacity and shipping from a plant.

Tennessee: Using a full-depth precast slab, it is hard to get horizontal shear capacity, and it becomes an issue of making full- or half-width pours (half are preferred). The biggest impediment is coming up with a durable, quick deck with few connections.

Texas: Some CIP allows for better control of curve.

Tennessee: Posttensioning is a problem.

Indiana: There is agreement that posttensioning is a concern. It is not done a whole lot, and the ability to keep people trained and inspection staff available is lacking.

Texas: There has been success with the "parking garage" detail, where edges are beveled to form a V groove and there are weld plate and grout channels. By using this method, they have avoided posttensioning decks. It has been used on bridges with 15,000 ADT, with observed conditions over a 6- to 7-year period.

Question/Topic

What about modular systems of sub/superstructure?

Discussion Points

Tennessee: There has been no trouble with that. Seismic connections are a problem with modular construction for substructure. There has been success with simple span made continuous for live load. We have built several bridges with this method and used splice plates to make it continuous for dead load.

Tennessee: There is a manual being made for Utah to standardize pier caps and columns.

Iowa: Too many steps that are time dependent were hurting the acceleration part of the project for half-width full-depth panels with overlay (on 24th Street).

Question/Topic

How was ABC done in Iowa?

Discussion Points

Iowa: Most of these were done as innovative bridge projects with IBRD research money, and the state can usually get some partners with research money. There is still a premium.

Tennessee: Unless the deck bulb tee system is used, one can crowd tee beams together with a narrow spacing to save on deck.

Texas: There has been success with precast deck panels, and they are now working on a precast overhang.

Tennessee: It is better to have a monolithic, well-cured slab than to have many splices and connections. It provides better durability and a better ride.

Texas: Precast caps are a success with repeatability, and work is currently under way on a set of standards for precast cap connections. This has been used on approximately 20 jobs.

Iowa: They have used steel piling and set precast abutments on top.

Texas: They have welded connections cap connections and prefabricated a wing wall as an extension of the back wall.

Illinois: There is interest in other somewhat standardized details, while they are trying to go forward with elements and connections. They find that user costs cannot be calculated effectively, and there is no convention to do so.

Question/Topic

How does support for ABC vary across the various levels of their agency?

Discussion Points

Tennessee: Those on the public relations side and in upper management who want to minimize traffic disruption are not thoroughly considering what it takes to build a bridge of good quality and durability. The user cost issue is real, but if it costs a third more to do ABC, the state is not recouping that extra third.

Question/Topic

What about detours and how they may affect the choice of ABC?

Discussion Points

Indiana: They are not a problem.

Texas: There has been some success when the detour is large and access to emergency services is cut off.

Illinois: Is there an effort, nationally, to address this lack of communication between all echelons of staff? If we could summarize the costs but highlight the advantages, it would be useful for some of the other bridge engineers.

Tennessee: ABC is not going to be magic, nor is it going to work for many sites, such as stream crossings.

Texas: Another issue is how to set up contracts to facilitate ABC.

Minnesota: With regard to funding issues, they are considering setting aside funds to encourage districts to use ABC as an "incentive pot."

Question/Topic

What does it take to bring down the costs of ABC? How do we make it economically viable?

Discussion Points

Texas: It costs the contractor more to build it faster, so we need to figure out how to set it up so that it takes fewer resources to accomplish.

Tennessee: If it is a viable system and is economically advantageous, the marketplace will discover it. In the end, it only matters that disruption to the public is mitigated.

Question/Topic

How about A+B bidding?

Discussion Points

Tennessee: It has its place, but B costs might be unrealistic, as the DOT won't get a rebate from the public.

Texas: Disincentives get thrown into the bid, and the DOT ends up paying for it.

Illinois: Illinois has had involvement with A+B bidding and finds it helpful, but it puts a lot of pressure on a small staff. Therefore, consultants would be helpful for the state.

Question/Topic

Do you have any innovative projects or research in the participating states?

Discussion Points

Minnesota: Minnesota is currently conducting research as a follow-up response to the 2004 FHWA scanning tour, specifically using the Poutre Dalle inverted-tee system with a CIP pour.

Tennessee: Seismic connections for substructure elements are currently being researched.

SHRP 2 Innovative Bridge Design Focus Group III

1:30–3:00 p.m. EST, July 20, 2009
Participants: Mark Leonard, Colorado; Mike Beauchamp, Paul Chung, California; Mark Elicegui, Nevada; Matt Farrar, Idaho; Greg Fredrick, Wyoming; Jugesh Kapur, Washington; Kent Barnes, Montana; Dave Fredrick, Arizona; Craig Shikes, Oregon

Question/Topic

What are some of the impediments hindering the implementation of ABC?

Discussion Points

California: ABC is something that is currently being developed and requires further research into technologies for seismic regions. There has been some education of business partners and promotion of ABC in California. Contractors in California are good and geared for CIP, but getting them into prefabricated elements is going to take some work.

Nevada: Like California, contractors are good with CIP, but Nevada has no established precast industry, no certified precasters; so precast girders come from Arizona or Utah, and there is an extra expense associated with shipping.

Wyoming: Wyoming is a rural state with a limited number of contractors and fabricators, so a lack familiarity with prefabricated elements has to be overcome. Cranes limit size and weight of installation. Recently two precast abutments that would have been welded to piling with a precast girder got changed to CIP. Precast is a benefit because you don't have to get fresh concrete out onto rural roads.

Washington: Seismic connections are the biggest concern. Traditionally, Washington is a precast–prestressed state and would like to extend that to substructure, but the weight is an issue. Even with SPMTs, contractors are hesitant to share profit; and getting elements to the site is a problem. Washington has completed a deck replacement using SPMTs.

Arizona: 90% of all bridges are CIP on prestressed I-girders. Traffic control is a problem for Arizona, and detours cost a lot of money. There is not a lot of knowledge about ABC, so they are not sure if local contractors can handle implementing ABC.

Oregon: Oregon has used ABC and has put some provisions in its design manual as a starting point. A challenge is that there are a lot more cases where it can be justified if user costs are taken into account, but the issue is how to take those into account; and those costs are not necessarily savings from the highway budget. It is perceived as a benefit to the taxpayer, but you have to look at the big picture. Seismic performance of ABC elements is a concern, and similarly to Washington, 80% of the bridges in the state are made of precast concrete girders. There is an aggressive trucking industry, which has ruled out CIP construction in most cases and created substantial vertical clearance requirements during construction.

Idaho: ABC has been tried, but user costs and best value are not true cost savings. The bridge budget is strapped, so the problem is completing more expensive projects versus user costs. Where do you draw the line between true cost savings and user cost savings? When ABC is both appropriate and cost-effective, it is ideal.

Montana: Montana is another rural state, but it is trying to define a class of projects that is fit for ABC, as there is a use for ABC in a rural environment. As part of that process Montana wants to look at projects early on to see if they're appropriate for ABC. There is an ABC project to replace several bridges upcoming. In the past, a 100% precast bridge was value engineered to 100% CIP. With limited detour ability, ABC will save money. Montana is looking hard at providing flexibility within the contract. Maybe lane rental or I/D contracting methods can promote rapid construction.

Oregon: User cost consensus would be useful for putting into contracts, and contractors would have to show the savings in user costs to justify value engineering proposals.

Colorado: A lot of time can be saved with precast deck systems. In urban areas, there is a worry about the corrosive resistance of bare decks, as well as their durability. In Colorado, there is no familiarity with precast substructure. Another impediment is the need to continue to think about innovative contracting methods that allow the contractor to come up with innovative solutions. Some of their greatest successes with ABC have come from contractor proposals.

Question/Topic

What are the challenges from a design production standpoint?

Discussion Points

Idaho: The new FHWA connection manual is useful, but transporting elements out to the site needs to be considered. There need to be practical, tested seismic details.

Montana: One of the problems often seen is consultants with timelines detailing that we need to do a bridge project conventionally, when it is clear that the project is right for ABC. Montana is a seismic state and can work through most of it in most situations.

California: Connections are the major issue; they need to be validated for seismic performance. There are standards for precast girders and other elements, but one of the other design issues is the design–bid–build contracting scheme, and ABC has to be assumed with limited input from contractors. Successful projects occurred when the contractor was fully engaged with designers leading to good collaboration. They are very limited as to what can be done prior to bidding, because they can't directly interact until the contractor is hired. Culturally, very comfortable with CIP prestressed box beam bridge construction; the culture is going to have to change.

Oregon: Many of the best designers at the DOT have been lost to design companies in the private sector, and now some designers struggle even on projects where there are clear cut codes and standards.

Wyoming: One of the issues from the design standpoint is standardization, but that too presents challenges: we can't even agree on precast girder sections, tons of standard precast girders. On-site quality control is difficult; and there is a need to guarantee the same quality when precasting is done in the field as when it's done in the shop. The industry is going to have to embrace ABC at the same time owners are, and there needs to be a driving force.

Question/Topic

Is there an industry push in the west for ABC techniques?

Discussion Points

Idaho: Idaho has had a couple bridges use SPMTs, including one under construction, and has found that SPMT use is not as big of a challenge as previously thought.

Washington: The major impediment to ABC on the West Coast is the issue of cost. ABC is not perceived as a more economical way of doing things all the time. Precasting is probably more expensive than CIP, and SPMTs add more costs—while designers are being told to keep the costs down. It is hard to adapt to something that is radically different, and hard to justify more expense, unless there is an effective way to bring user costs into the equation.

Question/Topic

What is the current level of support for ABC? Is it increasing or decreasing, and what needs to change for things to get better?

Discussion Points

Colorado: There is a fairly high degree of acceptance both in structure groups and in regions for innovative ideas to decrease the time of construction. Traffic disruption mitigation is not necessarily just ABC. Structurally, interest in ABC is growing and doing a good job with recognizing opportunities.

Oregon: The degree of acceptance has been pretty good but can probably be better. A trunnion replacement on I-5 was a success, and the public was happy with the short closure period. Acceptance must improve from a couple of standpoints: mobility, which seems to trump everything else (if a method minimizes disruption to mobility, people are willing to pay more), and environmental, because less environmental impact becomes the preferred option.

California: While we were promoting ABC in various districts, ROW [right-of-way], environmental, and traffic people were extremely interested in the applications of ABC to their respective fields, and this led to a lot of increased interest.

Washington: Washington is heavily motivated and has used precast elements, moved whole bridges on rollers, and used SPMTs. The state is currently engaged in research with the University of Washington to develop seismic connections and has formed a research partnership with CalTrans. Anywhere where they can, they will use ABC.

Nevada: The interest is high in Nevada, primarily to minimize traffic disruption. Major bridges in Las Vegas have had success with segmental construction as Las Vegas has lower seismicity and higher traffic. To date the state has not completed any ABC projects; because of a lack of precasters in Nevada, precasting is not particularly cost-effective. Use of precast elements is interesting, and Nevada will be looking into it for the future. There are concerns with full-depth precast deck element durability.

Oregon: They are currently doing research into full-depth panels, looking at durability. There are issues with concrete decks that use high-performance concrete; there have been cracking and shrinkage issues. They are concerned with two-part sandwich panels' delamination and are proposing full-depth deck panels with no overlay, extra thickness, and posttensioning to get durability. Research is still in progress, and next year they will do a trial project.

Idaho: Management likes less interference with traffic and keeping the public happy, but they are struggling with a lack of funding. ABC is still perceived to be more expensive.

Colorado: Acceptance is high among upper management, as long as the structure maintains the expected durability.

Nevada: Similar sentiments as Colorado and Idaho above.

Contractors Surveys

First Contractors Survey Results

Similar to the solicitation of owners, an electronic survey for contractors was also prepared. In contrast to the state survey, which was somewhat easy to distribute by using the list of SHRP 2 agency coordinators provided by TRB, solicitation of contractors was significantly more difficult. In the end, the assistance of the Associated General Contractors (AGC) in Washington, D.C., was sought; and it was able to distribute the request for responses through various state AGC chapters. Responses were sparse and uneven. Consisting of four pages of questions, the survey garnered 17 complete responses and 29 partial responses (submitted as complete), yielding 46 viable responses. The survey has 29 questions; as with the owners survey, questions are shown or hidden on the basis of answers, so only applicable questions are presented and answered.

Questions Summary

The first page of the contractors survey asks for information regarding a specific ABC project on which the respondent may have worked. Respondents who have no experience working on an ABC project are taken to the next page. For respondents with ABC experience, the survey poses questions similar to those asked in the specific project section of the owners survey, but from a contractor's point of view. Unlike the owners survey, these questions attempt to ascertain the contractors' thoughts on ABC relative to a specific project, rather than simply to learn of a specific ABC project for the purposes of screening and further follow-up.

The second page of the survey addresses various ABC elements and techniques. The questions here are designed to help the research team learn what sort of limitations exist for the design and transport of elements, gather information about specialized equipment, and determine what types of contract documents and deliverables contractors prefer for ABC projects. The third page of the survey asks questions on contracting issues to determine what procurement methods the contractors like and dislike, which work best with ABC, and whether they are effective. The fourth page asks some closing questions regarding the future profitability of ABC projects and the changing role of the contractor. Space is provided for free responses and any other information the respondent feels would be applicable; and as before, the respondent is asked for contact information and whether or not the research team has permission to contact them with any further questions.

Contractor ABC Experience

When queried about how a project was accelerated, the most-common responses were that the project used the following:

- Large-scale prefabricated elements (63%); or
- Special lifting/hauling/moving equipment (75%).

Most of these projects appear to use a combination of the two main ABC methodologies, by using special equipment to move prefabricated elements. Contractors indicated that they were motivated to employ ABC because the owner required it.

When asked about the success rate of these projects,

- 78% of the contractors indicated that they had some problems (nonspecific) in the field; and
- These problems could be traced to unclear or inadequate specifications (25%) or design by contractor's engineers (13%).

The team was curious as to contractor opinions about the engineer of record providing information on assumed as-designed erection sequences. One of the premises of this research is that engineers will be contemplating potential innovative erection sequences on future ABC projects and designing the structure for at least one unique erection method. When questioned whether this information would be valuable to the contractor, 90% of the respondents indicated that it would. When asked a related question, specifically, would the contractors be receptive to an owner requirement that a specific method of construction be used in ABC contracts, 56% of the respondents answered no, 44% answered yes. If agencies mandate a specific method of construction, the freedom of the contractor to develop the best solution is diminished, and that has the potential to limit the number of contractors able to bid on a certain project. However, as was the case with at least one agency the research team contacted, an agency might not specifically mandate the use of a certain piece of equipment but instead require that the project be completed within a finite amount of time (an amount small enough that the only feasible way to achieve the constraint is through the use of a certain piece of equipment). The contractor would then be at fault for choosing another method of construction should problems arise and the time constraint not be met; liquidated damages could then be applied.

The survey also raised the issue of how contractors would react to a changing world of construction involving potentially costly methods of erection. When asked under what circumstances they would invest in new equipment (lease or buy) specifically to be able to complete a project using rapid replacement techniques,

- 20% said that they would not consider purchasing new equipment at all;
- 30% said they would consider it if the expected use was greater than one project; and
- 50% said they would invest only if the owner committed to many projects.

This information is consistent with comments heard informally by the team from other contractors and from contractors who have participated in prior demonstration projects. The cost of projects will be excessively high when contractors believe that a serious initiative to change business practices is lacking. When asked whether they would positively consider such a purchase if bridge construction became more standardized and based on certain methods of erection to speed the assembly, 90% responded yes. That response affirms the need for solid owner commitments to distribute costs across many projects, thus enabling contractors to recapture investment costs.

The question of contracting terms and fairness was also raised. Contractors indicated first that many forms of contracts have been used on their accelerated projects, supporting the notion that various procurement methods are suited for ABC projects and no single method is preferred. When asked if the method of contracting and the contracting terms were appropriate, 67% of respondents answered that the method of contracting was appropriate. The survey also asked whether contracts included appropriate incentives to accelerate work; 67% indicated that they did not. However, further examining those data yields some interesting observations. For those indicating that there were appropriate incentives, all believed that design–bid–build is better for ABC projects, and all affirmed the contract also included meaningful disincentives or liquidated damages if the required schedule was not met. When asked for their preference as to contract method for ABC projects, 25% of the respondents chose design–build and 75% chose design–bid–build. Notably, the results of this set of questions could be affected by the presence or absence of alternative delivery legislation in a particular state.

Contractor Personal Interviews

In addition to the electronic surveys of contractor opinions and comments on various aspects of accelerated bridge construction, contractors—or engineers who support contractors on innovative projects—were interviewed by members of the project team. Names have been withheld as are most details that would identify the response as being from a specific contractor. The discussion is divided into two parts: the first focuses on impediments to ABC implementation and the second focuses on potential new technologies or advancements that were discussed in the various interviews.

ABC Implementation Concerns

This section includes detailed information from four separate contractors, some of whom were interviewed multiple times, and two engineers who work in the area of construction support for major contractors.

Engineering Firm A

Engineer A is a consultant whose background includes design services for complex bridge erection and who also has experience in forensic engineering and construction error investigation. This individual's perspective is that the use of standardized designs prohibits innovation.

Engineer A would resolve this problem with alternative contracting, specifically using contracting mechanisms that allow engineers and contractors to team up and give the client their best ideas. Such "best value" awards are effective in bringing innovative designs and construction technologies into projects. However, such integrated delivery isn't a good fit for all projects, and contractors who don't have access to in-house or on-call specialty engineering will potentially be unable to bring innovation to small projects. In addition, the process for teaming designers and contractors as it is currently practiced is not user-friendly. The cost of pursuing work is very high, and many small designers and contractors cannot afford it. Stipends and other methods for obtaining more bidders and better ideas across all sizes of projects are needed.

Another issue Engineer A sees is a lack of commitment to new technologies and innovative materials and methods. The pace of innovation is too slow for it to be cost-effective. Part of the reason is risk. The concept of risk is easy to understand, but measuring and budgeting for the costs of risk or associated benefits at the program level is much more difficult. If an agency agrees to commit to x number of bridges using a new technology, it needs a mechanism to account for the costs associated with early failures or inefficiencies that are likely to occur as the new innovation is brought on-line but that will disappear or be minimized in future projects. The long-term benefits may be clear, but the short-term costs may be difficult to allocate or defend. Working through the learning curve and distributing the risks will require a coordinated effort among contractors, designers, and owners. A commitment to multiple projects or repetitive designs will be needed so that contractors can spread the cost of new equipment over several projects.

Engineer B

The engineering services team for Contractor A (see the following section) was interviewed. This major national contracting firm has been involved in many fast-track projects on an emergency basis, as well as planned rapid replacements. The engineering team made several suggestions.

To improve project administration, project managers should use alliance contracts, which allow designers to get contractor input early in the project. This type of contract is useful when a lack of scope definition or early project complexity makes the use of design–build contracts problematic. In an alliance contract, the contractor is brought into the project at the 30% design stage with a fixed-fee cost-reimbursable contract. The award is made on the fee, and the incentive to reduce costs is created by looking at profitability from a margin (percentage) basis. If the fixed fee is earned at a lower overall cost, the margin, or return, is higher on a percentage basis, which financial backers like to see. Contractor A has successfully used alliance contracts on projects in Portland, Oregon.

Design–build contracts are very effective when the short list includes contractors with engineering expertise or knowledge. In design–build, the field personnel can be in direct contact with designers, which expedites the construction process. For many such projects, the design-builder will self-perform complex structures and highway work but subcontract more routine work. This allows for a division of labor that maximizes resources (including time) and capabilities. The size of the letting, the schedule, and the complexity of the project are all considered by large, sophisticated contractors when deciding on what work to bid. To be competitive, they have to be able to add value through innovation capabilities, financial strength, or speed.

Incentive contracts are effective. They allow for competitive bidding without sacrificing the need for speedy completion. However, agencies that have contracts containing incentive language must be ready to staff the projects to meet accelerated schedules from their end as well. If agencies strictly enforced significant liquidated damages, projects would be done more quickly. Currently, many extensions are granted for insignificant events (e.g., weather that could be worked through).

Contractor A

As described previously, Contractor A is a major national contracting firm that has worked on both fast-track emergency projects and planned rapid replacements. Several ideas emerged from the interview.

Bridge designs for "workhorse" bridges can be standardized to allow for repetition and prefabrication. However, realignment (to prevent skew) and fill or excavation (to allow for standard span lengths) of many existing bridges may be necessary to create an inventory of standard bridges. For structural designs, the best place to start is probably with AASHTO girders. The goal is not to design each bridge individually but to use repetitive design standards and adapt the conditions (alignment, span length, width) to the standard. Typical span lengths and abutment details, modular deck components, and prefabricated structural elements could be integrated to meet specific design needs for individual bridges. However, connection details and other design issues (moisture intrusion, freeze–thaw protection, hydraulics) would have to be analyzed for each application.

Another key design issue is to use a life-cycle framework for bridge design. Designers should consider how the deck, structure, and substructure will be replaced should expansion or rehabilitation be required. Every replacement and rehabilitation cycle costs money; and rehabilitation cycles probably never restore a bridge to 100% of initial design quality. So if those costs can be accounted for in the planning and concept stages, the justification for more-expensive bridges that are more easily "replaceable" can be made.

One of the major schedule constraints for current bridge rehabilitation can be utility relocation and coordination. Utility companies are critical project stakeholders but operate more or less independently from the project team. Allowable cut-over windows and down times drive many construction schedules, and missing a cut-over window can delay a project by as much as 6 months. Any current bridge design (new or replacement) should incorporate plans to optimize utility relocation or maintain utilities. That might involve redundancies, secondary vaults, or creation of a grid to allow for rerouting. These design concepts will not necessarily help with current bridge work, but if the practice is started now, bridge work in the long term (20 to 50 years) will become easier.

With regard to the design of new structures to facilitate rapid reconstruction, Contractor A noted that it is unrealistic to think that one or only a few technologies will become dominant. An array of solutions will be needed for different site constraints, soil conditions, bridge characteristics, traffic volumes, and so forth.

Contractor B

A vice president and an area manager for Contractor B, a major midwestern bridge contractor, were interviewed. They observed that the key to speeding the construction of bridges is to design them so that as many activities as possible can overlap. For instance, if piers could be built off site while piling is driven and foundations are placed, then piers could be placed in a matter of days once foundations are completed. Beam caps could be fabricated off site with posttensioning ducts cast in. Beam caps could be placed as soon as the piers are set. Such overlapping, using off-site fabrication, would reduce 3 weeks of pier and cap work on a typical bridge to 2 or 3 days. This is consistent with the goals of ABC as executed with large-scale prefabricated elements and rapid on-site erection with conventional equipment.

The process could be further accelerated by using prebuilt modular sections in lieu of off-site fabrication. The use of modular systems with standard designs is a proven method of accelerating bridge construction. Burlington Northern Railroad uses standard plans and connection details with modular precast systems such that they can combine them in configurations appropriate to each project (e.g., abutment type X, span type Y, and foundation type Z). The individual precast modules can be ordered in bulk on basis of the upcoming construction program. Burlington Northern holds the inventory, paying at fabrication, not at placement.

Real gains in project speed can be achieved by changing administrative processes to allow for contractor input during the planning and design phases. This is especially true as transportation projects become more complicated and agencies have to replace and rebuild, rather than focus on new construction. Value engineering after the bid slows the process down. A simple way to get contractor input at early stages of the project is to allow agency personnel and consultants to be in contact with contractors without disqualifying the contractor from subsequently bidding on the work. No real bidding advantage accrues from advising designers in early stages. This issue will become increasingly difficult to manage as more and more public owners and design consultants lose in-house construction expertise. The increasing complexities of projects, the frequent changes in construction markets, the predominance of integrated team delivery in the private sector, and retirement of current construction expertise are all contributing to the loss of construction knowledge on the owner–designer teams. The problem has been growing for some time, but has gotten dramatically worse in the last 5 years.

An administrative innovation worth considering is the creation of expert panels or review boards that would be available to agencies to help with constructability reviews and to provide construction input during planning and design. The "expert witness–friend of the court" system works reasonably well in providing technical expertise to nontechnical judges and jurors; a similar model could be adopted for construction, with some system in place to ensure qualifications.

Using more design–build procurement would speed up programs as well. On a recent long-span pedestrian bridge project, the job was readvertised as a design–build project after the original bids in the traditional design, low-bid award came in too high. For the design–build project, the contractor went through conceptualization, design, procurement, and construction in the same amount of time that the design phase took in the traditional method and for far less money than the original concept as it was engineered and bid.

Another beneficial change would require a culture adjustment for some agencies. Some public owners consider the bridge builder as a value-adding, technically proficient partner in the process; but others still view the builder as a contractor that needs to be told what to do. If agencies want advice and input from contractors, they need be viewed as professional service providers, and then held accountable to those professional standards.

Contractor C

Contractor C operates under the philosophy that time, cost, and quality are the fundamental performance characteristics, and two of the three can be achieved on any project. Since quality cannot be compromised, the trade-off is between time and cost. If agencies want to replace bridges faster, they will probably have to pay more.

Totally precast bridges are possible, but most contractors are currently operating their cranes at about 90% of capacity; so transformation to large-scale precast operations would require some changes in how equipment is used on projects within the industry.

Contractor C has developed various proprietary systems and concepts to accelerate bridge construction. These include a proprietary system that eliminates the overhang brackets with special edge conditions for deck sections. This placement technology is proprietary and must be allowable in the specifications before it can be used on projects. Contractor C has also developed a top-down construction system that allows for bridge placement without the need for any ground-mounted equipment. This system minimizes environmental disturbances and provides a more economical placement system on certain types of projects, specifically those with multiple long segments with site constraints. They have used this system on a 3-mile bridge system over wetlands in the eastern United States. The bridge elements do need to be designed for construction loads, not just final traffic loads, so the designer must be aware of the contractor's intention to use such a system before designing the bridge structure and substructure. For this reason, a project like this is best suited for design–build where integration of these decisions is possible. In a design–bid–build scenario, with many options for construction, presuming such a system from the outset might result in an over-engineered structure if others pursue alternate methods of erection. Contractor C points out that many time-saving technologies are proprietary and not conducive to an open-bid, low-cost award procurement system.

Contractor D

Even in an ABC world, contractors need to be provided with an opportunity to gain a competitive edge over their competitors. Projects that have unreasonable or unknown risks will have that level of risk translated directly to dollars on bid day. The perception of risk limits potential interest in projects.

The use of precast/prefabricated elements offer many advantages in an ABC context. They lower risks that are due to uncertainty, such as difficulties with labor availability or performance quality; they may mitigate poor access or site constraints and minimize many weather problems that can affect on-site construction. With precast/prefabricated construction, the bulk of the man-hours are spent in a climate-controlled predictable work environment and subcontracted out at a fixed price. There are additional benefits of self-control and pricing control if the contractor also takes on the role of producer. Precast specifications are generally clear and rarely contested. The erection of these components is generally well understood and simple to execute.

An important facet of ABC success is repeatability. With multiple applications, all contractor risks are lowered and costs are reduced commensurately. The fewer new operations that occur on any particular project, the more time can be used to improve existing processes.

Accelerated projects can be profitable. These projects should always include incentive/disincentive clauses. In accelerated contracts, an advantage to the contractor is the speed of revenue recognition. However, a project that provides earning opportunity should not require a similar outlay or investment to earn the bonus.

According to Contractor D, things that contractors generally don't like or that contribute negatively to project success include the following:

- The use of fast-track scheduling when it is not needed;
- Onerous specifications meant to make up for incomplete or poorly prepared plans;
- Projects that are ineffectively communicated, including scope, scheduling, and phasing requirements and details on innovative details that might be used on future projects; and
- One-of-a-kind designs that limit the possible reuse of forms, require new technology, have an unsure cost history, and generally require significant training of personnel.

With regard to traffic control, agencies typically seek to minimize traffic impacts—to limit closures, detours, user costs, and impacts on essential services. The result is inefficiencies for the contractor. Traffic closures are a dangerous field operation, and the resulting limited work windows are inefficient. Work is frequently shifted to less-productive shifts, and costs of cancellation are high. From both sides of the traffic control issue, the use of closures forces each side into an undesirable situation. ABC is an interesting solution in that it may benefit both sides of this issue.

ABC Ideas and Concepts from Contractor Interviews

The following ideas are a compilation of those offered up by the various engineers and contractors individually interviewed during this process.

Traffic Closures

Disruption may be reduced by rerouting traffic to local surface streets and working quickly within a completely closed road section. This approach speeds up the overall construction time.

Overhead Construction

New elevated lanes could be built down the median of existing roadways by first closing the center lanes, then shifting traffic to the outside and working overhead.

Self-Consolidating Concrete

Self-consolidating concrete has been used by the Colorado DOT on bridge piers, but it may not have resulted in significant time savings. It did help speed construction somewhat but was not a major time-saver.

Concrete Encasement

In composite designs, the use of "big steel" encased in concrete has not been used much but does show promise. Such designs may also be long-lived and require less maintenance.

Steel Shells

Encasing concrete in steel is an alternate approach to composite action. Piers and columns could use steel as stay-in-place forms to shield concrete, especially in high-impact areas. With conventional CIP concrete, rebar coverage can be compromised, and that is expensive and time-consuming to fix later.

Increased Efficiency with Conventional Means

Conventional means and methods can still be used in an ABC situation. Contractor B has placed a 450,000-lb straddle bent and eight lines of girders during a single 8-hour road closure, including crane erection and dismantling during the closure period. Achieving such speed requires innovative means and methods by the contractor, but it is possible and can be completed with traditional equipment.

Self-Propelled Modular Transporters

SPMTs are useful in certain conditions but are not a cookie-cutter solution. They need to be employed more consistently (repetitive work) to become cost-effective.

Sliding and Launching

Many placement and positioning systems have proven successful, such as horizontal skids to move an integral, completed bridge (deck, superstructure, and substructure) into place on caissons or some similar foundation system. This system is best suited for dry channel applications where skids can be placed adjacent to the bridge. A similar system using floats instead of skids can be employed for wet channel applications. It is also possible to flood a dry basin or even create a basin to float a bridge into place. Pivot placement systems have been used, but many of them are single-use designs and probably not feasible for standard bridge designs.

Other Special Equipment

SPMTs have their place, but cranes will probably be the standard for the near term. Designs should be adaptable to a number of placement options. Customized equipment like launching gantries requires standard, repetitive design to warrant the initial capital investment. Agencies might be able to own the custom equipment and make it available to contractors on special bridge projects where it is warranted. An analogy would be the way load test frames are currently managed.

Deck Panels

Full-depth precast deck panels reduce construction time. Many such designs require connections between panels or between panels and beams, and the schedule is driven by the number of connections. Creating a box-out in the deck system and working in congested stud pockets is low-productivity, intricate work. Using a less elaborate design would reduce costs. Speed is also slowed by the need for a wearing course overlay to meet specifications. An alternative would be to set panels high and grind to tolerance, including the pavement grooving. That might eliminate a step. Larger pocket spacing to develop full-composite action while minimizing “small work” is another option.

Partial-Depth Deck Panels

Partial-depth precast panels that double as stay-in-place forms are simple and save time, but reflective cracking is an issue.

Precast Segments

The Victory Bridge in New Jersey uses precast footings, piers, caps, bents, and decks. Precasting accelerates construction, but one downside is that the substructure goes from light work to heavy work and that may restrict the number of bidders. Using precast segments allows the contractor to set a section of work in a day that might take 3 weeks with conventional reinforced, cast-in-place concrete with traditional formwork and falsework requirements. The transportation issues involving weight and geometry of large segments should be considered in designing the overall system.

Lightweight Concrete

Lightweight concrete mixes can be used to lighten sections, enabling geometrically larger section placements by the same cranes.

Postgrouting

Use of postgrouted pier foundations has potential but is currently not a time-saving technology because many owners are not open to innovation. Onerous testing and verification requirements for innovative technologies (such as postgrouted piers) slow down the project, eliminating the potential time savings created by the innovation.

Semicontinuous Construction

Using threaded rods to make girders continuous is a good compromise between structural efficiency and construction speed. Threaded rods eliminate creep redistribution. From a construction viewpoint, it is best to have either simple spans or mechanical connections.

Standard Precast Sections

Standardization of sections is helpful in increasing productivity. Most major contractors have high-capacity cranes (250-ton) or can rent them easily, so designing each member of a section to weigh about the same at the high end of crane capacity would reduce the number of operations for a given bridge. In particular, precast box girders work well for long spans.

Composite Precast Construction

The Willis Avenue Bridge in New York City used precast boxes as so-called sacrificial forms, which also allowed for aesthetic features to be cast in. This process can have schedule benefits.

Contractor Limitations

Smaller contractors will be more challenged to use renewal technologies because of capital constraints, inability to spread risk across multiple projects, and lack of access to engineering knowledge. However, some new designs can be implemented by small contractors. For instance, the FHWA 2004 scanning report discussed inverted-tee bridges with loop connections (the Poutre Dalle System) that could be placed and cast by small contractors. However, the test projects went well over budget, so cost may be a limiting factor for this design.

Ultra-High-Strength Concrete

It may be possible to combine ultra-high-strength concrete chemistry with geofoam technology to develop an expanding, high-strength concrete material that could essentially be spray-applied.

Second Contractors Survey Results

In June and July of 2009, several additional outreach activities were completed, allowing further insight into hindrances to implementing ABC methods, their causes, and their solutions. New surveys and interviews concerned with impediments and the acceptance of ABC within the industry were conducted with contractors. Contractor questions pertained to the following topics:

- What problems are typically encountered in the field on ABC projects?
- Would the availability of standardized bridge elements and systems suitable for ABC encourage greater use of ABC and lower costs?
- What are the optimal weights and sizes for prefabricated substructure or superstructure units?
- For standardizing ABC, what weight limitations are suggested for large pieces to be erected?

The results of these surveys and interviews are summarized here.

Contractor 1

This contractor does not have much experience with ABC but is constructing a project that will make extensive use of prefabricated elements. For this job, mobilization is a problem, as the bridges are in remote areas. As a result, the prefabricated elements will be in much smaller pieces than is desirable because of limitations on the cranes that have access to the site. Specifically, a 150-ton crane is being used, and that capacity is the limiting factor in deciding how large precast elements can be. This contractor performs construction services on long-span and complex bridges, so the use of specialty engineers to check structures for movement or lifting is warranted.

Contractor 2

This contractor has used ABC on precast culverts and on railroad bridge replacements, specifically having employed precast pier caps, abutments, box culverts, and superstructure sections. In this contractor's experience, mobility is also a problem, as larger cranes require more space and delivery methods for large precast pieces can be complex and cause problems and delays. Therefore, this contractor prefers a site-specific design that acknowledges constraints, mobilization, and crane needs over a standardized design approach. In many cases the engineered ABC solutions resulted from collaboration between the designer and the fabricator so that the site-specific access issues were adequately addressed in the design. Again, limits on the weights of precast structural elements should be based on crane capacities. This contractor suggests using a 100-ton crane to move footings, columns, and cap beams, and a 200-ton crane for beam elements and superstructure modules—assuming the cranes can be mobilized given the specific site constraints of a given project.

Contractor 3

Contractor 3 used ABC methods to completely demolish and replace an existing four-span bridge over an interstate with a two-span precast box girder bridge in 60 days. In this case, the owner required use of ABC because the bridge carried a major access route for local businesses. The contractor believes that the availability of standardized bridge elements would help lower construction costs and allow fabricators and contractors to work together and develop an assembly-line approach to construction. As with the other contractors interviewed, Contractor 3 sees site access as a large problem and the use of larger cranes as more expensive. This contractor suggests a reasonable maximum weight of precast bridge elements is 100,000 lb for footings, columns, and cap beams; 125,000 lb for beam elements; and 150,000 lb for superstructure modules.

Contractor 4

This contractor appears to use the ABC methods of precasting fairly regularly without major problems. The contractor's use of ABC methods was motivated by time and cost, as well as the quality of precast items, which is higher than some CIP methods. The typical problem of using precast elements to accomplish ABC is "getting the engineering and detailing right." Having a senior designer with decision-making ability on site to answer questions quickly would help. With regard to standardization, several elements have the potential to be standardized easily: precast concrete box beams, precast concrete barriers, and precast vertically posttensioned pier elements. Foundations and pier caps may not be as easy to standardize as other elements, so this contractor suggests having a menu of options that could be used in various situations, such as different soil types and site constraints. This contractor reports that lifts between 50 and 100 tons are becoming quite common and might be a good range for prefabricated bridge-element weight limits on standard bridge replacement projects.

Contractor 5

As a Japanese contractor, this firm's experiences with ABC differ significantly from those of contractors based in North America. In Japan, ABC is seen as a method not for replacing deficient structures but for mitigating heavy traffic jams at specific at-grade crossings by building viaducts and bypasses rapidly. Unlike those in the United States, Japanese contracting methods generally do not contain incentive/disincentive clauses or liquidated damages that encourage the contractor to expedite construction; for that reason ABC in the North American sense is not common. This contractor believes that each ABC project requires a site-specific design because each project must satisfy different site constraints, such as working space, storage yard, and availability of erection equipment. Because Japan has highly congested and dense urban areas, Contractor 5 suggests that prefabricated units with a maximum weight of 30 tons are feasible for installation within the tight constraints of a typical Japanese metropolis. Given that Japan is an extremely seismically active region, this contractor has several options for foundations and connections between superstructure and substructure that could be of some use to precasting efforts in states such as California and Washington.

Contractor 6

This contractor has experience with ABC projects, having employed full-depth deck panels and precast abutments and approach slabs. Some problems have arisen with these projects, such as inadequate time for shop drawing approval and fabrication and availability of special materials. The contractor suggests that future ABC projects take these possible delays into account lest the full speed of ABC not be achieved. Contractor 6 feels that standardization of ABC bridge elements would be beneficial and could help lower costs. Precast concrete abutment blocks have been particularly successful, stacked 11 high (33 ft) and posttensioned to the footing. When lead time is provided to ensure the availability of fabricated blocks, a 300-ft abutment can be erected in as little as 5 days.

Contractor 7

Contractor 7 has extensive experience with ABC and is currently involved in a project that uses innovative automated machinery to accomplish ABC with precast elements. When using ABC, speed of construction is a factor, but as one project is a 3-mile bridge over wetlands, environmental impact mitigation is also a factor. One problem encountered when using ABC methods has been owner reluctance to embrace unproven or unfamiliar methods; for an ABC project to be successful, flexibility on the part of the owner is a necessity. Standardization as a concept can be useful for lowering construction costs, but imposing standard solutions may stifle innovative solutions. This contractor believes that the size and weight of prefabricated elements does not preclude their use in an ABC project, and every option should be studied and analyzed for efficacy in a given project environment. Bridge-element weight limits suggested by this contractor are 40 to 50 tons for an individual unit, and up to 1,200 tons for modular prefabricated units.

Contractor 8

This contractor has used stay-in-place concrete form panels to achieve ABC, due to both time and access restraints. Resistance to change by local officials has been an impediment to ABC use. Some have expressed concerns about long-term maintenance; education about the method helps relieve those concerns, but works only if "the student is willing to learn."

Contractor 9

This fabricator has done precasting for several emergency projects that required the use of ABC to open structurally sound bridges as quickly as possible. Because of varying substructure and superstructure components, this fabricator believes ABC projects require site-specific designs. Maximum limits for the weights of prefabricated components suggested by this fabricator are larger than those suggested by contractors: 190,000 lb for all types of components when being shipped over land, and 400,000 lb for all types of components when movement over water is an option.

APPENDIX C

UHPC Lab Testing Report

This report was written in conjunction with Hartwell (2011).

Introduction

Problem Statement

Many U.S. highway bridges are currently in need of repair or replacement. Increases in traffic are placing excess stress on highway systems and economic constraints are limiting their renewal. Technologies and solutions must be developed that rapidly and systematically produce long-lasting highway bridges in a way that presents minimal disruption to the public (SHRP 2, 2011). Currently, work is being done to develop accelerated bridge construction (ABC) standards and codes to implement a method of rapid renewal that will address these problems as a part of the Second Strategic Highway Research Program (SHRP 2). One step in the establishment of these new standards and code provisions for ABC's use across the country, is the design, construction, and testing of a demonstration bridge in Pottawattamie County, Iowa. The demonstration bridge is to be located on U.S. Hwy 6 near Council Bluffs. The design of the three-span precast modular demonstration bridge has been developed by using various details from multiple ABC projects across the country. The goal of this complete ABC design for rapid renewal is to reduce the estimated 6-month road closure for typical construction to a 14-day road closure.

The evaluation of promising technologies selected for this demonstration project includes the simultaneous laboratory testing of specific elements or details that are deemed critical to speed of construction and service life. The unique design of the transverse deck joint over the bridge piers is one such instance in this demonstration bridge. Because the transverse deck joint uses an emerging innovative material—ultra-high-performance concrete (UHPC)—and the UHPC joint is located in the negative moment region tensile zone, this detail is critical to bridge performance. The UHPC deck joints are the focus of the laboratory testing discussed in the following section.

Research Goals and Objectives

The main goal of this research is to evaluate the performance of UHPC deck joints in a typical ABC bridge project. Several different testing needs for the UHPC joints have been identified. While a few of these needs are being addressed by researchers around the country, some aspects were evaluated through lab testing intended to support the design process. The lab tests conducted and their objectives follow:

- Test 1: Grinding of the UHPC closure joint material for the longitudinal and transverse joint closures between the precast deck modules
  - Evaluate the grindability of the cast-in-place UHPC in relation to the accelerated construction schedule.
- Test 2: Placement, handling, and quality of the UHPC material at the intersecting closure joints
  - Evaluate the constructability of intersecting, cast-in-place UHPC joints with respect to the flow characteristics and properties of the material.
  - Qualitatively assess the feasibility of the UHPC joint placement procedure.
- Test 3: Serviceability and strength of the transverse bridge deck joint at the pier
  - Evaluate the negative bending performance of the module-to-module transverse connection detail at the piers.
  - Determine the cracking moment at this location.
  - Determine the ultimate moment capacity at this location.

Additional investigations for the demonstration bridge were conducted but not included in this report. They are live-load testing of the demonstration bridge and direct tensile bond testing of the UHPC.

Arrangement of the Report

The following section of this report summarizes the background for this project and presents a review of relevant literature and past work regarding UHPC in bridge design. Subsequent sections discuss the different test methods and materials used in this research, the qualitative and quantitative results, and the conclusions and recommendations from the testing program.

Background

This section presents a summary of the broader SHRP 2 Renewal Project R04 initiative and a review of relevant literature in the area of UHPC. The purpose of this section is to provide an overview that shows how this research fits theory and practice.

Project Background

Second Strategic Highway Research Program

The Second Strategic Highway Research Program, under which this project is funded, focuses on safety, renewal, reliability, and capacity. This project, Project R04, focuses on "developing technologies and institutional solutions to support systematic rehabilitation of highway infrastructure in a way that is rapid, presents minimal disruption to users, and results in long-lasting facilities" (SHRP 2, 2011). Further, the objective of SHRP 2 Project R04 is to "develop standardized approaches to designing, constructing, and reusing (including future widening) complete bridge systems that address rapid renewal needs and efficiently integrate modern construction equipment" (SHRP 2, 2011). The project includes four phases which provide for research and development/design of the rapid renewal demonstration bridge up through construction and field demonstration. Typical bridges, characterized as bridges with up to three spans and a maximum span length of 200 ft, are the focus because of the opportunity for widespread application. This research is the result of Project R04 Phase III Task 10C, which calls for laboratory testing of the handling and constructability of the UHPC joint material. While Task 10C was the primary research conducted by the team at Iowa State University, additional investigations for the demonstration bridge were performed, including live-load testing of the demonstration bridge and direct tensile-bond testing of the UHPC.

Bridge Description

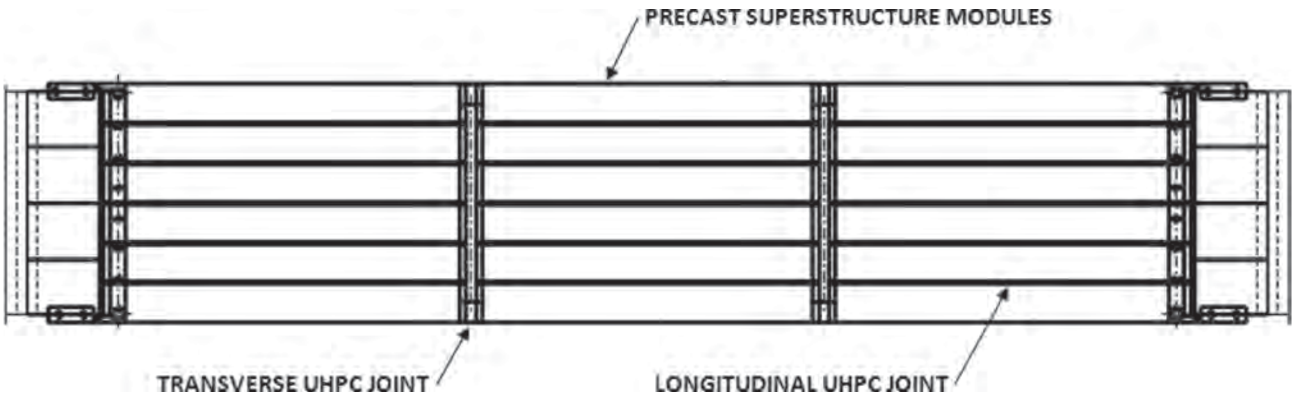
The design of the three-span precast modular demonstration bridge—which is the most prominent physical manifestation of the R04 project—was developed by the Iowa Department of Transportation (Iowa DOT) and the design engineer, HNTB Corporation, using various details from several other ABC bridges across the country and around the world. The new bridge will replace a concrete haunched girder bridge built over Keg Creek in 1953 (Figure C.1).

Figure C.1. Existing U.S. Hwy 6 bridge over Keg Creek.

The new demonstration bridge, which is to represent a typical, standardized ABC bridge design, is a precast modular bridge system which includes precast approach pavement slabs. The precast column and cap beam construction for the piers will be connected by using grouted couplers, while the precast superstructure deck modules will create a semi-integral abutment allowing rapid construction. On the deck, moment-resisting UHPC joints will connect the deck modules and create no open deck joints across the span. The SHRP 2 Project R04 demonstration bridge will be the first bridge in the United States with moment resisting UHPC joints at the piers. At 204-ft, 6-in. long and 47-ft, 2-in. wide, the new bridge will consist of a pair of 67-ft, 3-in. end spans and a 70-ft, 0-in. center span. Six precast deck modules connected by the longitudinal UHPC closure joints make up the bridge cross section (Figure C.2 and Figure C.3).

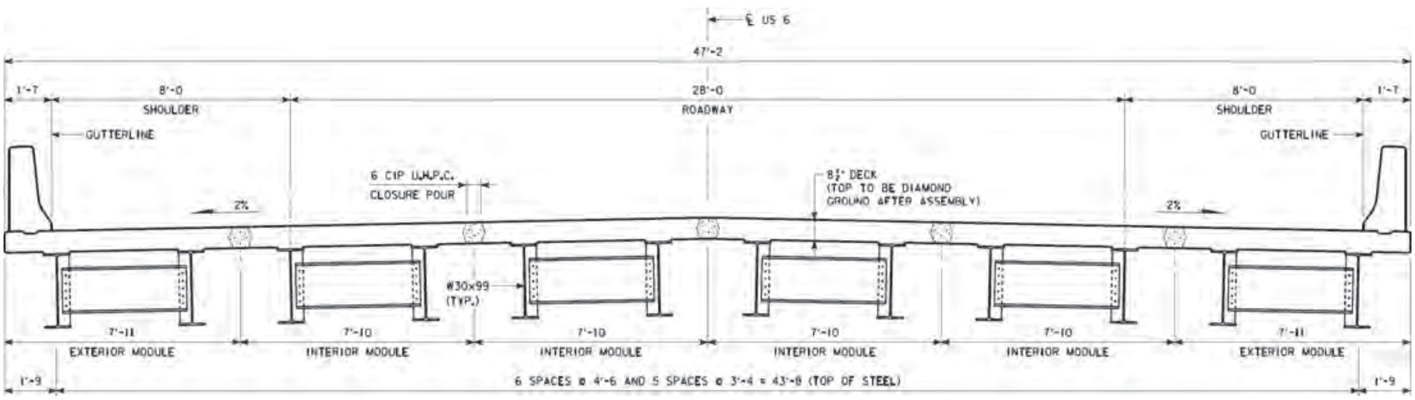
By creating a complete ABC design for rapid renewal, the design engineer has been able to reduce the estimated 6-month road closure required for typical construction to 14 days. Since US-6 in Pottawattamie County is a primary highway route, it provides an excellent platform to showcase the ABC rapid renewal concept for typical bridges.

The evaluation of design concepts contained in the demonstration project and the R04 standardized details includes the simultaneous laboratory testing of specific elements that are found to be critical to speed of construction and service life. The unique design of the transverse deck joint over the bridge piers is one such example in this demonstration bridge. Because the transverse deck joint employs an emerging innovative material in UHPC, and the UHPC joint is the tension reinforcement through the transverse connection over the piers, it is critical to bridge performance. Figure C.4 shows the precast bridge deck being added to the piers and Figure C.5 illustrates the complete bridge.



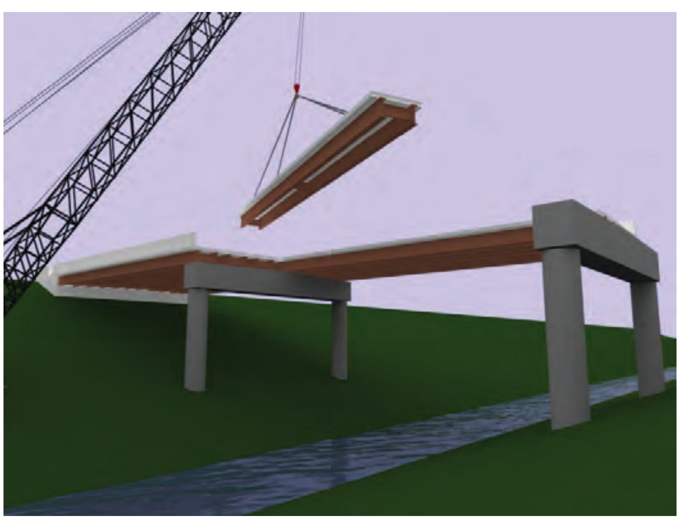
Source: Iowa Department of Transportation, 2011a.

Figure C.2. Bridge plan view.



Source: Iowa Department of Transportation, 2011a.

Figure C.3. Bridge deck cross section.



Source: HNTB Corporation.

Figure C.4. Precast deck modules.

Source: Iowa DOT Office of Bridges and Structures, 2011b.

Figure C.5. Completed SHRP 2 Project R04 demonstration bridge.

Relevant UHPC Material Background

As the United States faces the challenge of renewing its aging highway infrastructure, the longevity and durability of new structures are of particular interest. Research into many different materials and techniques to achieve durability has taken place. UHPC, a new class of cementitious material, is one technology that is increasingly being considered to provide durability and longevity in highway infrastructures because of its advanced material behaviors (Graybeal, 2009). While widespread use of UHPC has not yet taken place in the United States, multiple state departments of transportation have employed UHPC in recent demonstration projects. Project R04 uses the patented mix, Ductal, developed by Lafarge Canada. Material background, characteristics, and present applications of UHPC relating to the SHRP 2 Project R04 are discussed in the next section.

Material Background and Characteristics

The development of concrete materials known as UHPC is one of the newest advances in current concrete products. Developed in Europe beginning in the early 1990s, the first commercial UHPC product became available in the United States in 2000. In *Design and Field Testing of Tapered H-Shaped Ultra High Performance Concrete Piles* (Vande Voort, 2008), the author discusses the material characteristics that define the range of cementitious products known as UHPC. Generally, UHPC has a compressive strength that is greater than 22 ksi, contains fiber reinforcement, and uses powder components that help to eliminate some of the typical limitations of normal concrete. Additionally, Graybeal states that "UHPC has a discontinuous pore structure that reduces liquid ingress, significantly enhancing durability" when it is compared with normal strength and high-performance concretes (HPC) (2011). The low permeability is attributed to the fine powders and chemical reactivity which create an extremely compact matrix and small, discontinuous pore structure (Perry and Royce, 2010). A typical mix of UHPC contains silica fume, ground quartz, sand, cement, fibers, superplasticizer, and water.

Researchers in Europe have been at the forefront of the UHPC material testing and literature. However, increasingly, studies into this emerging material have commenced in the United States. Benjamin Graybeal has completed wide-ranging in-depth testing on the advanced material characteristics of UHPC. Graybeal (2006) reports that when compared with normal strength concrete, UHPC displays significantly enhanced material properties. Notably, compressive strength, tensile strength, rate of strength gain, and several durability properties significantly exceed those of normal concrete. While steam treatment of UHPC initially improves the material's properties considerably—increasing compressive strength by 53%, modulus of elasticity by 23%, and essentially eliminating long-term shrinkage—UHPC still shows very high compressive strengths no matter what type of curing is employed. The time needed to reach initial set for UHPC was between 12 and 24 hr, which is longer than normal concrete. However, once the initial set takes place, UHPC gains compressive strength rapidly. The speed of the setting time and strength gain can be controlled with different mix additives. The tensile strength was found to be higher than normal concrete (both pre- and posttensile cracking). With and without steam treatment, the tensile strength was found to be 1.3 ksi and 0.9 ksi, respectively. Cyclic testing for durability characteristics showed that the UHPC performs very well across the range of tested conditions, and even cracked UHPC cylinders exhibited extremely limited permeability and deterioration. Graybeal's broad material testing regimen provided a base of information critical to the development of current demonstration details that use UHPC in highway structures. Vande Voort (2008) extensively discusses UHPC material characteristics and explains how the low porosity of the UHPC microstructure is the significant factor behind the material's superior durability properties.

Horszczaruk (2004) and Graybeal and Tanesi (2007) conducted abrasion resistance testing on high-strength fiber-reinforced concrete and UHPC by using the ASTM C944 standard procedure (ASTM, 1999). Horszczaruk focused on 12.0 ksi to 14.5 ksi compressive strength concretes that included larger aggregates than are present in any UHPC mix design. Graybeal and Tanesi conducted abrasion resistance testing on UHPC after different curing protocols and differing surface treatments: cast, blasted, and ground. They found that steam-based curing significantly affects the abrasion resistance of the material. Untreated specimens lost nearly 10 times the amount of mass compared with specimens undergoing steam-curing treatment. Additionally, smoother textures tended to be more resistant to abrasion than is the blasted finish.

Ductal, the patented UHPC mix from Lafarge Canada that is commercially available in the United States, was developed by three companies—Lafarge, Bouygues, and Rhodia—nearly two decades ago. Ductal is composed of silica fume, ground quartz, sand, cement, high-tensile-strength steel fibers, high-range water reducer, and water. The high-tensile-strength steel fibers included in the mix are 0.008 in. in diameter and 0.5 in. long. When Ductal is used, the silica fume, ground quartz, sand, and cement are combined into a premix that arrives in bags along with the steel fibers and high-range water reducer. Ductal JS1000 is the specific mix recommended when UHPC is being used as a joint closure material. The product data sheet from Lafarge Canada contains further material information, batching, and placement guidelines for the Ductal JS1000 product (Lafarge Canada).

Current Bridge Applications

UHPC has been used in many different bridge components. Applications range from UHPC I-girders and complete redecking systems to UHPC joint closures and precast concrete piles. The most-prominent applications of UHPC in bridge superstructure components are discussed in this section.

I-Girder

In the United States, two simple-span prestressed concrete girder bridges have been constructed with UHPC I-girder shapes. The Mars Hill Bridge in Wapello County, Iowa, was the first to be constructed in 2005 by Wapello County, the Iowa DOT, and the FHWA (Figure C.6). The bridge is a 111-ft single-span bridge with a three-girder cross section at 9-ft, 7-in. spacing and a 4-ft overhang. The UHPC I-girders are modified Iowa 45-in. bulb tee sections. To save material in the section, the web width was reduced by 2 in., the bottom flange by 2 in., and the top flange by 1 in. (Bierwagen and Abu-Hawash, 2005).

The Virginia DOT constructed the second bridge using UHPC I-girder shapes. One 81.5-ft-long span of the 10-span Route 624 Bridge over Cat Point Creek was built with five UHPC I-girders. The girders were 45-in.-tall bulb tee beams and contained no conventional steel stirrups for shear reinforcement because the steel fibers present in the UHPC provided adequate shear resistance (Ozyildirim, 2011).

Bulb Double Tee Girder

Another deployment of UHPC in bridge construction in the United States involves the design and implementation of a UHPC bulb double tee girder or pi section in Buchanan County, Iowa (Figure C.7). The three-span bridge (112 ft, 4 in. long, 24 ft, 3 in. wide) used three pi sections in the 51-ft, 2-in. center span. The Iowa DOT worked in collaboration with the FHWA's Turner–Fairbank Laboratory, Iowa State University's Bridge Engineering Center, and Massachusetts Institute of Technology to develop the UHPC pi section (Keierleber et al., 2007).

Source: Bierwagen and Abu-Hawash, 2005.

Figure C.6. UHPC I-girder bridge, Wapello County, Iowa.

Source: Berg, 2010.

Figure C.7. Pi-girder bridge, Buchanan County, Iowa.

Decking System

A two-way ribbed precast slab system, or waffle slab, was developed to capitalize on the strength and durability characteristics of UHPC. The longevity of bridge decks could be increased by UHPC's low permeability, and the strength of the material helps reduce the mass of material required. This waffle slab system has undergone testing at Iowa State University's Bridge Engineering Center, and construction of the bridge in Wapello County by the Iowa DOT took place in the fall of 2011.

Field-Cast Joint Closures

In two instances UHPC has been used as a deck joint closure material in the United States (Figure C.8). Two bridges in New York State used UHPC as a joint closure material between precast deck panels. The New York State DOT used the UHPC joint fill material in the transverse deck joints of the Route 23 Bridge in Oneonta and the longitudinal deck

Source: Perry and Royce, 2010.

Figure C.8. UHPC deck closure joints.

joints of the Route 31 Bridge in Lyons. However, unlike the current demonstration project, the transverse deck joints in the Oneonta bridge were located in the positive bending moment region, placing the UHPC joint in compression.

Graybeal (2010) conducted testing on the UHPC bridge deck connections used in the New York State DOT's bridges under static and cyclic structural loading at the Turner–Fairbank Highway Research Center's Structural Testing Laboratory. Simulated wheel patch loads were applied adjacent to the UHPC joints for four transverse joint specimens. The test specimens were arranged and tested so that flexural stresses, which would be caused by traffic, were oriented parallel to the joint. It is important to note that the test setup focused on local flexural behaviors of the deck only. The test did not account for global flexural behaviors of the deck and girder system. Neglecting overall flexural effects of the deck and girder system, Graybeal found no significant UHPC or interface cracks parallel to the transverse joint. Graybeal also tested two specimens representing the longitudinal joint connection. Wheel patch loads were applied adjacent to the joint, and the specimen was oriented so that flexural stresses occurred perpendicular to the joint. Graybeal suggested that the field-cast UHPC joint wouldn't necessarily debond at the connection interface under the low loads and small direct flexure in the longitudinal direction.

Methods

The evaluation of promising technologies for the SHRP 2 accelerated bridge construction demonstration project includes the simultaneous laboratory testing of specific elements or details that are found to be critical to speed of construction and durability. The unique design of the transverse deck joint over the bridge piers is one such instance in the demonstration bridge. Because the transverse deck joint utilizes an emerging innovative material in ultra-high-performance concrete (UHPC) and the UHPC joint experiences high levels of flexural tension, the material is critical to the bridge's performance and thus the focus of the lab testing.

Several areas of testing for the UHPC joints were identified during the design of the demonstration bridge. While a few of these areas have been addressed by researchers around the country, some aspects of the demonstration bridge were examined through further lab testing to support the bridge's design.

The lab tests conducted for this study include abrasion testing of the UHPC closure joint material, constructability testing of the intersecting deck joints, and strength and serviceability testing of the transverse deck joint at the pier. The methods used in this testing regimen were selected to address these objectives: to determine the grindability of cast-in-place UHPC, to assess the feasibility of intersecting deck joint placement, and to evaluate the bending performance of the module-to-module transverse connection detail at the piers.

UHPC Abrasion Testing

Abrasion testing of cast-in-place UHPC was conducted to determine the early age grindability of the material when used in the demonstration bridge. Specifically, a period of time in which the contractor can grind the joint material without causing damage to the joints or equipment had to be identified. The demonstration bridge specifications indicated that the UHPC closure joint attain 10,000 psi of compressive strength before it should be ground. This test helped to determine the relative ease of grinding for the material after the 10,000 psi threshold has been reached. Experimental variables for this test included the maturity of the UHPC and the specimen surface finish.

Mixing and Casting UHPC Cylinders

To complete the abrasion testing, UHPC cylinders were cast. From one cylinder, four test specimens were produced. The batching and casting was completed in 1 day with three batches of the same UHPC mix design proportions. In total, 24 UHPC cylinders, each 4 in. by 8 in., were cast from three batches of UHPC to create the test specimens. Twelve of the cylinders were cut in half to create four surfaces per cylinder for testing. Each surface was then treated as a separate specimen. The other 12 cylinders were used in compressive strength tests that were used to correlate maturity and compressive strength of the UHPC to abrasion test results. Cylinders were cured at 40°F, 70°F, and 100°F and were tested 2, 4, 7, and 28 days after casting (Table C.1).

Ductal JS1000, from Lafarge Canada, was the UHPC premix material used for the abrasion test specimens. The UHPC mix designed by Lafarge Canada included Ductal JS1000 premix, water, Chryso Premia 150 superplasticizer, and steel fiber. UHPC mix design proportions are included in Table C.2 and Table C.3.

The batching procedure was adapted from the procedure recommended by Lafarge Canada for the Ductal JS1000 premix

Table C.1. Specimen Curing and Testing Matrix

Days After Placement	40°F Cure	70°F Cure	100°F Cure
2	4 specimens	4 specimens	4 specimens
4	4 specimens	4 specimens	4 specimens
7	4 specimens	4 specimens	4 specimens
28	4 specimens	4 specimens	4 specimens

Table C.2. Abrasion Test Specimen, UHPC Mix Design 1 (0.58-ft³ batch)

Material	Weight (lb)	Mix Proportion (%)
Ductal JS1000 premix	79.9	87.4
Water	4.7	5.2
Chryso Premia 150	1.1	1.2
Steel fiber	5.7	6.2
Total	91.4	100.0

Figure C.9. Lancaster Products mixer.

in an Imer Mortarman 750 mixer to batch the UHPC mix design (Lafarge Batching Procedure). A Lancaster Products 1.5-ft^3 mixer was used to mix the two 0.58-ft^3 batches and one 0.40-ft^3 batch of UHPC for the test specimens (Figure C.9).

Abrasion and Compressive Strength Tests

Three surface finish types were tested for grindability during the abrasion testing: a rough top surface, a diamond cut surface, and a smooth formed surface (Figure C.10). After curing the cylinders at the various temperatures, four specimens were produced by cutting each cylinder in half. For one cylinder at each cure temperature and time, one rough top surface, one smooth form surface, and two diamond cut surfaces were tested.

To evaluate the UHPC material for grindability, testing was completed following ASTM C944, the Standard Test Method for Abrasion Resistance of Concrete or Mortar Surfaces by the Rotating-Cutter Method (ASTM, 1999). For fabricated concrete or mortar-based specimens, this test helps indicate the relative wearing resistance. ASTM C944 uses a drill press with an abrading cutter rotating at 200 rotations per minute. The normal force exerted on each specimen surface in this test is 22 ± 0.2 lbf. The rotating cutter head leaves a 3.25-in.-diameter abraded circular area (Figure C.11 and Figure C.12).

Following the ASTM C944 standard, the initial mass of each specimen was determined to the nearest 0.1 g. The specimen was then clamped into the testing device such that no rotation could take place. To ensure the quality of each test, special care was taken with the shaft of the rotating cutter head to ensure that each specimen surface was normal. Once properly secured in the device, the motor was started and the cutter was slowly lowered into contact with the specimen. Following continual abrasion of the specimen for 2 min, the specimen was removed from the testing device, cleared of dust and debris, and massed again to the nearest

Table C.3. Abrasion Test Specimen, UHPC Mix Design 1 (0.40-ft³ batch)

Material	Weight (lb)	Mix Proportion (%)
Ductal JS1000 premix	54.8	87.4
Water	3.2	5.2
Chryso Premia 150	0.7	1.2
Steel fiber	3.9	6.2
Total	62.7	100.0

Figure C.10. Diamond cut and rough top surface finishes.

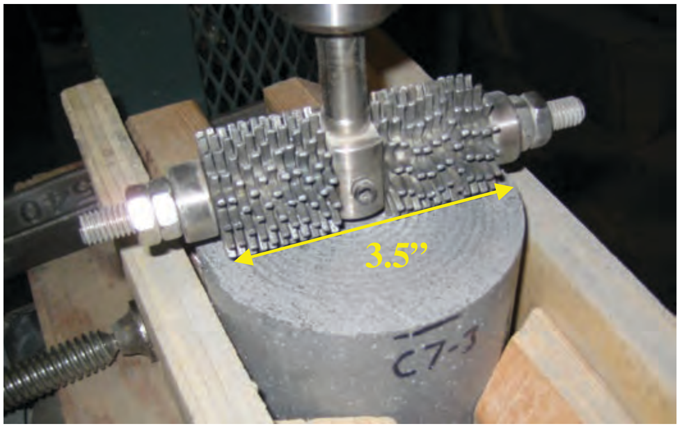


Figure C.11. Rotating cutter head.

0.1 g. This process was repeated two additional times for each individual specimen. In total, 12 abrasion tests were conducted on each test day. Four tests were completed for each of the three curing temperatures at 2, 4, 7, and 28 days after batching.

Compressive strength tests were simultaneously conducted on corresponding cylinders of the same age and curing temperature using ASTM's Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens (ASTM C39). To ensure the ends of the cylinders were smooth and parallel, the ends of each cylinder were cut before testing. Because of the high strength of the UHPC, capping compound was not used during the compressive strength testing.

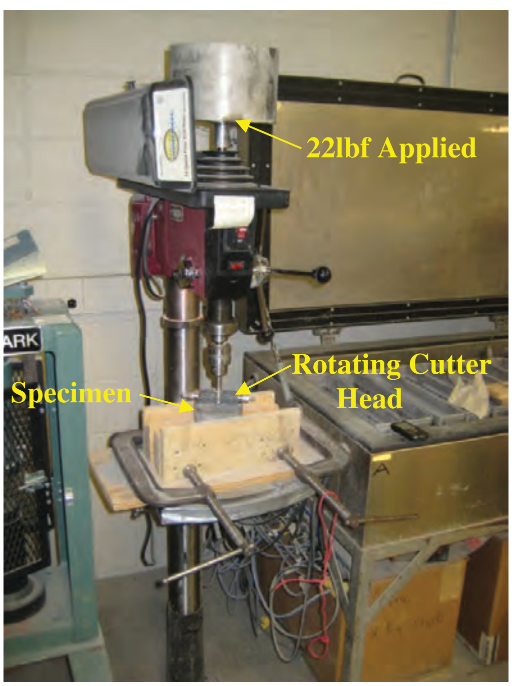


Figure C.12. ASTM C944 test setup.

Joint Constructability Testing

A full-scale deck joint mock-up was constructed to evaluate the constructability of intersecting cast-in-place UHPC joints, qualitatively assess the feasibility of the UHPC joint placement procedure, and provide an opportunity to demonstrate casting the material for bridge designers and contractors. Effectively casting the UHPC deck joints is essential to the construction schedule and performance of the SHRP 2 ABC demonstration bridge project. This mock-up, which replicated the conditions in the demonstration bridge, provided an opportunity to understand the flow characteristics and properties of the UHPC mix design with respect to the actual conditions. This helped the bridge designer and contractor plan material staging and placement.

Designing and Constructing Intersecting Joint Formwork

Formwork for a representative portion of the intersection region of transverse and longitudinal UHPC deck joint was designed and constructed for the constructability testing (Figure C.13). The finished intersecting joint specimen was 6 ft, 6 in. long by 7 ft, 4 in. long in the transverse and longitudinal joint directions, respectively. The transverse joint, which runs perpendicular to the bridge traffic, measured 16 in. wide, while the longitudinal joint—which runs parallel to the direction of traffic—measured 6 in. wide (Figure C.14). Each joint was 8.5 in. thick, matching the precast module concrete deck thickness.

The specimen contained all of the steel reinforcement in the joint detail, including those that protruded from the precast deck modules. To replicate the demonstration bridge conditions fully, the specimen was constructed with a 2% cross slope.

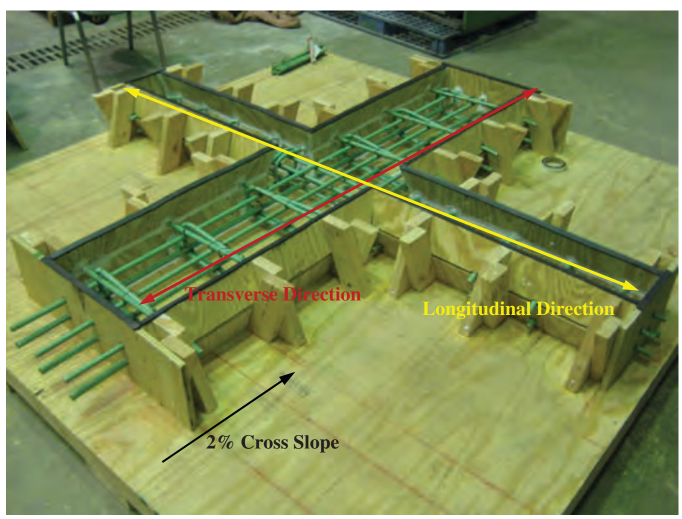
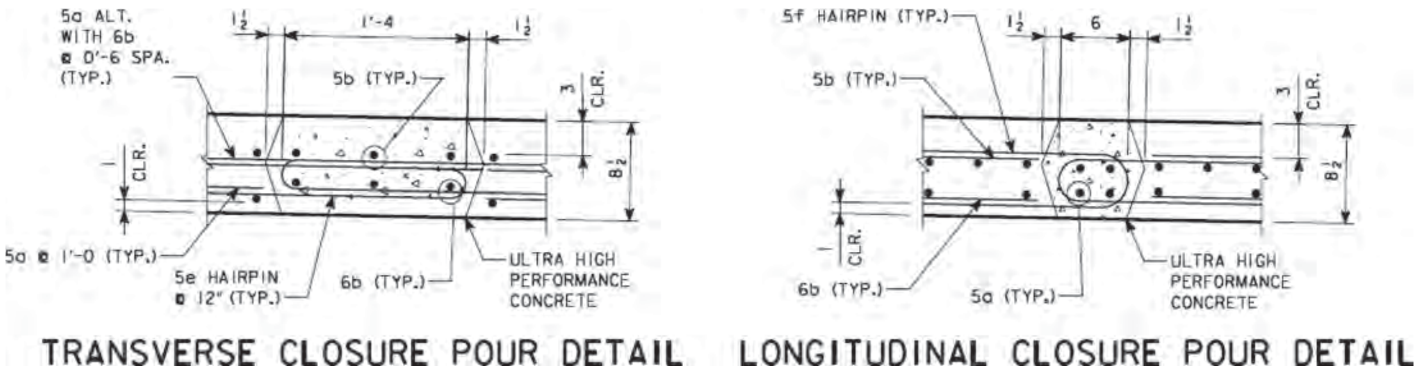


Figure C.13. Intersecting joint specimen formwork.



Source: Iowa Department of Transportation, 2011a.

Figure C.14. UHPC deck joint details.

To replicate the proposed demonstration bridge UHPC placement technique, acrylic glass bulkheads, which aimed to prevent the formation of cold joints, were fabricated and installed (Figure C.15). In the test specimen, the bulkheads were located in the longitudinal joint approximately 2 in. from the transverse joint. The placement of the vertical bulkheads in the longitudinal joints allowed for continuous placement of the transverse closure joint.

Mixing and Casting Intersecting Joint Specimen

The UHPC mix design that was specified by Lafarge Canada for the SHRP 2 ABC demonstration bridge was used during the constructability testing of the intersecting joint detail. The JS1100RS 60/40 mix design included Ductal Light Grey Premix, water, Chryso Premia 150, Chryso Optima 100, and steel fibers. A technical service engineer from Lafarge Canada was present during the batching and casting of this specimen (Table C.4).

Under the supervision of the Lafarge Canada representative, batching was completed for the entire intersecting joint specimen. For each 5.11-ft^3 batch, 14 50-lb bags of Ductal Light Grey Premix were emptied into the drum and mixed to gain homogeneity. Once completely mixed, the Premia 150, Optima 100, and water were added. The batch was then mixed until the turning point was reached. At the turning point, the wetted batch changed from its granular mix state to a semiplastic state; at that point the steel fibers were incorporated into the batch. After the steel fibers were fully integrated, the batch could then be discharged (Lafarge Batching Procedure).

The total volume of the intersecting joint specimen was 9.54 ft^3. Because the Imerman 750 mixer used in the lab is limited to a 5.11-ft^3 batch of UHPC and because cylinders for compressive strength tests needed to be cast as well, the specimen was cast in three batches. By using the acrylic glass vertical bulkheads, the transverse joint could be partially filled with the first batch and then completed with the second (Figure C.16). In an attempt to gain a homogeneous

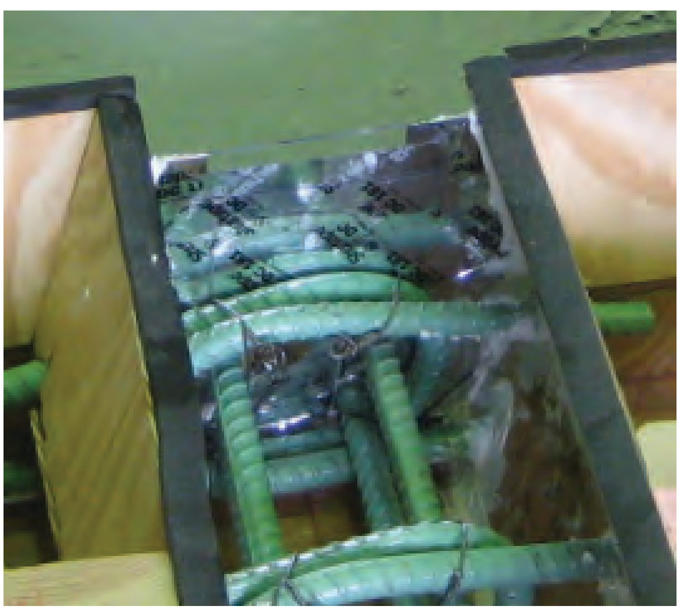


Figure C.15. Acrylic glass vertical bulkhead.

Table C.4. Constructability Test Specimen, UHPC Mix Design 2 (5.11-ft^3 batch)

JS1100RS 60/40—Light Grey Premix		
Material	**Weight (lb)**	**Mix Proportion (%)**
Ductal JS1100 premix	700.0	86.74
Water	47.8	5.92
Chryso Premia 150	5.7	0.71
Chryso Optima 100	3.8	0.47
Steel fiber	49.7	6.16
Total	807.0	100.00

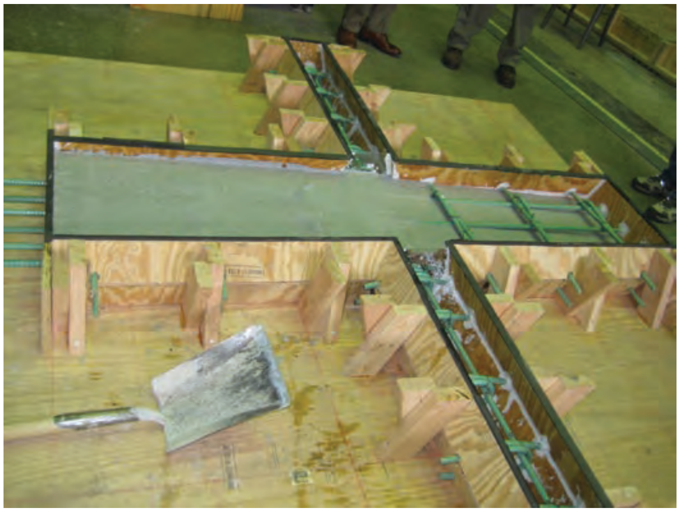


Figure C.16. Placement of transverse UHPC joint.

placement and eliminate a horizontal cold joint in the transverse joint, the first batch was agitated in the mold to disrupt the drying "skin" that began to form within 5 min of placement. The remaining UHPC from the second batch was then used to place one side of the longitudinal joint. Finally, the third batch filled the last portion of the longitudinal joint as well as the 18 cylinders, each 4 in. by 8 in., used for compressive strength testing.

The UHPC for the entire specimen was placed from the low end of the 2% cross slope to the high end. As recommended for the demonstration bridge, plywood top forms were attached as the formwork filled (Figure C.17). At the high end of the joint, "chimneys" were constructed in the top form to allow overfilling with UHPC, build up hydrostatic head pressure, and ensure that the entire joint was filled. While casting the specimen, no vibrating of the UHPC was needed because

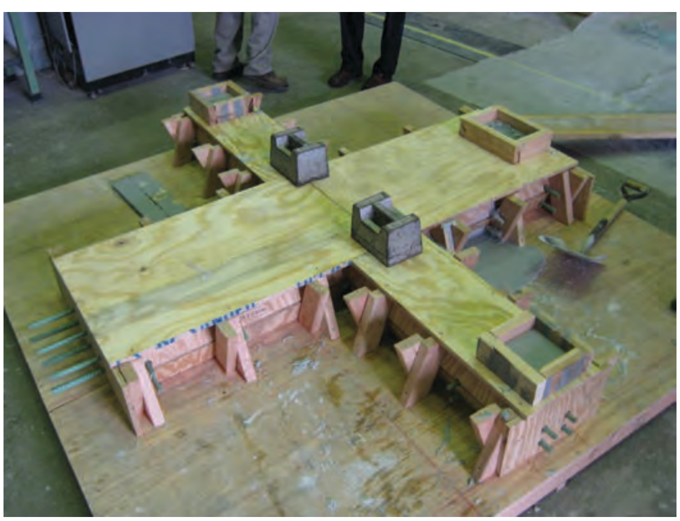


Figure C.17. Plywood top forms.

it is a self-consolidating material. After the mock-up was cured and removed from the forms, it was cut up into several sections to examine consolidation and locations of potential cold joints.

Transverse Joint Strength and Serviceability Testing

The module-to-module transverse connection used in the SHRP 2 ABC demonstration bridge was a unique and critical detail that had neither been implemented in a bridge nor tested to quantify structural performance (Figure C.18). To evaluate the negative bending performance of this detail, a mock-up of the connection was constructed and subjected to increasing levels of moment. Testing was completed through static and cyclic testing at service-level conditions, as well as static ultimate moment conditions.

Designing and Constructing Full-Scale Transverse Connection Specimen

A full-scale specimen replicating the module-to-module transverse connection at the pier was designed and constructed in the Iowa State University Structural Engineering Laboratory to allow for testing of the connection details. The specimen had a total length of 40 ft, 6 in. and comprised two prefabricated deck modules connected with the transverse UHPC joint under investigation. The specimen length was chosen so that the negative moment inflection points were located inside the specimen's ends. Each precast deck module consisted of two W30X99 steel girders cast compositely with a concrete deck that was 7 ft, 4 in. wide and 8.5 in. thick. The W30X99s used in the prefabricated deck modules were 20 ft long and were connected by two MC18X42.7 diaphragm members. These members constituted the steel frame of each module.

The steel frames for the prefabricated deck modules were fabricated in Muscatine, Iowa, and shipped to the laboratory at Iowa State University. In the laboratory, the modules were constructed in an upside-down orientation so that the concrete deck could be cast on the ground (Figure C.19). The two steel frames were placed in their respective deck slab forms on arrival. All epoxy-coated reinforcing steel bars present in the deck slab were placed and tied before setting the frames.

General requirements for the hardware, structural steel, and reinforcement bars used in the prefabricated deck modules were as follows:

- ASTM A709 grade 50W structural steel;
- High-strength ASTM A325 type III bolts;
- ASTM A563 heavy hex nut grade DH3;
- ASTM F436 type III washers; and
- Grade 60 epoxy-coated rebar.

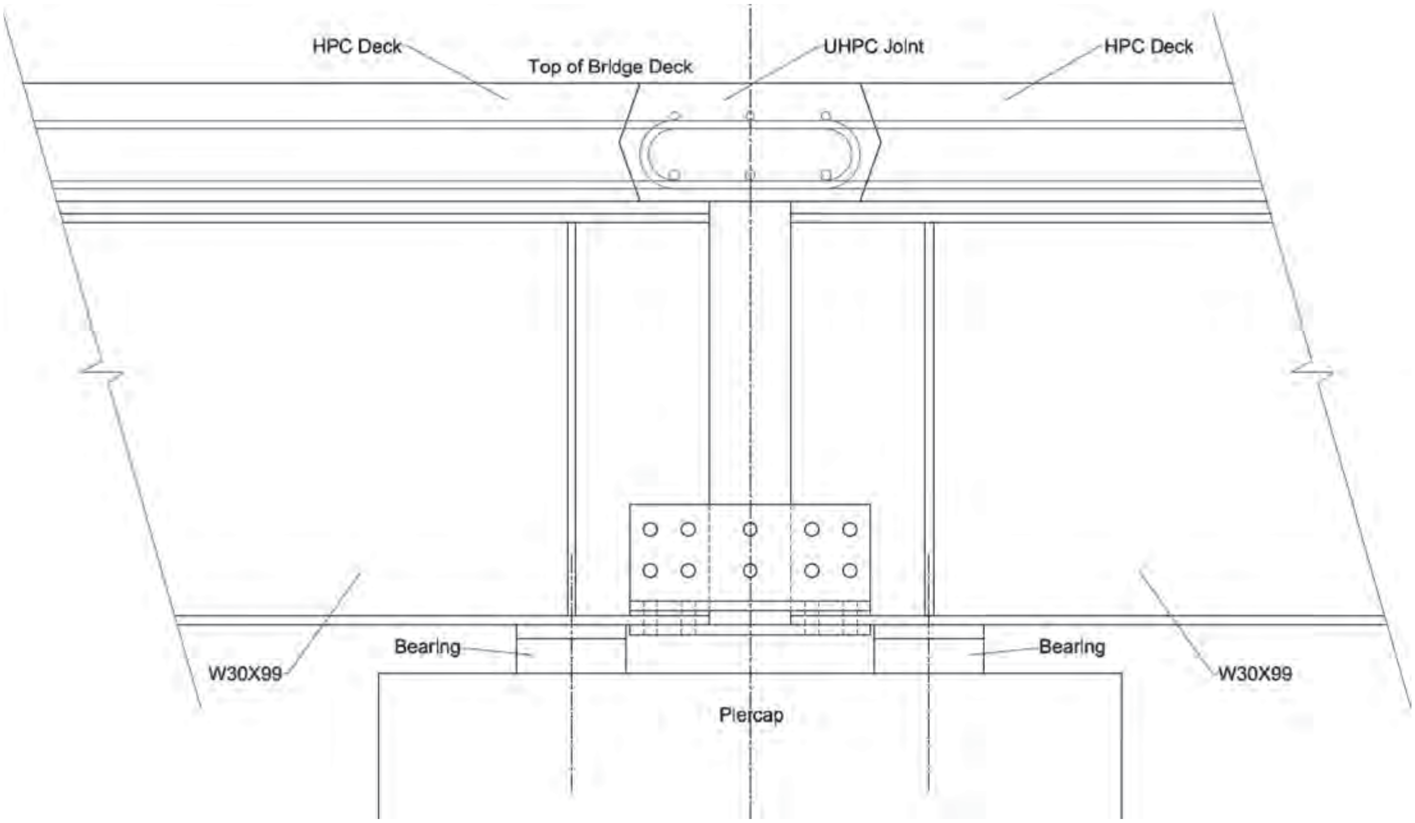


Figure C.18. Module-to-module transverse connection detail.

CV-HPC-D mix, an Iowa DOT high-performance concrete bridge deck mix specific for the western region of Iowa, was used for the specimen's deck slab. The mix (Table C.5) contains river rock commonly found in western Iowa.

Eight and a half cubic yards of HPC were needed for the two deck modules. In addition, 24 cylinders, each 4 in. by 8 in., and six beams, each 6 in. by 6 in. by 3 ft, were cast for compressive and flexural strength tests (ASTM C39 and ASTM C78).

For safety reasons, as noted previously, the specimen was cast in an upside-down orientation. Once cast, each deck module was positioned on supports, one temporary and one

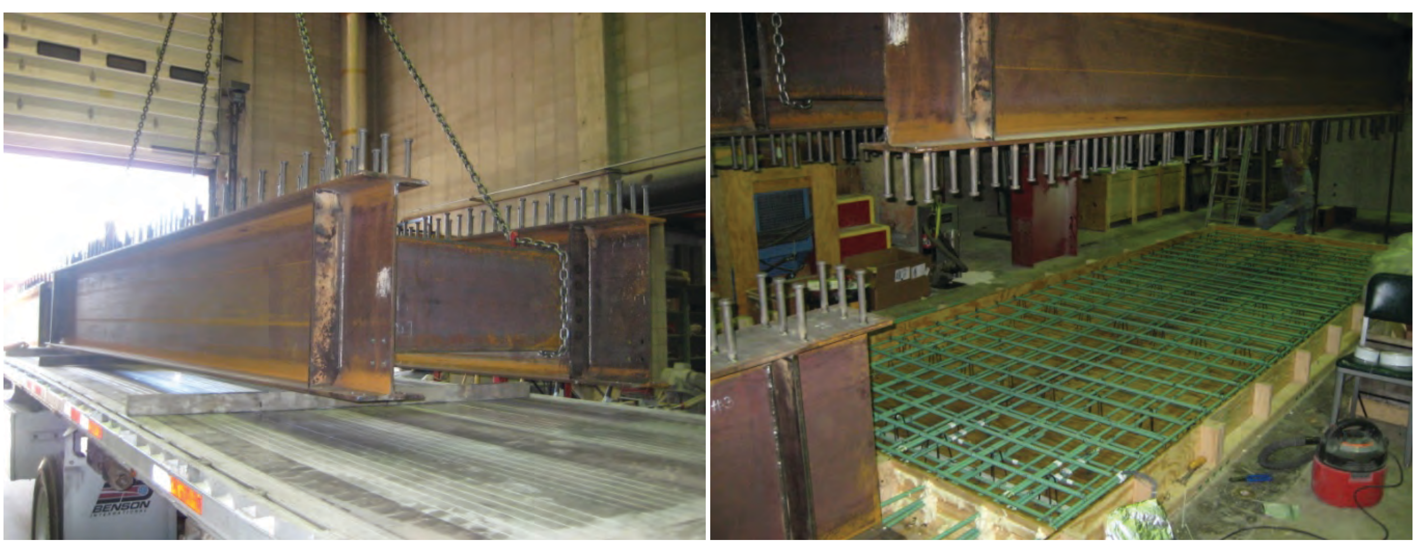


Figure C.19. Prefabricated deck module construction.

Table C.5. HPC Mix Proportions

Material	Relative Proportion (by volume)
Cement	0.126
Fly ash	20% max replacement by weight (mass)
Water	0.148
Coarse aggregate	0.300
Class V aggregate	0.366
Air	0.060

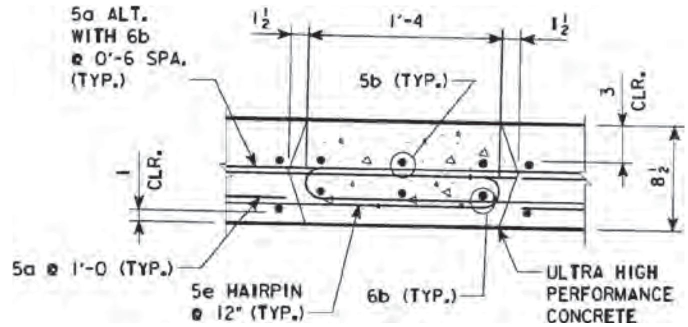


Figure C.21. Transverse UHPC deck joint detail.

permanent, then the two modules were connected via four steel angle plates (Figure C.20 and Figure C. 21). The steel angles were designed to be the compressive force path through the transverse module-to-module connection detail. Similarly, the transverse UHPC joint would act as the tension force path for the connection detail. The specimen was supported by temporary supports under the transverse joint formwork until the UHPC material was cured. This casting sequence was chosen to replicate the demonstration bridge design's simply supported condition for dead load and continuously supported condition for live load.

Casting the Transverse UHPC Joint

Using the UHPC mix design and batching procedure specified by Lafarge Canada, the transverse joint was cast in two 5.11-ft^3 batches (Table C.4). In addition, 45 cylinders, each 4 in. by 8 in., were cast for compressive strength testing at 1, 2, 4, 7, and 28 days. The cylinders were cured at 60°F, 70°F, and 90°F to evaluate strength at temperatures that could occur during the SHRP 2 ABC demonstration bridge construction period.

Figure C.20. Completed deck module.

Before placing the UHPC joint, the adjoining HPC deck surfaces were prepared as recommended by Lafarge Canada. The HPC surfaces were removed from the forms, brushed with a steel-brush grinding head, and pressure washed with water. On the morning of the UHPC placement, the HPC surfaces were wetted to attain saturated surface dry conditions during UHPC placement. Figure C.22 shows joint and form preparation and UHPC placement.

According to the project specifications, once the UHPC material reaches 14,000-psi compressive strength, traffic may be allowed on the demonstration bridge. This meant that specimen testing needed to commence immediately when the 14,000-psi compressive strength threshold was reached. On the basis of results from the UHPC in the constructability testing, the investigators expected the threshold would be reached 4 days after placement. The entire load testing frame for the specimen was constructed and load actuators positioned before placement of the transverse UHPC joint so that testing could commence as soon as the UHPC reached 14,000-psi compressive strength.

Instrumentation for the Transverse Module-to-Module Connection Specimen

To monitor the behavior of the deck and deck joint, strain levels were monitored throughout the thickness of the deck at locations on, in, or near the joint. A combination of embedded bonded strain gauges, surface-mounted strain gauges, and string potentiometers for deflection were installed on the specimen to analyze the performance of the entire transverse module-to-module connection (Figure C.23 and Figure C.24). Bonded strain gauges were affixed to reinforcing bars in both precast HPC deck slabs and in the transverse UHPC joint.

The specimen was instrumented at likely locations for cracks to occur. Those locations were as follows:

- To monitor strains in the prefabricated deck panels, embedded bonded gauges were placed on the steel hairpins where the longitudinal deck reinforcement terminates. Twelve

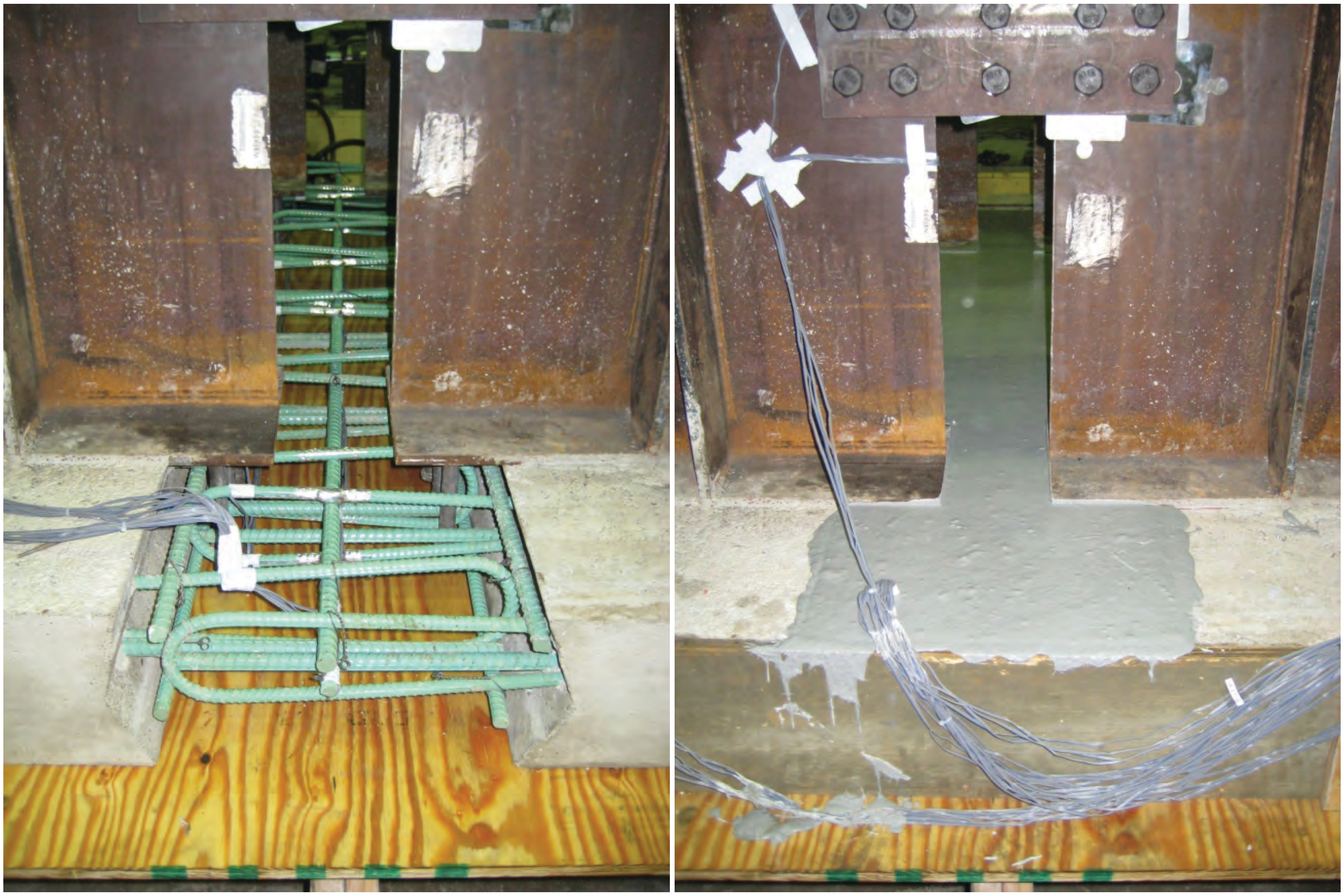


Figure C.22. UHPC joint preparation and placement.

embedded bonded strain gauges were used on the upper and lower legs of the hairpins at locations directly over the steel beams and at midspan of the deck between the beams on both superstructure modules. In addition, eight surface-mounted strain gauges (three on the top surface of the deck and one on the bottom for each module) were used as well. An additional six embedded bonded strain gauges (three in each deck module on the top mat of reinforcement) were placed on longitudinal reinforcing bars at the termination of the hairpin lap splice.

- To identify strain levels at the interface of precast HPC deck panel and the UHPC joint, 12 surface-mounted gauges (three on the top surface, three on bottom surface for each interface) were used.
- To quantify strain at the centerline of the UHPC joints, 12 embedded bonded strain gauges were used on the upper and lower legs of the hairpins at the transverse centerline of the joint corresponding to the locations of each steel beam and at the longitudinal centerline of the deck between the beams. In addition, six surface mounted strain gauges were mounted on the top and bottom surfaces of the UHPC joint at corresponding locations to the embedded strain gauges.
  - Eight embedded bonded strain gauges were placed on the straight transverse lacer bars within the UHPC joint.
  - Four embedded bonded strain gauges were mounted to the surfaces of the steel angle connectors between opposing steel beams across the joint.
- Specimen displacements were measured with seven string potentiometers mounted to the laboratory floor (three directly under the centerline of the transverse joint, one at each bearing location, and one at the transverse centerline of each module). (See Figures C.25 and C.26.)

Calculating Testing Load Levels

Design moment values were obtained from the demonstration bridge design engineer, HNTB. Using the AASHTO HS-20 vehicle and a conservative lateral live-load distribution factor of 1.0, Service Level I design live-load moment was calculated at −394 kip-ft. The design moment for Service Level II was −512.2 kip-ft. Ultimate design moment capacity for the

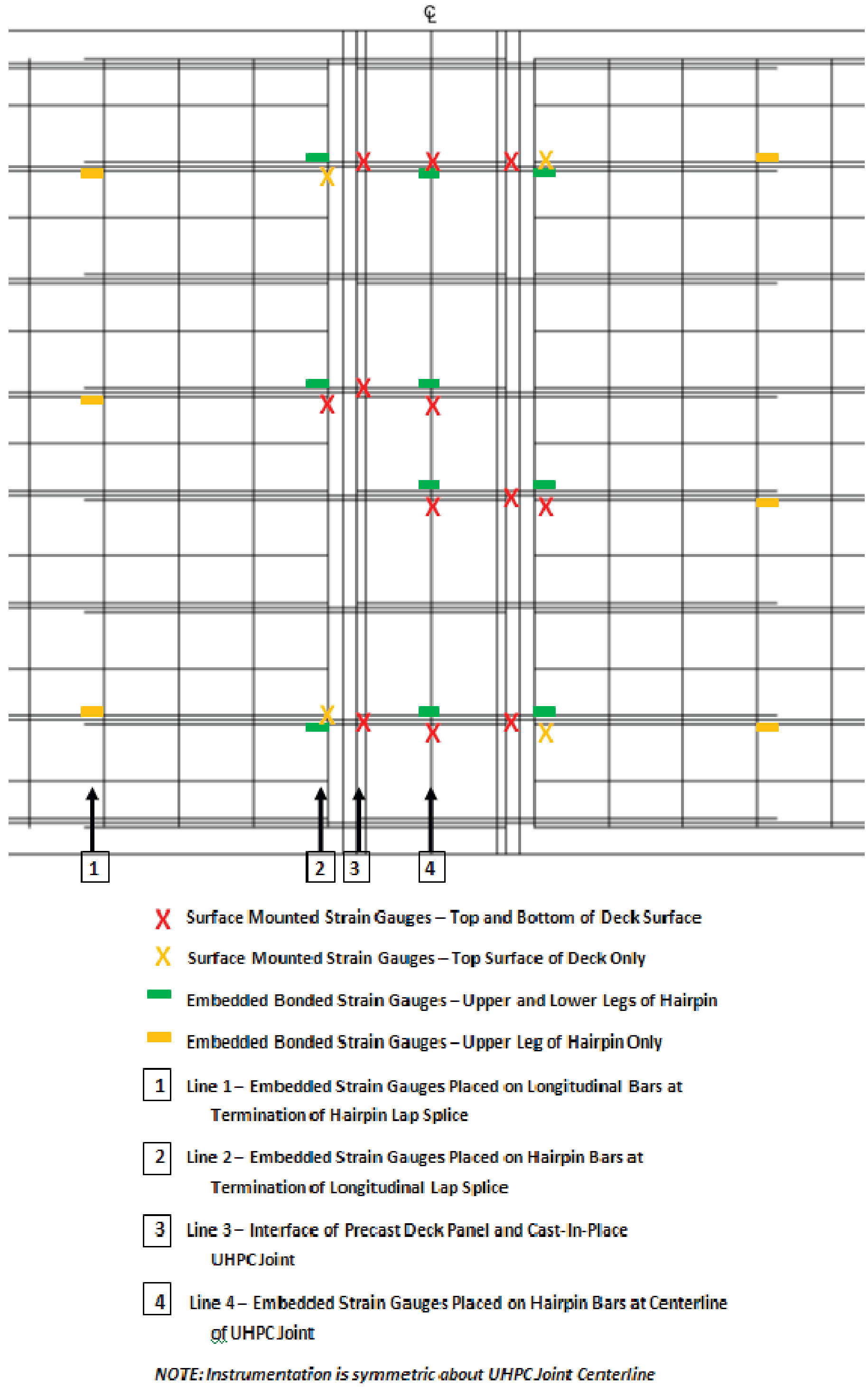


Figure C.23. Connection instrumentation locations, plan view.

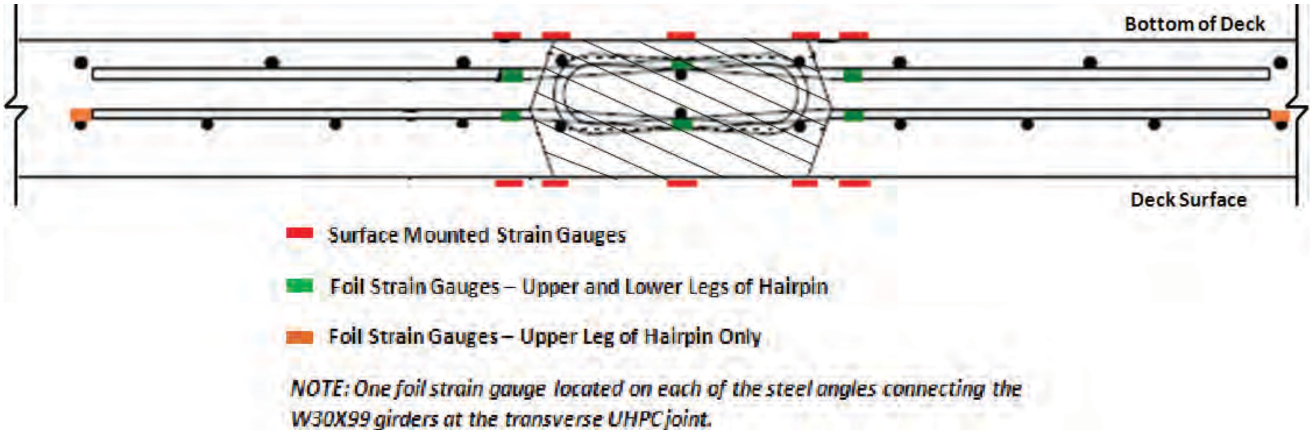


Figure C.24. Connection instrumentation locations, section view.

module-to-module transverse connection was –2,016 kip-ft. Because of the possibility of an HS-20 load on the other spans of the continuous bridge deck, a +74-kip-ft live-load moment was also possible.

Conducting Service-Level Static Testing

Testing of the module-to-module transverse connection specimen was performed up to Service Level II moment. Loads were applied with two hydraulic actuators each fitted with load cells and connected to spreader beams (Figure C.27 and Figure C.29). The spreader beams allowed for load application, replicating the demonstration bridge's bearing support conditions at the pier. Lubricated steel plates acted as the bearing points for load application (Figure C.28). The lubricated plates allowed the specimen and spreader beam to act independently. Four days after the placement of the transverse UHPC joint, when the cast-in-place UHPC reached the specified 14,000-psi compressive strength threshold, the actuators were placed in deflection control and the joint formwork removed.

Three incremental load tests were performed through Service Level I and up to Service Level II moment conditions. The incremental load test was completed with the actuators in load control. Using load control and lubricated steel bearing plates for the tests allowed for replication of the bearing conditions in the demonstration bridge. During the incremental tests, the specimen underwent visual inspection on the top and bottom deck surfaces.

On completion of the incremental load testing, the cyclic load testing through the full range of service-level conditions commenced. One million load cycles were completed over a period of 10 days. Visual inspection and marking of cracking took place every 250,000 cycles. This allowed for the detection of cracks and further monitoring for crack propagation in the HPC deck panels and in the transverse UHPC joint.

Figure C.25. Embedded bonded strain gauges in HPC deck.

Figure C.26. Surface-mounted strain gauges on top of deck.

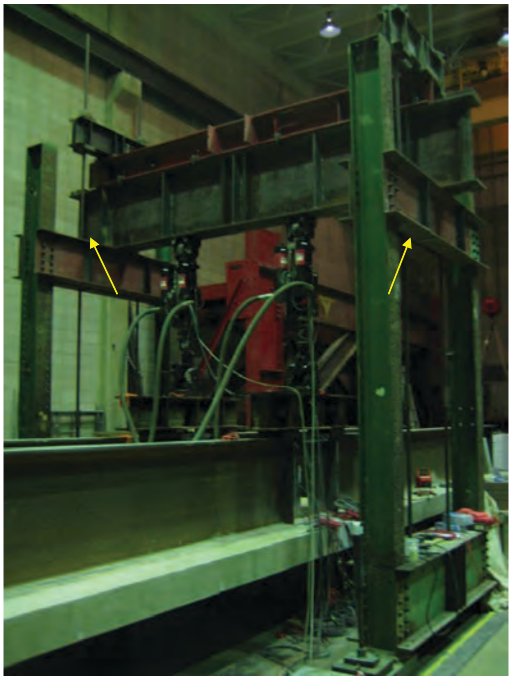


Figure C.27. Load frame for service-level testing.

Conducting Service-Level Testing with Connection Retrofit

After analyzing results from the initial service-level load testing and observing the formation of undesirable cracks, HNTB, the design engineer, designed a retrofit for the transverse module-to-module connection (Figure C.30 and Figure C.31). The retrofit employed high-strength steel rods mounted just under the deck surface to posttension the entire connection and lower total tensile strain levels present in the HPC deck and UHPC joint to below the expected cracking threshold.

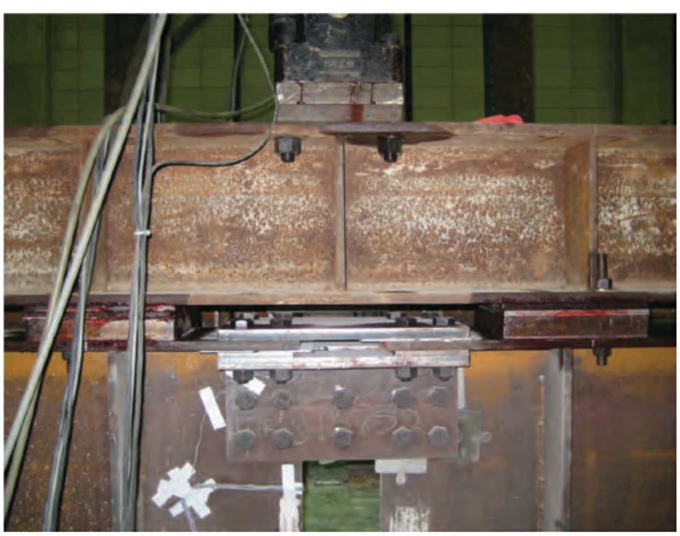


Figure C.28. Lubricated steel bearing plates.

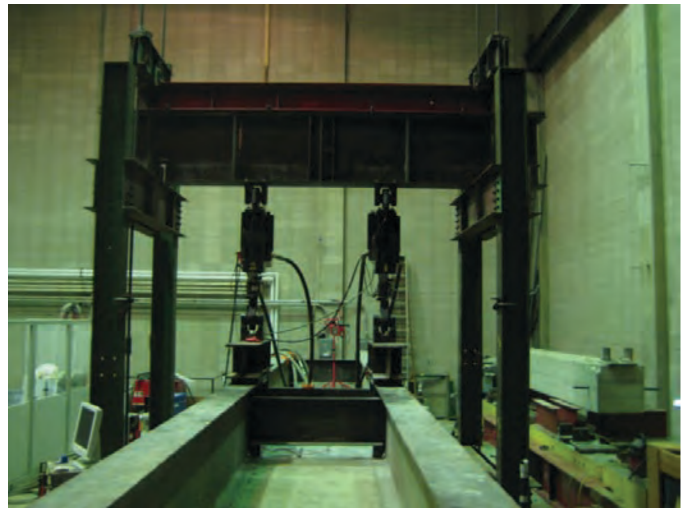


Figure C.29. Load test setup.

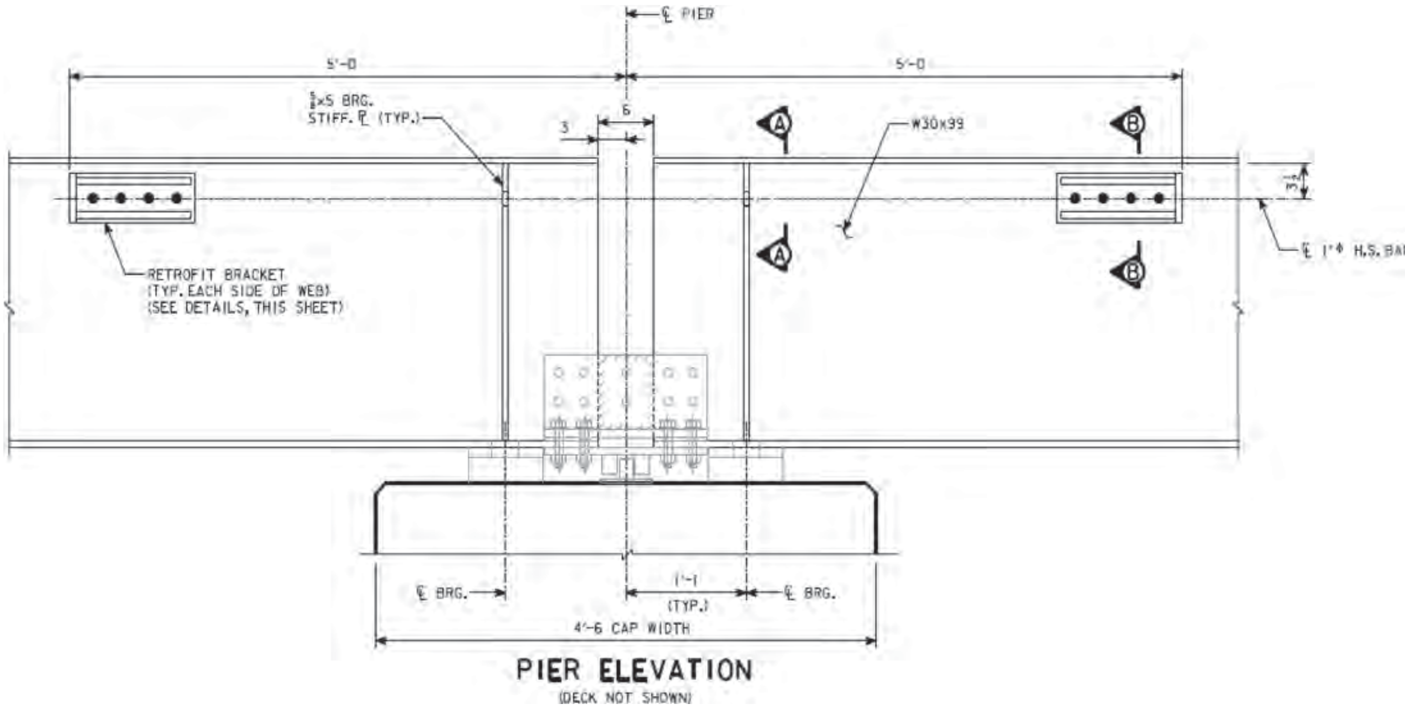
Four 1-in.-diameter high-strength threaded rods were installed following the attachment of the ASTM A709 grade 50W steel brackets to the W30X99 beams. The high-strength rods were initially posttensioned to achieve an effective force of 60 kips per rod at each location. Once the retrofit detail was successfully installed, the incremental static service-level load testing through Service Level II moment was conducted in the same manner as previously described. When service-level testing of the 60-kip-per-rod posttensioned retrofit was completed, the high-strength rods were posttensioned to 70 kips per rod. The service-level testing was repeated at the 70-kip posttensioning force level as previously described.

Conducting Ultimate Moment Capacity Testing

Once the static testing of the modified detail was completed, the posttensioning rods were removed. The final test was conducted to determine the ultimate moment capacity of the original transverse module-to-module connection at the pier. To complete the ultimate load testing, larger capacity actuators replaced those used for the service-level static and cyclic testing (Figure C.32). By incrementally loading the specimen in load control through its expected capacity (2,016 kip-ft) to failure, the performance of the connection could be studied, and the ultimate moment capacity of the connection was determined.

Results and Discussion

Quantitative and qualitative results of the primary laboratory testing regimen carried out for abrasion testing, joint constructability testing, and transverse joint strength and serviceability testing are presented in this section along with the results of various material tests (e.g., compressive strength of concrete, flexural strength of concrete, etc.) that accompanied the primary testing regimen.



Source: Iowa Department of Transportation, 2011a.

Figure C.30. Connection retrofit detail.

Figure C.31. Connection retrofit installed.

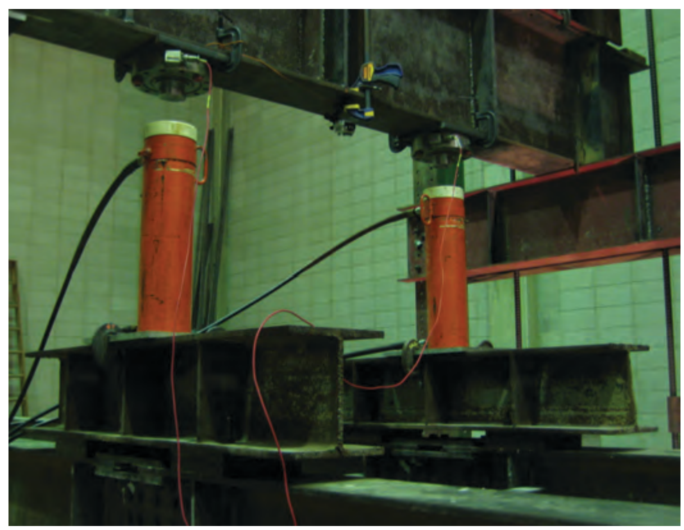


Figure C.32. Actuators for ultimate moment capacity testing.

Material Property Tests

UHPC Quality Control Tests

Quality control testing during the UHPC batching process in the laboratory at Iowa State University included temperature readings as well as static and dynamic flow testing. The testing was done according to Lafarge's flow testing procedure based on ASTM C230. Results of the quality control testing are presented in Table C.6 and Table C.7.

UHPC Strength Tests

Abrasion Testing Batch

Twelve 4-in.-diameter cylinders were tested for compressive strength in accordance with ASTM C39 to establish the maturity of UHPC Mix Design 1 (Table C.2) used in the abrasion testing. Compressive strength results from the 4-in. UHPC cylinders are presented in Figure C.33.

Table C.6. Joint Constructability: UHPC Quality Control Test Results

Batch	Time		Mix Temp Finish (°F)	Ambient Temp (°F)	Flow	
	Start	Finish			Static (in.)	Dynamic (in.)
1	9:55 a.m.	10:10 a.m.	85.0	64.0	8.50	9.25
2	10:27 a.m.	10:38 a.m.	84.0	64.0	9.13	10.00
3	11:07 a.m.	11:22 a.m.	82.0	66.0	9.13	10.00

Table C.7. Joint Strength and Serviceability: UHPC Quality Control Test Results

Batch	Time		Mix Temp Finish (°F)	Ambient Temp (°F)	Flow	
	Start	Finish			Static (in.)	Dynamic (in.)
1*	10:00 a.m.	10:23 a.m.	100.0	75.5	6.00	N/A
2	11:16 a.m.	11:36 a.m.	60.0	75.5	9.75	10.00
3	11:48 a.m.	12:08 p.m.	60.1	75.6	9.75	10.00
4	12:40 p.m.	1:02 p.m.	60.0	75.6	N/A	N/A

*Batch not used.

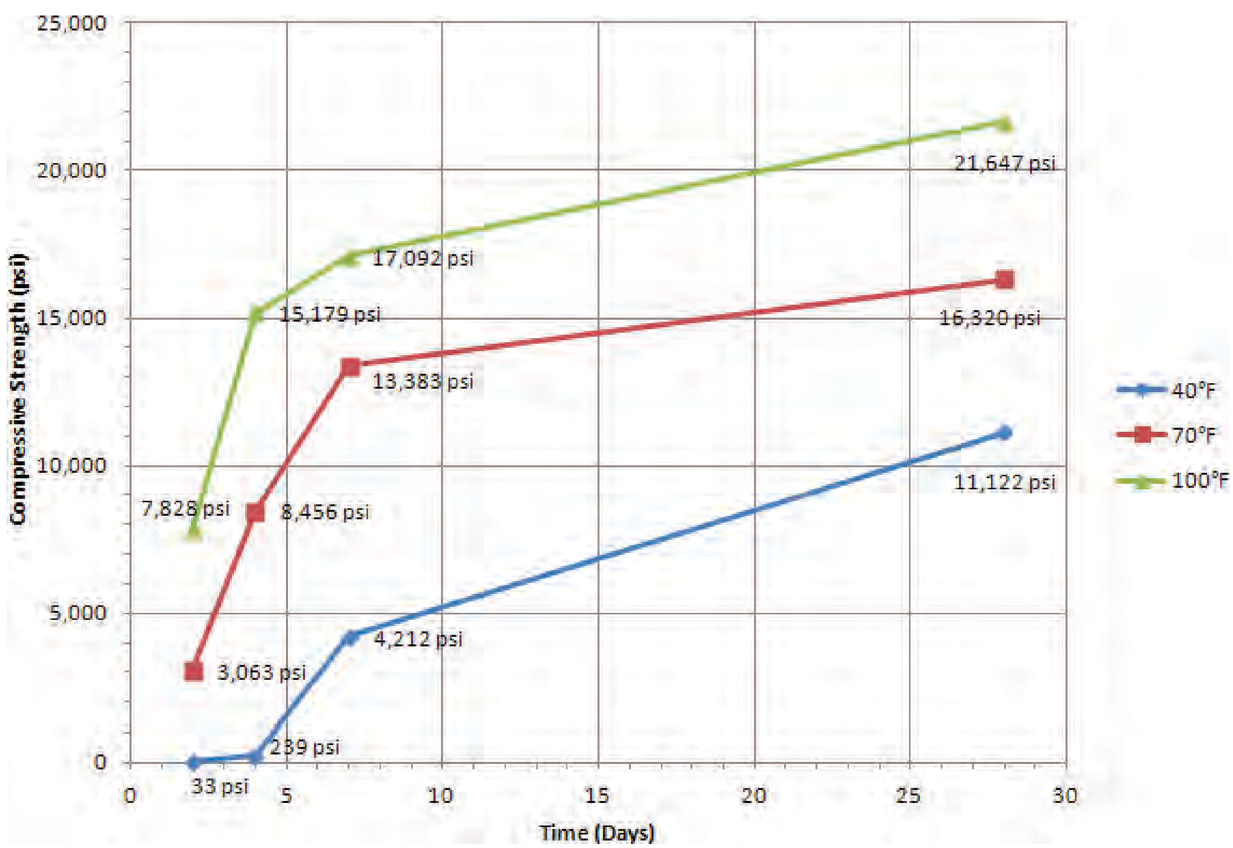


Figure C.33. Compressive strength of UHPC Mix Design 1 (abrasion testing).

The compressive strengths varied for the curing temperatures of 40°F, 70°F, and 100°F. The 28-day compressive strengths (f_c') for 40°F, 70°F, and 100°F were 11,100 psi, 16,300 psi, and 21,600 psi, respectively.

Joint Constructability Testing Batch

Eighteen 4-in.-diameter cylinders were tested for compressive strength in accordance with ASTM C39 to establish the maturity of the UHPC Mix Design 2 (Table C.4) used in the joint constructability testing. Compressive strength results from the 4-in. UHPC cylinders cured at 70°F are presented in Figure C.34.

Transverse Joint Strength and Serviceability Testing

Forty-five 4-in.-diameter cylinders were tested for compressive strength in accordance with ASTM C39 to establish the maturity of the UHPC Mix Design 2 used in the transverse joint strength and serviceability testing. Fifteen cylinders each were cured at 60°F, 70°F, and 90°F to replicate potential field curing temperatures and determine strength variations of the mix design. Compressive strength results from the 4-in. UHPC cylinders are presented in Figure C.35.

UHPC Mix Design 2, designed specifically for the SHRP 2 Project R04 demonstration bridge and used in the joint constructability and transverse joint strength and serviceability testing, reached 10,000-psi compressive strength in approximately 2 days and 14,000-psi compressive strength in 4 days. The 28-day strength (f_c') of the UHPC used in the final two testing procedures was approximately 17,000 psi.

HPC Strength Test

Transverse Joint Strength and Serviceability Testing Batch

Twenty-four 4-in.-diameter cylinders were tested for compressive strength in accordance with ASTM C39 to establish the maturity of the Iowa DOT CV-HPC-D mix design used in the prefabricated deck modules for the transverse joint strength and serviceability testing. Compressive strength results from the 4-in. HPC cylinders are presented in Figure C.36. The 28-day compressive strength (f_c') of the deck HPC for the prefabricated modules was approximately 5,800 psi.

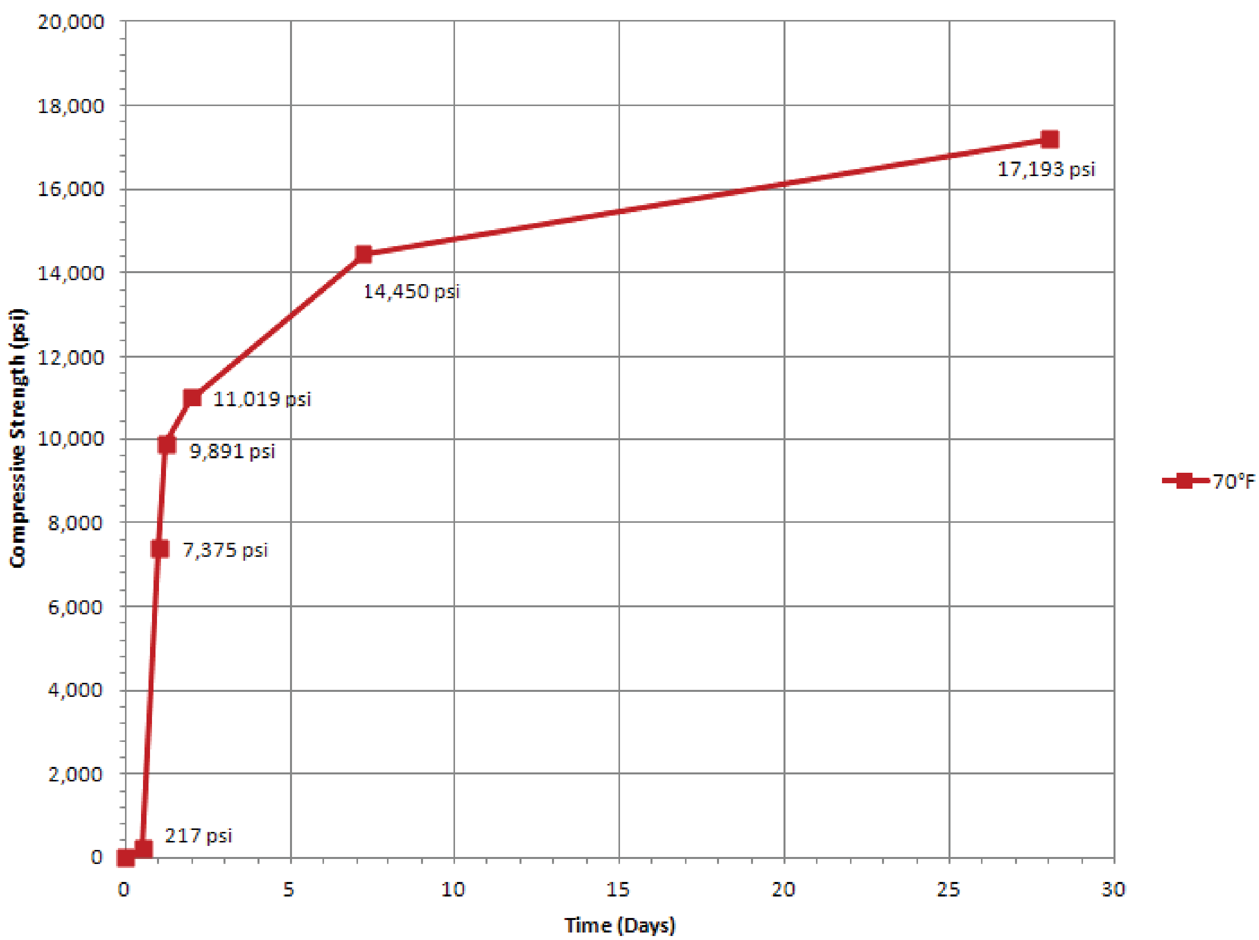


Figure C.34. Compressive strength of UHPC Mix Design 2 (joint constructability).

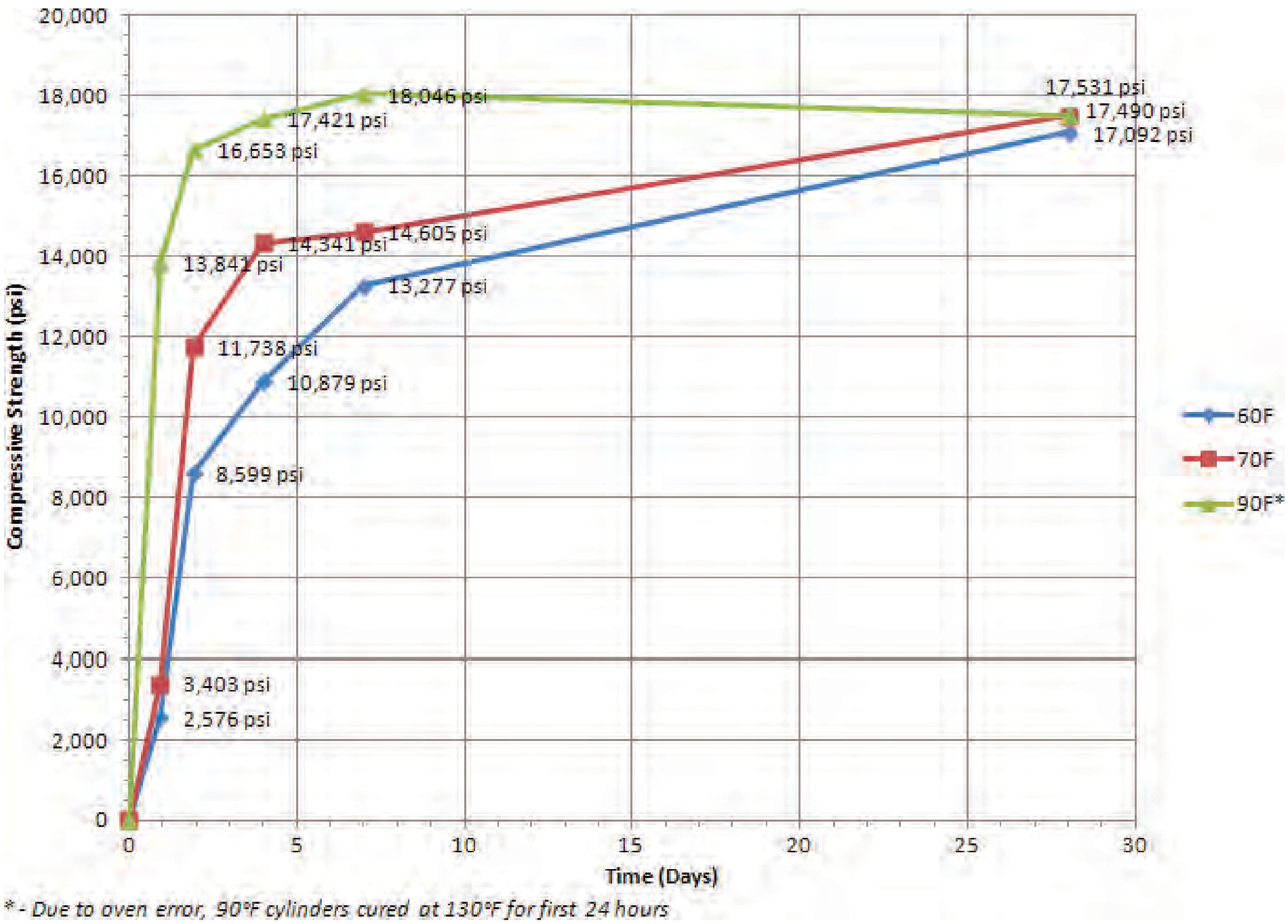


Figure C.35. Compressive strength of UHPC Mix Design 2 (strength and serviceability).

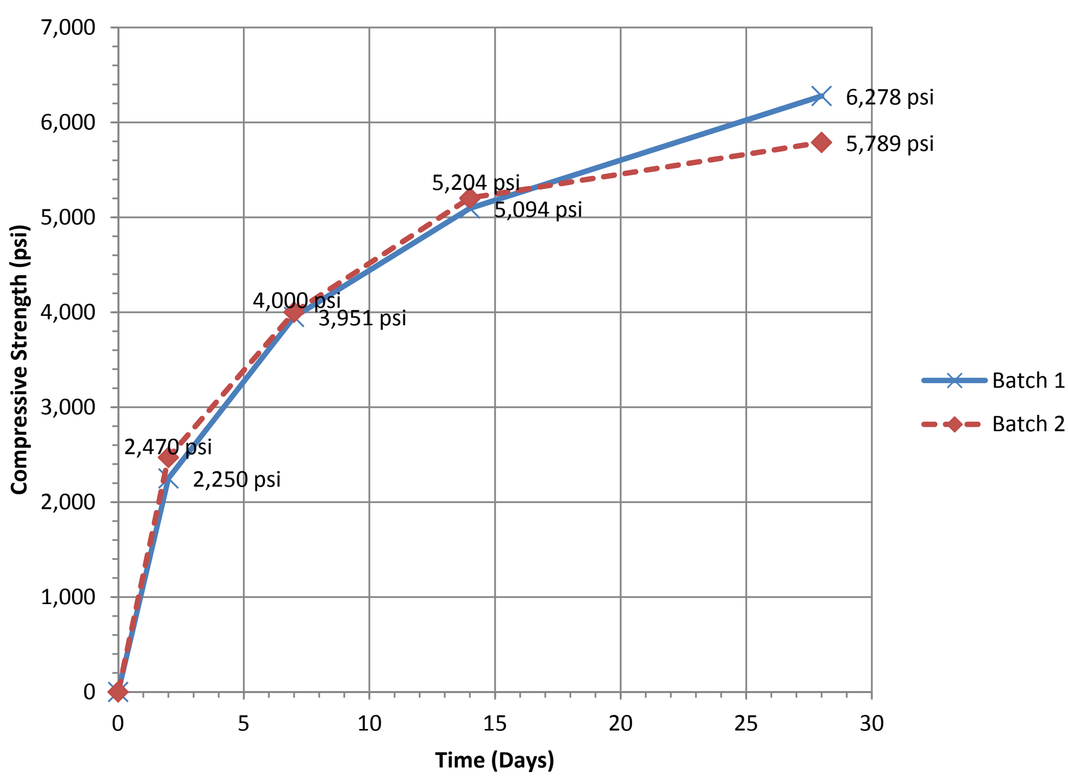


Figure C.36. Compressive strength of HPC (joint strength and serviceability).

Table C.8. Joint Strength and Serviceability—HPC Flexural Strength

Specimen	Max. Applied Load P (lb)	Span Length L (in.)	Width at Fracture b (in.)	Depth at Fracture d (in.)	Modulus of Rupture R (psi)
B1-28	5301	18	6	6	441.75
B2-28	5433	18	6	6	452.75
B3-28	5081	18	6	6	423.42
					Modulus of rupture = 439.31

Flexural Strength Test

TRANSVERSE JOINT STRENGTH AND SERVICEABILITY TESTING BATCH

Three beams were tested to determine the modulus of rupture of the HPC used in the prefabricated deck modules. The beams tested had nominal cross-sectional dimensions of 6 in. by 6 in. and a length of 18 in. and were tested in accordance with ASTM C78 to obtain the modulus of rupture. Flexural strength results for the 6-in. by 6-in. by 18-in. beams are presented in Table C.8. The modulus of rupture (f_r) for the HPC used in the deck modules was 439 psi.

Taking the modulus of elasticity (E_c) as 57,000 $\sqrt{f'_c}$ for the HPC, E_c was calculated to be 4,030 ksi (American Concrete Institute, 2005). Thus, with the modulus of rupture of 439 psi, the expected cracking strain for the precast HPC deck was 110με.

Because no flexural strength testing was completed on the UHPC material, the modulus of elasticity was calculated as 46,200 $\sqrt{f'_c}$ (Graybeal, 2007). That fell within Berg's E_c range of 5,800 ksi to 7,800 ksi, allowing the modulus of rupture to be approximated as 1,855 psi (Berg, 2010). The expected cracking strain of the UHPC was then calculated to be approximately 250με.

UHPC Abrasion Testing

Abrasion testing was completed on the UHPC material to determine the early age grindability of the joints in the demonstration bridge. Testing of the UHPC material for abrasion resistance was completed at Iowa State University in February and March 2011.

Abrasion Test Results

Twelve cylinders were cut into 36 specimens, resulting in three different surface finishes, and subjected to abrasion resistance testing in accordance with ASTM C944. The specimen identification matrix and identification terminology are presented in Table C.9. Results of the abrasion resistance testing at 2, 4, 7, and 28 days are presented in Table C.10 through Table C.13.

Taking the maturity of the UHPC into consideration, a plot of the percentage mass loss versus compressive strength for the three different surface finish conditions is presented in Figure C.37.

According to the compressive strength test results for the SHRP 2 Project R04 UHPC mix design used in the constructability and strength and serviceability testing (Mix Design 2), the UHPC will reach the 10,000-psi compressive strength required for grinding in the project specifications for the demonstration bridge at approximately 2 days if cured at 70°F. The 14,000-psi compressive strength threshold, required in the demonstration bridge project specifications for opening

Table C.9. Abrasion Specimen Identification Matrix

	2 Days	4 Days	7 Days	28 Days
A = 40°F	A2-1	A4-1	A7-1	A28-1
	A2-2	A4-2	A7-2	A28-2
	A2-3	A4-3	A7-3	A28-3
	A2-4	A4-4	A7-4	A28-4
	A2-5	A4-5	A7-5	A28-5
B = 70°F	B2-1	B4-1	B7-1	B28-1
	B2-2	B4-2	B7-2	B28-2
	B2-3	B4-3	B7-3	B28-3
	B2-4	B4-4	B7-4	B28-4
	B2-5	B4-5	B7-5	B28-5
C = 100°F	C2-1	C4-1	C7-1	C28-1
	C2-2	C4-2	C7-2	C28-2
	C2-3	C4-3	C7-3	C28-3
	C2-4	C4-4	C7-4	C28-4
	C2-5	C4-5	C7-5	C28-5

Example: In A2-1, A = curing temperature, 2 = number of days after pour in which test occurs, and 1 = specimen test number.
Note: Specimen Tests 1 through 4 are the rough, cut, or formed surface abrasion resistance tests. Specimen Test 5 is the compressive strength test.

Table C.10. 28-Day Abrasion Test Results: Day 2

Specimen Age:	2 Days	ASTM C 944: Abrasion Resistance of Concrete Surfaces by Rotating Cutter Method							
Test Date:	2/24/2011								
		Initial Mass	Mass 1	Mass 2	Final Mass	Wear Depth	Loss of Mass		
Specimen ID	Surface	g	g	g	g	mm	g	%	Additional Notes
A2-1	NA						0.00	0.00%	*too soft to test
A2-2	NA						0.00	0.00%	*too soft to test
A2-3	NA						0.00	0.00%	*too soft to test
A2-4	NA						0.00	0.00%	*too soft to test
B2-1	rough	1993.10	1991.08	1989.31	1987.85	0.41	5.25	0.26%	
B2-2	cut	1987.85	1986.63	1985.05	1983.48	0.76	4.37	0.22%	
B2-3	cut	2020.20	2017.20	2015.90	2014.10	0.59	6.10	0.30%	
B2-4	form	2016.88	2014.01	2008.08	2002.86	1.25	14.02	0.70%	
C2-1	rough	1951.30	1950.57	1949.88	1949.40	0.48	1.90	0.10%	
C2-2	cut	1949.37	1948.48	1948.22	1947.95	0.26	1.42	0.07%	
C2-3	cut	2016.96	2016.73	2016.52	2016.23	0.23	0.73	0.04%	
C2-4	cut	1771.24	1770.67	1770.26	1769.93	0.26	1.31	0.07%	

SHRP 2 Project No R04 - Phase III - Task 10C: Test 2

Table C.11. 28-Day Abrasion Test Results: Day 4

Specimen Age:	4 Days	ASTM C 944: Abrasion Resistance of Concrete Surfaces by Rotating Cutter Method							
Test Date:	2/26/2011								
		Initial Mass	Mass 1	Mass 2	Final Mass	Wear Depth	Loss of Mass		
Specimen ID	Surface	g	g	g	g	mm	g	%	Additional Notes
A4-1	rough	1454.66	1336.17		1336.17	7.29	118.49	8.15%	*Maxed out at 55 sec
A4-2							0.00	0.00%	*too soft to cut cylinder
A4-3							0.00	0.00%	*too soft to cut cylinder
A4-4	form	2242.74	2121.95		2121.95	6.96	120.79	5.39%	*Maxed out at 68 sec
B4-1	rough	1841.91	1841.25	1840.79	1840.36		1.55	0.08%	
B4-2	cut	1840.36	1840.14	1839.96	1839.78		0.58	0.03%	
B4-3	cut	2004.69	2004.54	2004.39	2004.24		0.45	0.02%	
B4-4	form	2004.24	2002.23	1999.73	1997.95		6.29	0.31%	
C4-1	cut	1779.75	1779.6	1779.51	1779.38		0.37	0.02%	
C4-2	cut	1779.38	1779.25	1779.17	1779.12		0.26	0.01%	
C4-3	cut	2094.86	2094.77	2094.61	2094.5		0.36	0.02%	
C4-4	form	2094.5	2094.3	2093.89	2093.39		1.11	0.05%	

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Table C.12. 28-Day Abrasion Test Results: Day 7

Specimen Age:	7 Days	ASTM C 944: Abrasion Resistance of Concrete Surfaces by Rotating Cutter Method							
Test Date:	3/1/2011								
Specimen ID	Surface	Initial Mass	Mass 1	Mass 2	Final Mass	Wear Depth	Loss of Mass		Additional Notes
		g	g	g	g	mm	g	%	
A7-1	rough	2128.75	2120.59	2116.72	2113.48		7.11	0.34%	
A7-2	cut	2113.48	2112.62	2111.56	2110.63		1.99	0.09%	
A7-3	cut	1955.9	1955.23	1954.52	1953.82		1.41	0.07%	
A7-4	form	1953.82	1951.01	1946.51	1943.64		7.37	0.38%	
B7-1	rough	1841.96	1839.46	1838.7	1838.17		1.29	0.07%	
B7-2	cut	1838.17	1837.96	1837.84	1837.75		0.21	0.01%	
B7-3	cut	2162.22	2162.15	2162.09	2162.02		0.13	0.01%	
B7-4	form	2162.02	2161.65	2161.09	2160.53		1.12	0.05%	
C7-1	rough	1983.47	1981.9	1981.03	1980.4		1.50	0.08%	
C7-2	cut	1980.4	1980.33	1980.22	1980.15		0.18	0.01%	
C7-4	cut	2103.96	2103.88	2103.84	2103.77		0.11	0.01%	
C7-3	form	2103.77	2103.63	2103.38	2103.12		0.51	0.02%	

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Table C.13. 28-Day Abrasion Test Results: Day 28

Specimen Age:	28 Days	ASTM C 944: Abrasion Resistance of Concrete Surfaces by Rotating Cutter Method							
Test Date:	3/22/2011								
Specimen ID	Surface	Initial Mass	Mass 1	Mass 2	Final Mass	Wear Depth	Loss of Mass		Additional Notes
		g	g	g	g	mm	g	%	
A28-1	rough	1700.7	1699.1	1698.8	1698.5		2.20	0.13%	
A28-2	cut	1698.5	1698.4	1698.2	1698.1		0.40	0.02%	
A28-3	cut	1855.3	1855.2	1855.1	1854.9		0.40	0.02%	
A28-4	form	1854.9	1854.5	1853.9	1853.4		1.50	0.08%	
B28-1	rough	1959.8	1959	1958.6	1958.3		1.50	0.08%	
B28-2	cut	1958.3	1958.1	1958	1957.9		0.40	0.02%	
B28-3	cut	2020.6	2020.4	2020.3	2020.2		0.40	0.02%	
B28-4	form	2020.2	2019.1	2017.8	2016.4		3.80	0.19%	
C28-1	rough	1985.4	1984.7	1984.4	1983.7		1.70	0.09%	
C28-2	cut	1803.7	1803.3	1803.2	1803.1		0.60	0.03%	
C28-3	cut	2054.1	2053.7	2053.3	2053.1		1.00	0.05%	
C28-4	form	2053.1	2053	2052.9	2052.6		0.50	0.02%	

SHRP 2 Project No R04 - Phase III - Task 10C: Test 2

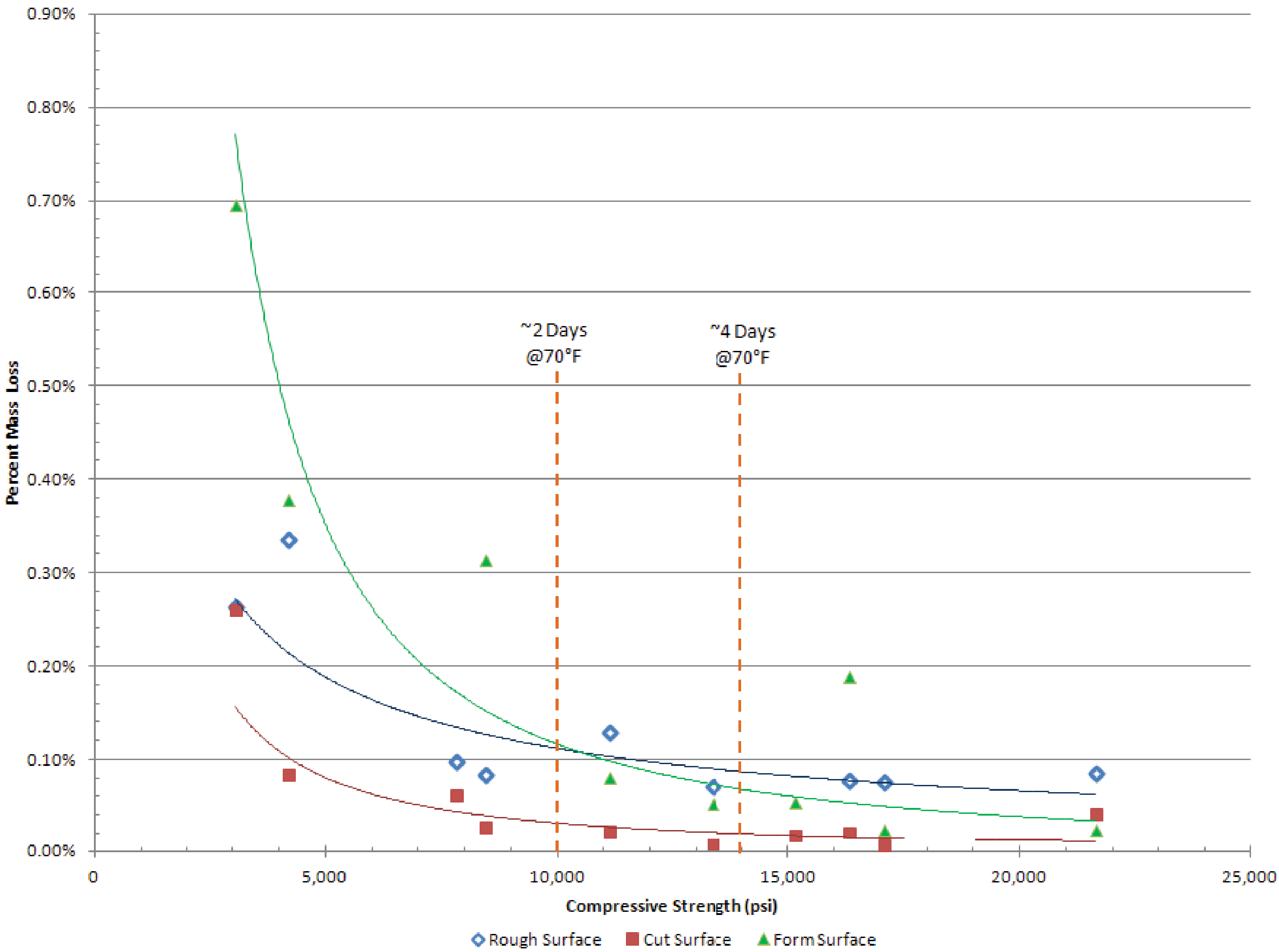

Figure C.37. Abrasion testing: Percentage mass loss versus strength.

the bridge to traffic, will likely be reached 4 days after placement. Thus, the contractor will have roughly 2 days to perform grinding of the joints from the time the 10,000-psi threshold is reached until opening of the bridge to traffic at 14,000-psi compressive strength. The percentage mass loss for both formed and top finishes at the 10,000-psi compressive strength threshold is approximately 0.12%. At 14,000-psi compressive strength of the UHPC mix, the percentage mass loss is approximately 0.07%. Over that 2-day duration, the UHPC's resistance to abrasion increases by approximately 40%. That would be a significant factor for the contractor in terms of grinding time and accelerated scheduling.

Figure C.37 shows that the formed surface and rough surface finishes displayed the lowest abrasion resistance. Specimens with formed surface finishes exhibited lower abrasion resistance than cut surfaces because of the steel fibers present in the UHPC. At the formed surface, the steel fibers were aligned preferentially, parallel with the surface. Thus, the fibers tended to pull off easily. The fibers lay parallel with the form surface because, as the UHPC flowed along the bottom of the form, the fibers tended to align and lie flat. The rough surface finish generally also included small, entrapped air bubbles, which allowed for easier removal of the UHPC material. As was expected, the cut surface finish had the highest abrasion resistance. Because the cast-in-place UHPC joints in the Project R04 demonstration bridge are a plywood-top formed surface, the abrasion resistance in the field is expected to most nearly resemble that of the formed surface finish seen in the abrasion tests.

Joint Constructability Testing

Joint constructability testing was done to qualitatively evaluate the intersecting, cast-in-place UHPC deck joints to be used in the demonstration bridge. Specifically, a full-scale mock-up of the intersection between one longitudinal and one transverse UHPC deck joint was constructed to investigate issues relating to casting sequence, material mixing and placement rates, effects of ambient temperature on construction, flow characteristics of the UHPC, and consolidation of material at congested locations. Testing of the UHPC joints for constructability was completed at Iowa State University in April 2011.

Constructability Test Results

Casting Sequence

The original proposal for the construction sequence of the demonstration bridge outlined continuous placement of the entire grid of UHPC deck joints (longitudinal and transverse). During discussions with the engineer, contractor, and material supplier, several logistical issues arose which challenged the feasibility of full-deck continuous placement. Typical mixers used by Lafarge Canada for UHPC placement mix 5.11 ft^3 per batch. On the job site, the mixers are used in pairs to provide a continuous supply of UHPC. Each batch is then discharged into buggies and transported onto the bridge to the placement location.

Given the large volume of UHPC needed for the bridge deck joints, continuous placement could be achieved only by using a large number of mixers and laborers. Otherwise, cold joints could potentially form in the UHPC deck joints. As an alternative, Lafarge proposed using stay-in-place acrylic vertical bulkheads to control the location of potential cold joints.

As a result of these discussions, a new construction sequence—limiting continuous placement to the transverse joints and allowing vertical cold joints in the longitudinal joints—was suggested for the joint constructability testing and demonstration bridge. A prototype of the stay-in-place acrylic vertical bulkheads (see Figure C.15) was fabricated and used during the joint constructability testing so its performance could be evaluated. The acrylic vertical bulkheads were used successfully.

Ambient-Temperature Effects on UHPC

The extent of the susceptibility to variations in temperature for the workability and flow characteristics of the UHPC mix design was observed during batching of the joint constructability test specimen and the transverse joint strength and serviceability test specimen to follow. Ambient air temperatures, seen previously in Table C.6, were steady at around 65°F at the time of batching for the intersecting joint specimen. However, during the batching for the transverse joint strength and serviceability specimen, ambient temperatures were 75.5°F (see Table C.7). Without compensating for the change in ambient air temperature, the workability and flow characteristics of the mixes were much different.

When ambient temperatures were 65°F, the temperature of the UHPC on discharge from the mixer ranged from 82°F to 85°F for the intersecting joint specimen's three batches. Within this range, the UHPC had acceptable flow characteristics for placement. When ambient temperatures were around 75.5°F, the temperature of the UHPC on discharge from the mixer was over 100°F. At this ambient temperature, the UHPC never reached its anticipated flow characteristics in the mixer, thus the batch was rejected. To correct the problem, water in the mix design was replaced by mass with ice and the UHPC material temperature was reduced. When ice was used, the batches could be successfully discharged and placed. The temperature upon discharge from the mixer for batches using ice was 60°F. This modification—the replacement of water by mass with ice—enabled extended working time and improved the flow relative to the previous batch.

Flow Characteristics and Consolidation of UHPC

Evaluating the flow of the UHPC around the corners at the intersection of the longitudinal and transverse deck joints was a critical aspect of this test. Adequate consolidation of the UHPC in the joint cross section around steel reinforcement is important to the deck joint performance. During UHPC placement, when the final-mix temperature was limited to a maximum of 85°F, the UHPC material appeared to have adequate flow characteristics to achieve good consolidation and flow around corners at the intersections of longitudinal and transverse joints (Figure C.38).

After the specimen was cured and removed from the forms, it was cut into several sections to examine consolidation and potential cold joints. No significant voids around the steel reinforcing bars were observed (Figure C.39 and Figure C.40).

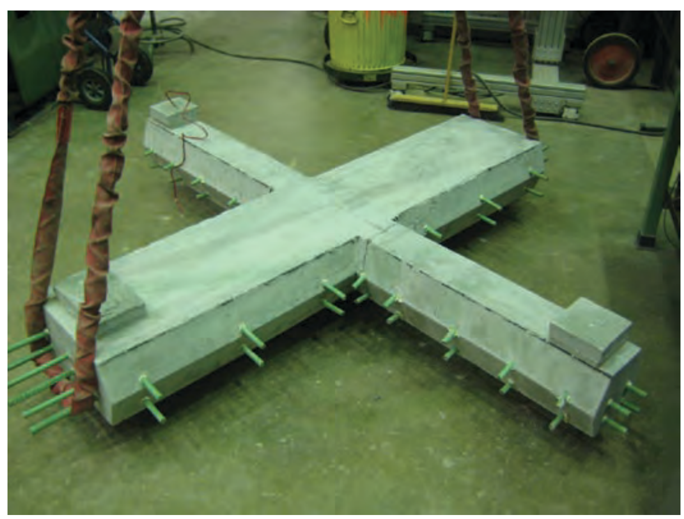

Figure C.38. Joint intersection specimen.

Figure C.39. Section of transverse joint, Example 1.

The test also validated the use of top forms and chimneys at the high end of the 2% cross slope of the bridge deck at transverse joints. The top forms were applied sequentially as the joint was filled from the lowest elevation to the highest. The chimneys (see Figure C.17) provide additional hydrostatic head in the freshly placed UHPC to aid in consolidation within the joint. Top forms and chimneys were suggested for use in the demonstration bridge. Instead, three-quarter-inch-high spacer boards were placed below the top forms to build up small hydrostatic head and produce similar results.

Joint Intersection Detail Recommendations

Final inspection of the specimen upon removal from the forms allowed for additional observations and recommendations.

Figure C.40. Section of transverse joint, Example 2.

Figure C.41. Stay-in-place acrylic bulkhead.

The proposed stay-in-place acrylic bulkhead successfully allowed for sequential placement of the UHPC, but it also created a possible infiltration plane where water and chemicals could access the embedded steel joint reinforcement (Figure C.41).

To maintain sequential placement of UHPC in the deck joint grid and avoid possible infiltration planes, a detail for a partial-height, removable acrylic bulkhead was developed and suggested for use in the demonstration bridge (Figure C.42). The removable acrylic bulkheads should be used in the longitudinal joint, and in compression zones where possible (Figure C.43). Placing the bulkheads at those locations will provide better continuity at the interface between the hardened and freshly placed UHPC, which will help prevent the ingress of water and other chemicals. In addition, the

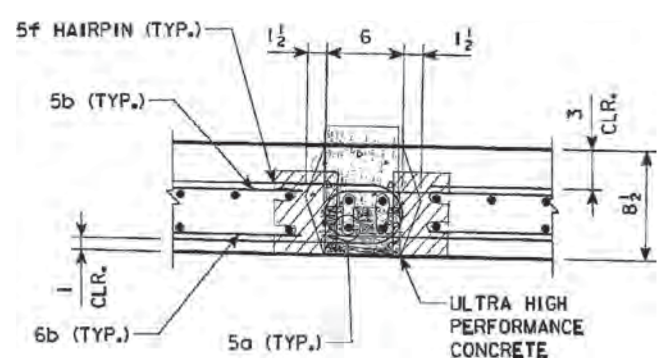

Figure C.42. Removable acrylic bulkhead.

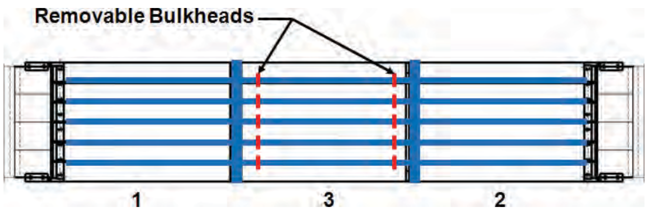

Figure C.43. Proposed placement plan.

placement sequence of the UHPC (Figure C.43) will be controlled, starting at the lowest elevations through the transverse joints over the piers up to the bulkheads. The center-span UHPC joints will be placed last.

Transverse Joint Strength and Serviceability Testing

Strength and serviceability testing of the module-to-module transverse connection for the SHRP 2 Project R04 demonstration bridge was performed to evaluate the negative bending performance of this detail over the piers, determine its cracking moment, and verify the ultimate moment capacity. Testing of the module-to-module transverse connection was completed at Iowa State University from July to October 2011.

Results Terminology

Because of the orientation of the testing specimen in the laboratory, the driving surface (or top-of-deck surface) was located on the bottom of the specimen and the bottom-of-deck surface was located on the top of the specimen (Figure C.44 and Figure C.45).

Refer to Figure C.23 and Figure C.24, which display the locations of all instrumentation for the test specimen presented in the following sections.

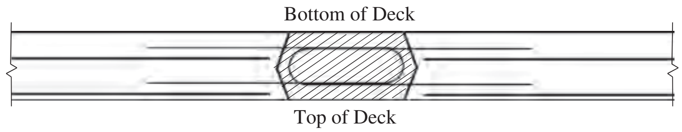

Figure C.45. Deck surface terminology diagram.

Service-Level Static Test Results

Load testing through live-load Service Levels I and II moment was completed on the specimen (Figure C.46). The range of expected service-level moments for the module connection varied from +74 kip-ft to −538 kip-ft. Loading was completed at 5,000 lbf increments to complete visual inspection of the specimen and check for the appearance of cracks and accrual of damage.

Strain levels were monitored with the embedded and surface-mounted strain gauges located throughout the specimen (Figure C.23 and Figure C.24). Strain levels for surface-mounted strain gauges at locations that spanned the HPC-UHPC interface exceeded 110$\mu\epsilon$, the HPC cracking strain, at approximately halfway to Service Level I moment (Figure C.47).

By selecting only one longitudinal line of surface mounted strain gauges, it can be seen that, immediately adjacent to the gauges spanning the interface, surface strain levels were well below the HPC cracking strain (Figure C.48). Surface-mounted strain gauges at these locations registered negligible strains throughout. The disparity between immediately adjacent gauges and the strains registering in excess of the HPC cracking strain across the interface suggested debonding and an opening at the interface between the precast HPC deck and the UHPC joint. Note that the intent of the design for the demonstration bridge was to avoid all cracking in the deck at the transverse joint over the pier, as that would be detrimental to the durability of the deck.

Visual inspection of the joint interface at Service Level II confirmed the debonding and substantial opening of the interface suggested in the strain gauge data (Figure C.49).

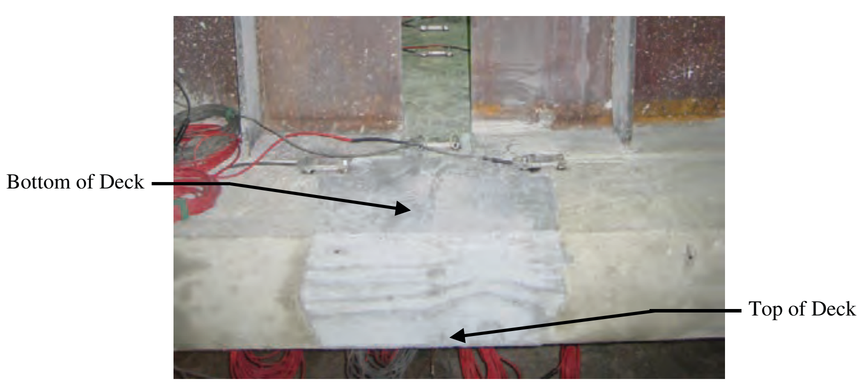

Figure C.44. Deck surface terminology photograph.

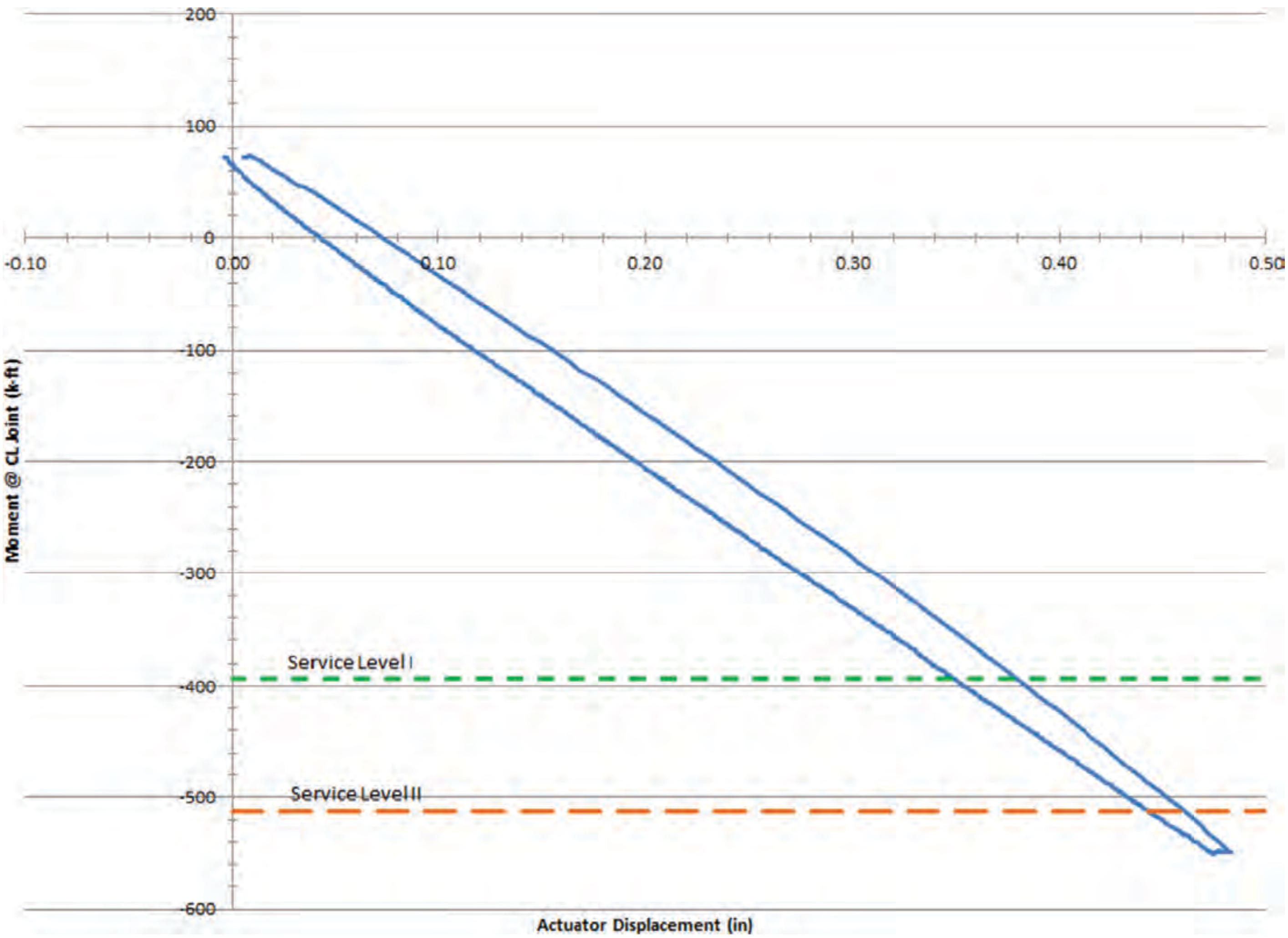

Figure C.46. Applied moment versus actuator displacement.

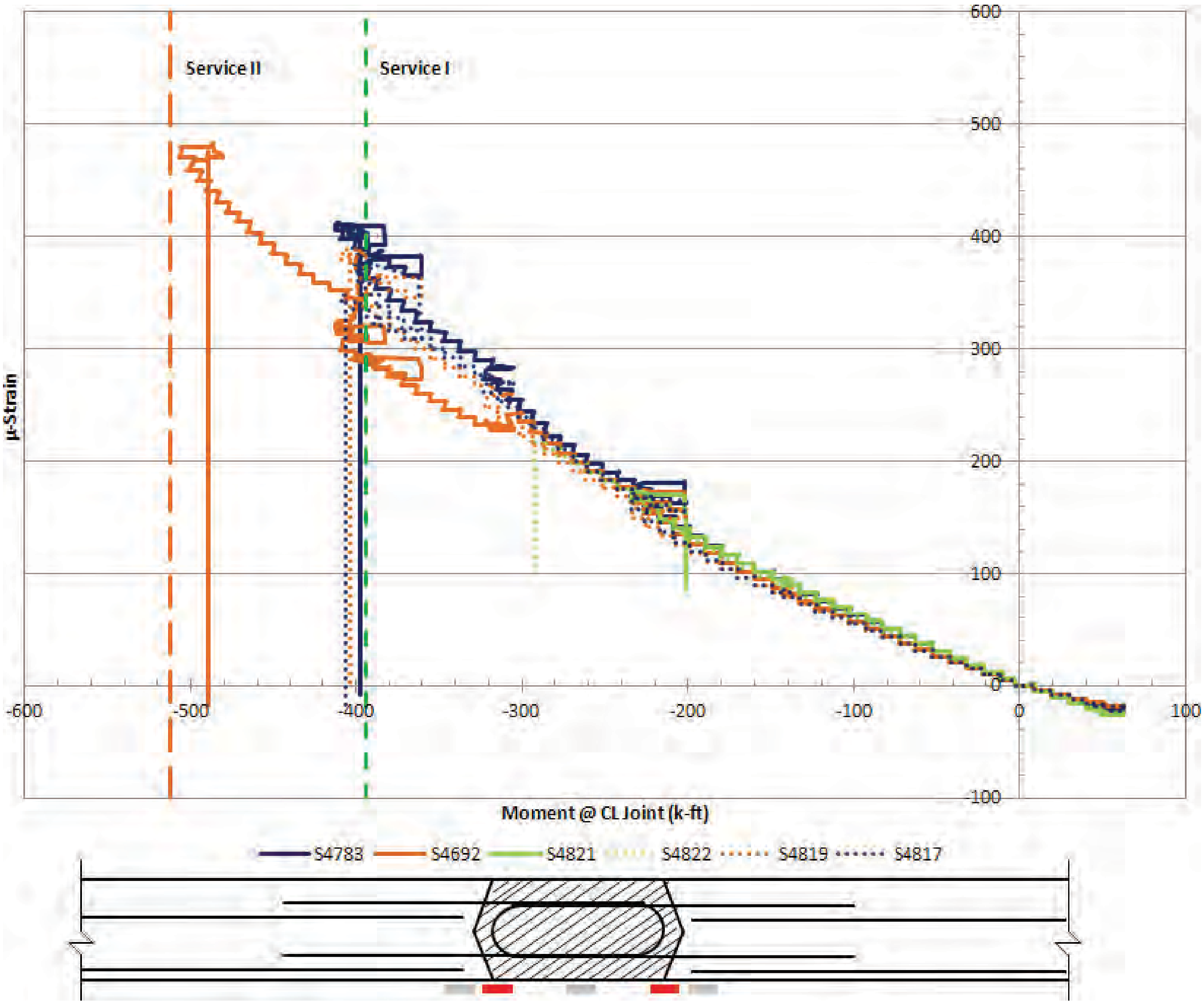

Figure C.47. Top-of-deck surface mounted strain gauges over the joint interface.

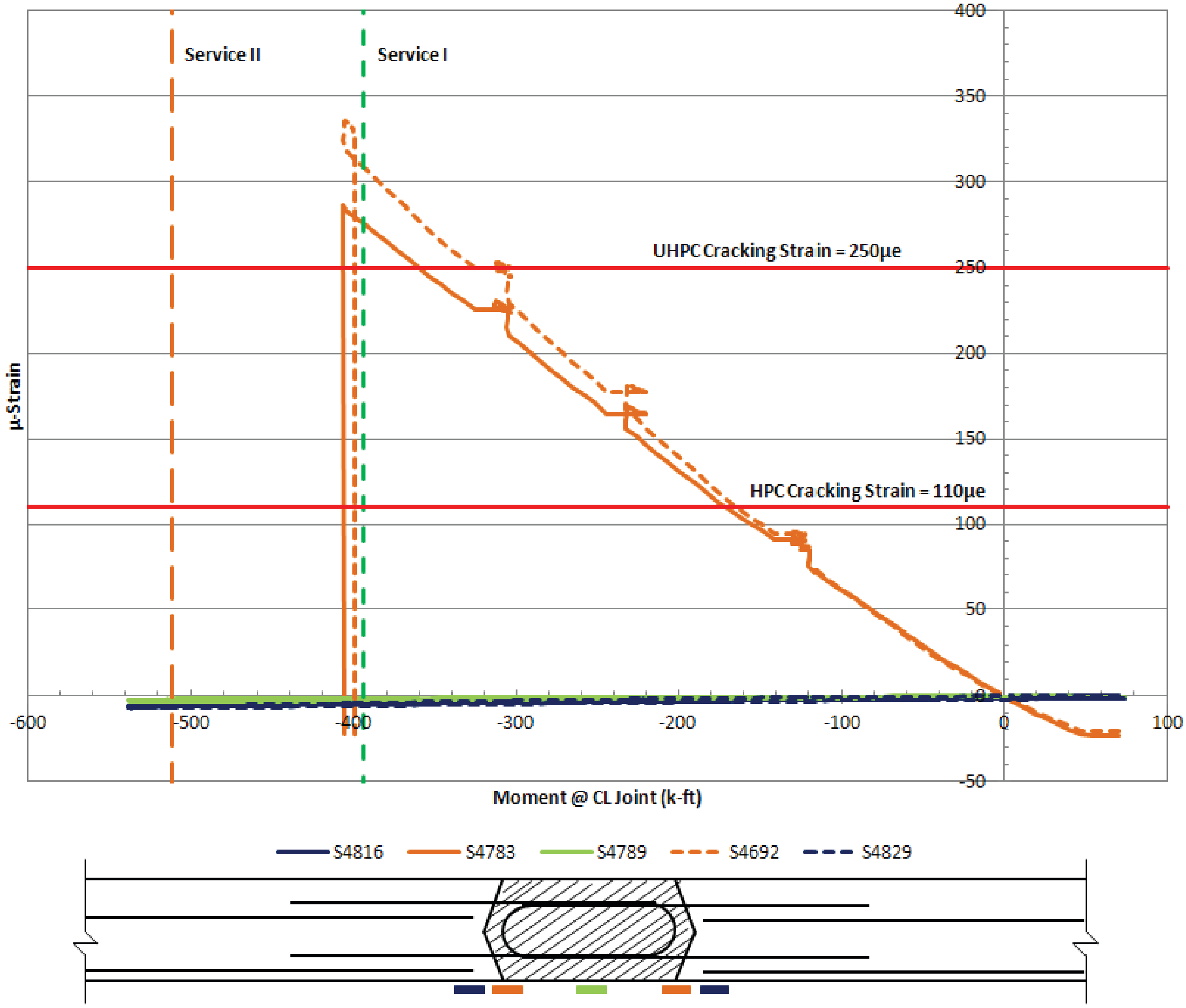

Figure C.48. Selected surface-mounted strain gauges adjacent to the joint interface.

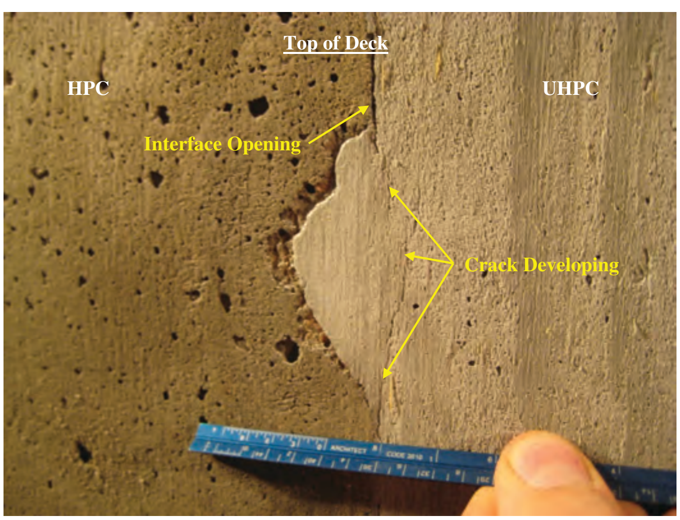

Figure C.49. Joint interface opening.

Later, inspection during fatigue testing further confirmed the interfacial debonding and opening occurring below service-level conditions.

In addition to joint interface debonding and substantial opening, strain levels in the embedded strain gauges registered above the expected HPC cracking strain as well. Figure C.51 through Figure C.53 show the embedded strain gauge data for top-of-deck gauges along longitudinal reinforcement under the two girder lines in the specimen. Figure C.54 through Figure C.56 display strain data for bottom-of-deck gauges along the same longitudinal reinforcement lines. Embedded strain gauge locations and identifications along with row groupings are shown in Figure C.50.

At the top of the deck, the groupings of embedded strain gauges show the varying strain seen within the UHPC joint, near the joint interface, and at the hairpin bar termination

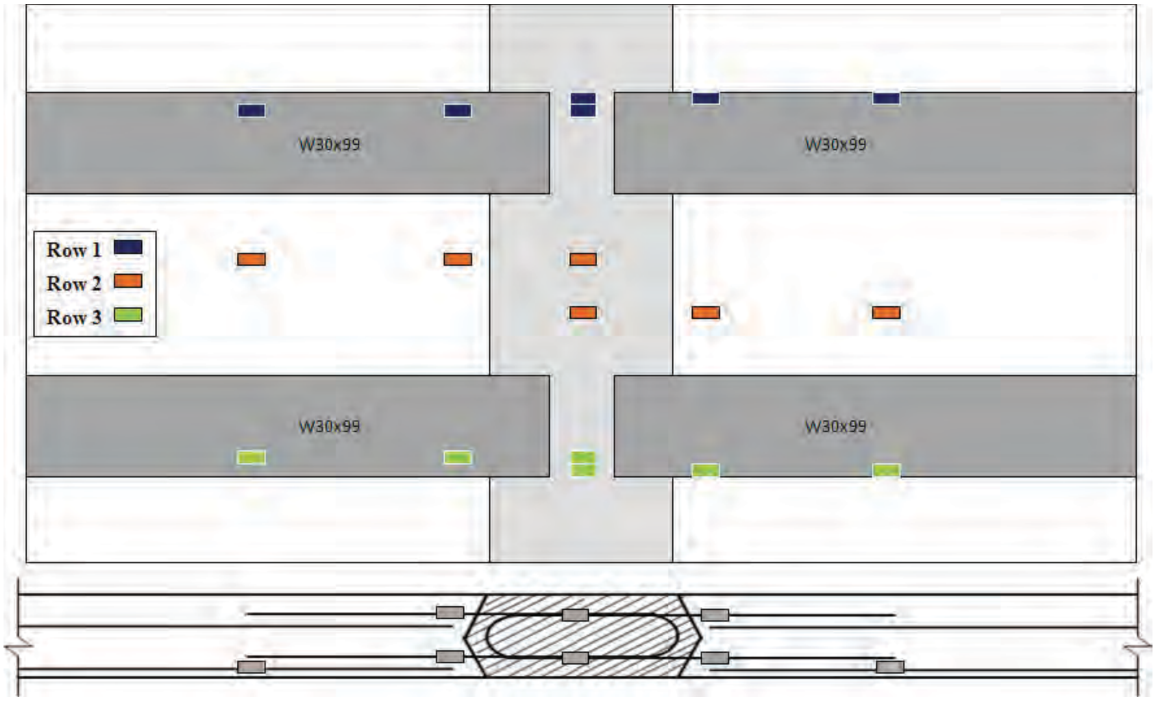

Figure C.50. Embedded strain gauge location and identification.

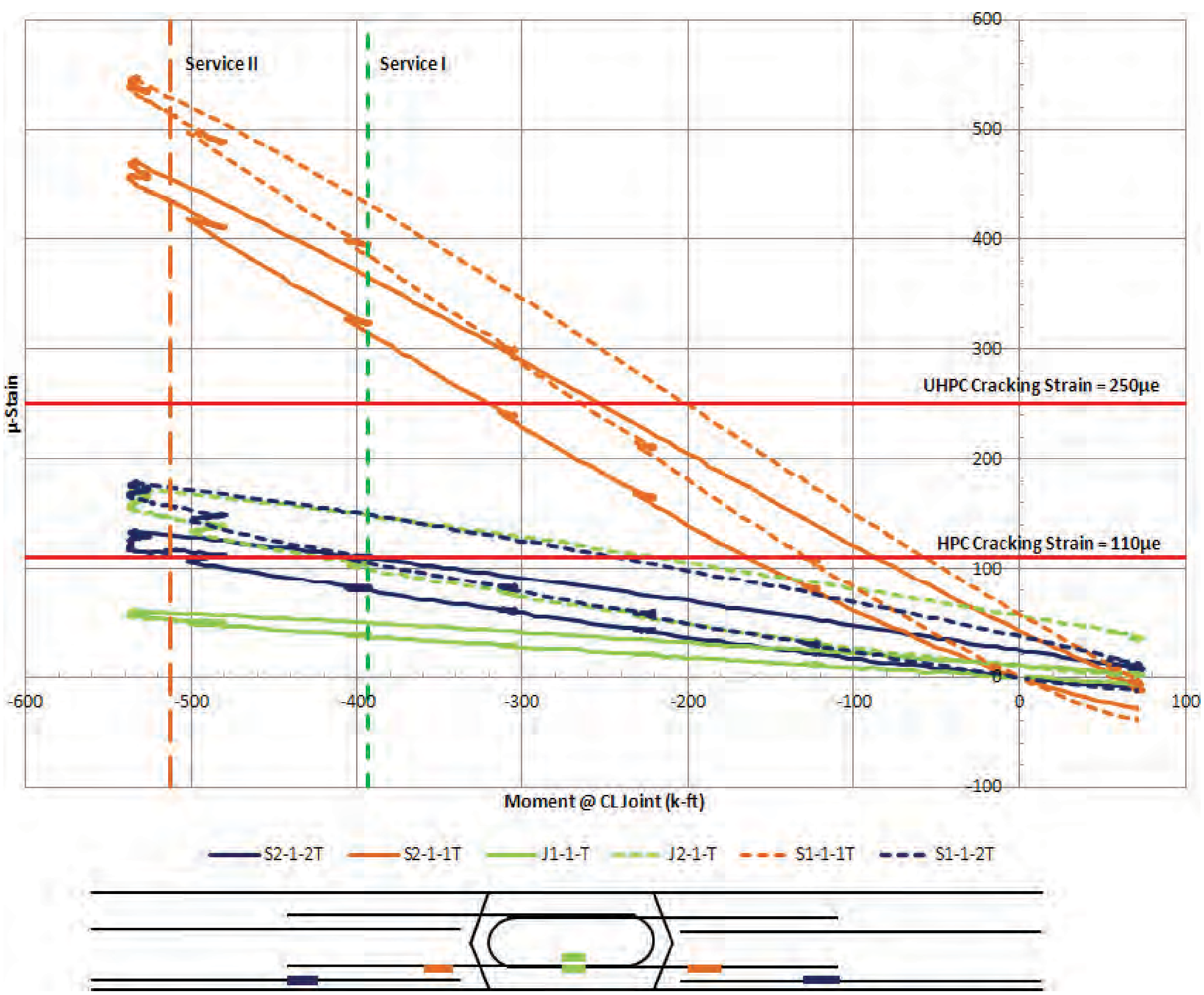

Figure C.51. Row 1, top-of-deck embedded strain gauges (static).

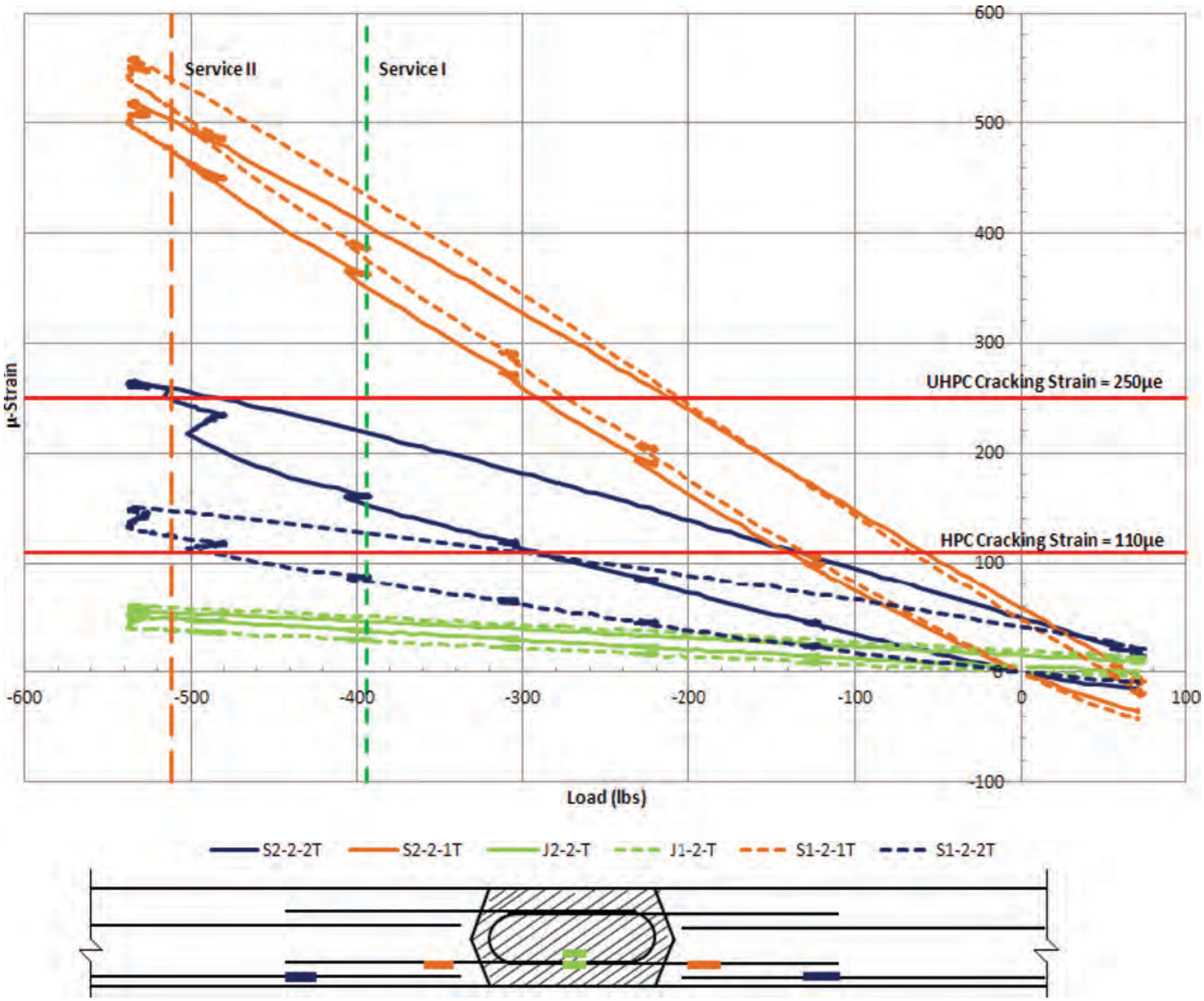

Figure C.52. Row 2, top-of-deck embedded strain gauges (static).

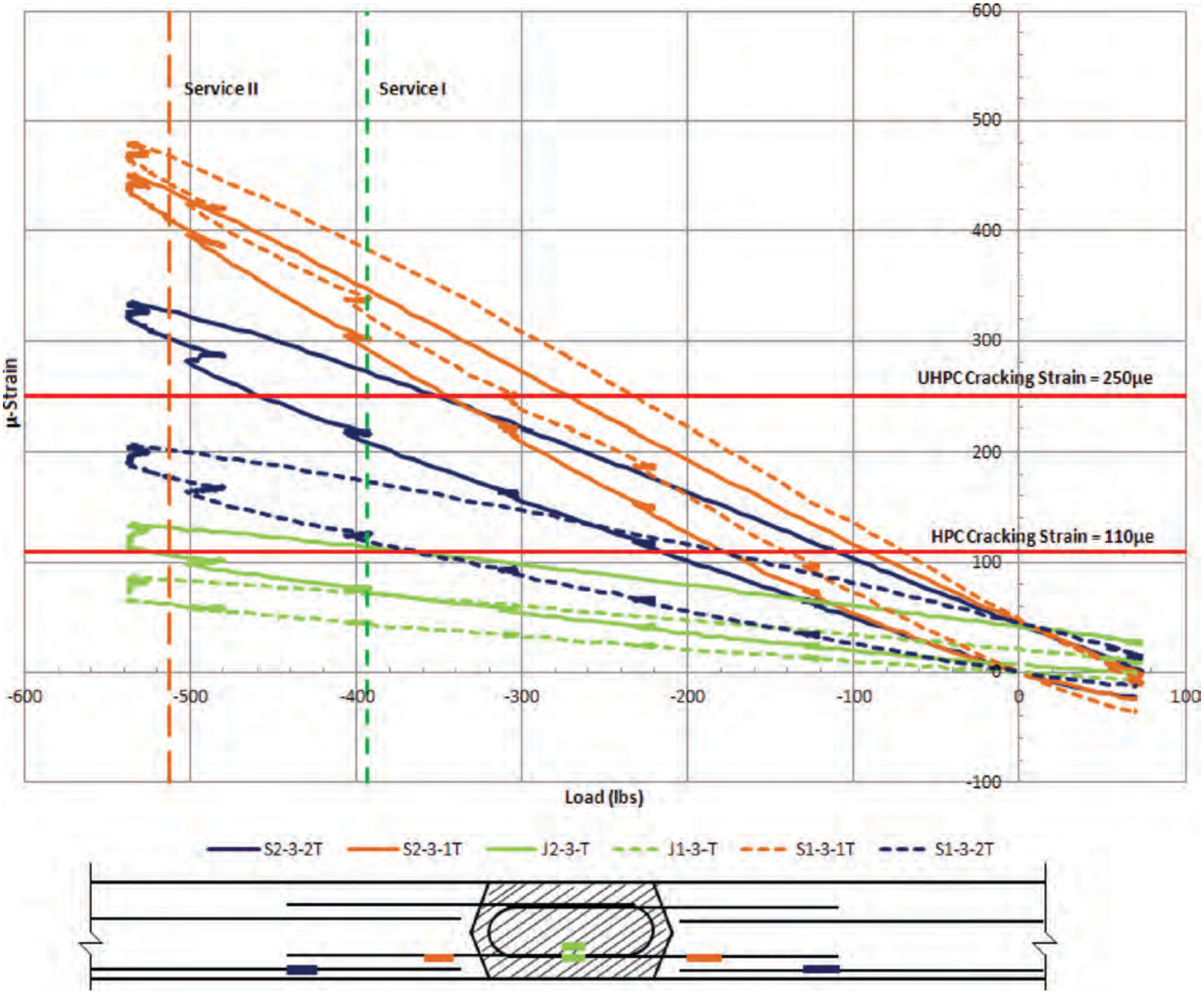

Figure C.53. Row 3, top-of-deck embedded strain gauges (static).

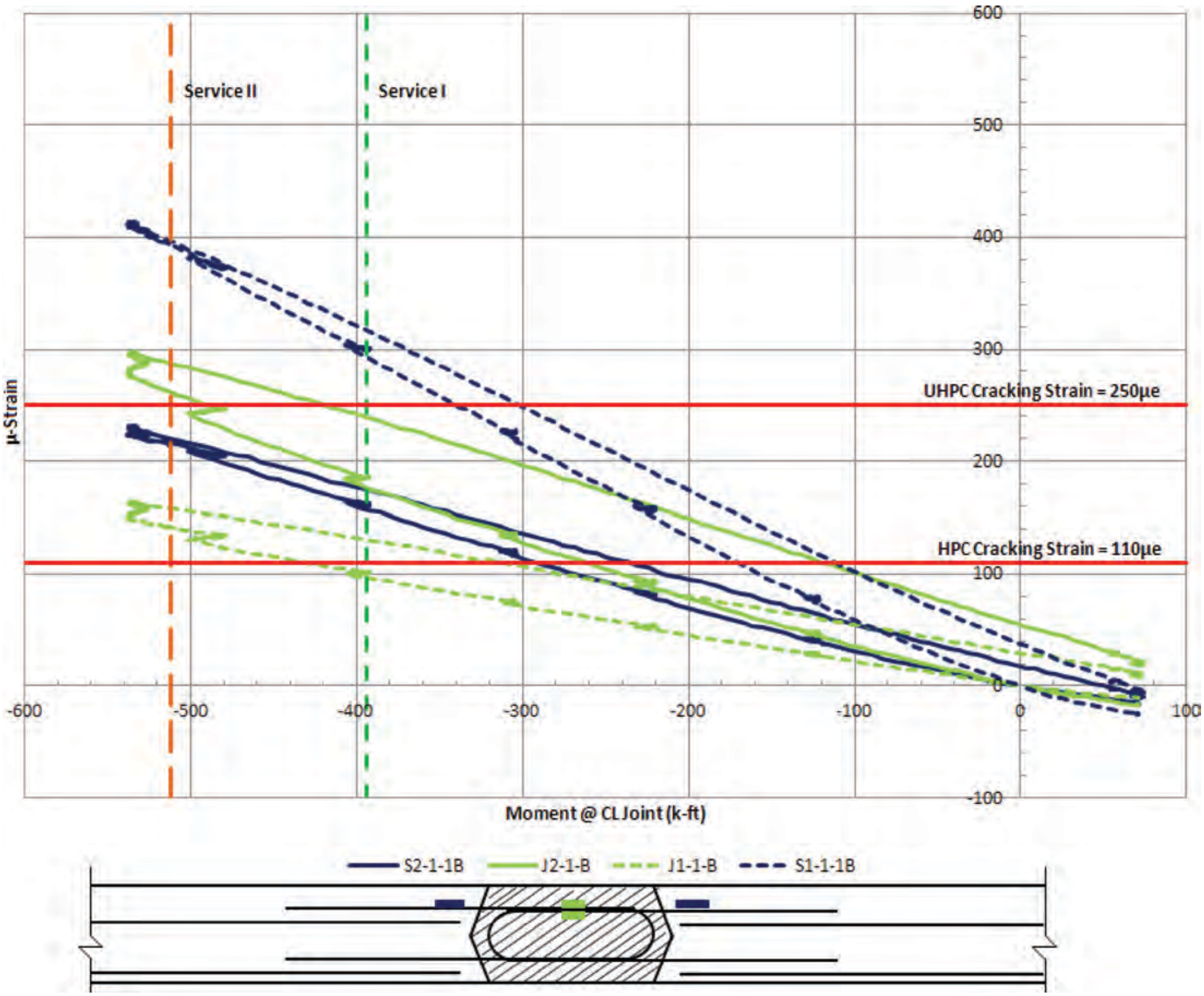

Figure C.54. Row 1, bottom-of-deck embedded strain gauges (static).

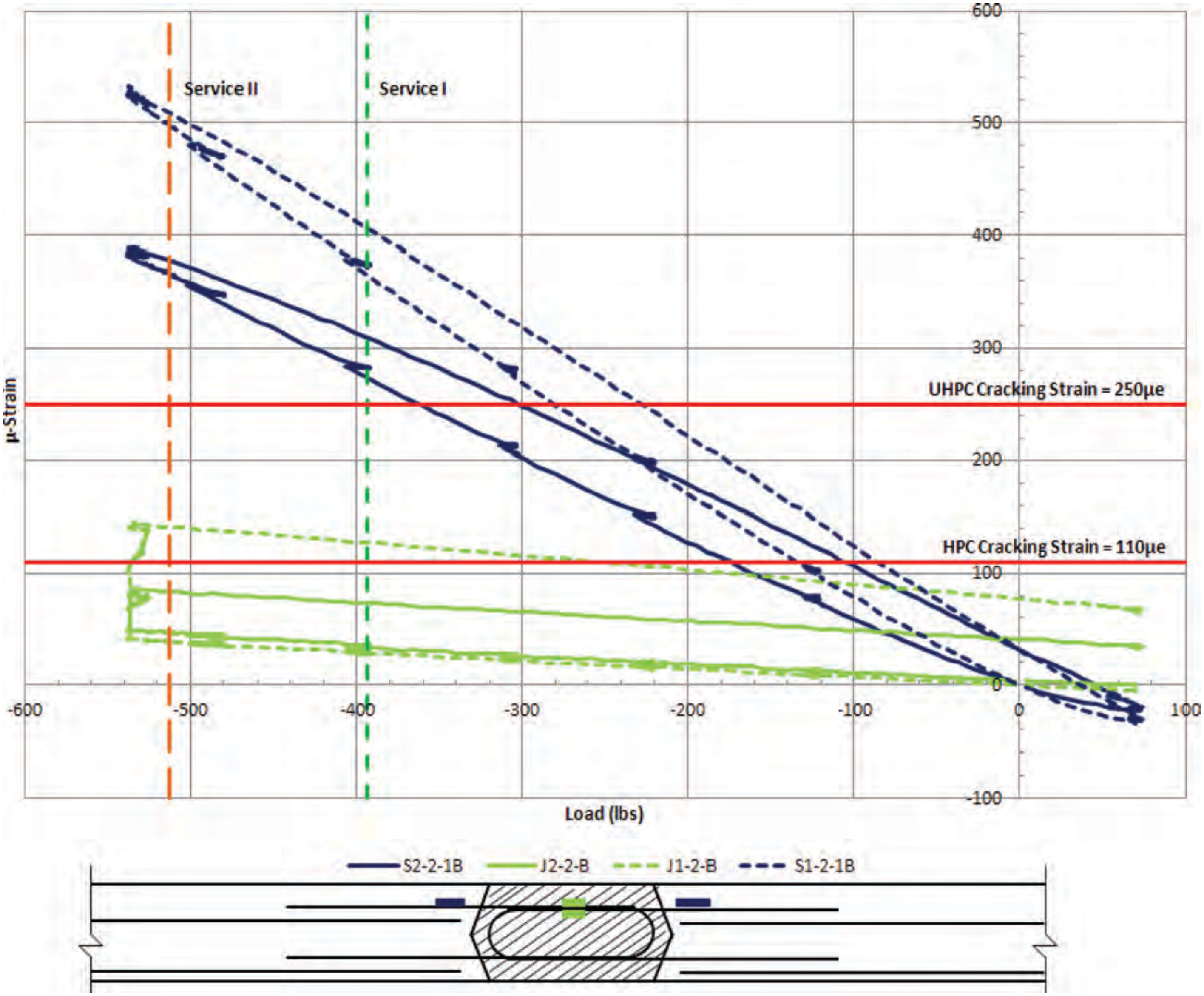

Figure C.55. Row 2, bottom-of-deck embedded strain gauges (static).

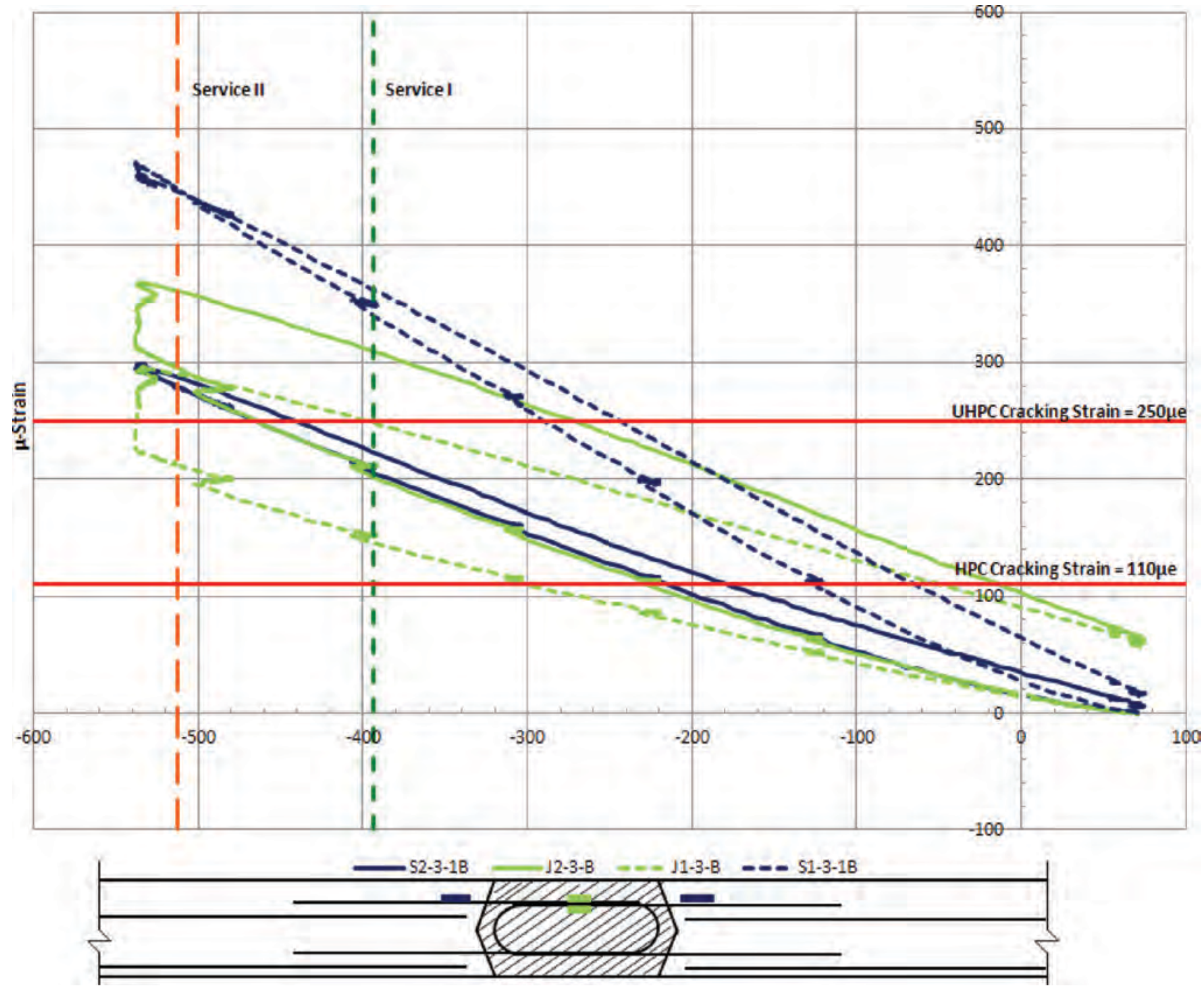

Figure C.56. Row 3, bottom-of-deck embedded strain gauges (static).

location that is 3 ft from the interface (Figure C.51 through Figure C.53). As observed with the surface-mounted strain gauges, the embedded gauges near the interface all exceed the HPC cracking strain before reaching Service Level I conditions. In the top-of-deck reinforcement, maximum strains of 540με, 550με, and 475με were recorded in S1-1-1T, S2-2-2T, and S2-3-2T, respectively. Strain in the UHPC joint (J1 and J2 gauges) were relatively lower in the top of the deck, not exceeding 160με, which is below the expected UHPC cracking strain level of 250με. Nearly all gauges located at the termination of the joint hairpin bar registered strain levels exceeding 110με before the Service Level II conditions. Those data suggested cracking in the prefabricated HPC deck modules under service-level loading. Cracking was not visually confirmed near the joint in the HPC deck during the incremental static loading, but opening and closing of the cracks during cyclic loading made cracking in the HPC clearly visible.

The groupings of embedded strain gauges on the bottom-of-deck reinforcement were located within the UHPC joint and near the joint interface only (Figure C.54 through Figure C.56). Results similar to those in the top of the deck were observed. Maximum strains of 460με, 520με, and 420με were registered near the joint interface reinforcement in the bottom of the deck at S1-3-1B, S1-2-1B, and S1-1-1B, respectively. Localized prying effects of the girders could potentially be responsible for the higher strain levels in the bottom-of-deck reinforcement for the UHPC joint in Rows 1 and 3. Strains at two of those locations were observed to exceed the UHPC cracking strain at or before reaching the Service Level II condition.

In general, internal strains in the UHPC were lower than in the HPC precast deck at each instrumentation location. As previously discussed, the gauges within 2 in. of the interface in the HPC deck registered the highest strains for all rows in both the bottom and the top of deck. In addition, the prevalence of the high strains at the termination of the hairpin reinforcement in the top of deck means cracking of the HPC is expected. These data suggest that the transverse connection detail was not satisfying the original project aim to avoid cracking in the deck over the pier.

Service-Level Fatigue Test Results

After the static tests were completed, fatigue testing commenced. Fatigue tests consisted of loading the specimen through the full service-level moment range for 1,000,000 cycles. The loading rate was one cycle per second, requiring approximately 2 weeks to complete. Strain data for embedded gauges on the top-of-deck reinforcement after the completion of 1,000,000 cycles are presented for gauges in Rows 1, 2, and 3 (Figure C.57 through Figure C.59).

The embedded strain results for the fatigue testing generally resembled those from the static testing. Similarly, the gauges near the interface consistently exhibited the highest strains, while the gauges within the UHPC registered the lowest in each of the instrumentation rows. When compared with the static testing results, some higher strain levels at 1,000,000 cycles suggested propagation of cracking and damage accrual within the specimen.

Visual inspection at the onset of cyclic loading revealed cracking in the precast HPC deck around the joint at roughly half of Service Level I conditions (Figure C.60). Inspection at 250,000 cycles identified cracks in the precast deck up to 10 ft away from the joint.

Damage accrual to the specimen during the fatigue testing was analyzed by comparing strain values at various cycle counts. Strain accrual data is presented for gauge groupings in Rows 1, 2, and 3 (Figure C.61 through Figure C.66 and Table C.14 through Table C.19). Increases in strains at embedded gauge locations throughout the specimen suggested the initial cracks were propagated from the incremental static service-level load tests during the fatigue testing. In the bottom-of-deck data for Rows 1 and 3 (Figure C.62 and Figure C.66), the high strain levels within the UHPC joint likely stem from the localized prying effects of the girders protruding into the joint on the bottom of the deck.

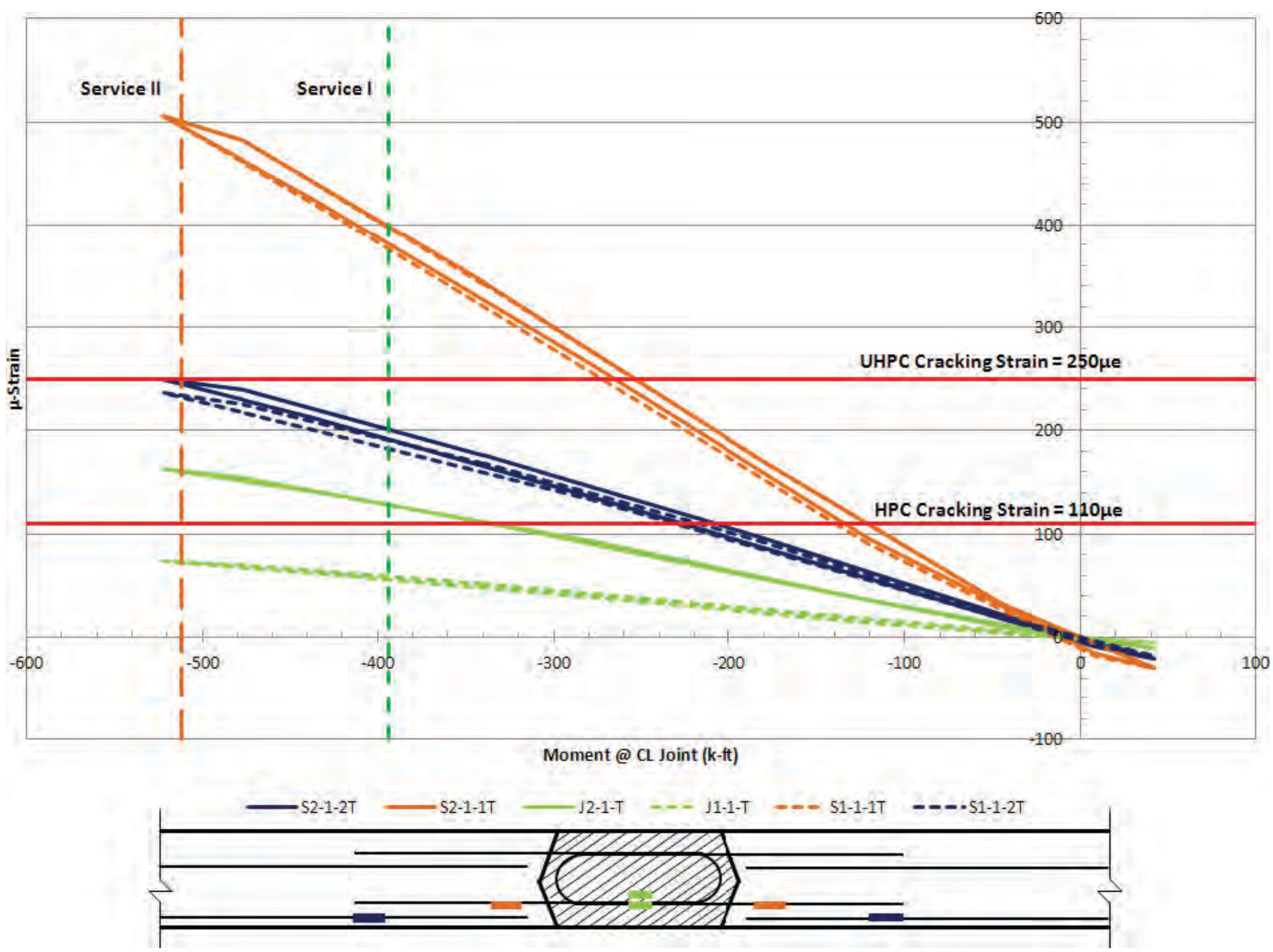

Figure C.57. Row 1, top-of-deck embedded strain gauges (1,000,000 cycles).

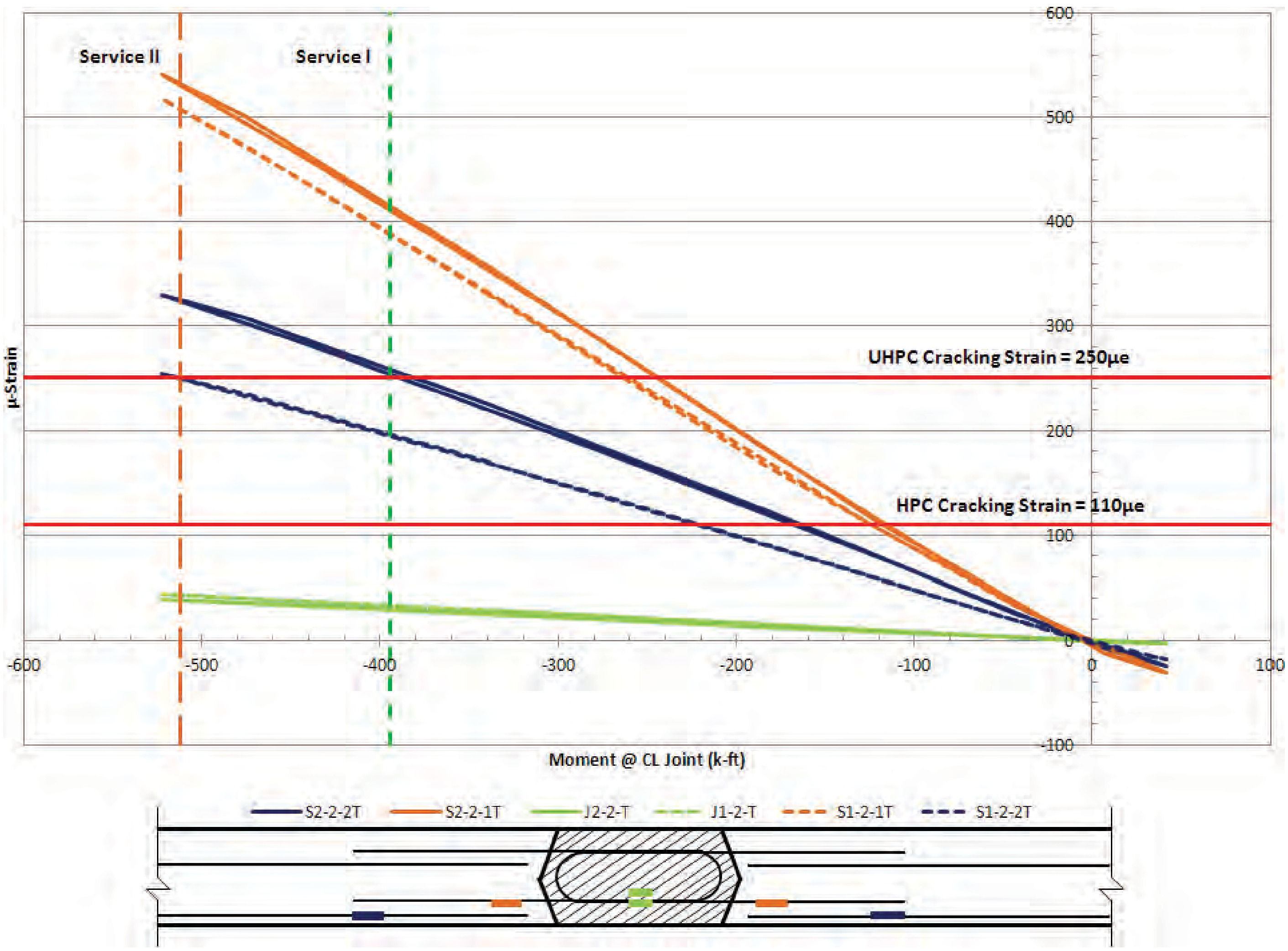

Figure C.58. Row 2, top-of-deck embedded strain gauges (1,000,000 cycles).

Table C.14. Row 1, Top-of-Deck Strain Accrual

Gauge	S2-1-2T με	S2-1-1T με	J2-1-T με	J1-1-T με	S1-1-1T με	S1-1-2T με
3,000 cycles	210	440	146	58	474	172
1,000,000 cycles	250	506	163	73	505	236
Strain increase	19.3%	15.0%	12.1%	27.4%	6.6%	37.6%

Table C.15. Row 1, Bottom-of-Deck Strain Accrual

Gauge	S2-1-1B με	J2-1-B με	J1-1-B με	S1-1-1B με
3,000 cycles	208	278	155	336
1,000,000 cycles	259	329	200	336
Strain increase	24.4%	18.4%	28.6%	0.0%

Table C.16. Row 2, Top-of-Deck Strain Accrual

Gauge	S2-2-2T με	S2-2-1T με	J2-2-T με	J1-2-T με	S1-2-1T με	S1-2-2T με
3,000 cycles	288	472	38	42	512	159
1,000,000 cycles	330	542	39	44	518	255
Strain increase	14.5%	14.7%	1.5%	3.6%	1.2%	60.2%

Table C.17. Row 2, Bottom-of-Deck Strain Accrual

Gauge	S2-2-1B με	J2-2-B με	J1-2-B με	S1-2-1B με
3,000 cycles	366	47	78	500
1,000,000 cycles	411	52	88	492
Strain increase	12.3%	10.8%	12.1%	–1.7%

Table C.18. Row 3, Top-of-Deck Strain Accrual

Gauge	S2-3-2T με	S2-3-1T με	J2-3-T με	J1-3-T με	S1-3-1T με	S1-3-2T με
3,000 cycles	317	414	113	81	458	203
1,000,000 cycles	331	460	127	95	488	314
Strain increase	4.5%	11.1%	12.6%	16.4%	6.5%	54.2%

Table C.19. Row 3, Bottom-of-Deck Strain Accrual

Gauge	S2-3-1B με	J2-3-B με	J1-3-B με	S1-3-1B με
3,000 cycles	241	326	254	365
1,000,000 cycles	288	377	297	349
Strain increase	19.5%	15.9%	16.9%	–4.2%

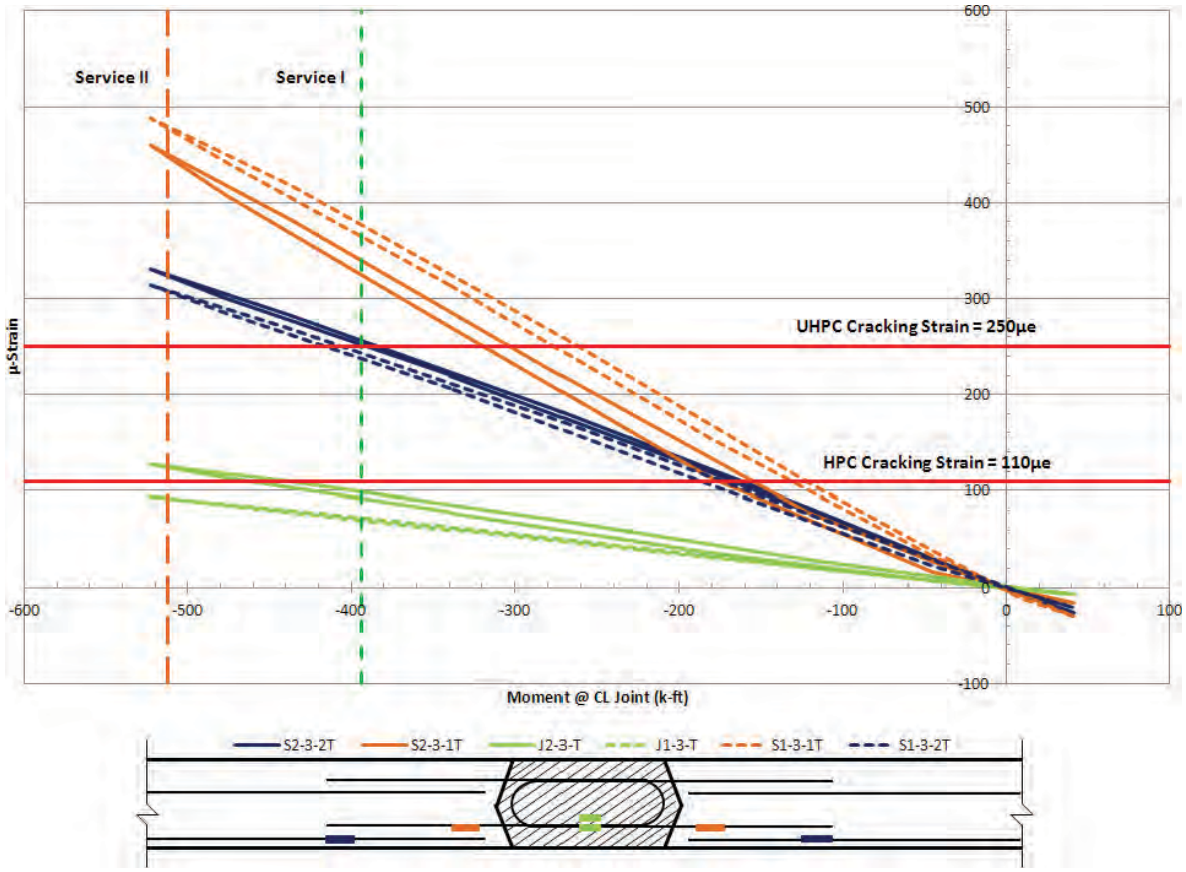

Figure C.59. Row 3, top-of-deck embedded strain gauges (1,000,000 cycles).

At 500,000, 750,000, and 1,000,000 cycles, further visual inspection confirmed propagation of the existing cracks and formation of new full-depth cracks in the precast deck panels within 10 ft of the joint.

The strain accrual throughout the fatigue testing varied at each location. Generally, the strain levels increased to 28%.

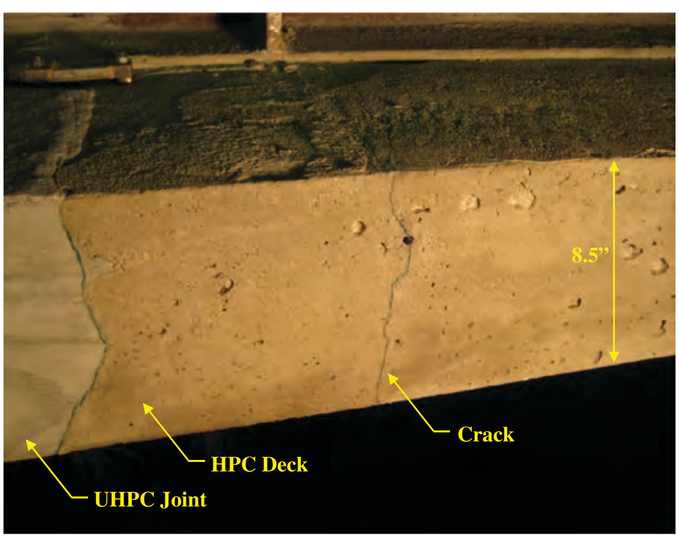

Figure C.60. Full-depth cracking in precast deck.

At one location, the strain level increased 60%, while at others only a small decrease was observed. These outliers most likely result from the highly sensitive nature of embedded strain gauges to localized cracking. In addition, because of the cyclic nature of loading and the frequency of data recordings, the peak strain readings for some gauges could have been missed, causing an apparent decrease in strain at certain locations.

As suggested by static testing results, visual inspection during fatigue testing confirmed early debonding and significant opening at the interface between the precast HPC deck panels and UHPC joint. In addition to debonding at the deck joint interface, cracking in the precast deck panels near the transverse joints was observed below Service Level I conditions. To mitigate these serious durability concerns, a modified detail was devised to posttension the deck in this region and minimize tensile stresses in the concrete throughout Service Level II without compromising the accelerated construction aspect of the SHRP 2 R04 project.

Connection Retrofit Test Results

Following fatigue testing, the specimen was modified to include high-strength steel rods mounted just under the deck surface to

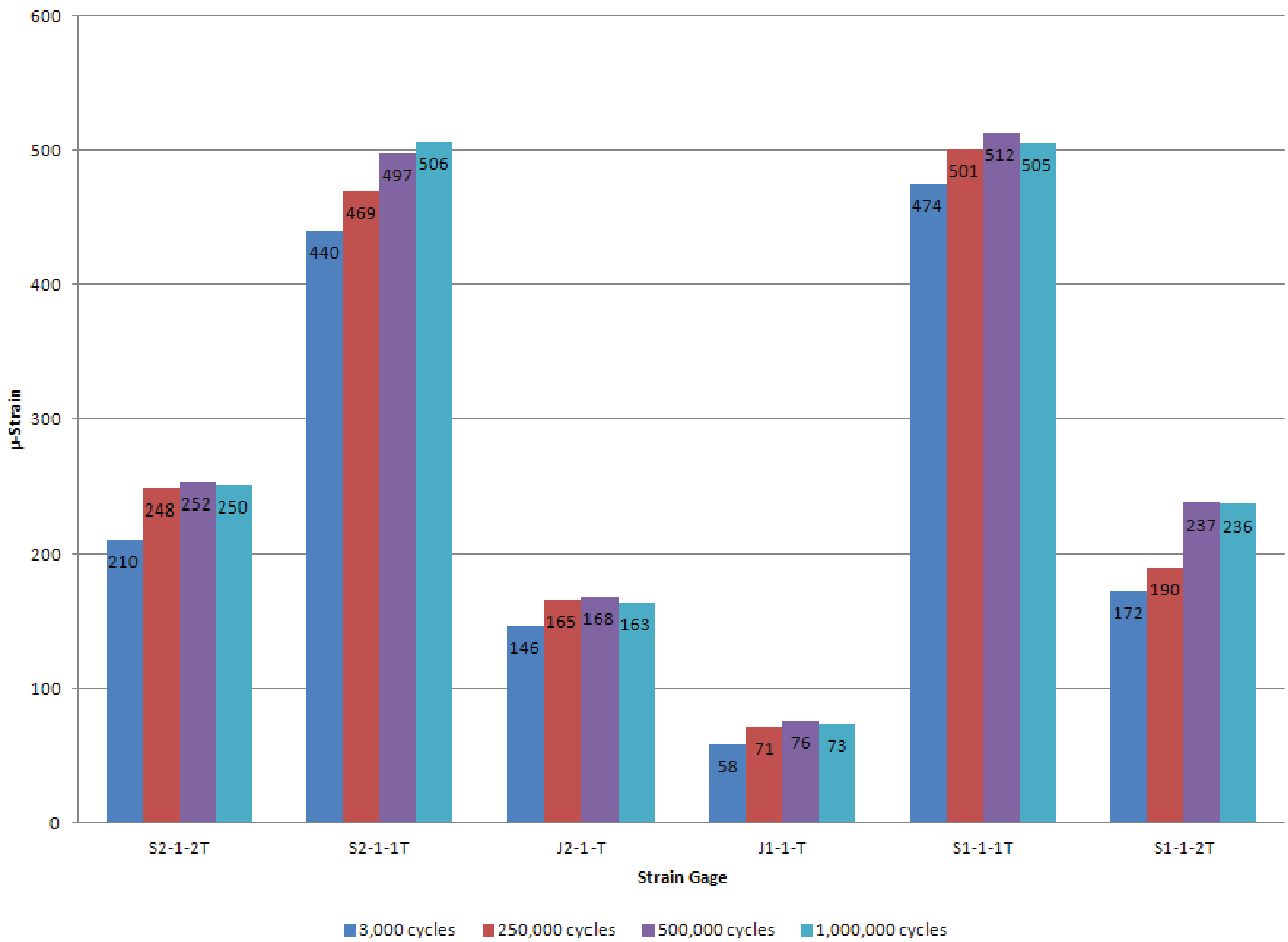

Figure C.61. Row 1, top-of-deck strain accrual.

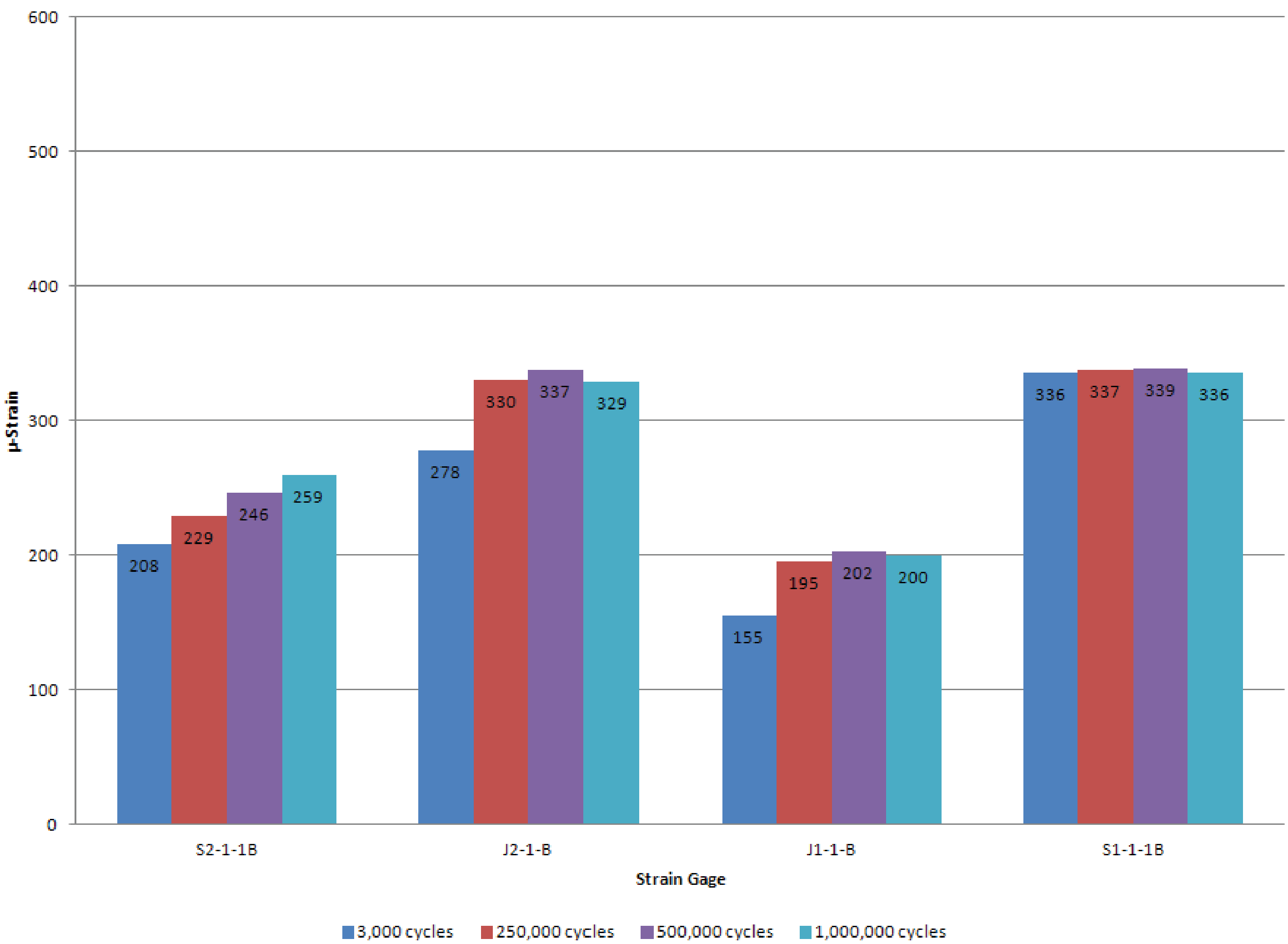

Figure C.62. Row 1, bottom-of-deck strain accrual.

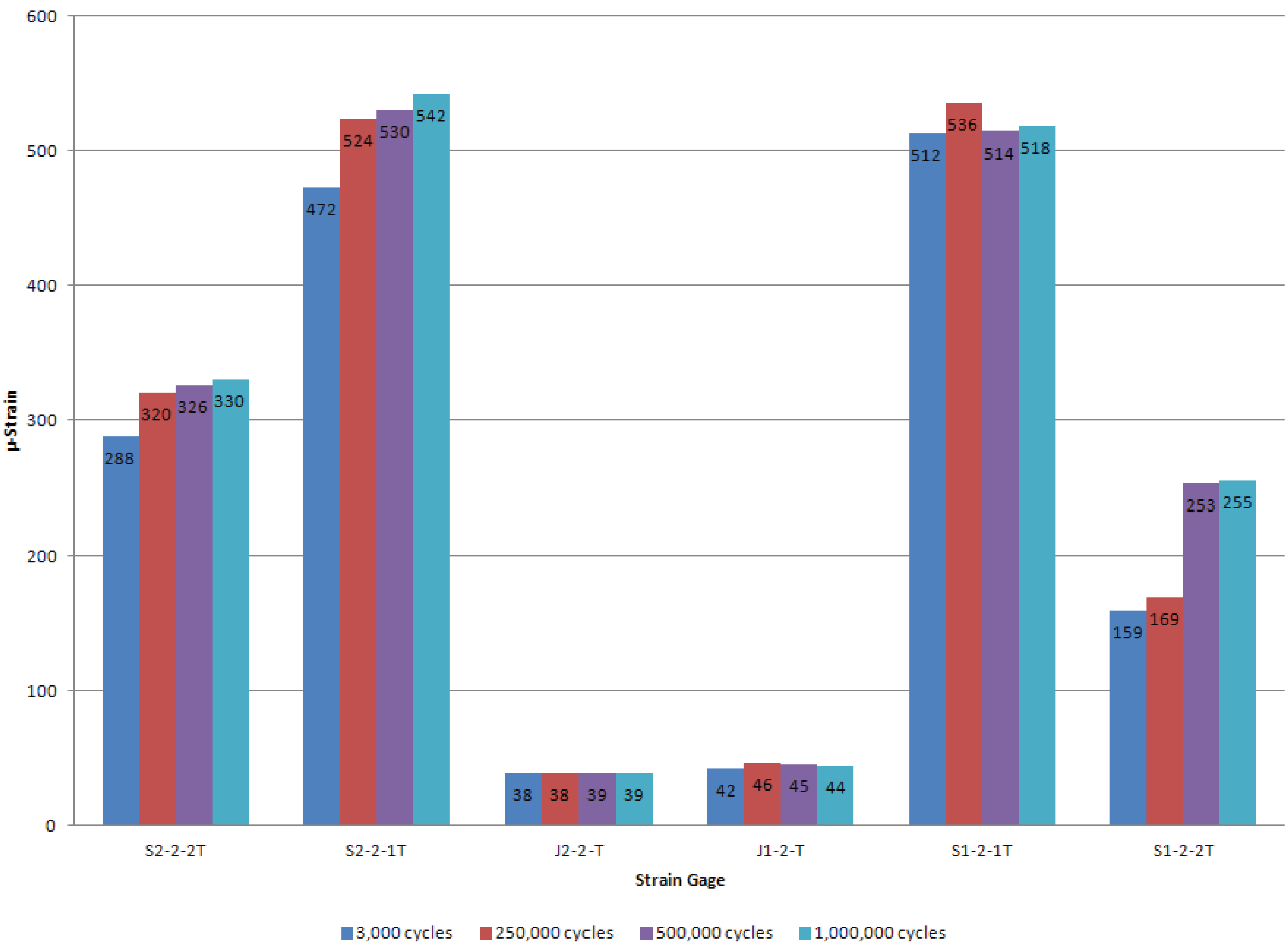

Figure C.63. Row 2, top-of-deck strain accrual.

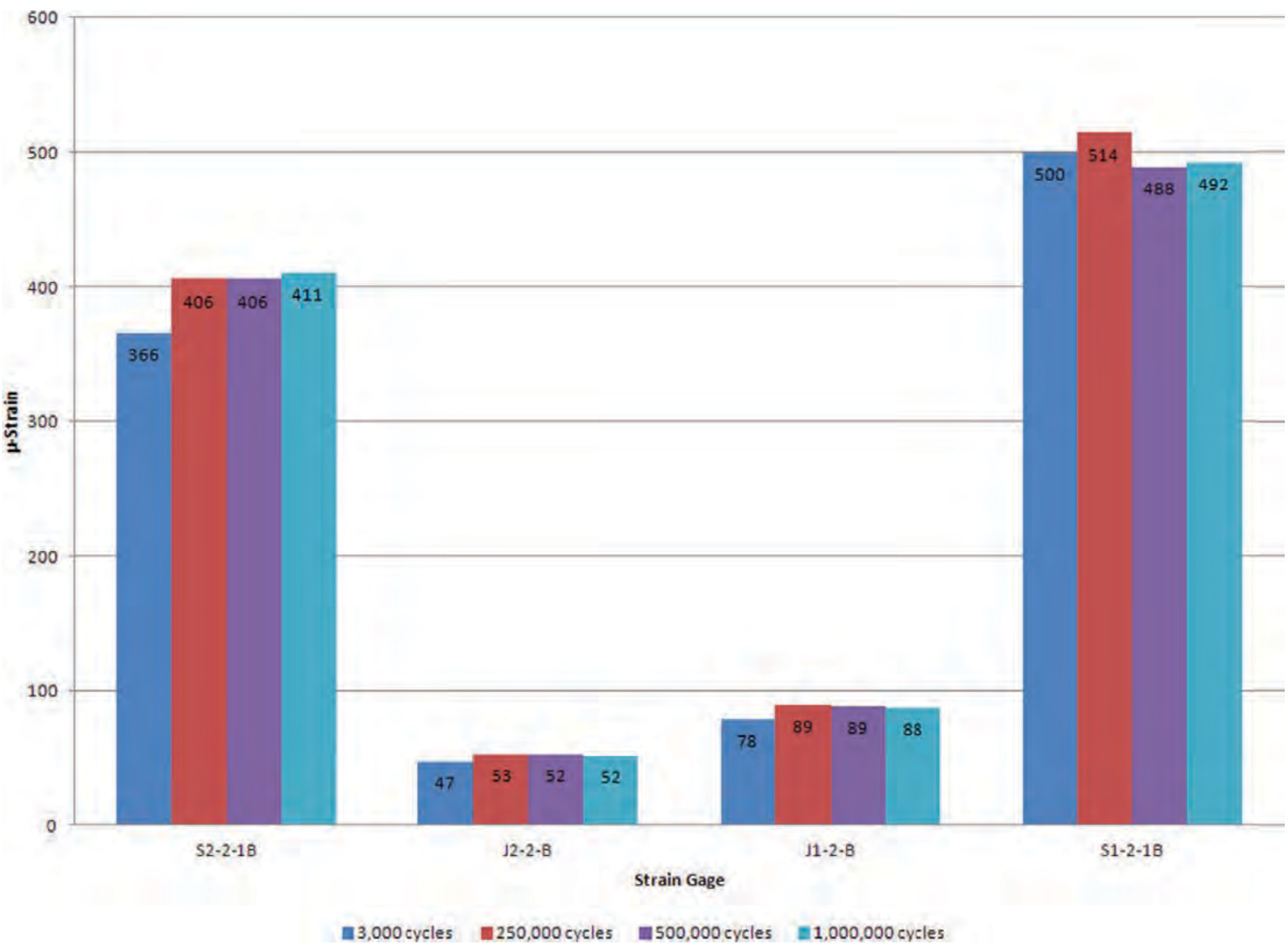

Figure C.64. Row 2, bottom-of-deck strain accrual.

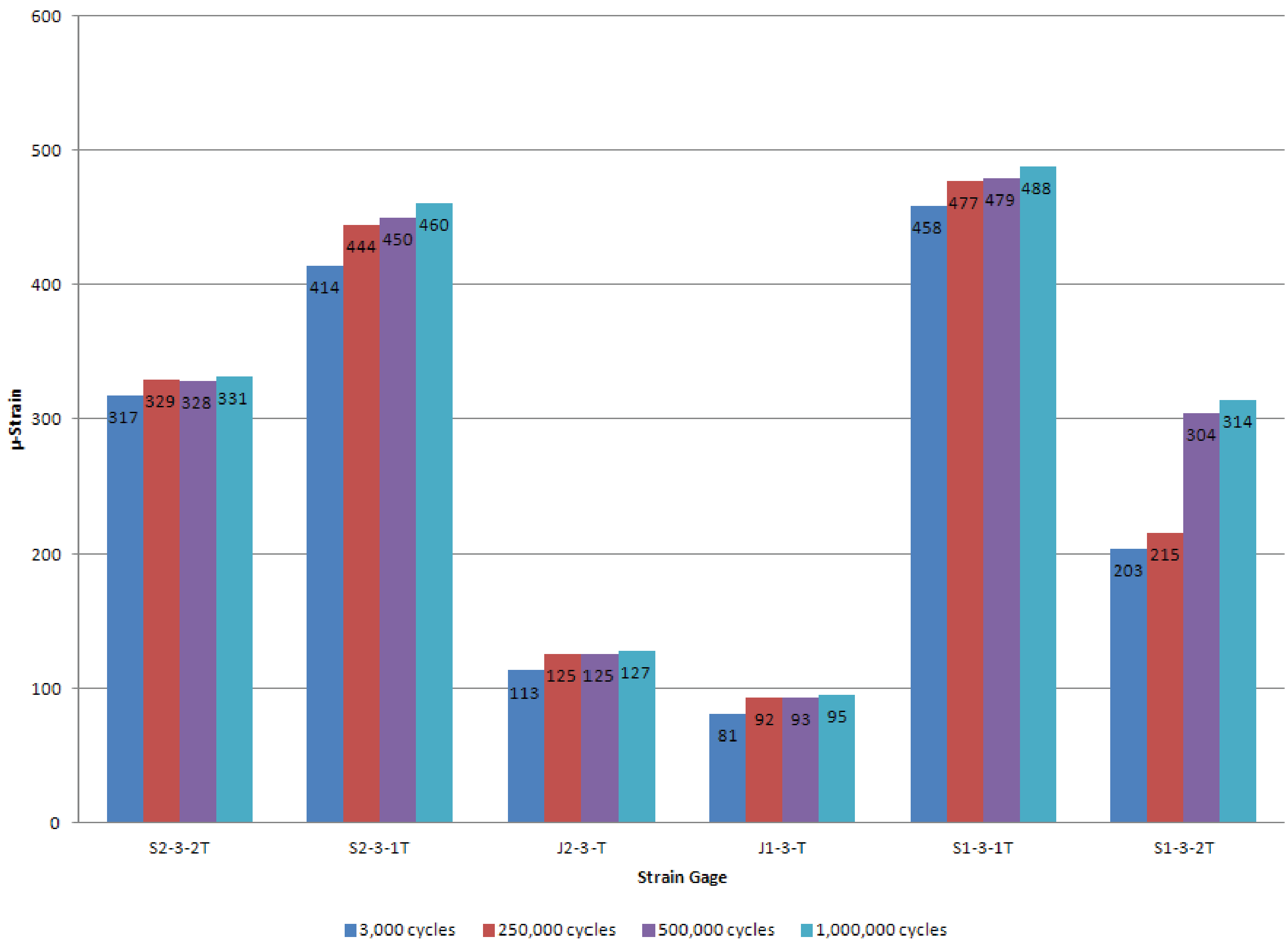

Figure C.65. Row 3, top-of-deck strain accrual.

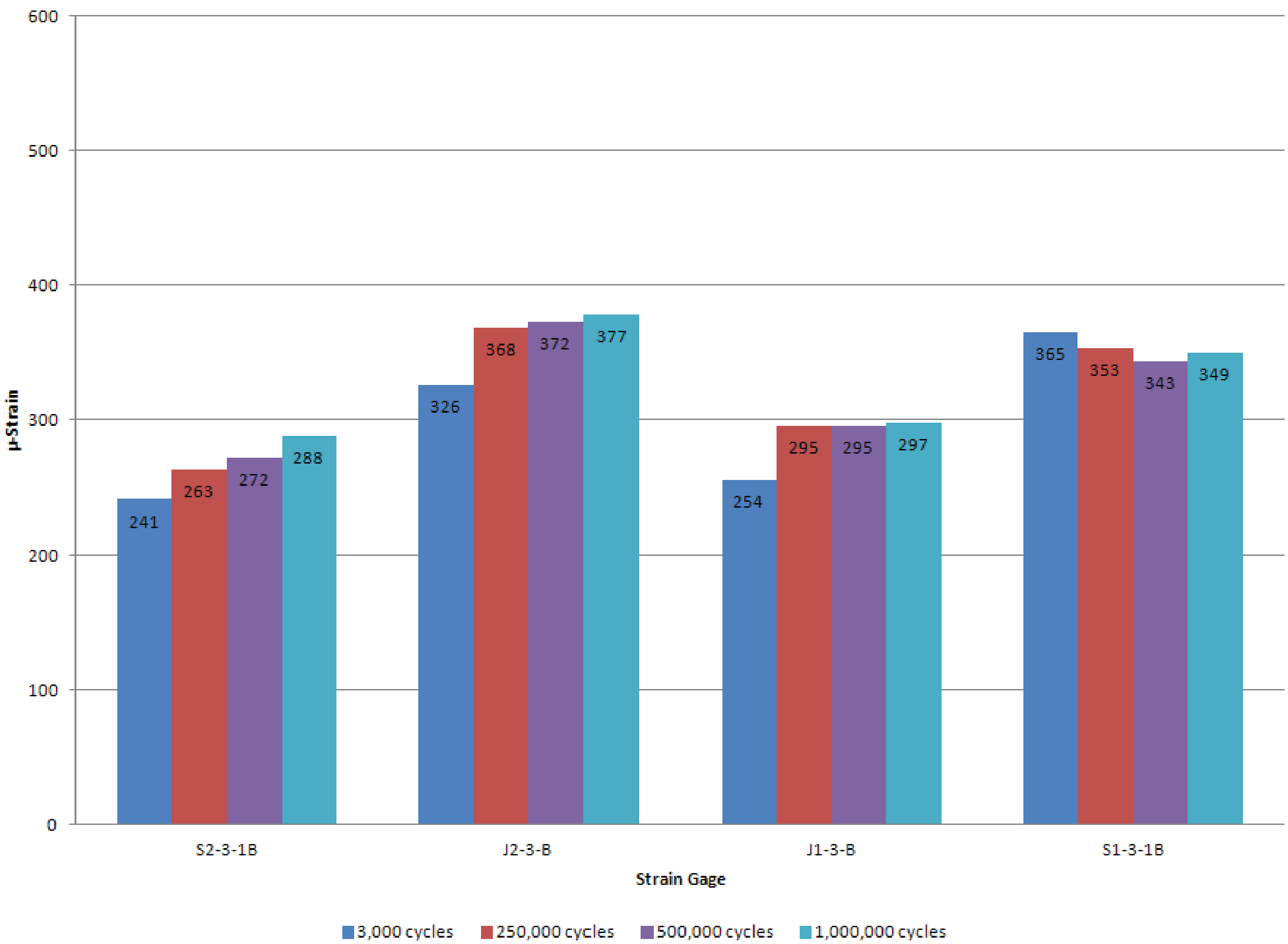

Figure C.66. Row 3, bottom-of-deck strain accrual.

posttension the entire joint region and prevent any possible cracking of the deck or joint in this region (Figure C.30). The retrofit detail was tested through the full range of service-level moments with a 60-kip posttensioning force per rod and again with a 70-kip posttensioning force per rod. The static test results for surface-mounted strain gauges across the joint interface (Figure C.67 and Figure C.74) and embedded strain gauges in Rows 1, 2, and 3 (Figure C.68 through Figure C.71 and Figure C.75 through Figure C.80) are presented in this section.

The 60-kip posttensioning force in each of the rods reduced tensile strain across the joint interface such that the HPC cracking strain was not reached until Service Level I conditions (Figure C.67). A maximum tensile strain of 200με was recorded across the joint interface at Service Level I moment. Embedded strain gauges never exceeded 110με before Service Level I conditions. However, strains did exceed the HPC cracking strain in the top-of-deck embedded gauges before reaching Service Level II (Figure C.68 through Figure C.71).

By contrast, applying 70 kips posttensioning force in each of the rods minimized or negated the tensile strain across the interface entirely when loaded to Service Level I. All surface-mounted strain gauges spanning the interface registered below the HPC cracking strain until after the Service Level I conditions were applied (Figure C.74). Tensile strain data across the interface revealed a maximum 29με at Service Level I moment. None of the embedded strain gauges, top- and bottom-of-deck, exceeded 110με until Service Level II conditions were applied (Figure C.75 through Figure C.78). The 70-kip-per-rod posttensioning force was recommended for application in the SHRP 2 Project R04 demonstration bridge.

Once again, embedded strain gauges within the UHPC joint consistently registered strains below those in the HPC deck for each of the instrumentation rows in both the top- and bottom-of-deck reinforcement. The 60 kips of posttensioning force per rod reduced strain levels from the previous incremental static testing but not completely below the HPC

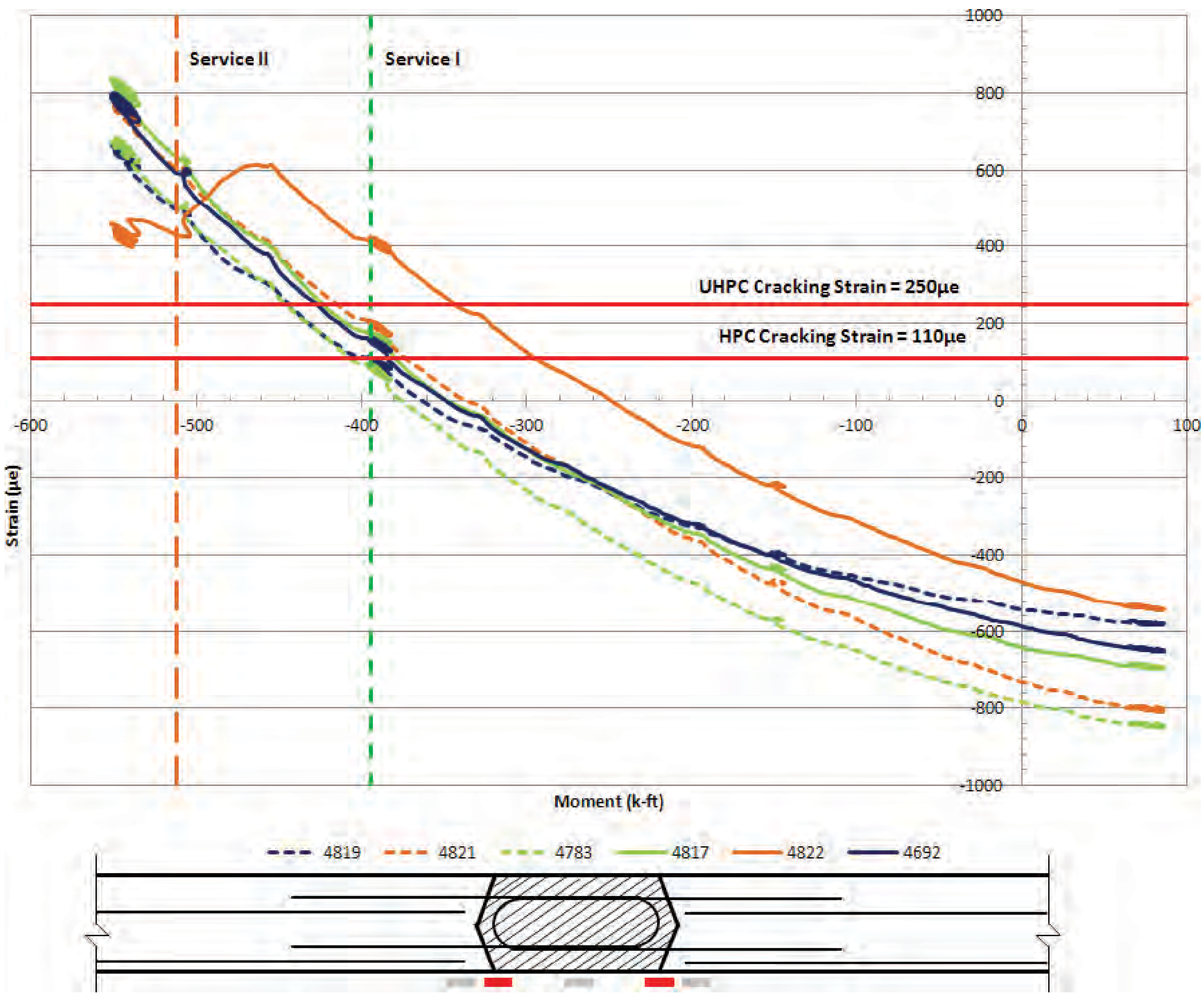

Figure C.67. Top-of-deck surface-mounted strain gauges over interface (60-k retrofit).

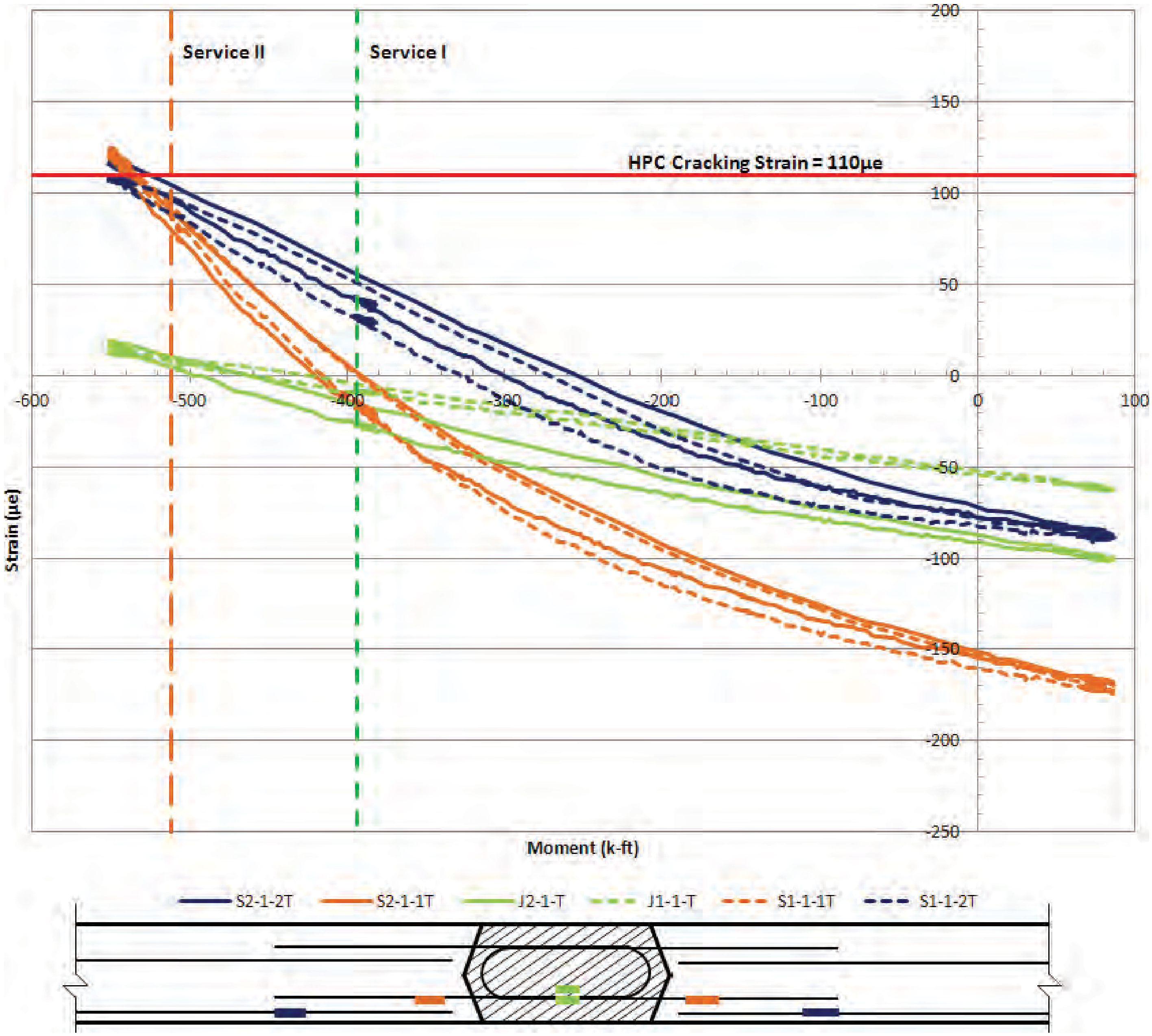

Figure C.68. Row 1, top-of-deck embedded strain gauges (60-k retrofit).

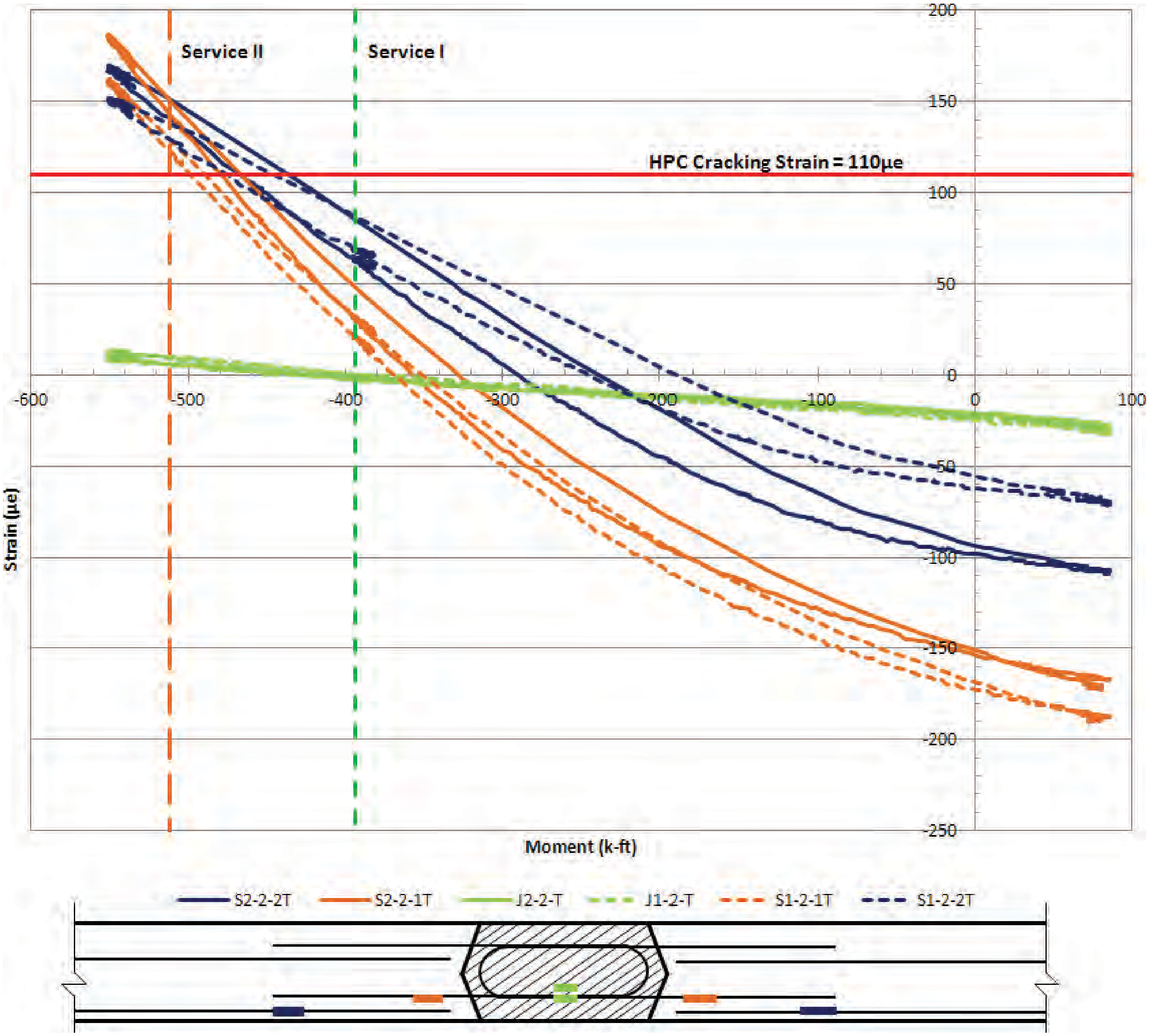

Figure C.69. Row 2, top-of-deck embedded strain gauges (60-k retrofit).

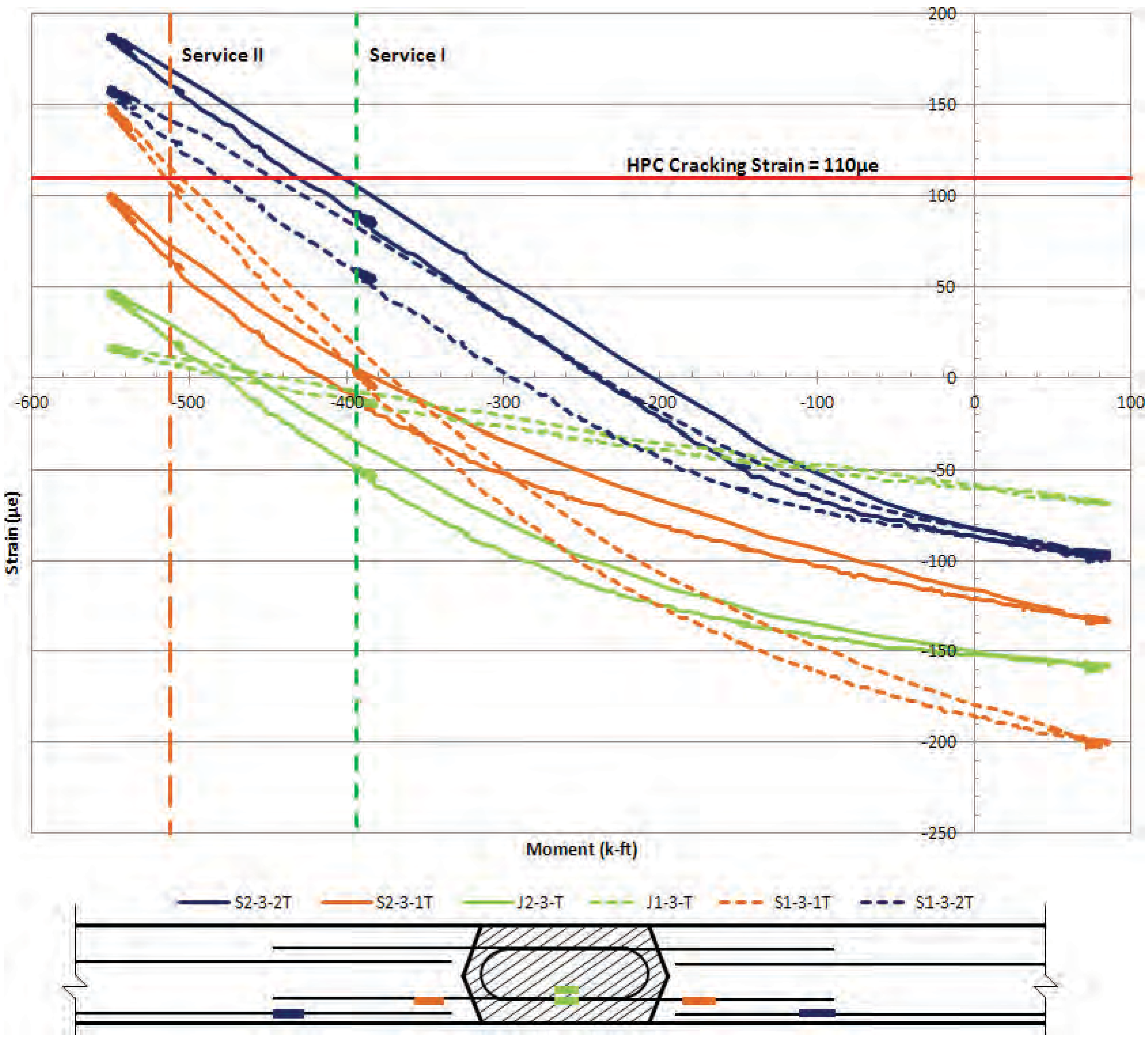

Figure C.70. Row 3, top-of-deck embedded strain gauges (60-k retrofit).

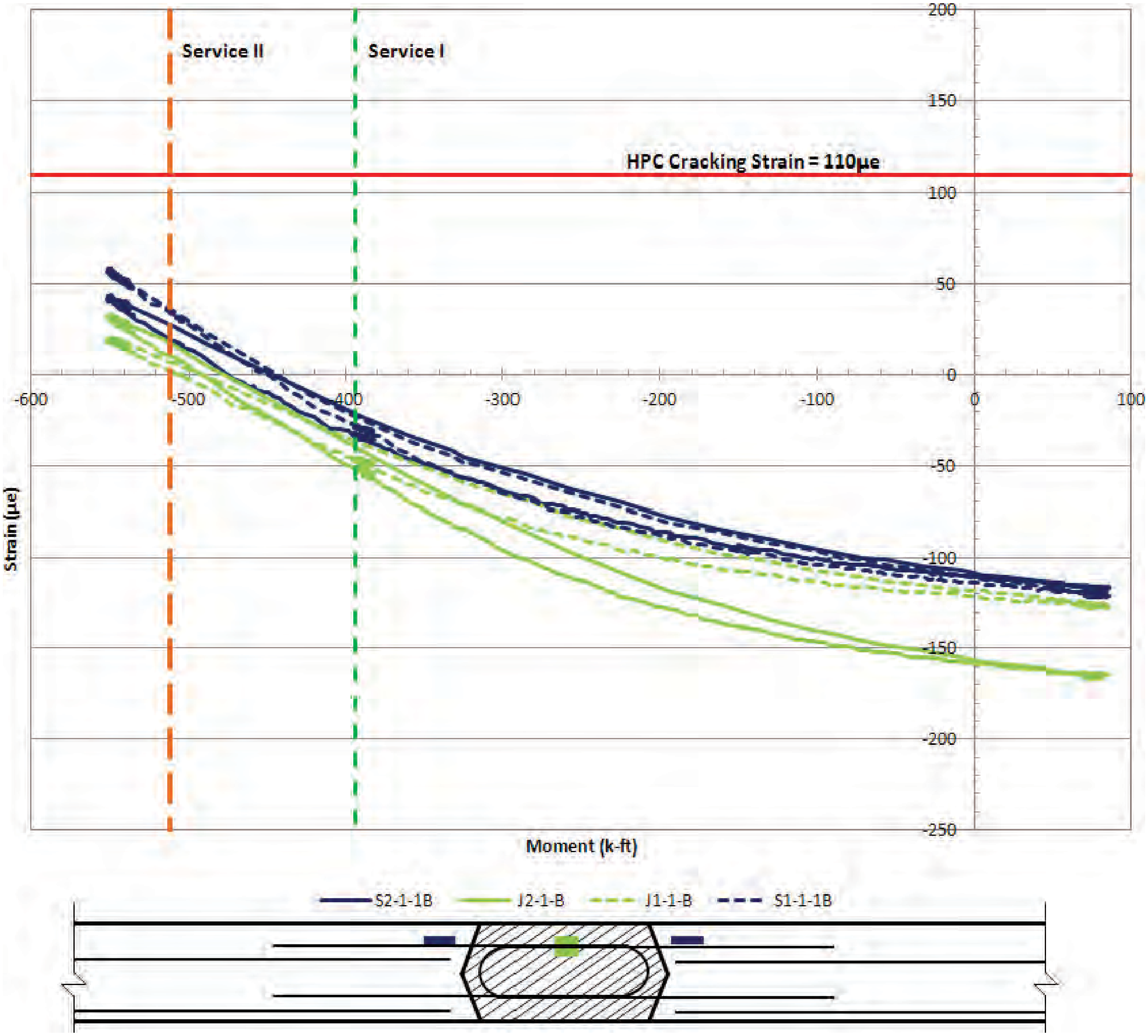

Figure C.71. Row 1, bottom-of-deck embedded strain gauges (60-k retrofit).

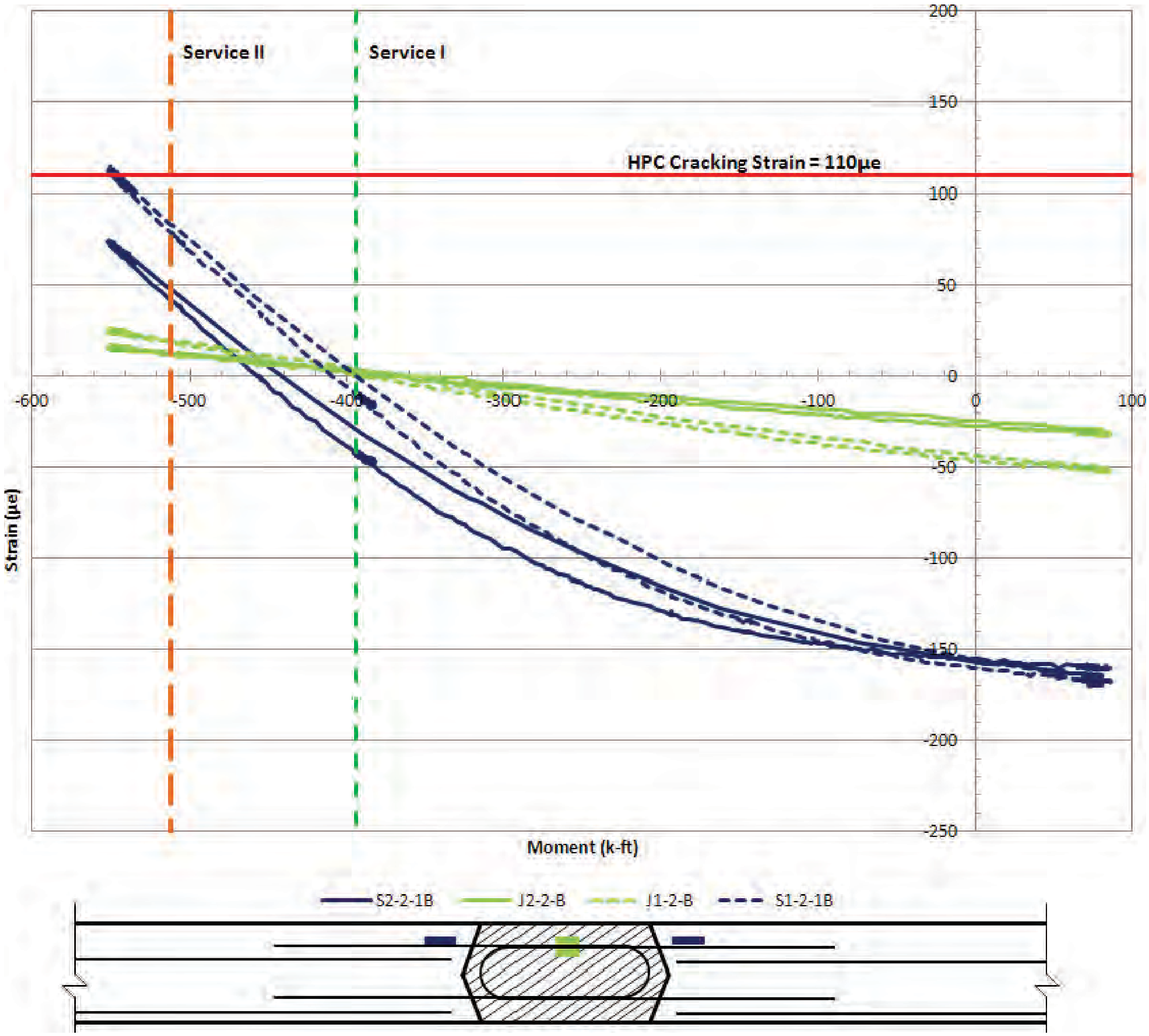

Figure C.72. Row 2, bottom-of-deck embedded strain gauges (60-k retrofit).

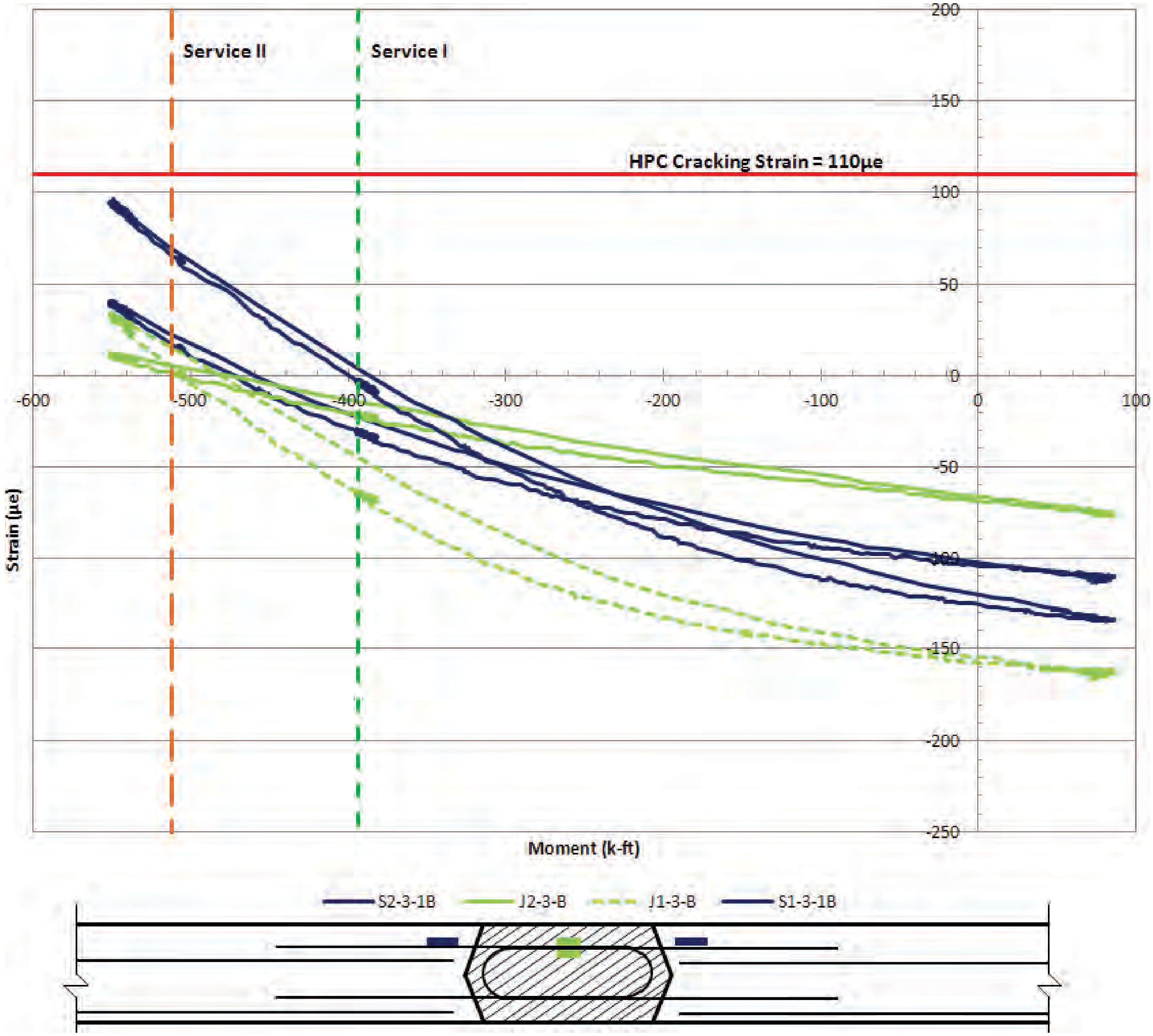

Figure C.73. Row 3, bottom-of-deck embedded strain gauges (60-k retrofit).

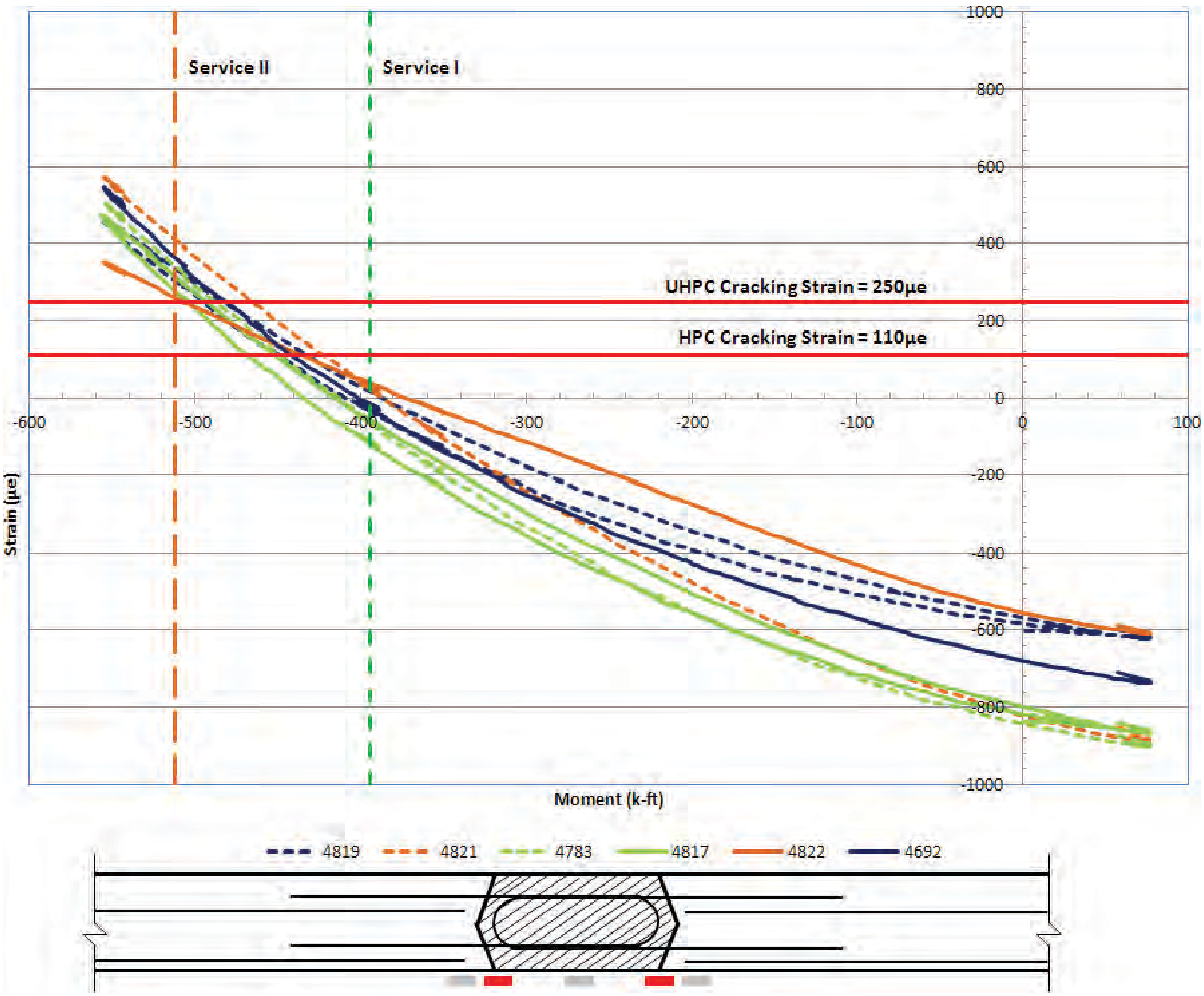

Figure C.74. Top-of-deck surface-mounted strain gauges over interface (70-k retrofit).

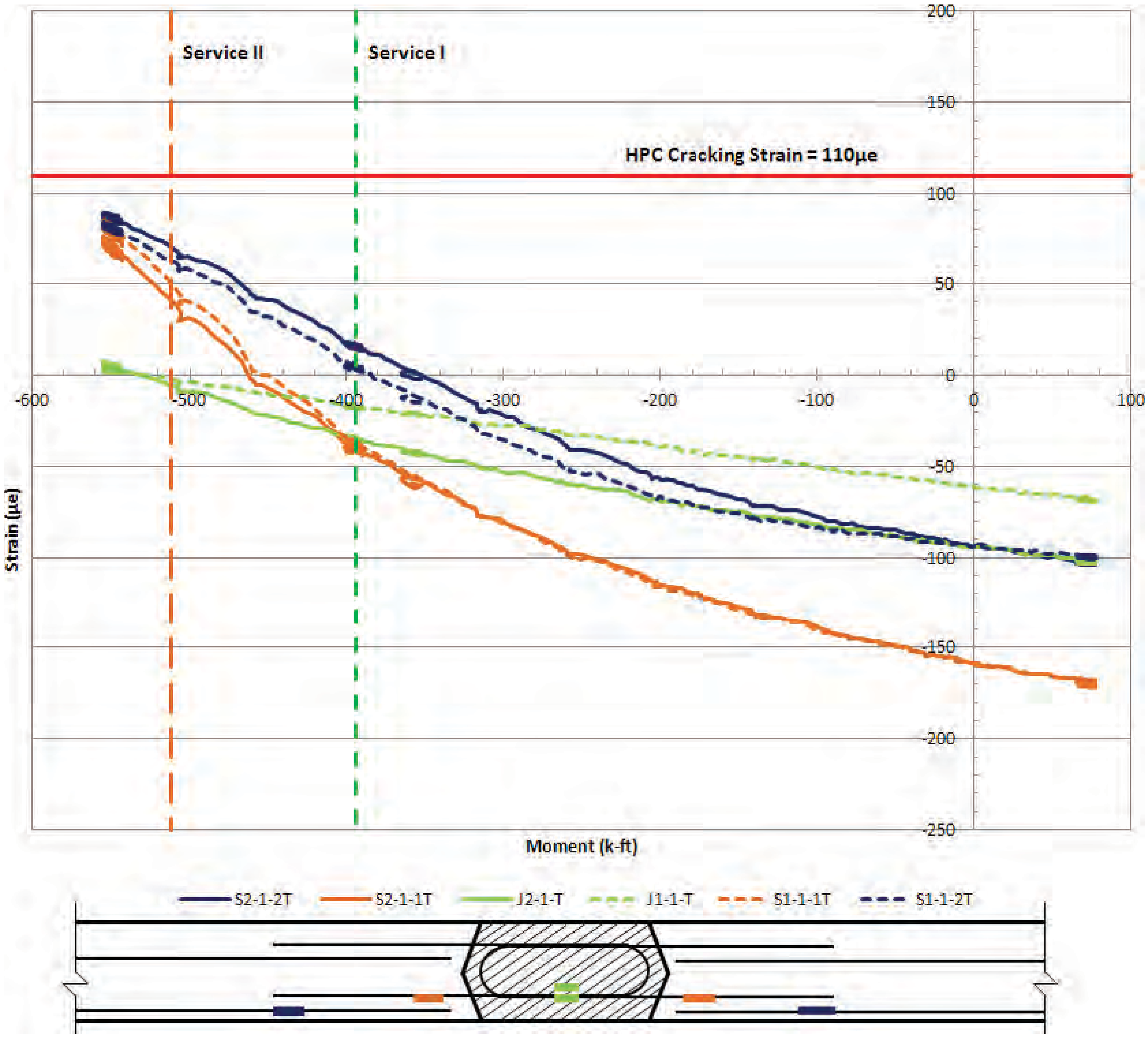

Figure C.75. Row 1, top-of-deck embedded strain gauges (70-k retrofit).

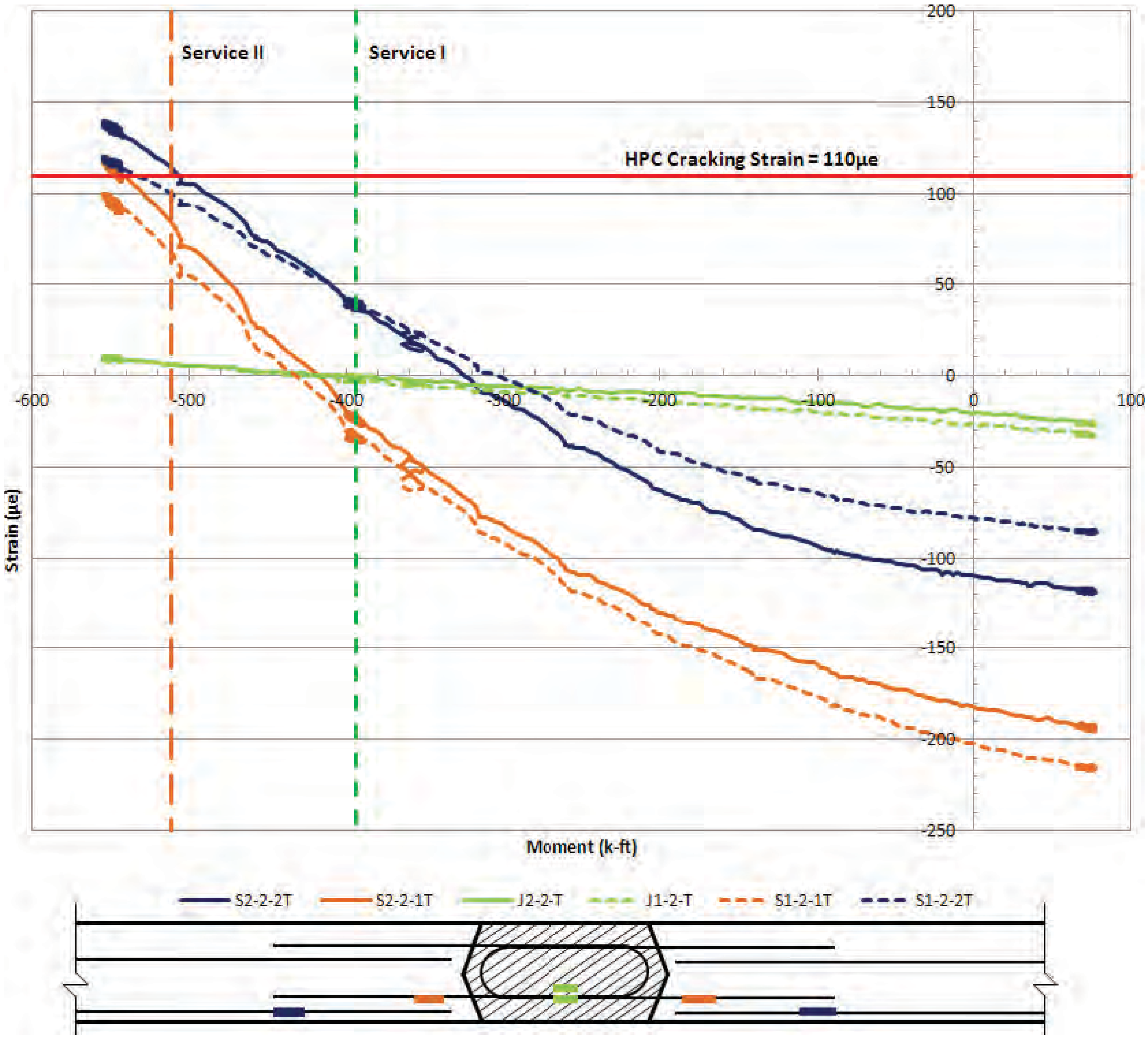

Figure C.76. Row 2, top-of-deck embedded strain gauges (70-k retrofit).

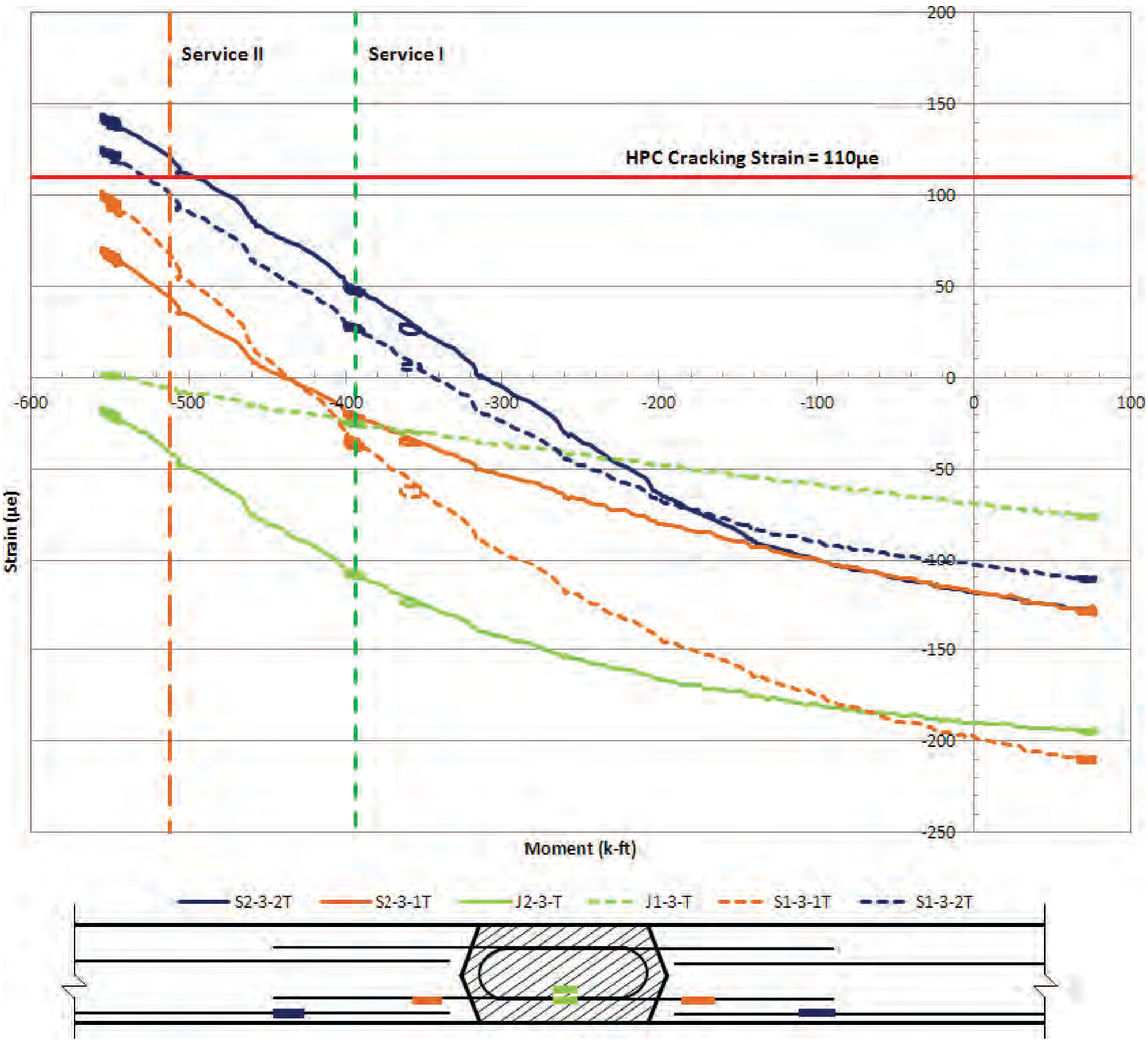

Figure C.77. Row 3, top-of-deck embedded strain gauges (70-k retrofit).

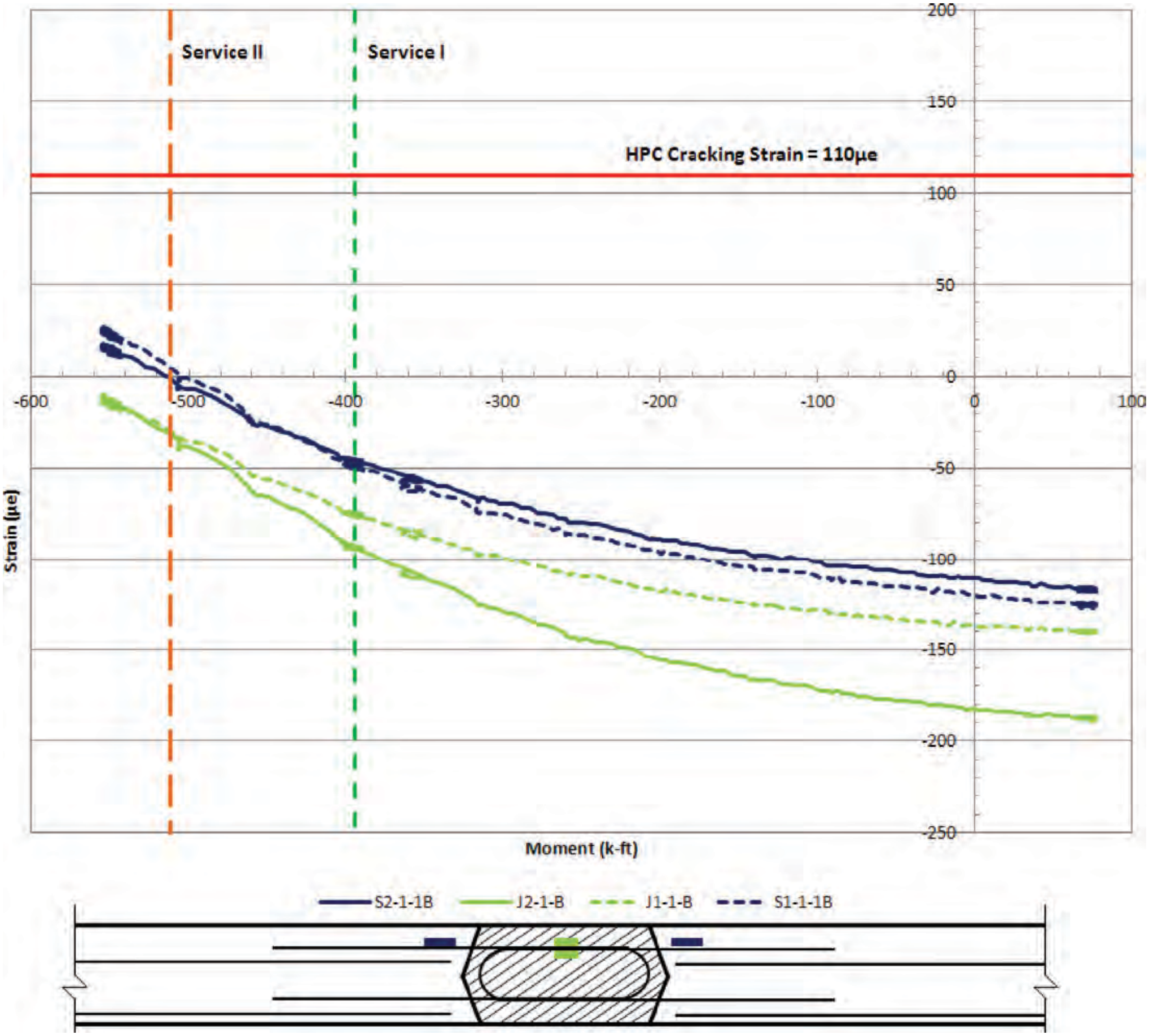

Figure C.78. Row 1, bottom-of-deck embedded strain gauges (70-k retrofit).

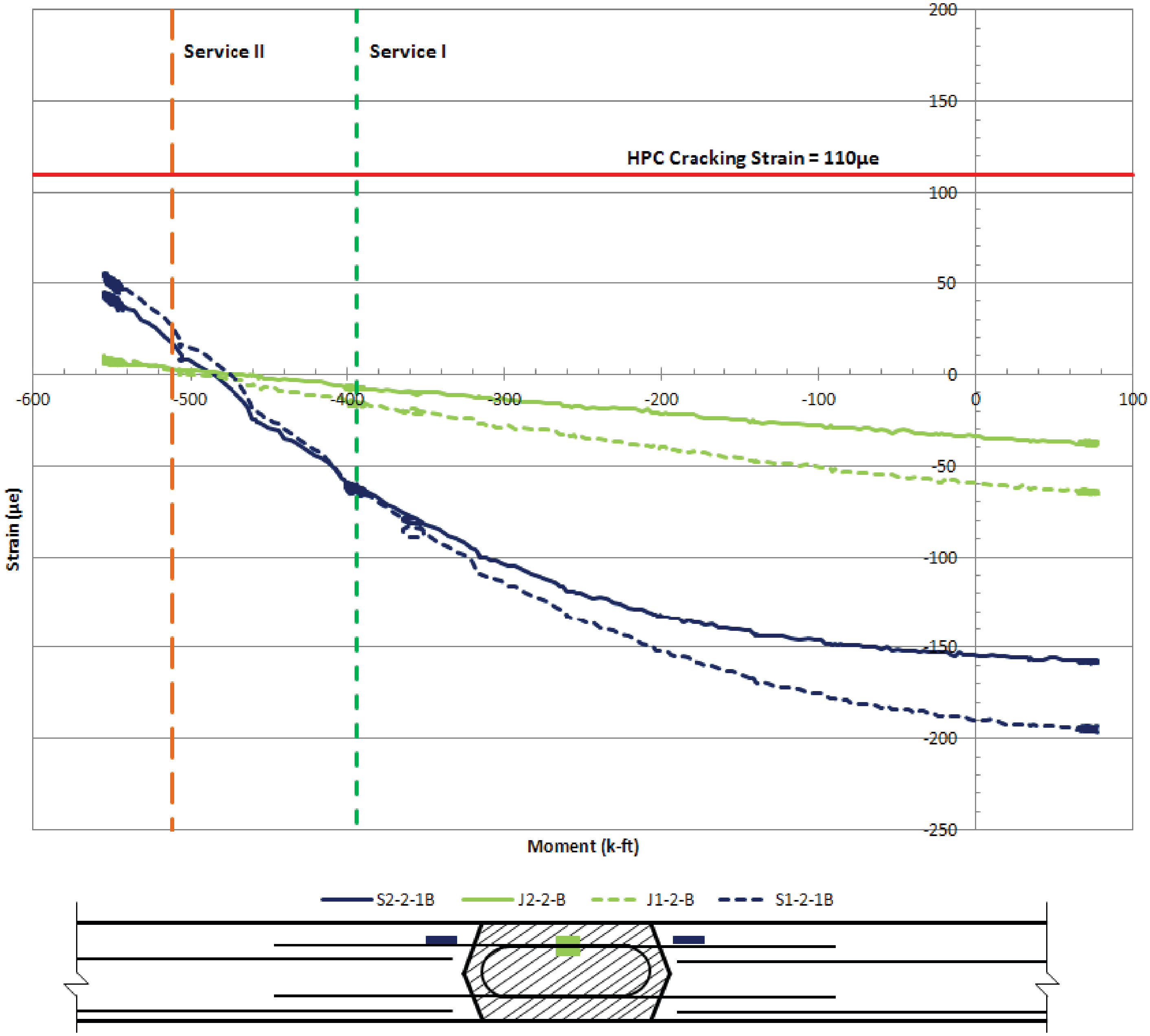

Figure C.79. Row 2, bottom-of-deck embedded strain gauges (70-k retrofit).

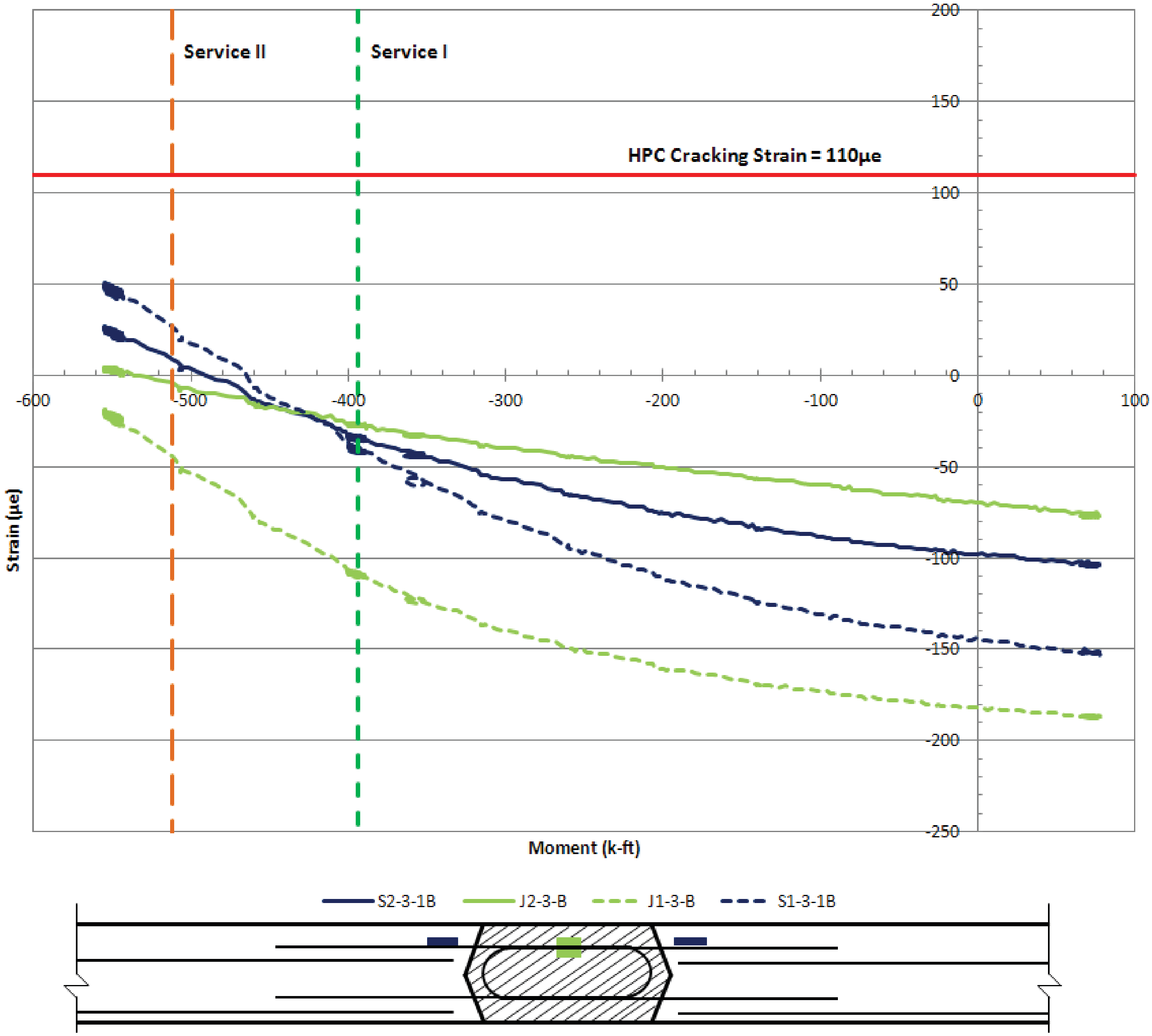

Figure C.80. Row 3, bottom-of-deck embedded strain gauges (70-k retrofit).

cracking strain before Service Level I moment. The 70 kips of posttensioning force per rod, however, did lower strains below the HPC cracking strain until Service Level II at each instrumentation row.

Ultimate Capacity Test Results

On completion of static testing for the modified detail, the posttensioning rods were removed and the transverse module-to-module connection detail was tested to ultimate moment capacity. Figure C.81 shows the moment-displacement curve for the specimen during testing. Strain data for the embedded gauges along reinforcement Rows 1, 2, and 3 (Figure C.82 through Figure C.87) were analyzed in combination with qualitative observations to determine the failure mechanism for the transverse module-to-module connection detail.

Embedded strain gauge data immediately adjacent to the joint interface in the top-of-deck reinforcement entered the inelastic range, suggesting yielding (2,000με) at approximately 1,500 kip-ft to 1,600 kip-ft of applied moment (see Figure C.82 through Figure C.84). All gauges embedded on the top-of-deck reinforcement at those locations behaved in

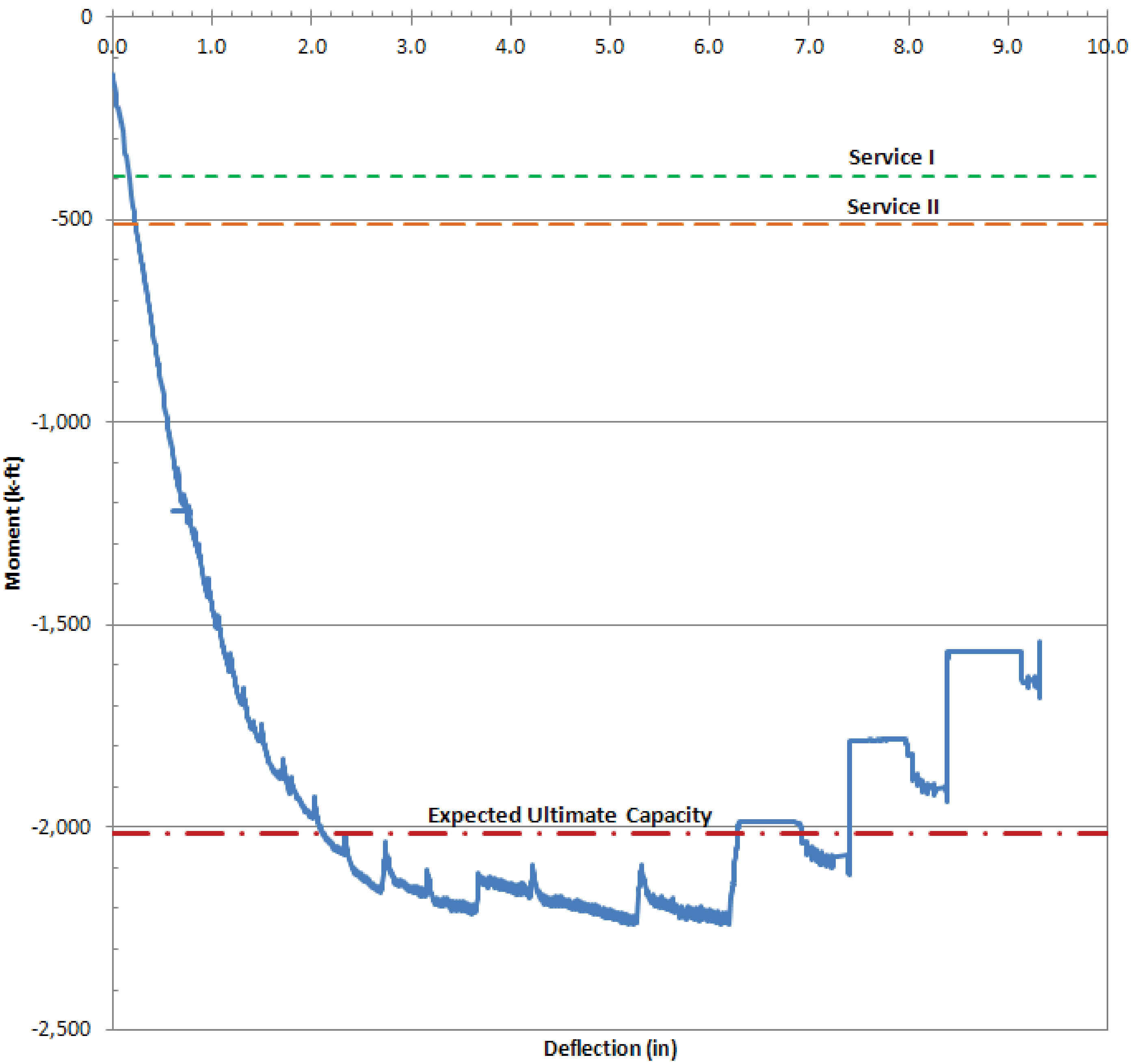

Figure C.81. Ultimate capacity moment versus deflection.

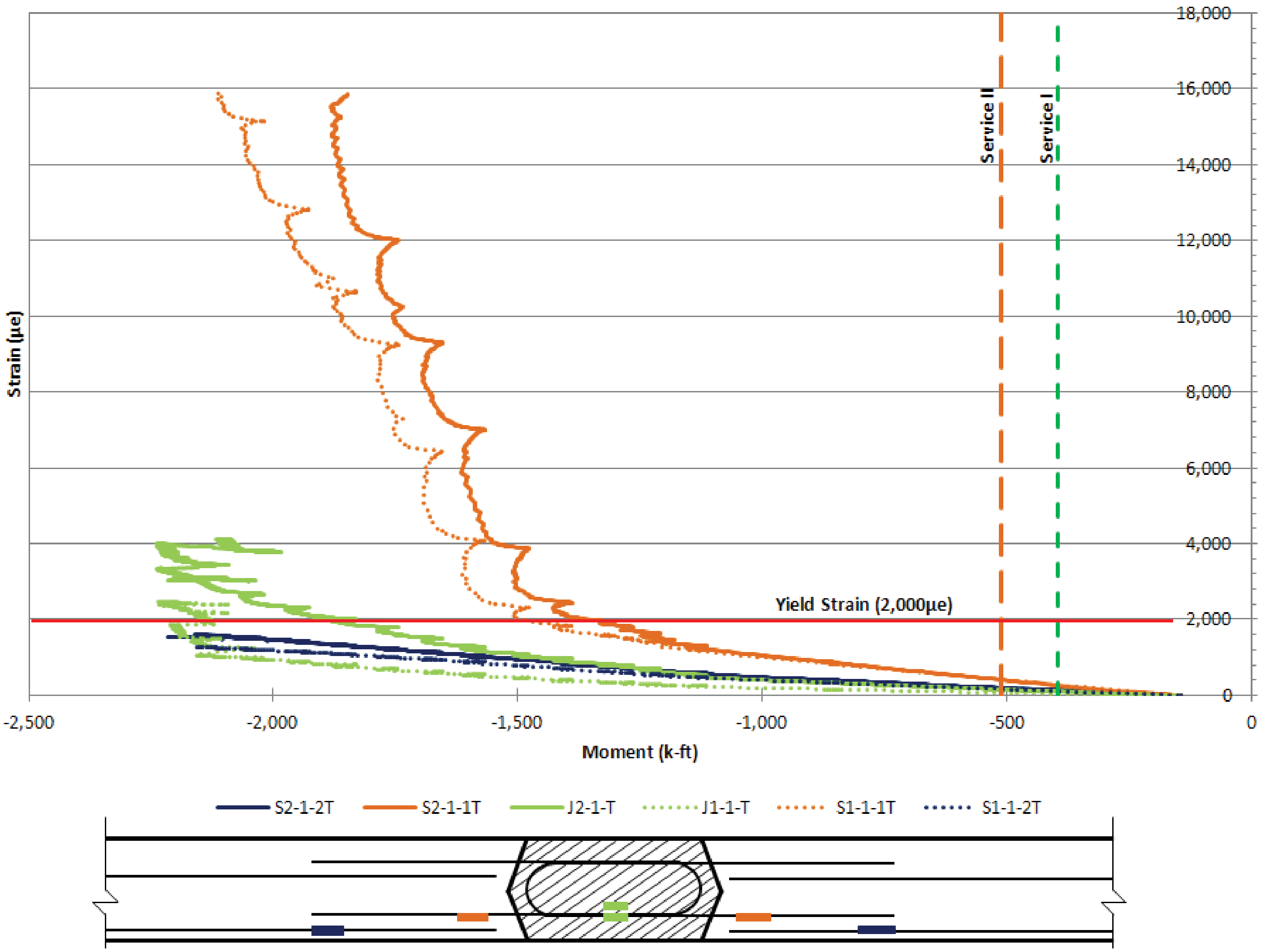

Figure C.82. Row 1, top-of-deck embedded strain gauges (ultimate).

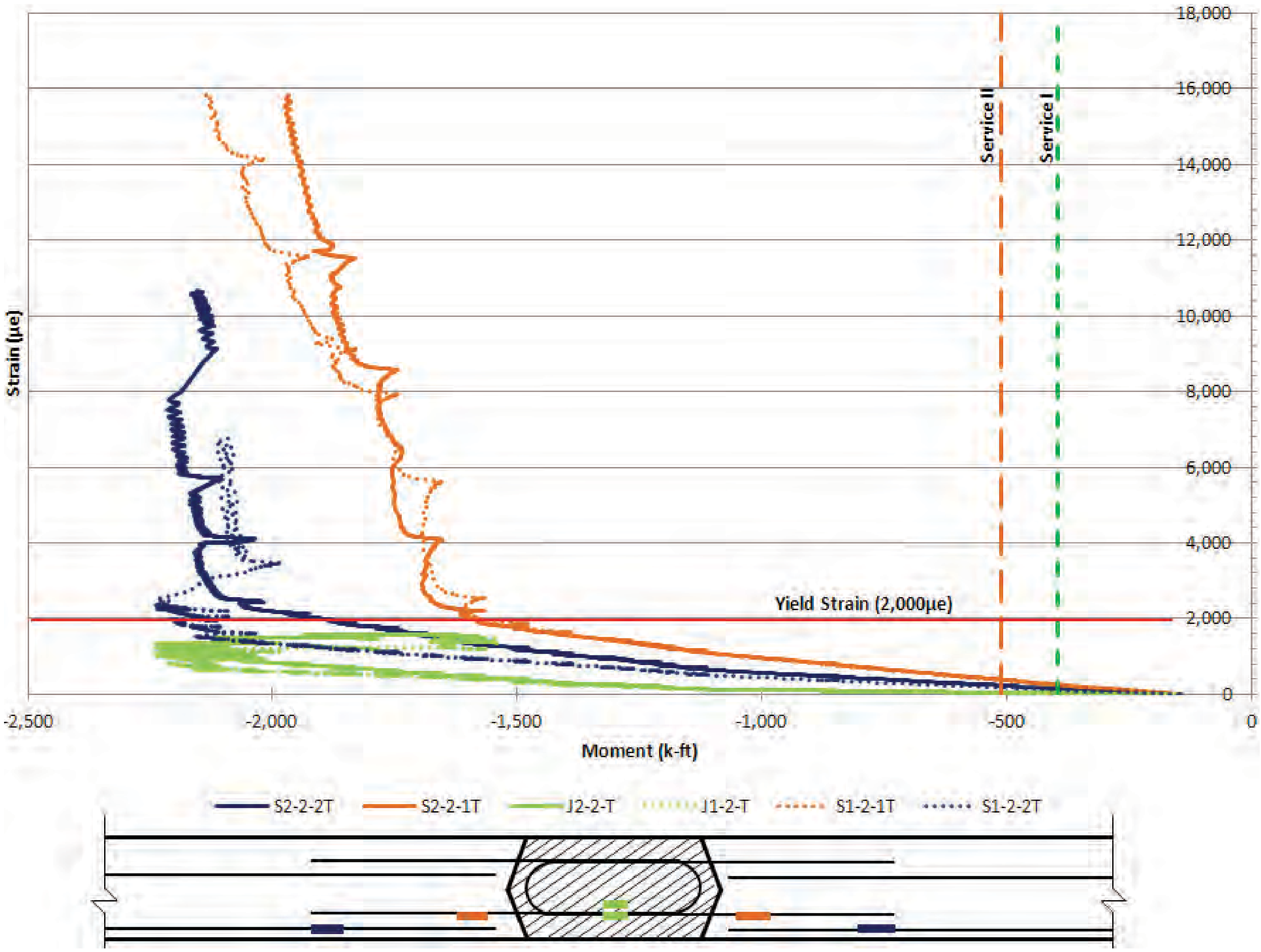

Figure C.83. Row 2, top-of-deck embedded strain gauges (ultimate).

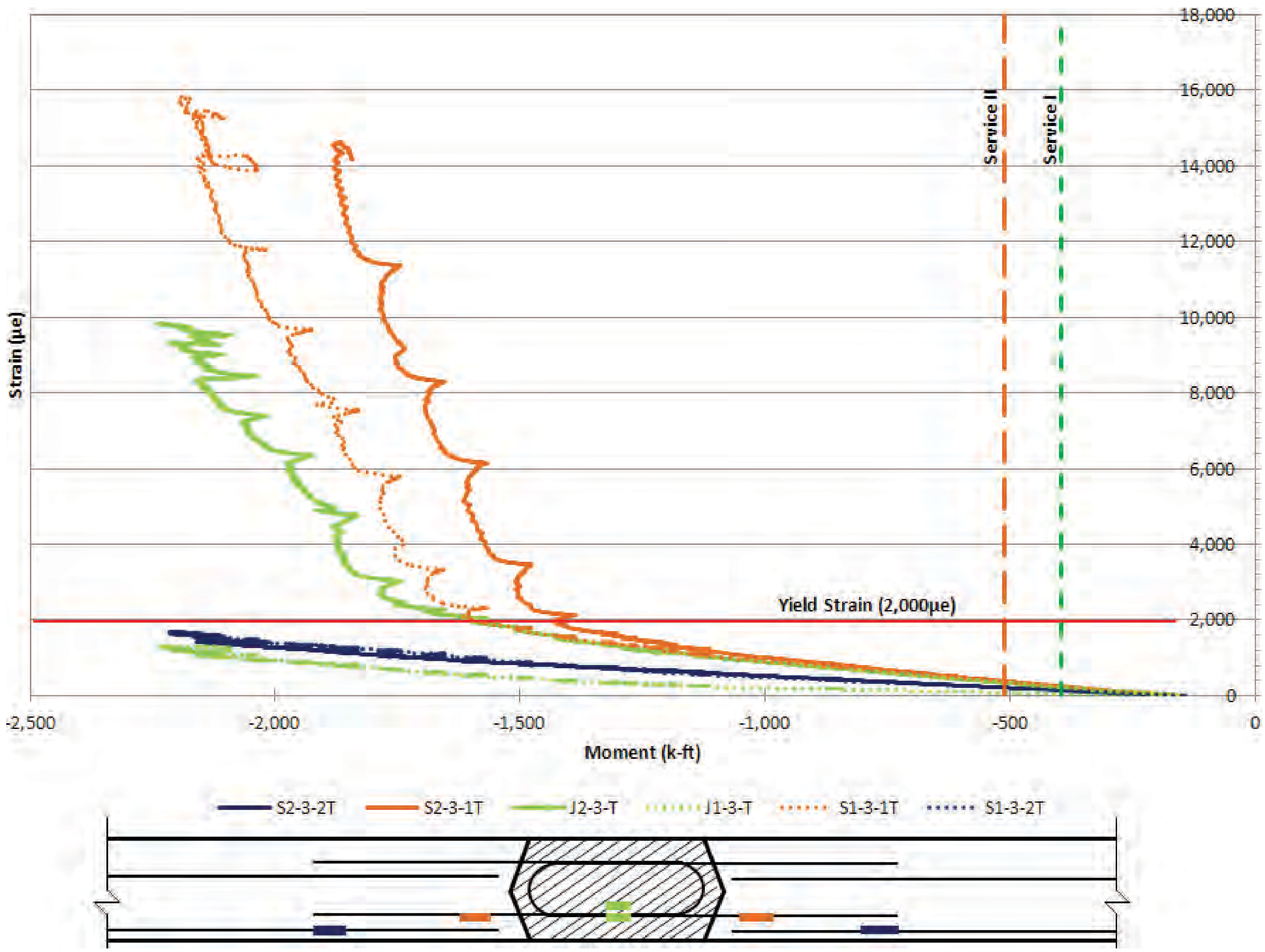

Figure C.84. Row 3, top-of-deck embedded strain gauges (ultimate).

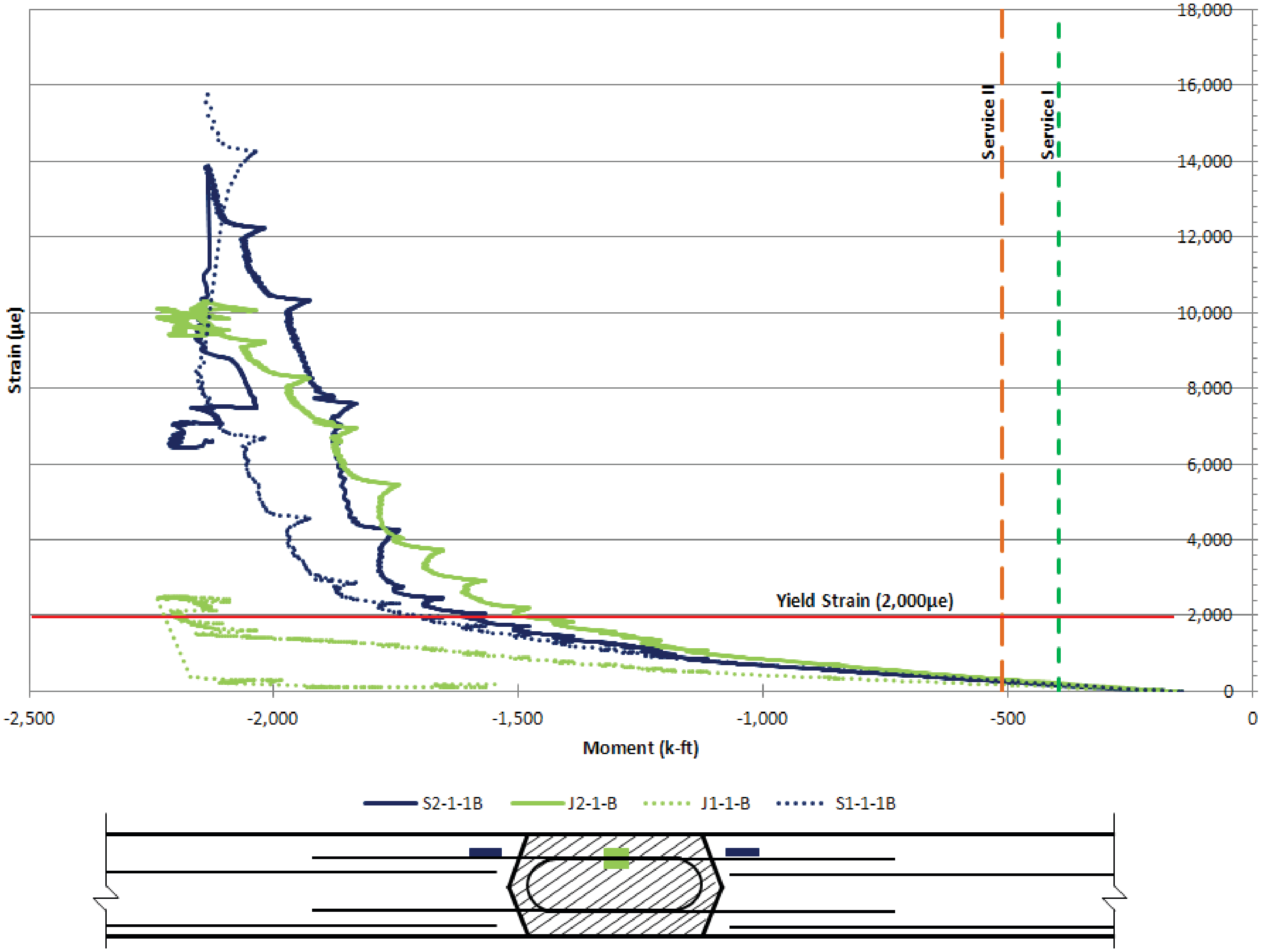

Figure C.85. Row 1, bottom-of-deck embedded strain gauges (ultimate).

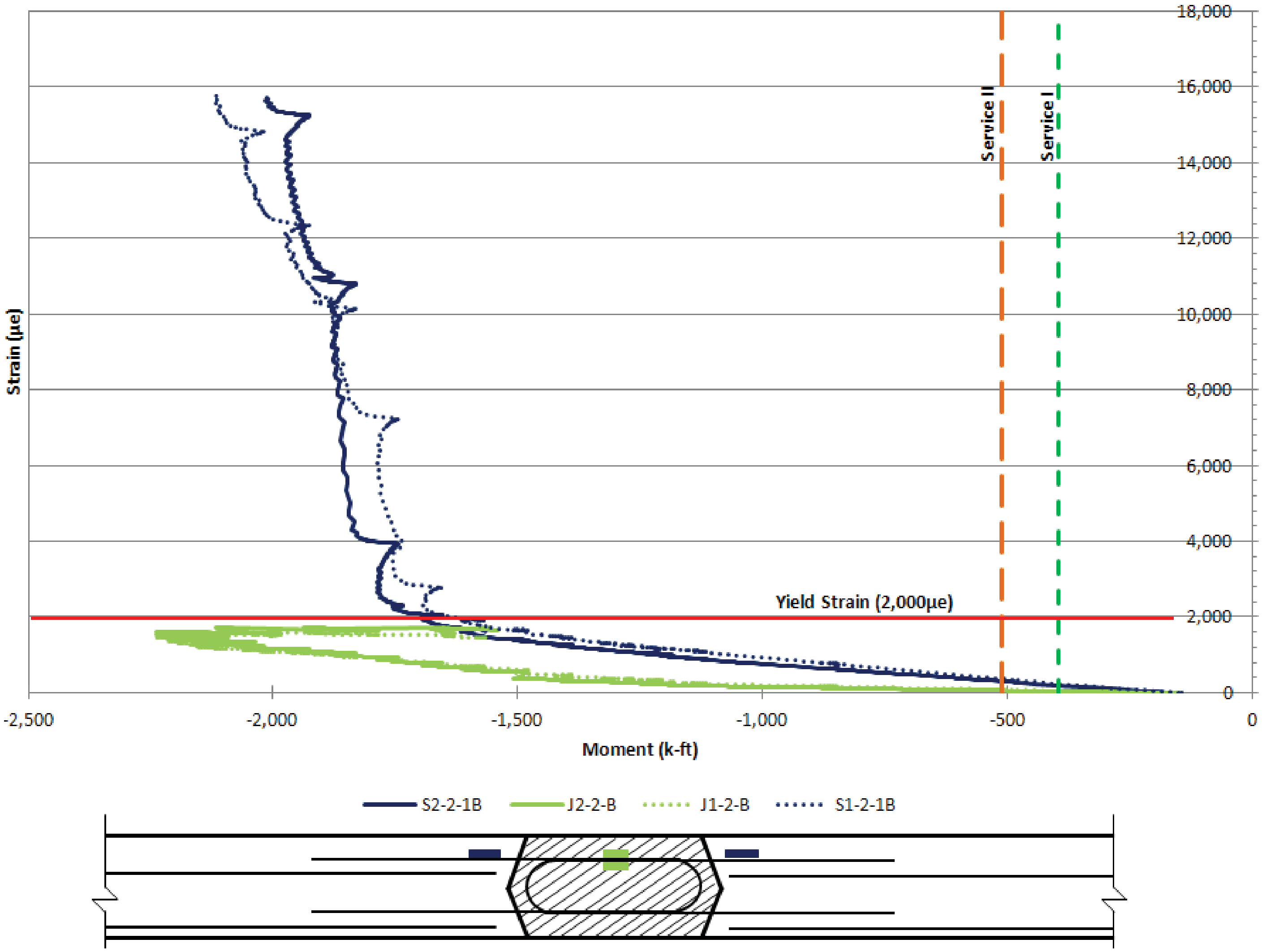

Figure C.86. Row 2, bottom-of-deck embedded strain gauges (ultimate).

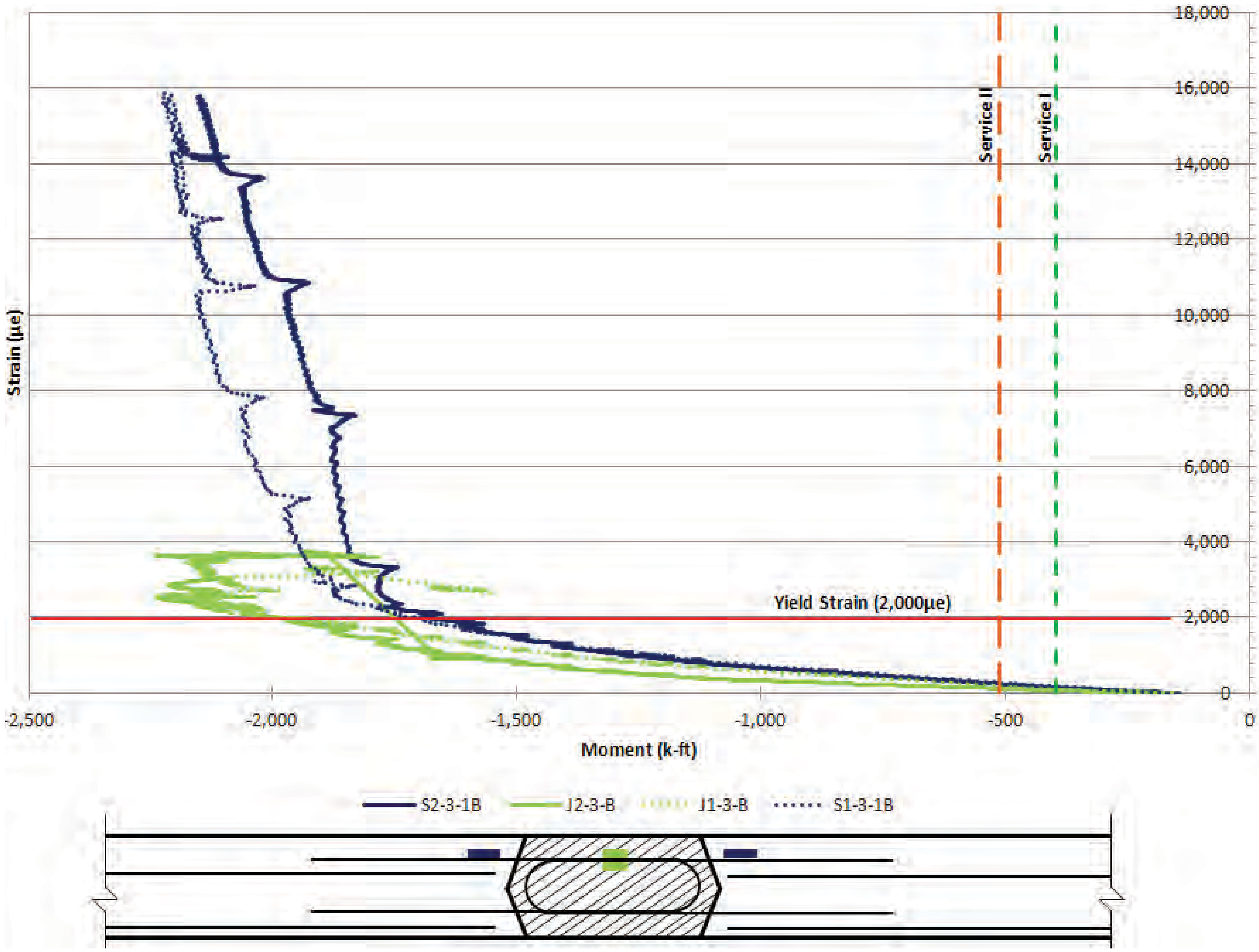

Figure C.87. Row 3, bottom-of-deck embedded strain gauges (ultimate).

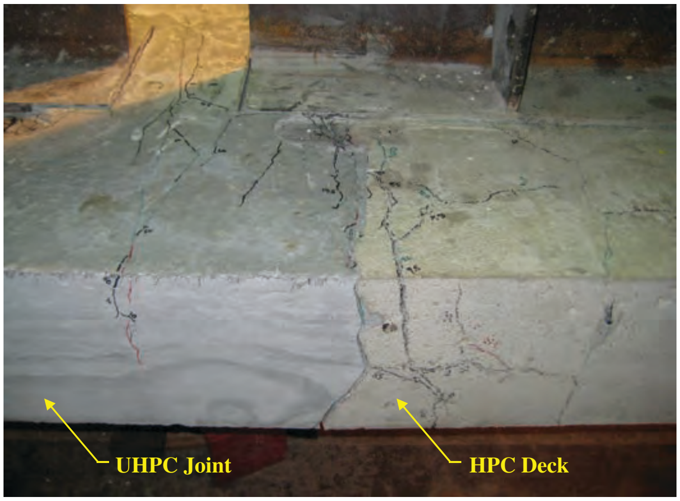

Figure C.88. Interface opening and crack propagation.

this manner. This result corresponded to the specimen entering into inelastic deformation around the same applied moment in Figure C.81. Similarly, bottom-of-deck embedded strain gauge data immediately adjacent to the joint interface indicated yielding at approximately 1,600 kip-ft to 1,800 kip-ft of applied moment (see Figure C.85 through Figure C.87).

With increased loading, the opening at the interface between the HPC deck and the UHPC joint widened. Cracks from service-level testing propagated and widened throughout the precast deck (Figure C.88). At 1,660 kip-ft, two large cracks (one in each deck module) spanning the entire width of the specimen became apparent approximately 1.5 in. from the joint interface on the bottom-of-deck surface. As the specimen was pushed well beyond service-level moments, reinforcement in the HPC deck near the UHPC interface began to yield. Eventually, the moment-displacement curve entered into the nonlinear region, and correspondingly, strains in reinforcement near the joint began to deform plastically (see Figure C.82 through Figure C.87).

Figure C.89. Girder–deck interface.

Throughout ultimate moment capacity testing, the W30X99 girders appeared to be slowly pulling away from the joint. All of the cracking in the UHPC joint and HPC deck could be seen accumulated locally where the girder appeared to pull away (Figure C.89).

The two large cracks parallel to the joint interface continued to widen, and eventually the UHPC suffered tensile rupture near the shear studs located in the joint (Figure C.90). See Figure C.91 for even more fractures. Cracking in the precast deck exposed the outermost reinforcement hairpins that entered into the joint allowing for pullout (Figure C.92). Load application continued, and the specimen reached a peak moment of 2,239 kip-ft before successive fractures of multiple

Figure C.90. UHPC rupture (top- and bottom-of-deck).

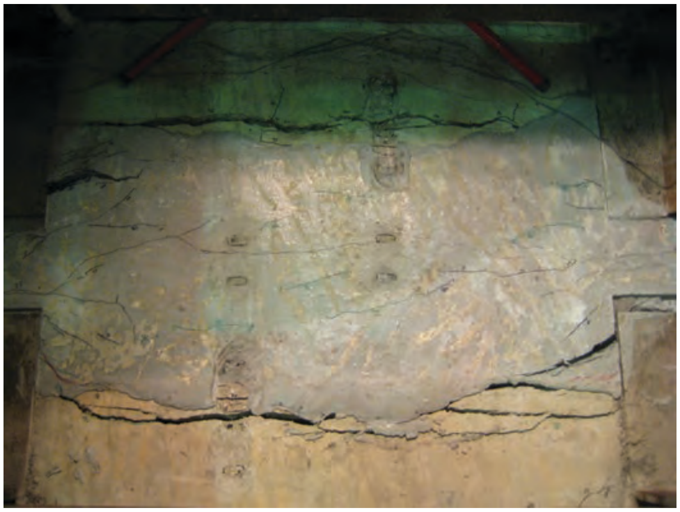

Figure C.91. Bottom-of-deck at failure.

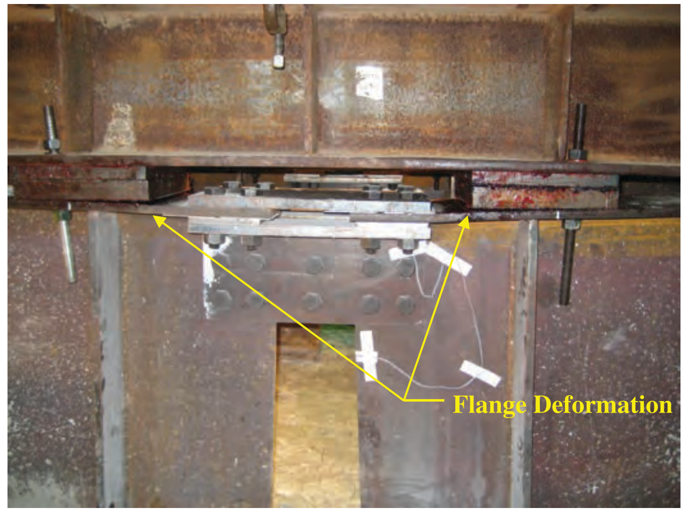

Figure C.93. Bottom flange deformation.

hairpin reinforcement bars acted as the ultimate mode of failure for the transverse connection.

Deformation in the bottom flange of the W30X99 girders was identified as the test progressed and the specimen underwent large deflections (Figure C.93). In addition, during the ultimate moment capacity testing, the slip critical bolted connections went into bearing and caused local deformation in the flange holes. Upon failure, the total deformation of the specimen at the centerline of the UHPC joint was 9.3 in. (Figure C.94).

Conclusions

This section presents conclusions for the abrasion, constructability, and strength and serviceability tests. The different test conclusions summarize the important issues and recommendations from the qualitative and quantitative testing data.

UHPC Abrasion

The objectives for the abrasion testing were to determine the early age grindability of the UHPC material to help the contractor with accelerated scheduling on the demonstration project. Several conclusions and recommendations for the joint material were made.

- Assuming a 70°F curing temperature, the UHPC will reach the 10,000-psi compressive strength required for grinding at approximately 2 days.
- Assuming a 70°F curing temperature, the material will reach the 14,000-psi compressive strength threshold required to open the bridge to traffic after 4 days. Thus, the contractor will have roughly 2 days to grind the UHPC joints for the bridge deck surface before the bridge reopens.

Figure C.92. Exterior hairpin reinforcement (opposite sides).

Figure C.94. Overall specimen deflection.

- Specimens with a formed surface finish exhibited less abrasion resistance than specimens with cut surfaces because the steel fibers in the UHPC lay parallel with the surface and tended to pull off easily. Fiber alignment was attributed to material flow on the bottom surface of the mold.
- Specimens with a rough surface finish generally included small entrapped air bubbles, which allowed for easy removal of the UHPC material.
- If the demonstration bridge's field cast joints have a formed surface finish because of a plywood top form, the abrasion resistance in the field is expected to most nearly resemble that of the abrasion resistance results of formed surface specimens.
- If the field-cast joints have an unfinished top surface, the abrasion resistance in the field is expected to most nearly resemble that of the abrasion resistance results of rough surface specimens.
- For the formed surface finish, abrasion resistance of the UHPC at 10,000-psi compressive strength will likely be about 40% lower than the abrasion resistance roughly 2 days later when the UHPC reaches 14,000-psi compressive strength.
- For the rougher, unfinished surface, abrasion resistance of the UHPC at 10,000-psi compressive strength will likely be about 27% lower than the abrasion resistance when the UHPC reaches 14,000-psi compressive strength roughly 2 days later.

Joint Constructability

The completed construction and casting of the intersecting deck joint mock-up specimen helped formulate a proposed UHPC placement plan.

- Ambient temperatures at the time of batching are very important to the flow characteristics of the UHPC.
- At an ambient temperature of 65°F, the temperature of the UHPC on discharge from the mixer ranged from 82°F to 85°F. Within that range, adequate flow characteristics to achieve good consolidation and flow around corners were observed.
- At an ambient temperature of 75.5°F, the temperature of freshly mixed UHPC reached 100°F and the flow characteristics were inadequate for placement and consolidation.
- Consequent replacement of water with ice by mass in the batch reduced the temperature of freshly mixed UHPC to 60°F and once again allowed for acceptable flow characteristics of the UHPC.
- At satisfactory discharge temperatures, the acceptable flow characteristics created no significant voids around steel reinforcing bars at the intersection of longitudinal and transverse deck joints.
- The UHPC should be placed from areas of lowest to highest elevation while applying top forms as the deck joints are filled. A small chimney should be constructed at the highest elevation to provide hydrostatic head in the UHPC and aid material consolidation.
- Full-depth stay-in-place acrylic bulkheads create a possible infiltration plane for water and chemical access to the embedded steel joint reinforcement and should be avoided if possible.
- To maintain controlled sequential placement of the UHPC and avoid infiltration planes, a partial-height removable acrylic bulkhead should be used in the longitudinal joint at locations where the UHPC material will likely be in compression.

Transverse Joint Strength and Serviceability

Testing of the transverse module-to-module connection over the pier identified the likely cracking moment and determined the ultimate capacity of the section. Many results and recommendations were made from the testing regimen regarding serviceability of the deck over the connection.

Service-Level Static Testing

- During both static and fatigue testing, surface-mounted strain gauges spanning the interface between the prefabricated deck modules and the UHPC joint indicated early debonding and significant opening at the interface.
- Visual observation of the interface at and below service-level load conditions confirmed the early debonding and opening of the HPC-UHPC interface.
- In addition to debonding at the joint interface, embedded strain gauges near the interface registered strains above the HPC cracking strain level (110με) at approximately half of Service Level I moment conditions, suggesting cracking is likely to occur in the prefabricated deck modules.

Service-Level Fatigue Testing

- Visual inspection at the onset of the 1,000,000-cycle service-level fatigue testing confirmed cracking in the precast HPC deck near the joint interface.
- Strain accrual during fatigue testing suggested propagation of existing cracks in the specimen. Visual inspection throughout the fatigue testing confirmed propagation of existing cracks and formation of new full-depth cracks in the prefabricated deck modules within 10 ft of the joint.

Connection Retrofit Testing

- To mitigate the serious durability concerns at the transverse module-to-module connection over the pier, a modified detail—which would not compromise the accelerated construction aspect of the project—was devised and implemented. The modified detail posttensioned the deck in this region to minimize tensile stresses in the concrete through service-level conditions.
- Static service level testing when 60 kips of posttensioning force was applied in each of the four rods for the modified module connection detail reduced the tensile strain across the interface, but not sufficiently to reduce strains below HPC cracking levels before Service Level I conditions.
- Static service-level testing indicated that the application of 70 kips of posttensioning force negated the tensile strain across the interface entirely until after Service Level I conditions were reached.
- Strains measured with surface-mounted strain gauges did not exceed the HPC cracking strain until after Service Level I conditions were reached. Strains measured with embedded strain gauges throughout the specimen did not exceed the HPC cracking strain until after Service Level II conditions were reached with 70 kips per rod of posttensioning force.
- The 70-kip posttensioning force per rod was recommended for application in the demonstration bridge to reduce the likelihood of deck cracking over the piers and increase deck durability at the transverse joint interface.

Ultimate Capacity Testing

- The overall specimen moment versus deflection plot indicated inelastic deformation of the specimen around 1,500 kip-ft to 1,600 kip-ft of applied moment.
- Top-of-deck reinforcement began yielding at approximately 1,500 kip-ft to 1,600 kip-ft of applied moment, corresponding to the inelastic deformation of the entire specimen.
- Bottom-of-deck reinforcement suggested yielding between approximately 1,600 kip-ft and 1,800 kip-ft of applied moment.
- The W30X99 girders slowly pulled away from the joint. UHPC tensile rupture occurred near the shear studs located in the joint and connected with two large cracks in the HPC deck parallel to the joint interface that had formed and widened as load increased.
- Spalling at the edges in the precast deck exposed the exterior module hairpins and allowed for rebar pullout. Successive fracture of multiple hairpin reinforcement bars entering the transverse joint was the ultimate mode of failure for the connection.
- The actual ultimate moment capacity of the transverse module-to-module connection (2,239 kip-ft) was determined to be approximately 10% greater than the expected ultimate moment capacity (2,016 kip-ft).

Through the project's full-testing regimen, the UHPC deck joints were evaluated for their use in the ABC demonstration bridge. Provided that the UHPC mix design's sensitivity to ambient temperature effects were accounted for, the UHPC provided excellent flow characteristics, workability, and consolidation during placement of the intersecting deck joints. In addition, the accelerated rate of compressive strength gain and higher cracking strain level of the UHPC made it well suited for its application in this ABC project.

While the UHPC displayed several superior material characteristics with respect to the durability and strength of the deck joints themselves, the direct tensile bond strength between the UHPC and the HPC deck on display during the strength and serviceability testing was a concern. Testing revealed that the interface between the transverse UHPC joint and HPC deck underwent early debonding and significant opening well below service-level moment conditions. This raised concerns as to the durability of the module-to-module transverse joint connection for the demonstration bridge. Consequently, a posttensioning retrofit detail was developed and tested to eliminate opening at the interface and cracking in the HPC deck around the transverse joint over the pier. With an adequate posttensioning force per rod, the retrofit successfully limited strains levels to below the HPC cracking strain.

Because of the interfacial bond issues observed over the course of this testing, further investigation into the direct tensile bond strength between the UHPC and HPC is recommended. This testing would better evaluate the durability of the longitudinal and transverse UHPC deck joints present in the ABC demonstration bridge and help determine the long-term viability of this UHPC deck joint detail as a solution in future ABC projects.

APPENDIX D

Field Demonstration Project Construction

Introduction

Phase III of SHRP 2 Renewal Project R04 required the construction of a demonstration bridge using the most-promising bridge details identified in earlier research and the modular systems being incorporated into accelerated bridge construction (ABC) standards. The US-6 bridge, which crosses Keg Creek near Council Bluffs, Iowa, is similar in size and length to a large majority of bridges across the United States. As a demonstration, it was replaced with a bridge that incorporates proven ABC bridge construction details with the innovative use of ultra-high-performance concrete (UHPC) to shorten the normal bridge replacement period from 6 months to only 2 weeks of traffic disruption. The improvements consist of replacing the bridge located on US-6 over Keg Creek in Pottawattamie County, Iowa. The existing 180-ft by 28-ft continuous concrete girder bridge (with spans of 81 ft, 48 ft, and 81 ft) was constructed in 1953 and was classified as structurally deficient with a sufficiency rating of 33. The replacement structure is a three-span (67 ft, 3 in.; 70 ft, 0 in.; and 67 ft, 3 in.) 210-ft, 2-in. by 47-ft, 2-in. composite steel modular bridge with precast substructures and precast bridge approaches. The bridge replacement is intended to increase the structural capacity of the bridge, improve roadway conditions, and enhance safety by providing a wider roadway.

This application provided a unique opportunity to effectively promote ABC for rapid renewal of the bridge infrastructure and also demonstrate various ABC technologies being advanced in the R04 project. The steel modular option was chosen as the most cost-effective on the basis of early discussions with local contractors and fabricators. Although it will not be fully detailed on the design plans, the contractor was allowed to propose a precast concrete modular alternative under a value engineering (VE) option if it can be constructed within the same ABC schedule and at a lower cost—none was proposed. The bridge was originally designed in-house to be constructed with a 13-mi detour (average daily traffic [ADT] = 7,000) and an estimated construction duration of 6 months. HNTB Corporation, a design engineering firm, redesigned the bridge by using ABC techniques and standard designs developed for this project so that the replacement could be completed in a 2-week period. The ABC period of 2 weeks pertains only to the time that traffic was disrupted. The total duration for the project, including time for prefabrication, was about 7 months, but the traveling public was affected for only just over 2 weeks. A daylong workshop, including a site visit, provided an opportunity to promote the dissemination of information to bridge owners from around the country.

The demonstration bridge features precast concrete semi-integral abutments, precast columns and pier caps connected with high-strength grouted couplers, and an innovative modular superstructure constructed with prefabricated concrete decked, steel stringer units and field-cast UHPC joints. The enhanced durability provided by the elimination of all open deck joints is seen as a major advance in long-life ABC projects; and the assembly of precast units without the need for any posttensioned connections avoids the need for specialized contractors.

The project was the first in the United States to use ultra-high-performance concrete (UHPC) to provide a full, moment-resisting transverse joint at the piers. This detail allows the prefabricated superstructure elements to be erected as a simple span and, once the UHPC joints are constructed, perform as continuous joints. The project team is performing full-scale laboratory testing of the critical field-cast UHPC continuity joints to ensure their long-term reliability and ultimate load capacity. These UHPC joints provide simple construction, additional load-carrying capacity, and a durable joint that prevents moisture intrusion and long-term maintenance problems.

Figure D.1. Original Keg Creek bridge.

Figure D.2. New Keg Creek bridge.

Demonstration Project Innovative Features

This demonstration project implements a series of innovations. It incorporates details drawn from diverse locations and applies them in a single demonstration project that was visited by Federal Highway Administration and department of transportation personnel from numerous states. Project innovations include the following:

- Overall, a complete bridge system was designed and constructed by using superstructure and substructure systems comprising prefabricated elements. The bridge approach slab also consists of precast elements.
- The superstructure units incorporate precast suspended backwall elements to create a semi-integral abutment.
- Ultra-high-performance concrete was used in the joints between the modular superstructure units and between the approach slab panels. UHPC was used for longitudinal joints and transverse joints over the piers. This project was the first in the United States to use UHPC to provide a full, moment-resisting transverse joint at the piers. The elimination of open deck joints provides for a more durable, low-maintenance structure in the final condition.
- Self-consolidating concrete (SCC) was used to improve consolidation and increase the speed of construction for abutment piles (fill pockets) and abutment to wingwall connections. Abutments consist of prismatic, precast concrete elements that feature a series of open holes to accommodate driven steel H-piles.
- Fully contained flooded backfill was used at the abutments. This proven construction method, ideally suited for ABC, involves placement of a granular wedge behind the abutment backwall, which is flooded to achieve early consolidation and significantly reduce the potential for formation of voids beneath the approach pavement.
- A structural health monitoring system (HMS) plan was implemented to evaluate and document the innovative aspects of accelerated construction. The monitoring plan included health monitoring instrumentations to assess the integrity of the structure and deck panel system during and after construction.
- ABC entails prefabricating as many of the bridge components as feasible given site and transportation constraints. This project took the approach that, for ABC to be successful, ABC designs should provide maximum opportunities for the general contractor to do its own precasting at a staging area adjacent to the project site or in its yard with its own crews. The components were designed so that a local contractor could perform all or almost all the precasting work and outsource little to precasters. The winning bidder chose to do that by leasing a temporary casting yard next to the bridge site.
- The technologies incorporated into this bridge project have been successfully used in constructed projects drawn from around the United States. Several diverse structural systems were assembled and incorporated into a single project, reinforcing the concept that innovation does not necessarily mean creating something completely new, but rather facilitating incremental improvements in a number of specific bridge details to fully leverage previously successful work.

This demonstration project can affect the future practices of the industry and the state departments of transportation. New technologies that are implemented successfully on this project will accelerate the adoption of the innovations in the United States. This will be accomplished by creating awareness and education related to the innovative features and increasing confidence among government and other stakeholders in recommending their use on other projects.

Demonstration Project Construction

The US-6 bridge, a three-span continuous structure that crosses Keg Creek near Council Bluffs, Iowa, is similar in size and length to a large majority of bridges across the United States and was replaced as a demonstration bridge in Phase III. The Iowa DOT has significant experience in accelerated bridge construction, including projects on both the primary and secondary road system. A challenge in identifying this type of demonstration project is often presented when the project must be constructed in line with both the owner's program schedule and the research team's schedule to deliver a project. The research team identified a project that met both of these critical objectives.

The following sections will present the construction process for the bridge: prebid meeting; contractor bids; site preparation and prefabrication; construction process for bridge, including work before the ABC period; and the postconstruction review meeting.

Prebid Meeting

A prebid meeting was conducted on January 21, 2011, at the Iowa DOT central complex in Ames. A wide range of primarily Iowa-based contractors attended. All were interested in learning more about the project before submitting a construction bid. Representatives from Iowa DOT and the SHRP 2 R04 project design team made detailed presentations on the innovative design aspects of the bridge and answered questions from potential bidders.

A number of questions were raised about UHPC and the differences in how it is mixed and placed compared with conventional concrete. Another significant point of interest to the contractors was the potential need to adjust for differential camber between adjacent superstructure modules.

Significant attention was also paid to the three-phase project schedule that would be followed by the successful bidder. Essentially, the contract was separated into three phases: certain work could be performed before the existing bridge was closed, some tasks could be performed only during the specified 14-day ABC period, and the remaining tasks could be done after the bridge reopened. The Iowa DOT chose this three-phase schedule to keep the specific ABC research work separate from the non-ABC-related work items.

During the 14-day ABC period, the contractor would be subject to liquidated damages at a rate of $22,000 per day. That amount was calculated on the basis of the user costs for the site given the measured traffic volume and 13-mi detour length required. Although the amount is much higher than the nominal amounts often used by bridge owners, the Iowa DOT believed that it was a measureable indicator of the importance of meeting the 14-day schedule.

In summary, the work under this contract was organized as follows:

- Stage 1: Before bridge closure
  - Construct drilled shafts to ground level.
- Stage 2: ABC period (14 days)
  - Close bridge and demolish existing bridge.
  - Construct wingwalls on piles.
  - Assemble precast piers.
  - Assemble semi-integral abutments.
  - Assemble modular superstructure.
  - Assemble precast approach slabs.
  - Cast UHPC closure joints and grind deck.
  - Cast in place pavement, shoulder, and guardrail.
  - Reopen bridge to traffic—end of ABC period.
- Stage 3: After bridge opening
  - Make channel improvements.
  - Construct reinforced-concrete flume.

Contractor Bids

The construction letting for the project was held on February 15, 2011. Seven fully responsive bids were received on the Keg Creek bridge project, with a low bid of $2.66 million submitted by Godbersen-Smith Construction of Ida Grove, Iowa. A summary of submitted bids follows:

- $2,658,823.35, Godbersen-Smith Construction.
- $3,202,409.35, A. M. Cohron Son Inc.
- $3,245,342.21, Cramer and Associates, Inc.
- $3,495,701.97, Hawkins Construction.
- $3,614,301.52, United Contractors Inc. Subsidiaries.
- $3,925,936.43, Jensen Construction Company.
- $3,990,723.50, Kiewit Infrastructure Co.

The bids included several additional items, such as stream stabilization and drainage improvements that were not actually part of the ABC project itself. Although the low bid slightly exceeded the Iowa DOT budget and the engineer's estimate for the project, the owner agreed to proceed with the project knowing the critical importance of the project.

Following the bid opening, the Iowa DOT performed an analysis of the bids to better understand how the costs for this ABC project differ from those of a conventional bridge. The Iowa DOT's typical method for comparing bid prices excludes mobilization and bridge removal and results in an average cost of $175/ft^2 of bridge area. However, because of the specialized requirements for accelerated construction, these work tasks were bid at a much higher price than for a typical bridge. When mobilization ($\$25/\text{ft}^2$) and bridge removal ($\$20/\text{ft}^2$) are included in the summary, the average bridge price is approximately $\$220/\text{ft.}^2$

The higher bid prices for the ABC demonstration bridge can be greatly offset by a significant savings in user costs. In addition, the 2-week construction duration greatly reduced the period of time that drivers and construction field personnel were subjected to additional risks. Likewise, since this bridge was constructed on a closed road, rather than on an on-site detour, the risk of potential public–worker collisions was eliminated. The actual dollar cost savings for these types of risk reduction measures are difficult to quantify, but certainly provide additional justification for future ABC projects.

Site Preparation

Because of a number of other ongoing projects and unanticipated work generated by record flooding in the summer of 2011, the contractor did not begin substantial mobilization until July. The bulk of the mobilization work involved removal of a few small to moderate-size trees and grubbing the work areas.

The contractor acquired a short-term lease on approximately 4 acres of farmland immediately adjacent to the southeast corner of the bridge site. This land was used as an on-site fabrication and casting yard and was prepared with a number of 12-in.-thick timber crane mats, supported on a sand bedding to provide a uniform bearing surface and a level area to build forms and cast the bridge components. The close proximity of the casting yard was a tremendous benefit to the contractor's operations and is a distinct advantage for bridges in a rural area where space is available.

Figure D.3. Contractor mobilization and casting yard.

Figure D.4. Aerial view of bridge site.

Preclosure Fabrication

To prepare for the 14-day ABC period when the existing bridge would be demolished and replaced, the contractor performed a number of off-line operations that could be completed without affecting the traffic on US-6. Each of these operations will be briefly discussed in the following sections.

Structural steel fabrication for the bridge was performed by DeLongs of Jefferson City, Missouri. During the shop drawing phase of the project, the contractor elected to construct the steel rolled beams without camber to simplify the fabrication. Although the dead-load deflection due to the deck concrete would cause a visible sag in the bottom flange, given the rural location of the project site, this was not seen as objectionable. The contractor assembled all of the structural steel on timber falsework in the assembly yard. The falsework was constructed to simulate the exact geometry of the permanent piers, including the same cap beam cross slope and elevation differences between piers and abutments.

Figure D.5. Timber falsework bents and steel beams.

Figure D.6. Steel superstructure on falsework bents.

Structural steel assembly was performed using bolted splice plates to connect adjacent modules at pier location. Although the spans were designed with sufficient moment capacity to function as simple spans, the transverse deck joints were also designed with sufficient capacity to provide continuity between adjacent spans with the impermeable UHPC bonded to the precast concrete to eliminate the intrusion of water into those joints. The bolted splices provide the compression flange connection at each location, and the UHPC joints in the deck provide the tension connection. The welded connection of the splice plates at each pier was modified during construction to eliminate the need for fillet welds on both sides of the L-shaped connection plates. The double-sided filled weld was replaced with a partial-penetration weld that provides an equal capacity.

Drilled Shaft Construction

Drilled shaft construction was performed by Longfellow Drilling of Clearfield, Iowa. The drilled shafts were constructed just outside the footprint of the existing bridge, which allowed traffic to continue throughout.

During construction of the shaft on the northwest corner of the bridge, the drilling operator observed the remnants of a timber pile as it was brought up from the bottom of the shaft. An investigation could not determine whether this pile was part of the existing bridge foundation, part of a previous bridge, or even a falsework pile. As a precaution, the Iowa DOT closed the westbound lane of the bridge for a few days while the drilled shaft concrete was placed to avoid any potential vibrations to the early-age shaft concrete.

Reinforcing cages for the drilled shafts were fully tied in the assembly area and moved down to the pier locations for insertion and concrete placement. To accommodate the grouted coupler connections to the precast column sections, a set of #14 reinforcing bar dowels were embedded in the top of the

Figure D.7. Drilled shaft construction adjacent to existing bridge.

Figure D.8. Installation of dowel bars for drilled shaft.

drilled shaft concrete before initial set. It was absolutely critical that these dowels be accurately set to match the columns, so the contractor constructed laser-cut templates for both halves of the connection. The dowels were placed in the correct orientation and elevation and secured in place until the shaft concrete was cured.

Deck Construction and Concrete Casting

The contractor elected to cast the entire deck as one continuous concrete placement rather than a series of individual

Figure D.9. Deck concrete placement using finish machine.

superstructure modules, as might be considered in a typical precast concrete plant. To create space for the joints between each module and permit separation of the pieces and future reassembly, the contractor constructed a grid of plywood blockouts that would allow the installation of the overlapping hairpin reinforcing but would also be easy to disassemble after the concrete was cured.

Before placement of the bridge deck concrete, researchers from Iowa State University installed a series of strain gauges, attached to the reinforcing steel at several locations near the west pier. The gauges were used not only during load testing before the bridge was opened but also as part of a health monitoring system that will be used to document in-service bridge performance (this instrumentation is not part of SHRP 2 Project R04).

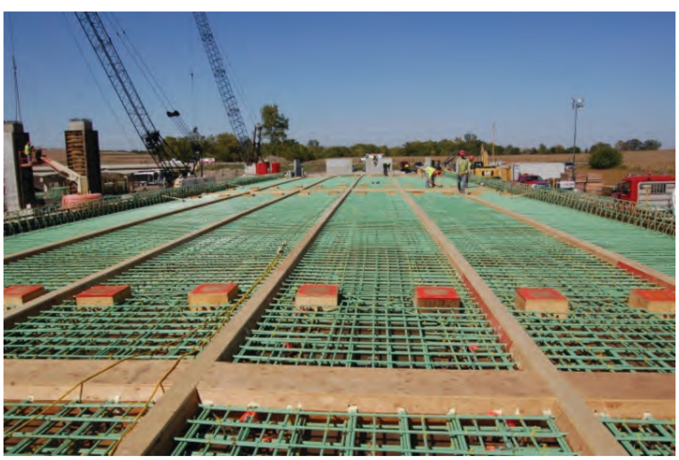

Figure D.10. Deck reinforcement and plywood blockouts.

The bridge deck concrete was placed very much like any other bridge deck by using a full-width Gomaco finishing machine. Concrete placement was started at one end of the deck and proceeded continuously to the opposite end of the deck without any significant incidents or delays. Although this construction method required quite a bit of handwork to finish the concrete around each of the plywood blockouts, it allowed the contractor to construct a new type of modular bridge and to do it with tools and equipment with which the crew was very familiar. Curing of the bridge deck was performed per Iowa DOT standards with a 7-day wet cure that used burlap, soaker hoses, and thermal blankets.

Substructure Components

Pier columns were constructed vertically on a forming bed consisting of 12-in.-thick timber crane mats. The forms were guyed and braced before concrete placement and remained so until the columns were moved to their permanent location. Lifting and guying of pier columns was performed by using strands embedded in the top center of each column and threaded inserts in the top of each face. To accurately place the grouted reinforcing couplers before pouring concrete, the contractor built laser-cut templates to match similar templates in the drilled shafts and cap beams.

Abutment and wingwall components were also cast on a forming bed consisting of 12-in.-thick timber crane mats. To ensure fit-up of the abutment and superstructure pieces

Figure D.11. Pier columns formed in casting yard.

Figure D.12. Pier cap beam casting adjacent to existing bridge.

Figure D.13. Posttensioned retrofit for modules.

Figure D.14. Epoxy adhesive being applied to joint faces.

during the ABC period, the precasting work had to be accurately measured and verified before placing concrete.

Pier cap beams were formed and placed on temporary casting beds located beneath and immediately adjacent to the existing bridge on a temporary creek crossing. Given the heavy weight of the caps, approximately 168,000 lb each, the location was selected to reduce the distance that the cap beams would be moved after curing. The contractor elected to use cast anchor bolts for bearing devices into the concrete rather than drilling/grouting or using anchor bolt wells.

Posttensioning Retrofit Design and Testing

Laboratory testing of the full-scale bridge superstructure modules was performed at Iowa State University before construction commenced. The results of those lab tests are presented in Appendix C of this report. Those test results showed that the bond between the UHPC joints and the precast deck concrete was not adequate to prevent the debonding of the two materials under tension loads. The modules were designed to allow them to perform as a simple span without the need for a supplemental connection between spans. However, to prevent intrusion of moisture into the deck, it was greatly preferable that the joints remain closed during service loads.

The R04 team developed a simple, posttensioned retrofit that could be installed after the modules were installed and the UHPC was cast. This retrofit included simple brackets mounted to the top of the beam webs, a 1-in.-diameter 150-ksi threaded rod, and anchorage hardware.

Initially, the posttensioning force at each rod was specified at 60 kips per rod. This force level was selected because, according to the manufacturer, a contractor could apply that level of force simply by applying the required torque on the anchor nuts with a torque wrench and multiplier. Following subsequent load testing at Iowa State University, the level of posttensioning was increased to 70 kips per rod to ensure that the deck joints remained fully compressed during service loads.

In addition to the posttensioning retrofit, the team decided to provide an epoxy bond between the UHPC and precast deck concrete. Although this retrofit could not be tested in the laboratory because of time constraints, the additional bond between UHPC and precast concrete was critical to the long-term success of the project. In the actual Keg Creek bridge, this bond was provided through the use of Rezi-Weld adhesive applied to faces of the transverse deck joints immediately before placement of UHPC.

Fourteen-Day ABC Period

As part of the normal project submittals, the contractor was required to submit a detailed schedule of operations to be conducted during the ABC period. The Iowa DOT and the SHRP 2 team carefully reviewed this schedule to ensure that the contractor would have sufficient equipment and manpower available to complete the work on time.

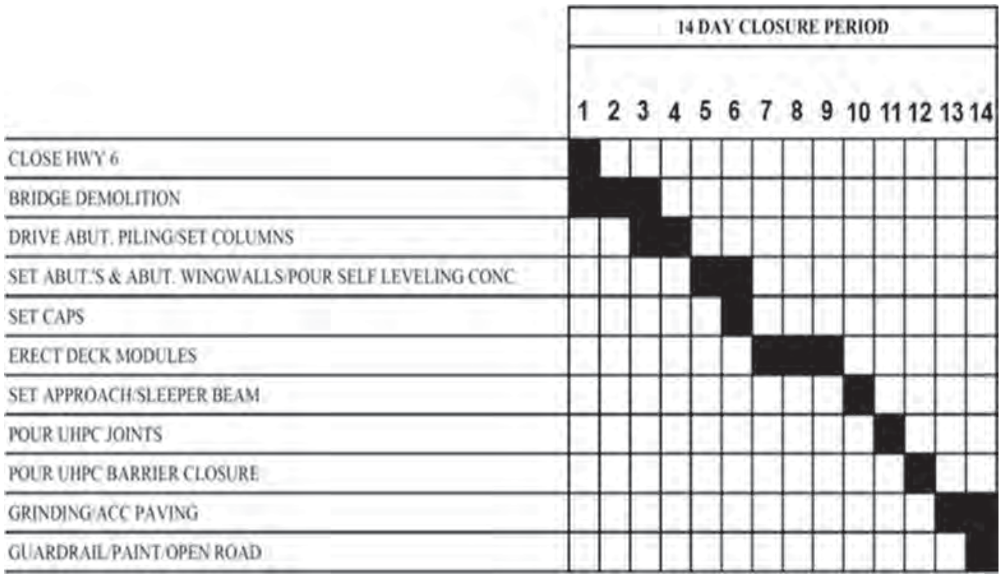

Figure D.15. CPM schedule during ABC period.

Contractor working hours during the ABC period were typically from 6:30 a.m. until 8:00 p.m. unless critical operations had to be completed to maintain the schedule. The contractor was careful to ensure that all operations requiring especially precise work, such as lifting and placement of large bridge components, were completed during daylight hours. These working hours, especially on a 7-days-per-week schedule, were challenging for the on-site workers. Because of the shorter daylight hours during the fall season, the productive working hours were somewhat less than what would be available during June or July. With longer working days, or even using a split shift for workers to staff the project nearly round-the-clock, the work required to replace a similar bridge could potentially be completed in a shorter closure period.

The contractor did not use any unusually large or specialized equipment for the demonstration project. At times, as many as seven cranes were working on the site. Most cranes were of moderate size, typically 110-ton capacity. During the erection of the large abutment and superstructure module components, a large 200-ton hydraulic crane was used. Two other types of equipment proved invaluable during the ABC period: hydraulic boom lifts and portable lighting units. The contractor commonly used as many as six boom lifts at any one time and often had up to 10 lighting generators available to permit safe working conditions during all hours of the day or night.

Demolition of Existing Bridge

The existing bridge was removed within a single day by using two hydraulic breakers mounted on tracked excavators and an American 7250 crane with wrecking ball. During demolition, the precast pier cap beams were protected from falling debris with additional timber crane mats. Concrete from the existing bridge was cleaned of reinforcing steel and crushed for use as channel protection material on site. The salvaged reinforcing steel was removed and recycled at an off-site facility.

Abutment Construction

Abutment construction consisted of a series of relatively simple and conventional operations:

- Excavate and create earthwork bench for piles.
- Drive steel H-piles to the appropriate bearing capacity at locations matching pocket voids.
- Cut off piles to the final elevation—2 ft above the top of bench elevation.
- Attach welded studs to the tops of the piles.

Figure D.16. Demolition of existing bridge deck.

- Transport two abutment barrels and four wingwall units from the casting yard to the bridge site.
- Lift and place precast abutment barrel and wingwall sections over the protruding steel H-piles.
- Place SCC in annular spaces around steel H-piles and between the joints connecting the abutment wing walls to the abutment footing.

Pile driving presented no particular problem for the contractor. To save time, abutment piles were simultaneously driven at both abutments. The contractor elected to provide an additional 10 ft of steel piling at each plan location to avoid potential delays during the ABC period (this stipulation may be provided in the contract to avoid delays). Ultimately, the piles reached the desired bearing capacity very near the anticipated elevation, but the minimal amount of waste was deemed “cheap insurance” against potential problems.

The contractor requested and was given approval to change from the steel shear studs welded to the upper section of the piles to a drilled-through, high-strength threaded rod at each location. This change was made to speed the construction and eliminate the need for an automatic stud welder on site.

Movement of the abutment components from the prefabrication yard to the bridge site required the use of a six-wheel-drive straight truck and a pair of moderate-size bulldozers to provide additional traction on the steep grade over the temporary creek crossing. This crossing was not designed by the project team and was installed by the contractor as part of the site preparation work.

Each abutment consists of a barrel section and two wingwall sections that are combined to form a U-shaped configuration. Installation of these components was slightly complicated by the need to place all three sections at the same time while aligning the corrugated metal pipe (CMP) pile pockets with the driven piles in each section. Overlapping hairpin reinforcing in the joints connecting these sections would not permit installation of one component at a time. This construction sequence did not present a major hurdle at the demonstration project site; however, the contractor observed that future projects in a more congested area might face a greater challenge if limited access was available to position cranes near the abutment.

Figure D.17. Abutment components moving to bridge site.

Figure D.18. Installation of abutment and wingwalls.

One significant problem occurred during the pile driving at the west abutment. Following installation of the abutment pieces, but before placement of the SCC in the pile pockets, the abutment was found to be approximately 26 in. east of the correct location. A survey error had led to the mistaken pile installation. After study and consultation with designer and owner, Godbersen-Smith decided to cut off and abandon the original set of piles and drive a new set in the correct location. This error and the resulting rework cost the contractor an estimated 2 days on the critical path schedule.

Placement of the SCC presented no particular problems for the contractor. To support the abutment sections at the correct elevation while the SCC gained strength, the contractor placed a series of 3-ft by 4-ft unreinforced concrete pads beneath the abutment barrel and wingwall sections. Before the contractor moved forward with the deck placement, the SCC compressive strength was verified by compression cylinder tests after being cured in accordance with the Iowa DOT SCC material specifications.

Pier Construction

Before the ABC period, the drilled shaft foundations were constructed adjacent to the existing bridge. After the existing bridge was demolished, pier construction consisted of the following operations:

- Cut off temporary casing at top of drilled shaft.
- Prepare top of shaft concrete.
- Move four column sections from casting yard to bridge site.

- Place grout bed at top of drilled shaft.
- Install precast column on top of drilled shaft, and install guy wires.
- Place grout in reinforcing steel couplers at bottom of column, and allow it to cure.
- Move two cap beam sections into position for lifting onto columns.
- Place grout bed at top of precast columns.
- Lift cap beam sections, and set atop precast columns.
- Place grout in reinforcing steel couplers at top of column, and allow to cure.

Rather than attempt to lift and carry the precast column sections from the casting yard to the bridge, the contractor constructed a temporary bench approximately halfway down the slope from the yard. A simple three-step process was used to move the columns to the bridge: a crane working from the upper yard lifted the column and placed it on the bench, the column was temporarily guyed, and a crane from the lower area lifted the column from the bench and moved it to the bridge site.

The tops of the drilled shafts were prepared by grinding away any roughness in the top surface of the shaft and using a hand-held band saw to cut the reinforcing dowels to the correct final length.

A bed, ½ in. to ¾ in. thick, consisting of nonshrink W. R. Meadows 588-10k grout was constructed on top of the drilled shaft in preparation for setting the precast column. The grout bed was designed to completely fill any irregularities between the mating components; any excess grout would be squeezed out when the column was placed.

Precast columns were placed by aligning the female end of the grout couplers with the dowel bars projecting from the drilled shaft. Once the column position was confirmed (by using survey) and verified (by using a cloth tape measure), a ½-in.-diameter guy wire was installed near the top of each

Figure D.20. Reinforcing couplers before concrete placement.

Figure D.21. Couplers in the bottom of precast column.

Figure D.19. Movement of precast columns.

Figure D.22. Placing precast column on drilled shaft.

Figure D.23. Injecting grout into reinforcing steel couplers.

face of the column and anchored to an immovable object to maintain column position.

The grouted couplers at the bottom of the column were injected with Dayton-Superior S-L grout in accordance with the manufacturer's specifications: Grout is injected with a hand pump through the port near the bottom of each coupler. Injection is continued until a steady stream of grout is observed oozing from the upper port of the same coupler. The ports are capped with a plastic plug, and the grout is allowed to cure approximately 18 hr until the required strength is obtained. Before the contractor moved forward with the superstructure module installation, the grout strength was verified by testing site-cured cube specimens in accordance with the manufacturer's recommendations.

The heaviest precast components in the bridge were the pier cap beams. These pieces weighed approximately 168,000 lb

Figure D.24. Pier column grout bed and dowels.

Figure D.25. Installation of pier cap beam.

and required two 110-ton cranes positioned on the temporary creek channel crossing to safely lift them into position. To help reduce the weight of these large pieces, two voided blockouts were created within the interior of the cap beam concrete by filling this volume with hollow plywood boxes. When the forms for the west pier cap beam were stripped, an unconsolidated area of concrete was observed near one of these plywood boxes. The owner expressed concern that this could indicate a potential shift in position by the plywood boxes. A series of ¾-in.-diameter test holes were drilled in the top and side faces of the cap beam to confirm that the boxes were in the correct position and no additional corrective action was required.

A grout bed similar to that used at the top of the drilled shaft was constructed on top of each column. In addition, a 4-in.-wide strip of compressible foam rubber was placed around the perimeter of each column before the grout bed was placed. This rubber served to confine the grout and allow only excess to squeeze out when the cap beam was placed.

The cap beams were lifted and placed with very little difficulty thanks to the accuracy of the templates used during the casting process. Following confirmation of the cap geometry, the couplers at the top of the column were grouted by using the same injection process as described earlier. This grout was again allowed to cure for approximately 18 hr before the first superstructure module was set.

Superstructure Module Assembly

Movement of the superstructure modules from the casting yard to the bridge site presented a number of challenges because of the large size and weight of the panels. In addition, the eccentric load caused by the integral barrier rail on the exterior modules, along with the 90° left turn and steep slope on the road to the bridge site further complicated the operation. The contractor was very careful to block and strap the modules to the truck bed to avoid any tipping and to position

Figure D.26. Movement of superstructure module.

the panel on the truck to maintain the center of gravity of the load between the wheel lines.

Two cranes performed the lifting of the modules. For the end spans, a 200-ton truck crane was positioned behind the abutment and a 110-ton crane was positioned on the channel crossing near the pier. For the center-span modules, two 110-ton cranes were positioned side by side near each pier, and the cranes "walked" forward to place the module in its final location.

The fit-up of modules at each abutment presented a bit of challenge. Although the pieces were originally fit together in the casting yard, after being moved to the final location, a few reinforcing bars needed adjustment when the modules were

Figure D.27. Overhanging backwall at semi-integral abutment.

Figure D.28. Assembly of modules creates "piano hinge."

set. The contractor used a handheld rebar bender to adjust bars as necessary. The final module at each abutment had to be lifted so that it hung perfectly plumb from the cranes to better fit into the opening between the wingwalls. In addition, the liberal use of liquid dish soap provided much needed lubrication to allow the final module to slide past the wingwall and into position.

The use of overlapping hairpin bars provides a very strong joint between modules but does complicate the construction somewhat because of tight tolerances for installation. Future applications of this bridge should evaluate other options (such as straight bars) that might simplify construction. The installation of adjacent modules with overlapping hairpin bars created what the contractor called a "piano hinge" for assembly. Following installation of the final module, straight reinforcing steel bars were slipped through the hairpins in each joint.

The reinforcing steel in the joints between adjacent modules was an ongoing point of discussion during the construction phase of the project. For this project, the contractor elected to cast the modules on site in one large operation and thus had the opportunity to make slight adjustments in the placement of reinforcing steel to avoid conflicts between modules. In future applications, where modules might be cast individually, project designers should provide accurate templates and evaluate potential details to reduce the number of reinforcing bars in these joints.

Mixing, Transportation, and Placement of UHPC

UHPC mixing was performed by a pair of ½-m^3-capacity electric mixers positioned at the east end of the bridge. The

Figure D.29. Adding steel fibers to UHPC mixing process.

Figure D.31. Placing UHPC in deck joints using Georgia buggy.

component materials for each batch were provided in bulk packages: an 1,800-lb "super sack" of powder mix and three boxes of steel fibers. In addition to the dry ingredients, water and the required admixtures were carefully weighed with a digital scale.

UHPC was transported to the bridge by using several mobile "Georgia buggies." The contractor fabricated plywood hoppers to minimize spillage and waste. UHPC was placed in the joints and allowed to flow around the reinforcing steel and completely fill the joint. The contractor used strips of ¾-in. plywood on each side of the joint to allow a slight overfilling. Once the concrete was cured, the excess concrete was ground flush by the diamond milling machine in the same operation as the profile and smoothness grinding. This grinding was performed within 48 hours of casting the UHPC to maintain the ABC schedule and to ensure that the UHPC had not gained full design strength at the time of grinding.

Because of the very flat, but still measurable, crest vertical curve on the bridge, UHPC placement was started at each end of the bridge and moved toward the center ("working uphill"). To control the flow of UHPC during placement, each longitudinal joint in the end span was filled simultaneously until the pier was reached. Just uphill from the transverse pier joints, a series of removable acrylic bulkheads were installed in the longitudinal joints to ensure that the entire end span and the transverse joint at the pier were completely filled before moving on to filling the longitudinal joints in the center span. Because of the relatively slow mixing and transportation process for the site-mixed UHPC, the placement of UHPC in the longitudinal joints was not completely efficient, and the contractor struggled at times to maintain a consistent level of UHPC in these joints. Future projects should consider filling only one or two longitudinal joints at a time and using bulkheads or other means to control the flow of UHPC until the joints were completely filled.

The ambient temperature at the time of UHPC placement varied from 40°F to 50°F. During the overnight hours following placement, the temperature dropped into the low 30s. To accelerate the curing process, the contractor used flexible ground heaters and insulating blankets to maintain the temperature in each joint at approximately 85°F to 90°F.

Iowa State University performed laboratory testing of UHPC specimens after casting. Results of that testing are presented in Appendix C. The test results were immediately

Figure D.30. Mixing UHPC with super sack ingredients.

Figure D.32. Pier joint detail with posttensioning.

communicated to the contractor so that grinding of the deck concrete could be performed within the ABC schedule.

Posttensioning of Superstructure Modules

Installation of posttensioning rods was performed while the UHPC was curing, and stressing was completed after the UHPC had reached a minimum 14,000-psi compressive strength as verified by Iowa State University with specimen testing. The 1-in.-diameter 150-ksi rods were inserted through each pair of brackets (a total of 48 locations) and stressed to a force of 70 kips. The stressing sequence followed was one in which all four rods within a given module were stressed simultaneously to minimize any potential for eccentric loads that would crack the still-curing UHPC.

Field-Welded Connections at Piers

Because of the survey errors noted previously, the bridge essentially was constructed with a very slight horizontal kink at each pier where the abutments are approximately 4 in. south of the correct alignment. The design plans showed the use of an anchor bolt well at each pier to provide some construction tolerance for setting of the swedged anchor bolts; however, the contractor elected to cast the anchor bolts into the pier cap beam concrete. Given the lack of tolerance combined with the kinked alignment, the contractor was unable to complete the bolted flange splices at several of the girder locations at each pier. Web splices were fully bolted and tensioned per the Iowa DOT standard specifications.

At locations where the bolted splice could not be completed, the contractor requested and received approval to substitute a fillet-welded connection at the piers. Although not the ideal solution, the field welding provided the necessary strength to ensure the compression component of the continuous girder spans at each pier as well as sealing each of the splice locations against future moisture intrusion.

Precast Approach Pavement Construction

Following installation of the final superstructure module, the next step was to construct the precast approach pavement at each end of the bridge. The precast approach pavement consisted of four doubly reinforced concrete panels, each 20 ft long and 10 ft, 7½ in. wide.

Before installing the panels themselves, a floodable backfill system was installed behind each abutment. That consisted of a geotextile fabric liner to separate the bridge embankment from the porous backfill material, a perforated subdrain running the length of the abutment and wrapping around each wingwall, and a porous backfill poured in layers approximately 1 ft thick. As each layer of porous backfill was placed, it was flooded with water and subjected to vibratory compaction. This method has been quite successful in eliminating approach settlement.

A precast reinforced-concrete sleeper slab was installed and leveled using a bed of fine sand. The sleeper was cast with an integral 2-in. crown, matching the bridge deck and the approach roadway. The precast approach panels were installed and leveled to fill the space between the abutment wingwalls. This task was made more difficult and time-consuming by the contractor's decision to place the approach pavement panel concrete on a prepared—but not completely smooth—subgrade in the casting yard. This caused the bottom surface of the approach panels to be somewhat rougher than anticipated.

Although the plans called for the joints between the approach pavement panels to be filled with UHPC, the contractor requested, and was granted, a substitution to use SCC instead. The change was requested because the supply of UHPC component materials on site was low after overruns

Figure D.33. Installation of precast approach.

during the installation of the superstructure. The approach pavement joints and the lifting pockets in the superstructure modules were filled with SCC in a straightforward operation. By the following day, the SCC mixture, which contained Type III portland cement for rapid strength gain, had allowed enough shrinkage in the lifting pockets to permit the intrusion of water around the perimeter. This seepage was later repaired with an epoxy injection.

Following completion of the approach panels, the contractor placed reinforced-concrete shoulder panels and an asphalt leveling course to precisely match the existing roadway pavement. In hindsight, the use of precast approach panels in this application may have been less than ideal. Given the need to construct a cast-in-place tie-in section anyway, a simpler solution might have been to place the entire approach as a cast-in-place area by using an accelerated curing process.

Deck Grinding for Profile and Smoothness

One of the final steps before opening the bridge to traffic was to grind the entire deck surface with a diamond grinder. The grinding had three purposes: to remove any irregularities on the deck surface especially at the UHPC joints, to correct any discrepancy in the longitudinal profile, and to provide increased skid resistance.

Although the UHPC material is a much higher strength than conventional concrete and contains an infinite number of high-strength steel fibers, it did not appear to present any significant challenge in the grinding process. Following the grinding operation, the smoothness of the deck was measured with a profilometer. Final results of that test are not yet available, but the bridge deck and approaches appear to provide a smooth, quiet ride, and the transitions from approach pavement to bridge are quite good as well.

Following the bridge grinding, in the UHPC concrete placed in the deck joints, in at least one area, the steel fibers were not uniformly mixed with the reactive powder components. The largest observed “fiber ball” was approximately 1.0 in. to 1.5 in. in diameter. Although this location might permit the intrusion of water beneath the surface of the UHPC joint, the low permeability of the material should prevent substantial seepage of water into the deck.

Figure D.34. Fiber ball on surface of UHPC joint.

Load Testing by Iowa State University

Before opening the bridge to traffic, researchers from Iowa State University performed a live-load test on the bridge for the Iowa DOT to document the as-built performance of the structure (this work is not part of SHRP 2 Project R04).

Highways for LIFE Workshop

To more widely disseminate information about the construction process and lessons learned from the Keg Creek bridge demonstration project, a national Highways for LIFE showcase was held in Council Bluffs, Iowa, on October 28, 2011. The showcase was held at the Hilton Garden Inn in Council Bluffs, a site that provided convenient access to the Omaha airport, the Interstate highway system, and the project site—which was approximately 15 miles away.

Nearly 80 people from 14 states attended the showcase. The participants represented state DOTs, FHWA, designers, and contractors—all of whom shared an interest in accelerated bridge construction.

Planning

Planning for the showcase began in July and included a series of conference calls and online meetings to develop an agenda, suggest and confirm speakers, and organize the logistics for a meeting location and food service. The planning committee included representatives from the following:

- National Academy of Sciences (SHRP 2);
- SHRP 2 R04 project team;
- Iowa DOT (Bridge Office and District 4);
- FHWA (Iowa, Washington, and Atlanta); and
- University of Florida.

The University of Florida, working under a support contract with FHWA, produced the invitations and managed the registration process.

The showcase schedule had to remain somewhat flexible until only 3 weeks before the event. Because of the contractor’s activities and potential weather delays, the start date for the critical 14-day ABC period was not finalized until that point. Given the long distances travelled by many of the attendees, the planning committee wanted to ensure that the bridge construction would have reached an “interesting” phase on the day of the showcase.

Travel Funding

A limited number of travel scholarships were provided by the SHRP 2 program to encourage participation from state DOTs across the United States. A number of bridge owners were given financial support for travel expenses for the showcase.

Agenda

The showcase agenda included presentations from a variety of viewpoints, providing an overview of the Highways for LIFE program, both national and Iowa perspectives on accelerated bridge construction, and a detailed presentation on the design and construction of the Keg Creek bridge. Following the event, the speaker presentations were made available in PDF format for all participants. The presentations are also available for download on the Highways for LIFE website.

Figure D.35. ABC showcase in Council Bluffs, Iowa.

Figure D.36. ABC showcase visit to Keg Creek bridge site.

Site Visit

Following lunch at the conference hotel, showcase attendees were encouraged to visit the project site during the afternoon. Bus transportation was provided to the project site, and attendees were allowed to freely observe all aspects of the construction progress. Weather was cool and windy with temperatures in the low 50s in the afternoon.

On the day of the showcase, the contractor was placing the UHPC material for all of the superstructure deck joints, which was an ideal time for the attendees to arrive. Visitors were able to observe the mixing, transporting, placing, finishing, and curing operations. For most of the showcase attendees, this was their first in-person exposure to UHPC.

During the site visit, countless small group meetings were held to share ideas and recommendations for future projects that might use some or all of the technologies observed.

Evaluation

Showcase attendees were asked to complete a brief evaluation to allow the organizers to assess the quality and usefulness of the showcase content, as well as the potential for applying ABC technologies on upcoming projects in their own jurisdictions.

Overall, the showcase event received very good reviews from the participants, with an average rating of 4.1 on a scale of 1 (poor) to 5 (excellent). Several participants provided contact information and agreed to participate in follow-up communications 6 months after the showcase to document how their future ABC projects are advancing.

Project Website

Given the significant national interest in the Keg Creek bridge project, the Iowa DOT established a project website (http://www.iowadot.gov/us6kegcreek/) to provide access to a variety of project-related materials. These include the following:

- Photo gallery;
- Video gallery with time-lapse video of entire ABC period and project animations;
- Detour information;
- Plan drawings and technical details; and
- Links to FHWA and Highways for LIFE websites.

SHRP 2 Project R04 Video Documentary

The SHRP 2 research team is currently producing a set of documentary videos under some additional funding from the SHRP 2 Program. Two videos will result—an approximately 10-min version intended for an upper-level administrator at a DOT that might be considering an ABC project, and a longer version, approximately 25 min, geared toward a more technical viewer such as a DOT bridge designer, consultant, or contractor.

The videos will be made available online through a variety of sources, including SHRP 2 and FHWA websites.

Postconstruction Review Meeting

The Iowa DOT hosted a postconstruction review meeting in Ames on November 17, 2011. In attendance were representatives from the following:

- Iowa DOT (Bridge Office, District 4, and Council Bluffs Residency);
- SHRP 2 Project R04 research team (HNTB and Iowa State University);
- SHRP 2 program manager;
- FHWA (Iowa division);
- Applied Research Associates (documenting project for SHRP 2); and
- Godbersen-Smith Construction.

The purpose of the meeting was to review the design and construction process for the demonstration bridge and to document not only the successful elements of the project but also those aspects that could be improved for future projects. The SHRP 2 Project R04 research team has developed standard bridge details for this type of modular bridge, and many suggestions have been incorporated into those standards. A summary of lessons learned from this demonstration project is provided in the next section.

Summary and Lessons Learned

Overall, the Keg Creek bridge project was a tremendous success. The bridge was completely replaced in 16 days by using only conventional equipment and labor and without significant problems. All parties (owner, designer, and contractor) worked closely together to resolve challenges as they arose during the ABC period. A SHRP 2 Project R04 representative was on site during the ABC period to make immediate decisions when questions arose. This was a critical component to the overall success. Following the postconstruction review meeting, a summary of lessons learned was compiled:

- On-site prefabrication of bridge components can be performed by contractors and result in a high-quality product. On-site inspection staff should be prepared for work that is not exactly like their normal projects.
- On-site mixing and placement of trial batches of UHPC should be considered to help eliminate fiber balling issues. Early and proactive communication with the UHPC provider is critical to the success of the on-site placement operations.
- Project special provisions should be carefully written to provide for both on-site and more-traditional precast concrete operations. The special provisions should describe casting, quality assurance, and inspection.
- The bond between UHPC and conventional precast concrete is critical. Surface preparation before placement of UHPC should be performed according to the manufacturer's recommendations. Future direct tension testing of bond specimens at Iowa State University will be beneficial in understanding this condition.
- Field placement of UHPC in large quantities can be challenging to manage. For future projects, cold joint bulkheads should be strategically placed to manage UHPC pours efficiently. Separating the UHPC pour in the suspended backwall from the slab joint pour might also be beneficial.
- Joint reinforcement that uses hairpin bars should be carefully evaluated for future projects. Simplifying the joint construction with reinforcement details that allow the joints to be more easily constructed may be possible. Bars should be staggered and projecting bars shortened if possible.
- Joint reinforcement congestion should be carefully evaluated for future projects. Reducing the number of longitudinal bars to allow these joints to be more easily constructed may be possible. Bars crossing at the joint intersections create congestion and time-consuming placement methods.
- Surveying is a critical element of fast-track bridge replacement projects. To avoid critical and time-consuming errors, two sets of independent surveys should be used to verify accurate pile driving and foundation placement during the ABC period.
- Precast approach pavement may not be the most efficient means of connecting an ABC bridge to the adjacent roadway. Placing a section of cast-in-place approach concrete by using accelerators may be faster since a small closure pour will almost inevitably be needed in any case.
- Additional isometric views should be included in plans to allow contractor and inspection personnel to better understand how the bridge components fit together.
- Although no backup plan was needed on this project, the contractor should have one in the event that a bridge component is damaged during the ABC period. At the very least, a repair plan should be agreed upon in advance.
- Ideally, the designer should be present on site during the ABC period to facilitate quick decision making.

On the basis of the lessons learned from the Keg Creek demonstration bridge project, several adjustments were incorporated into the proposed ABC standards presented elsewhere in this report. These adjustments were intended to improve constructability and reliability and to provide improved opportunities for future plant-cast and site-cast ABC projects.

APPENDIX E

ABC Standard Plans

STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

STANDARD PLANS FOR ABC MODULAR SYSTEMS

INDEX OF DRAWINGS

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DESIGN TEAM | SHEET NUMBER 01

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STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

STANDARD PLANS FOR ABC MODULAR SYSTEMS

ABC CONSTRUCTION CONCEPTS

SHEET NO.	DESCRIPTION
CC1	GENERAL NOTES
CC2	GENERAL NOTES
CC3	CONVENTIONAL ERECTION REPLACEMENT SINGLE SHORT SPAN BRIDGE
CC4	CONVENTIONAL ERECTION WIDEN SHORT SPAN BRIDGE OVER ROADWAY
CC5	CONVENTIONAL ERECTION WIDEN SHORT SPAN BRIDGE OVER ROADWAY
CC6	CONVENTIONAL ERECTION REPLACEMENT SHORT SPAN BRIDGE OVER ROADWAY
CC7	CONVENTIONAL ERECTION REPLACEMENT SHORT SPAN BRIDGE OVER ROADWAY
CC8	CONVENTIONAL ERECTION REPLACEMENT SHORT SPAN BRIDGE OVER WATERWAY (OPT 1)
CC9	CONVENTIONAL ERECTION REPLACEMENT SHORT SPAN BRIDGE OVER WATERWAY (OPT 1)
CC10	CONVENTIONAL ERECTION REPLACEMENT SHORT SPAN BRIDGE OVER WATERWAY (OPT 2)
CC11	CONVENTIONAL ERECTION REPLACEMENT SHORT SPAN BRIDGE OVER WATERWAY (OPT 2)
CC12	CONVENTIONAL ERECTION WIDEN LONG SPAN BRIDGE OVER ROADWAY
CC13	CONVENTIONAL ERECTION WIDEN LONG SPAN BRIDGE OVER ROADWAY
CC14	CONVENTIONAL ERECTION WIDEN LONG SPAN BRIDGE OVER ROADWAY
CC15	CONVENTIONAL ERECTION REPLACEMENT LONG SPAN BRIDGE OVER ROADWAY
CC16	CONVENTIONAL ERECTION REPLACEMENT LONG SPAN BRIDGE OVER ROADWAY
CC17	CONVENTIONAL ERECTION REPLACEMENT LONG SPAN BRIDGE OVER ROADWAY
CC18	STRADDLE CARRIERS ON PERMANENT BRIDGE - SHORT SPAN BRIDGE
CC19	STRADDLE CARRIERS ON PERMANENT BRIDGE - SHORT SPAN BRIDGE
CC20	STRADDLE CARRIERS ON PERMANENT BRIDGE - STAGED CONSTRUCTION

SHEET NO.	DESCRIPTION
CC21	STRADDLE CARRIERS ON LAUNCH BEAMS - SHORT SPAN BRIDGE
CC22	STRADDLE CARRIERS ON LAUNCH BEAMS - SHORT SPAN BRIDGE
CC23	STRADDLE CARRIERS ON LAUNCH BEAMS - STAGED CONSTRUCTION
CC24	ADDC CONCEPT - PLAN & ELEVATION
CC25	ADDC CONCEPT - TYPICAL CROSS SECTION
CC26	ADDC CONCEPT - STAGED CONSTRUCTION
CC27	LTTB CONCEPT - PLAN & ELEVATION
CC28	LTTB CONCEPT - TYPICAL CROSS SECTION
CC29	LTTB CONCEPT - STAGED CONSTRUCTION
CC30	TYPICAL ERECTION TRUSS MODULE
CC31	TYPICAL ROLLING GANTRY CONCEPTS
CC32	ERECTION OF PREFABRICATED CONCRETE SUBSTRUCTURE ELEMENTS

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GENERAL INFORMATION: SUBSTRUCTURE

PREFABRICATED COMPONENTS PRODUCED OFF-SITE CAN BE QUICKLY ASSEMBLED AND CAN REDUCE CONSTRUCTION TIME, COST, MINIMIZE LANE CLOSURE TIME AND THE NEED FOR A TEMPORARY BRIDGE. THE INTENT OF THESE DESIGN STANDARDS IS TO PROVIDE INFORMATION THAT APPLIES TO THE DESIGN, DETAILING, FABRICATION, HANDLING AND ASSEMBLY OF PREFABRICATED COMPONENTS USED IN ACCELERATED BRIDGE CONSTRUCTION, ACCORDING TO AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS.

SUBSTRUCTURES ARE THE PORTIONS OF THE BRIDGE LOCATED BETWEEN THE SUPERSTRUCTURE AND THE FOUNDATION (SUPPORTING SOIL, PILES, OR DRILLED SHAFTS). GEOTECHNICAL DESIGN, PILE DESIGN, AND DETAILING ARE NOT CONSIDERED SUBSTRUCTURES AND ARE NOT COVERED IN THESE DESIGN STANDARDS. THE SUBSTRUCTURE DETAILS DEPICTED CAN BE ADAPTED TO FIT OTHER FOUNDATION TYPES. THE SYSTEMS PRESENTED IN THESE DESIGN STANDARDS CONSIST OF PREFABRICATED CONCRETE PIER BENTS, INTEGRAL ABUTMENTS, SEMI INTEGRAL ABUTMENTS AND WINGWALLS.

THE PREFABRICATED SUBSTRUCTURE SYSTEMS PRESENTED IN THESE PLANS ARE INTENDED TO BE USED WITH THE PREFABRICATED SUPERSTRUCTURE SYSTEMS THAT ARE A PART OF THESE DESIGN STANDARDS. THE REINFORCING DETAILS AND CONNECTION DETAILS SHOWN ARE SUITABLE FOR USE IN NON-SEISMIC AREAS.

TYPICAL DESIGNS FOR THE SUBSTRUCTURE SYSTEMS HAVE BEEN GROUPED INTO THE FOLLOWING SPAN RANGES:

40 FT < SPAN < 70 FT
70 FT < SPAN < 100 FT
100 FT < SPAN < 130 FT

THE SUBSTRUCTURE SYSTEMS SUPPORT A TYPICAL TWO LANE BRIDGE WITH SHOULDERS HAVING AN OUT-TO-OUT WIDTH OF 47'-2". WHILE THE BRIDGE CROSS-SECTION WAS CHOSEN TO REPRESENT A ROUTINE BRIDGE STRUCTURE, THE DESIGN CONCEPTS, DETAILS, FABRICATION AND ASSEMBLY METHODS ARE EQUALLY APPLICABLE TO OTHER BRIDGE WIDTHS.

THE DETAILS PRESENTED IN THESE PLANS ARE INTENDED TO SERVE AS GENERAL GUIDANCE IN THE DEVELOPMENT OF DESIGNS SUITABLE FOR ACCELERATED BRIDGE CONSTRUCTION. THESE DETAILS SHALL NOT BE PERCEIVED AS STANDARDS THAT ARE READY TO BE INSERTED INTO CONTRACT PLANS. THEIR IMPLEMENTATION SHALL WARRANT A COMPLETE DESIGN BY THE "ENGINEER OF RECORD (EOR)" IN ACCORDANCE WITH THE REQUIREMENTS FOR THE PROJECT SITE AND DOT STANDARDS AND SPECIFICATIONS. THE DESIGNER SHALL VERIFY THAT ALL REQUIREMENTS OF THE LATEST AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, INCLUDING INTERIM PROVISIONS, ARE SATISFIED AND PROPERLY DETAILED IN ANY DOCUMENTS INTENDED OR PROVIDED FOR CONSTRUCTION.

SKEWED SUPERSTRUCTURES:

THESE PLANS PRESENT A CONCEPT WELL-SUITED TO BRIDGES SUPPORTED ON BEARING LINES NORMAL TO THE CENTERLINE OF THE STRUCTURE. LOW TO MODERATE SKEWS CAN BE ACCOMMODATED WITH DUE CONSIDERATION GIVEN TO THE DESIGN, FABRICATION, AND ERECTION.

DESIGN SPECIFICATIONS:

AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, 5TH EDITION. LRFD LIMIT STATE LOAD COMBINATION STRENGTH I, STRENGTH III, STRENGTH V, AND SERVICE I.

DESIGN LIVE LOAD: HL-93
FUTURE WEARING SURFACE = 25 PSF

LIFTING AND HANDLING STRESSES:

1. THE POSITIONS OF LIFTING INSERTS ARE CALCULATED TO LIMIT LIFTING STRESSES AND TO ENSURE THAT THE PREFABRICATED ELEMENT HANGS IN THE CORRECT ORIENTATION DURING LIFTING.
2. THE DESIGNER SHALL ANALYZE SPANS ON THE ASSUMED TEMPORARY/LIFTING SUPPORTS BASED ON THE STRENGTH I LIMIT STATE WITH A LOAD FACTOR EQUAL TO 1.25.
3. MAXIMUM STRESSES IN PREFABRICATED COMPONENTS DURING LIFTING, HANDLING AND ERECTION SHALL BE CHECKED UNDER THE SERVICE I LOAD COMBINATION. A 25% HANDLING IMPACT FACTOR ON DEAD LOADS SHALL BE ASSUMED. LIFTING AND HANDLING STRESSES ARE TO BE SPECIFIED ON THE PLANS.
4. PREFABRICATED ELEMENTS AND MODULAR SYSTEMS ARE TO BE ANALYZED BASED ON ELASTIC BEHAVIOR FOR LIFTING AND HANDLING. INELASTIC ANALYSIS WILL NOT BE PERMITTED.
5. NO PERMANENT DISTORTION (TWIST) AS A RESULT OF LIFTING AND HANDLING WILL BE ALLOWED.

SHOP DRAWINGS AND ASSEMBLY PLAN

THE CONTRACTOR SHALL PREPARE AND SUBMIT SHOP DETAILS, ASSEMBLY PLAN, AND ALL OTHER NECESSARY WORKING DRAWINGS FOR APPROVAL IN ACCORDANCE WITH THE REQUIREMENTS OF PROJECT SPECIFICATIONS. THE CONTRACTOR WILL PROVIDE THE SPACING AND LOCATION OF THE LIFTING DEVICES ON THE SHOP DRAWINGS AND CALCULATE HANDLING STRESSES. THE ASSEMBLY PLAN SHALL INCLUDE, BUT NOT NECESSARILY BE LIMITED TO, THE FOLLOWING:

DETAILS OF ALL EQUIPMENT THAT WILL BE EMPLOYED FOR THE ASSEMBLY; METHOD OF ERECTION
COMPUTATIONS TO INDICATE THE MAGNITUDE OF STRESSES DURING ERECTION
DETAILED SEQUENCE OF CONSTRUCTION AND SCHEDULE.
METHODS OF PROVIDING TEMPORARY SUPPORT OF THE COMPONENTS.
METHODS FOR VERTICAL ADJUSTMENT.
METHODS OF FORMING AND CURING CLOSURE POURS AND INSTALLATION OF COUPLERS.

FABRICATION TOLERANCES

FABRICATION TOLERANCES SHALL BE DETAILED IN THE PROJECT PLANS AND SPECIFICATIONS. UNLESS OTHERWISE SHOWN ELSEWHERE IN THE PLANS OR SPECIFICATIONS, LIMIT VARIATIONS FROM DIMENSIONS SHOWN IN THE CONTRACT DOCUMENTS TO NO MORE THAN $\frac{1}{8}$ INCH.

SITE CASTING

IF THE CONTRACTOR ELECTS TO FABRICATE THE NON-PRESTRESSED BRIDGE COMPONENTS AT A TEMPORARY CASTING FACILITY, THE CASTING SHALL COMPLY WITH THE PROVISIONS OF THE PROJECT SPECIFICATIONS.

GEOMETRY CONTROL

CONSTRUCTION GEOMETRY CONTROL FOR DIFFERENTIAL CAMBER, SKEWNESS, AND CROSS-SLOPE ARE KEY TO ENSURING PROPER FIT UP OF PREFABRICATED SYSTEMS.

THE CONTRACTOR SHALL CHECK THE ELEVATIONS AND ALIGNMENT OF THE STRUCTURE AT EVERY STAGE OF CONSTRUCTION TO ASSURE PROPER ERECTION OF THE STRUCTURE TO THE FINAL GRADE SHOWN ON THE DESIGN PLANS. USE VERTICAL ADJUSTMENT DEVICES TO PROVIDE GRADE ADJUSTMENT TO MEET THE ELEVATION TOLERANCES SHOWN ON THE PLANS.

BRIDGE CROSS SLOPES CAN BE ACCOMMODATED BY TILTING THE SUPERSTRUCTURE MODULES WITH RESPECT TO PLUMB. THE SLOPE OF THE BRIDGE SEAT SHALL CONFORM TO THE BRIDGE CROSS SLOPE. CORRECTIONS FOR GRADE BY SHIMMING OR NEOPRENE PADS CAN BE DONE WHEN APPROVED BY THE ENGINEER.

THE PREFABRICATED SUBSTRUCTURE ELEMENTS SHALL BE PRE-ASSEMBLED TO ASSURE PROPER MATCH BETWEEN ELEMENTS TO THE SATISFACTION OF THE ENGINEER BEFORE SHIPPING TO THE JOB SITE. THE PROCEDURE FOR LEVELING AND VERTICAL ADJUSTMENT SHALL BE ESTABLISHED DURING THE PRE-ASSEMBLY AND APPROVED BY THE ENGINEER.

MECHANICAL GROUTED SPLICES

A TEMPLATE WILL BE REQUIRED FOR ACCURATE MECHANICAL SPLICE PLACEMENT DURING COMPONENT FABRICATION AND/OR FIELD CAST CONDITIONS TO ENSURE FIT-UP BETWEEN JOINED COMPONENTS. PLACEMENT TOLERANCES SHOULD BE AS RECOMMENDED BY THE MANUFACTURER. THE GROUTING PROCESS SHOULD FOLLOW THE MANUFACTURER'S RECOMMENDATIONS FOR MATERIALS AND EQUIPMENT. ALL CONNECTIONS SHALL BE DRY FIT IN THE FABRICATION YARD PRIOR TO INSTALLATION OF THE ELEMENTS AT THE BRIDGE SITE.

ELEMENT SIZES

THE SIZE AND WEIGHT OF PRECAST CONCRETE SUBSTRUCTURE ELEMENTS CAN BECOME AN ISSUE FOR SHIPPING AND HANDLING. IT IS PREFERABLE TO KEEP THE WEIGHT OF EACH ELEMENT TO LESS THAN 50 TONS, HOWEVER HIGHER WEIGHTS MAY BE ACCEPTABLE FOR CERTAIN SUBSTRUCTURE ELEMENTS.

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

STANDARD PREFABRICATED
SUPERSTRUCTURE

GENERAL INFORMATION SHEET I

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GENERAL PROCEDURE FOR INSTALLATION OF SUBSTRUCTURE MODULES

A. GENERAL PROCEDURE FOR INSTALLATION OF PRECAST ELEMENTS

1. DRY FIT ADJACENT PRECAST ELEMENTS IN THE YARD PRIOR TO SHIPPING TO THE SITE.
2. ESTABLISH WORKING POINTS, WORKING LINES, AND BENCHMARK ELEVATIONS PRIOR TO PLACEMENT OF ALL PRECAST ELEMENTS.
3. PLACE PRECAST ELEMENTS IN THE SEQUENCE AND ACCORDING TO THE METHODS OUTLINED IN THE ASSEMBLY PLAN. ADJUST THE HEIGHT OF EACH PRECAST ELEMENT BY MEANS OF LEVELING DEVICES OR SHIMS.
4. USE PERSONNEL THAT ARE FAMILIAR WITH INSTALLATION AND GROUTING OF SPLICE COUPLERS. FOLLOW THE RECOMMENDATIONS OF THE MANUFACTURER FOR THE INSTALLATION AND GROUTING OF THE COUPLERS.

B. GENERAL PROCEDURE FOR PIER COLUMNS AND CAPS

1. LIFT THE PRECAST ELEMENT AS SHOWN IN THE ASSEMBLY PLAN USING LIFTING DEVICES AS SHOWN ON THE SHOP DRAWINGS.
2. SURVEY THE ELEVATION OF THE COMPLETED STRUCTURE DIRECTLY BELOW THE ELEMENT. PROVIDE SHIMS TO BRING THE BOTTOM OF THE ELEMENT TO THE REQUIRED ELEVATION.
3. SET THE ELEMENT IN THE PROPER HORIZONTAL LOCATION. CHECK FOR PROPER HORIZONTAL AND VERTICAL ALIGNMENT WITHIN SPECIFIED TOLERANCES. REMOVE AND ADJUST THE SHIMS AND RESET THE ELEMENT IF IT IS NOT WITHIN TOLERANCE.
4. CHECK THE GROUTED SPLICE COUPLERS BETWEEN ADJACENT ELEMENTS THAT WILL SUPPORT COMMON PRECAST ELEMENTS IN FUTURE STAGES OF CONSTRUCTION. SET THE ELEMENT AND INSTALL THE COUPLERS ONCE THE CONNECTION GEOMETRY IS ESTABLISHED AND CHECKED.
5. INSTALL TEMPORARY BRACING IF SPECIFIED IN THE ASSEMBLY PLAN.
6. ALLOW THE GROUT IN THE COUPLER TO CURE UNTIL THE COUPLER CAN RESIST 100 PERCENT OF THE SPECIFIED MINIMUM YIELD STRENGTH OF THE BAR PRIOR TO REMOVAL OF BRACING AND PROCEEDING WITH INSTALLATION OF COMPONENTS ABOVE THE ELEMENT.

C. GENERAL PROCEDURE FOR ABUTMENT STEM AND WINGWALLS (SUPPORTED ON PILES)

1. LIFT ABUTMENT STEM PRECAST ELEMENT OR WINGWALL PRECAST ELEMENT AS SHOWN IN THE ASSEMBLY PLAN USING LIFTING DEVICES AS SHOWN ON THE SHOP DRAWINGS.
2. SET THE PRECAST ELEMENT IN THE PROPER HORIZONTAL LOCATION. CHECK FOR PROPER ALIGNMENT WITHIN SPECIFIED TOLERANCES.
3. ADJUST THE DEVICES PRIOR TO FULL RELEASE FROM THE CRANE IF VERTICAL LEVELING DEVICES ARE USED. CHECK FOR PROPER GRADE WITHIN SPECIFIED TOLERANCES.
4. ALL CLOSURE POUR SURFACES PRIOR TO CONNECTING THE MODULES, SHALL BE SATURATED SURFACE DRY.
5. PLACE HIGH EARLY STRENGTH SELF CONSOLIDATING CONCRETE AROUND PILE TOPS AND BETWEEN PRECAST MODULES AS SHOWN ON THE PLANS. ALLOW CONCRETE TO FLOW PARTIALLY UNDER THE PRECAST ELEMENT.
6. DO NOT PROCEED WITH THE INSTALLATION OF ADDITIONAL PRECAST ELEMENTS ABOVE UNTIL THE COMPRESSIVE TEST RESULT OF THE CYLINDERS FOR THE PILE CONNECTION CONCRETE HAS REACHED THE SPECIFIED MINIMUM VALUES.

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

STANDARD PREFABRICATED SUBSTRUCTURE

GENERAL INFORMATION SHEET II

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GENERAL INFORMATION: SUPERSTRUCTURE

PREFABRICATED COMPONENTS PRODUCED OFF-SITE CAN BE QUICKLY ASSEMBLED, AND CAN REDUCE CONSTRUCTION TIME, COST, MINIMIZE LANE CLOSURE TIME AND/THE NEED FOR A TEMPORARY BRIDGE. THE INTENT OF THESE DESIGN STANDARDS IS TO PROVIDE INFORMATION THAT APPLIES TO THE DESIGN, DETAILING, FABRICATION, HANDLING AND ASSEMBLY OF PREFABRICATED COMPONENTS USED IN ACCELERATED BRIDGE CONSTRUCTION, ACCORDING TO AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS.

THE SYSTEMS PRESENTED IN THESE DESIGN STANDARDS CONSIST OF A PRESTRESSED CONCRETE GIRDER WITH AN INTEGRALLY CAST DECK AND A COMPOSITE DECKED STEEL STRINGER MODULE. BOTH SYSTEMS INCLUDE A FULL DEPTH FLANGE THAT SERVES AS THE RIDING SURFACE TO ELIMINATE THE NEED FOR A CAST-IN-PLACE DECK.

THE PREFABRICATED SUPERSTRUCTURE SYSTEMS (SUPERSTRUCTURE MODULES) PRESENTED IN THESE PLANS MAY BE USED WITH THE PREFABRICATED SUBSTRUCTURE SYSTEMS THAT ARE A PART OF THESE DESIGN STANDARDS, OR THEY MAY BE USED WITH OTHER NEW OR EXISTING SUBSTRUCTURES THAT HAVE BEEN ADAPTED TO SUPPORT THE LOAD REQUIREMENTS FOR THESE SUPERSTRUCTURE MODULES.

TYPICAL DESIGNS FOR SUPERSTRUCTURE MODULES HAVE BEEN GROUPED INTO THE FOLLOWING SPAN RANGES:

20 FT < SPAN < 40 FT
40 FT < SPAN < 70 FT
70 FT < SPAN < 100 FT
100 FT < SPAN < 130 FT

THE SUPERSTRUCTURE CROSS-SECTION AND MODULE WIDTHS HAVE BEEN SHOWN FOR A TYPICAL TWO LANE BRIDGE WITH SHOULDERS HAVING AN OUT-TO-OUT WIDTH OF 47'-2". WHILE THE BRIDGE CROSS-SECTION WAS CHOSEN TO REPRESENT A ROUTINE BRIDGE STRUCTURE, THE DESIGN CONCEPTS, DETAILS, FABRICATION AND ASSEMBLY ARE EQUALLY APPLICABLE TO OTHER BRIDGE WIDTHS.

THE DETAILS PRESENTED IN THESE PLANS ARE INTENDED TO SERVE AS GENERAL GUIDANCE IN THE DEVELOPMENT OF DESIGNS SUITABLE FOR ACCELERATED BRIDGE CONSTRUCTION. THESE DETAILS SHALL NOT BE PERCEIVED AS STANDARDS THAT ARE READY TO BE INSERTED INTO CONTRACT PLANS. THEIR IMPLEMENTATION SHALL WARRANT A COMPLETE DESIGN BY THE ENGINEER OF RECORD (EOR) IN ACCORDANCE WITH THE REQUIREMENTS FOR THE PROJECT SITE AND DOT STANDARDS AND SPECIFICATIONS. THE DESIGNER SHALL VERIFY THAT ALL REQUIREMENTS OF THE LATEST AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, INCLUDING INTERIM PROVISIONS, ARE SATISFIED AND PROPERLY DETAILED IN ANY DOCUMENTS INTENDED OR PROVIDED FOR CONSTRUCTION.

ALL CONSTRUCTION AND ASSEMBLY PLANS, INCLUDING THE DESIGN OF LIFTING POINT, HARDWARE, AND RIGGING, SHALL BE SIGNED AND SEALED BY A LICENSED PROFESSIONAL ENGINEER.

ALL FORMWORK FOR THE DECK SHALL BE SUPPORTED FROM THE LONGITUDINAL GIRDERS SIMILAR TO CONVENTIONAL CONSTRUCTION METHODS. SHORED CONSTRUCTION SHALL NOT BE ASSUMED. DECKED GIRDER SYSTEMS SHALL ALLOW FUTURE DECK REPLACEMENT WITHOUT THE USE OF SHORING.

SKEWED STRUCTURES:

THESE PLANS PRESENT A CONCEPT WELL-SUITED TO BRIDGES SUPPORTED ON BEARING LINES NORMAL TO THE CENTERLINE OF THE STRUCTURE. LOW TO MODERATE SKEWS CAN BE ACCOMMODATED WITH DUE CONSIDERATION GIVEN TO DESIGN, FABRICATION, AND ERECTION. LARGER SKEWS REQUIRE DUE CONSIDERATION OF DESIGN AND DETAILING REQUIREMENTS. THE ENGINEER OF RECORD SHALL BE RESPONSIBLE FOR COMPLETELY INCORPORATING THE EFFECTS OF ANY DEGREE OF SKEWED SUPPORTS IN ACCORDANCE WITH ALL APPLICABLE DESIGN SPECIFICATIONS.

DESIGN SPECIFICATIONS:

AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, 5TH EDITION

DESIGN LIVE LOAD: HL-93
FUTURE WEARING SURFACE = 25 PSF

THESE CONCEPT DESIGNS DO NOT CONSIDER PERMIT OR OVERLOAD VEHICLES AT THE STRENGTH LIMIT STATE THAT MAY BE REQUIRED BY THE GOVERNING AGENCY.

FABRICATION TOLERANCES:

FABRICATION TOLERANCES SHALL BE DETAILED IN THE PROJECT PLANS AND SPECIFICATIONS. UNLESS OTHERWISE SHOWN ELSEWHERE IN THE PLANS OR SPECIFICATIONS, LIMIT VARIATIONS FROM DIMENSIONS SHOWN IN THE CONTRACT DOCUMENTS TO NO MORE THAN $\frac{1}{8}$ INCH.

SITE CASTING:

IF THE CONTRACTOR ELECTS TO FABRICATE THE NON-PRESTRESSED BRIDGE COMPONENTS AT A TEMPORARY CASTING FACILITY, THE CASTING SHALL COMPLY WITH THE PROVISIONS OF THE PROJECT SPECIFICATIONS.

GENERAL INSTALLATION PROCEDURE:

1. DRY FIT ADJACENT PRECAST ELEMENTS IN THE YARD PRIOR TO SHIPPING TO THE SITE.
2. DO NOT PLACE MODULES ON PRECAST SUBSTRUCTURE UNTIL THE COMPRESSIVE TEST RESULTS OF FOR THE PRECAST SUBSTRUCTURE CONNECTION CONCRETE HAS REACHED THE SPECIFIED MINIMUM VALUES.
3. SURVEY THE TOP ELEVATION OF THE SUBSTRUCTURES. ESTABLISH WORKING POINTS, WORKING LINES, AND BENCHMARK ELEVATIONS PRIOR TO PLACEMENT OF ALL MODULES.
4. LIFT AND ERECT MODULES USING LIFTING DEVICES AS SHOWN ON THE SHOP DRAWINGS IN CONFORMANCE WITH THE ASSEMBLY PLANS.
5. SET MODULE IN THE PROPER LOCATION. SURVEY THE TOP ELEVATION OF THE MODULES. VERIFY PROPER LINE AND GRADE WITHIN SPECIFIED TOLERANCES. APPROVED STEEL SHIMS SHALL BE USED BETWEEN THE BEARING AND THE GIRDER TO COMPENSATE FOR MINOR DIFFERENCES IN ELEVATION BETWEEN MODULES AND APPROACH ELEVATIONS. FOLLOW MATCH-MARKS.
6. TEMPORARILY SUPPORT, ANCHOR, AND BRACE ALL ERECTED MODULES AS NECESSARY FOR STABILITY AND TO RESIST WIND OR OTHER LOADS UNTIL THEY ARE PERMANENTLY SECURED TO THE STRUCTURE. SUPPORT, ANCHOR, AND BRACE ALL MODULES AS DETAILED IN THE ASSEMBLY PLAN.
7. DIFFERENCES IN CAMBER BETWEEN ADJACENT MODULES SHIPPED TO THE SITE SHALL NOT EXCEED THE PRESCRIBED LIMITS. IF THERE IS A DIFFERENTIAL CAMBER, THE CONTRACTOR SHALL APPLY DEAD LOAD AS NEEDED TO BRING ADJACENT BEAMS WITHIN THE CONNECTION TOLERANCE. A LEVELING BEAM CAN ALSO BE USED TO EQUALIZE CAMBER. THE LEVELING PROCEDURE SHALL BE DEMONSTRATED DURING THE PRE-ASSEMBLY PROCESS PRIOR TO SHIPPING TO THE SITE. THE ASSEMBLY PLAN SHALL INDICATE THE LEVELING PROCESS TO BE APPLIED IN THE FIELD. IF A LEVELING BEAM IS TO BE USED, HAVE AVAILABLE A LEVELING BEAM AND SUITABLE JACKING ASSEMBLIES FOR ATTACHMENT TO THE LEVELING INSERTS OF ADJACENT MODULES. EQUIP ALL MODULES WITH LEVELING INSERTS FOR FIELD ADJUSTMENT OR EQUALIZING OF DIFFERENTIAL CAMBER. THE INSERTS WITH THREADED FERRULES SHALL BE CAST IN THE DECK, CENTERED OVER THE BEAM WEB(S). A MINIMUM TENSION CAPACITY OF 5,500 LBS IS REQUIRED FOR THE INSERTS.
8. FORM, CAST AND CURE UHPC CLOSURE POURS AS DETAILED IN THE PLANS AND SPECIFICATION.
9. DIAMOND GRIND THE DECK TO ACHIEVE A SMOOTH PROFILE. DIAMOND GRINDING OF THE BRIDGE DECK SHALL NOT BEGIN UNTIL THE UHPC CLOSURE POUR CONCRETE HAS REACHED THE SPECIFIED MINIMUM COMPRESSIVE STRENGTH OF 10 KSI.

REQUIREMENTS FOR UHPC JOINTS:

PRIOR TO CONCRETE PLACEMENT DURING FABRICATION, THOROUGHLY COAT THE BEVELED FACES OF THE FORMWORK AT ALL CLOSURE JOINTS WITH AN APPROVED CONCRETE RETARDING ADMIXTURE.

AFTER FORMS ARE STIPPED DURING FABRICATION, USE A HIGH-PRESSURE STREAM OF WATER TO ROUGHEN THE BEVELED FACES AT ALL CLOSURE JOINTS TO AN AMPLITUDE OF $\frac{1}{4}$ INCH WITHOUT DISPLACING COARSE AGGREGATE.

EDGES OF CLOSURE POUR SHALL BE SATURATED SURFACE DRY PRIOR TO PLACING UHPC. ALL CONCRETE FACES TO BE IN CONTACT WITH UHPC SHALL BE CLEANED AND COATED WITH AN APPROVED EPOXY BONDING AGENT PRIOR TO PLACING UHPC.

MOCKUPS OF EACH UHPC POUR SHALL BE PERFORMED PRIOR TO ACTUAL UHPC CONSTRUCTION.

ALL THE FORMS FOR UHPC SHALL BE CONSTRUCTED FROM PLYWOOD. USE CONTINUOUS TOP AND BOTTOM FORMS FOR UHPC JOINTS.

TWO PORTABLE BATCHING UNITS SHOULD BE USED FOR MIXING OF THE UHPC.

EACH UHPC PLACEMENT SHALL BE CAST USING ONE CONTINUOUS POUR. COLD JOINTS ARE PERMITTED ONLY AS APPROVED BY THE ENGINEER. UHPC SHALL BE PRODUCED TO FILL ANY ONE CONNECTION AREA WITHIN 30 MINUTES.

THE UHPC SHALL BE CURED ACCORDING TO MATERIALS SUPPLIER RECOMMENDATIONS.

WEATHER CONDITION DURING UHPC PLACEMENT, INCLUDING TEMPERATURE AND WIND, SHOULD BE TAKEN INTO CONSIDERATION INACCORDANCE WITH SUPPLIER RECOMMENDATIONS.

DIAMOND GRIND BRIDGE DECK:

AN ADDITIONAL THICKNESS OF $\frac{1}{2}$ INCH HAS BEEN INCORPORATED IN THE DECK TO PERMIT CORRECTION OF THE DECK PROFILE BY DIAMOND GRINDING.

SAW CUT GROOVE TEXTURE FINISH:

SAW CUT LONGITUDINAL GROOVES INTO TOP OF BRIDGE DECK USING A MECHANICAL CUTTING DEVICE AFTER DIAMOND GRINDING.

GEOMETRY CONTROL:

CONSTRUCTION GEOMETRY CONTROL FOR DIFFERENTIAL CAMBER, SKEWNESS, AND CROSS-SLOPE ARE KEY TO ENSURING PROPER FIT UP OF PREFABRICATED SYSTEMS.

THE CONTRACTOR SHALL CHECK THE ELEVATIONS AND ALIGNMENT OF THE STRUCTURE AT EVERY STAGE OF CONSTRUCTION TO ASSURE PROPER ERECTION OF THE STRUCTURE TO THE FINAL GRADE SHOWN ON THE DESIGN PLANS. USE VERTICAL ADJUSTMENT DEVICES TO PROVIDE GRADE ADJUSTMENT TO MEET THE ELEVATION TOLERANCES SHOWN ON THE PLANS.

BRIDGE CROSS SLOPES UP TO 4 PERCENT CAN BE ACCOMMODATED BY ERECTING THE SUPERSTRUCTURE MODULES OUT OF PLUMB. THE SLOPE OF THE BRIDGE SEAT SHALL CONFORM TO THE BRIDGE CROSS SLOPE. CORRECTIONS FOR GRADE BY SHIMMING OR NEOPRENE PADS CAN BE DONE WHEN APPROVED BY THE ENGINEER.

CAMBER CONTROL:

DIFFERENTIAL CAMBER CAN CAUSE DIMENSIONAL PROBLEMS WITH THE CONNECTIONS. CONTROL OF CAMBER DURING FABRICATION IS REQUIRED TO ACHIEVE RIDE QUALITY. CAMBER DIFFERENCES BETWEEN ADJACENT DECK SECTIONS AT THE TIME OF ERECTION SHALL NOT EXCEED THE LIMITS SHOWN ON THE PLANS.

THE PREFABRICATED SUPERSTRUCTURE SPAN SHALL BE PRE-ASSEMBLED TO ASSURE PROPER MATCH BETWEEN MODULES TO THE SATISFACTION OF THE ENGINEER BEFORE SHIPPING TO THE JOB SITE. THE PROCEDURE FOR LEVELING ANY DIFFERENTIAL CAMBER SHALL BE ESTABLISHED DURING THE PRE-ASSEMBLY AND APPROVED BY THE ENGINEER.

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STANDARD PREFABRICATED GIRDER SUPERSTRUCTURE

GENERAL INFORMATION

HNTB
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OCTOBER 2011

DESIGN TEAM

SHEET NUMBER G3

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GENERAL NOTES:

THESE PLANS PRESENT AN ACCELERATED BRIDGE CONSTRUCTION (ABC) CONCEPT FOR ERECTION OF A PRECAST CONCRETE ABUTMENT.

THE SYSTEM PRESENTED IN THESE CONCEPT PLANS CONSISTS OF A PRECAST ABUTMENT SYSTEM AND A PRECAST APPROACH SLAB SYSTEM. BOTH SEMI-INTEGRAL ABUTMENTS AND INTEGRAL ABUTMENTS ARE PRESENTED IN THESE CONCEPT PLANS. EACH HAS ITS OWN BENEFITS AND THE SELECTION OF THE FINAL SYSTEM SHALL BE MADE BY THE ENGINEER OF RECORD WITH DUE CONSIDERATION TO THE DESIGN SPECIFICATIONS AND SITE CONSTRAINS FOR ANY GIVEN PROJECT.

CONSTRUCTION LOADS:

CONCRETE STRESSES DURING HANDLING SHALL NOT EXCEED ALLOWABLE STRESSES PER DESIGN SPECIFICATIONS AS FOLLOWS: TENSION $0.24\sqrt{f'c}$ (ksi) COMP: 0.85 f'c. LIFT POINTS AND TEMPORARY SUPPORTS SHALL BE DETAILED AND LOCATED ON FINAL DESIGN PLANS.

THE EFFECTS OF DEAD LOAD STRESSES AT THE ERECTION STAGE SHALL BE INCREASED BY 25 PERCENT TO ACCOUNT FOR DYNAMIC EFFECTS DURING HANDLING AND TRANSPORTATION.

ALL PRECAST ELEMENTS SHALL BE REMOVED FROM THE FORMS IN SUCH A MANNER THAT NO DAMAGE OCCURS TO THE ELEMENT. ANY MATERIALS FORMING BLOCKOUTS IN THE PRECAST ELEMENTS SHALL BE REMOVED SUCH THAT DAMAGE DOES NOT OCCUR TO THE PRECAST ELEMENTS OR THE BLOCKOUT. PRECAST ELEMENTS SHALL BE STORED IN SUCH A MANNER THAT ADEQUATE SUPPORT IS PROVIDED TO PREVENT CRACKING OR CREEP-INDUCED DEFORMATION (SAGGING). DURING STORAGE FOR LONG PERIODS OF TIME (LONGER THAN ONE MONTH), ALL PRECAST ELEMENTS SHALL BE MEASURED AT LEAST ONCE PER MONTH TO ENSURE CREEP-INDUCED DEFORMATION DOES NOT OCCUR.

ALL PRECAST ELEMENTS SHALL BE HANDLED IN SUCH A MANNER AS NOT TO DAMAGE THE PRECAST ELEMENTS DURING LIFTING OR MOVING. LIFTING ANCHORS CAST INTO THE PRECAST ELEMENTS SHALL BE USED FOR LIFTING AND MOVING THE PRECAST ELEMENTS AT THE FABRICATION PLANT AND IN THE FIELD. THE ANGLE BETWEEN THE TOP SURFACE OF THE PRECAST ELEMENTS AND THE LIFTING LINE SHALL NOT BE LESS THAN SIXTY DEGREES, WHEN MEASURED FROM THE TOP SURFACE OF THE PRECAST ELEMENTS TO THE LIFTING LINE. DAMAGE CAUSED TO ANY PRECAST ELEMENTS SHALL BE REPAIRED AT THE EXPENSE OF THE CONTRACTOR TO THE SATISFACTION OF THE ENGINEER.

REPAIRS OF DAMAGE CAUSED TO THE PRECAST ELEMENTS DURING FABRICATION, LIFTING AND HANDLING, OR TRANSPORTATION SHALL BE ADDRESSED ON A CASE-BY-CASE BASIS. DAMAGE WITHIN ACCEPTABLE LIMITS CAUSED TO SURFACES OR TO KEYED EDGES OF THE PRECAST ELEMENTS SHALL BE REPAIRED USING APPROVED MATERIALS AT THE FABRICATION PLANT AT THE EXPENSE OF THE FABRICATOR. REPETITIVE DAMAGE TO PANELS SHALL BE CAUSE FOR STOPPAGE OF FABRICATION OPERATIONS UNTIL THE CAUSE OF THE DAMAGE CAN BE REMEDIED. ALL PROPOSED REPAIRS SHALL BE APPROVED BY ENGINEER IN ADVANCE.

SPECIFICATIONS:

DESIGN: AASHTO LRFD BRIDGE DESIGN SPECIFICATION 5TH EDITION, 2010
DESIGN LIVE LOAD: HL-93

CONSTRUCTION: AASHTO LRFD BRIDGE CONSTRUCTION SPECIFICATIONS, 5TH EDITION, WITH INTERIMS

MATERIAL PROPERTIES:

CONCRETE: HIGH PERFORMANCE CONCRETE (HPC) WITH MINIMUM 28 DAY COMPRESSIVE STRENGTH OF 5000 PSI.

SELF CONSOLIDATING CONCRETE: HIGH EARLY STRENGH WITH MINIMUM 28 DAY COMPRESSIVE STRENGTH OF 5000 PSI AND 1 DAY STRENGTH OF 3000 PSI

ULTRA HIGH PERFORMANCE CONCRETE (UHPC): IN ACCORDANCE WITH PROJECT SPECIAL PROVISIONS

REINFORCING STEEL: GRADE 60

CONCRTETE COVER: 3" ON ALL SURFACES IN GROUND CONTACT
2" ALL OTHER SURFACES

TOLERANCES:

TOLERANCES FOR THE FABRICATION OF PRECAST CONCRETE COMPONENTS ARE GENERALLY IN ACCORDANCE WITH APPENDIX B OF PCI MANUAL MNL-116.

RECOMMENDED TOLERANCES FOR ABUTMENT COMPONENTS AND APPROACH SLABS ARE SPECIFICED ON THESE PLANS.

LIMITATIONS:

THESE GUIDELINES ARE BASED ON THE GENERAL INFORMATION (DIMENSIONS, MATERIALS, LOADS, ALLOWABLE STRESSES, ETC.) PRESENTED IN THESE CONCEPT PLANS AND ARE INTENDED TO ASSIST THE DESIGN ENGINEER IN THE DEVELOPMENT OF A SET OF CONTRACT PLANS. THESE GUIDELINES SHALL NOT BE INTEREPRETED AS UNIVERSALLY APPLICABLE TO ANY DESIGN PROBLEM, NOR DO THEY RELIEVE THE ENGINEER OF RECORD OF ANY DUTIES PERTAINING TO THE RESPONSIBLE DESIGN OF THE TYPE OF BRIDGE FOR WHICH GUIDELINES HAVE BEEN PREPARED.

INDEX OF DRAWINGS

NOTES:

1. FOR GENERAL INFORMATION ON THESE GUIDELINES, SEE SHEET G1.

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STANDARD PREFABRICATED
CONCRETE SUBSTRUCTURE

GENERAL NOTES 1

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SHEET NUMBER A1

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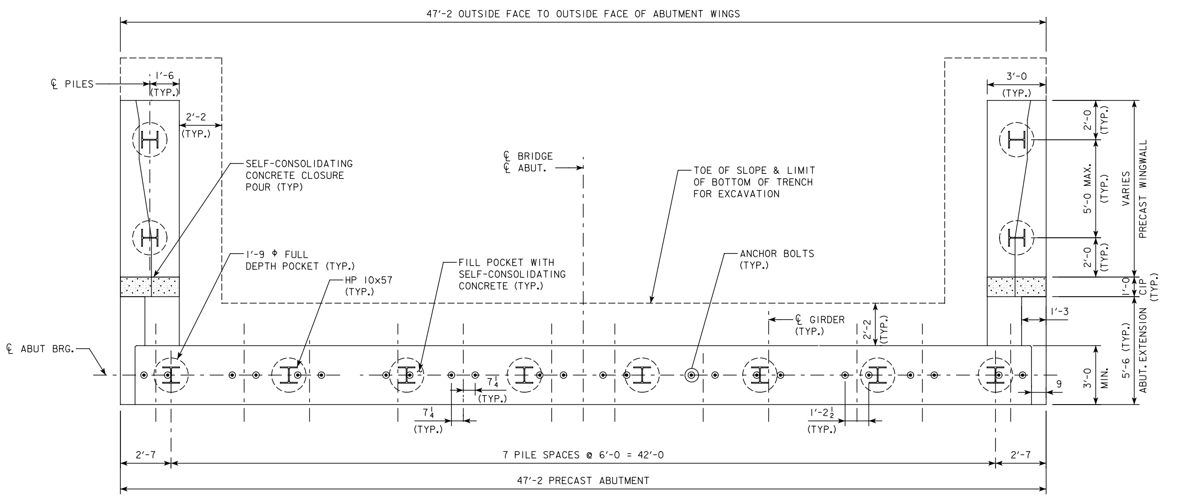
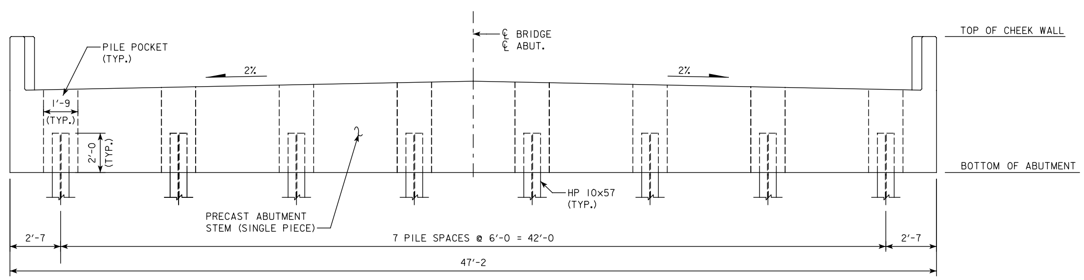


ABUTMENT PLAN AND PILE LAYOUT

ABUTMENT FABRICATION TOLERANCES	
LENGTH	+/- ¾"
WIDTH (OVERALL)	+/- ¼"
DEPTH (OVERALL)	+/- ¼"
VARIATION FROM SPECIFIED PLAN END SQUARENESS OR SKEW	+/- ⅛" PER 12 WIDTH +/- ½" MAXIMUM
VARIATION FROM SPECIFIED ELEVATION END SQUARENESS OR SKEW	+/- ⅛" PER 12 WIDTH +/- ½" MAXIMUM
SWEEP, FOR MEMBER LENGTH: UP TO 40 FEET 40 FEET TO 60 FEET	 +/- ¼" +/- ½"
LOCATION OF GROUTED SPLICE COUPLER MEASURED FROM A COMMON REFERENCE POINT	+/- ¼"
LOCAL SMOOTHNESS OF ANY SURFACE	+/- ¼" IN 10 FEET

NOTES:

1. FINAL PILE POSITIONS SHALL NOT DEVIATE FROM THE LOCATIONS DESIGNATED IN THESE PLANS BY MORE THAN 3" IN ANY DIRECTION.
2. FOR TYPICAL ABUTMENT SECTION, SEE SHEET A6.
3. USE 1'-9" GALVANIZED CORRUGATED METAL PIPE FOR PILE POCKETS.
4. GALVANIZED METAL PT DUCTS SHALL BE USED AT ALL ANCHOR BOLT LOCATIONS EXCEPT THOSE THAT FALL IN PILE POCKETS. THESE LOCATIONS SHALL BE FIELD DRILLED ONCE POCKET CONCRETE HAS ACHIEVED 3000 PSI COMPRESSIVE STRENGTH.
5. ABUTMENT AND WINGWALL DIMENSIONS ARE SHOWN FOR EXAMPLE BRIDGE AND SHOULD BE ADJUSTED TO FIT THE SIZE AS PART OF THE COMPLETE DESIGN FOR SITE CONDITIONS AND PROJECT REQUIREMENTS.
6. THESE PLANS ILLUSTRATE INDEPENDENTLY SUPPORTED U-WINGWALLS RIGIDLY ATTACHED TO THE ABUTMENT TO MAINTAIN ABUTMENT STABILITY. FLARED WINGWALLS ARE ALSO A FEASIBLE OPTION.
7. BEARING LOCATIONS SHOWN ARE FOR THE STEEL SUPERSTRUCTURE MODULE SYSTEM.



ABUTMENT ELEVATION

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STANDARD PREFABRICATED CONCRETE SUBSTRUCTURE

SEMI-INTEGRAL ABUTMENT PLAN & ELEVATION

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DESIGN TEAM

SHEET NUMBER A2

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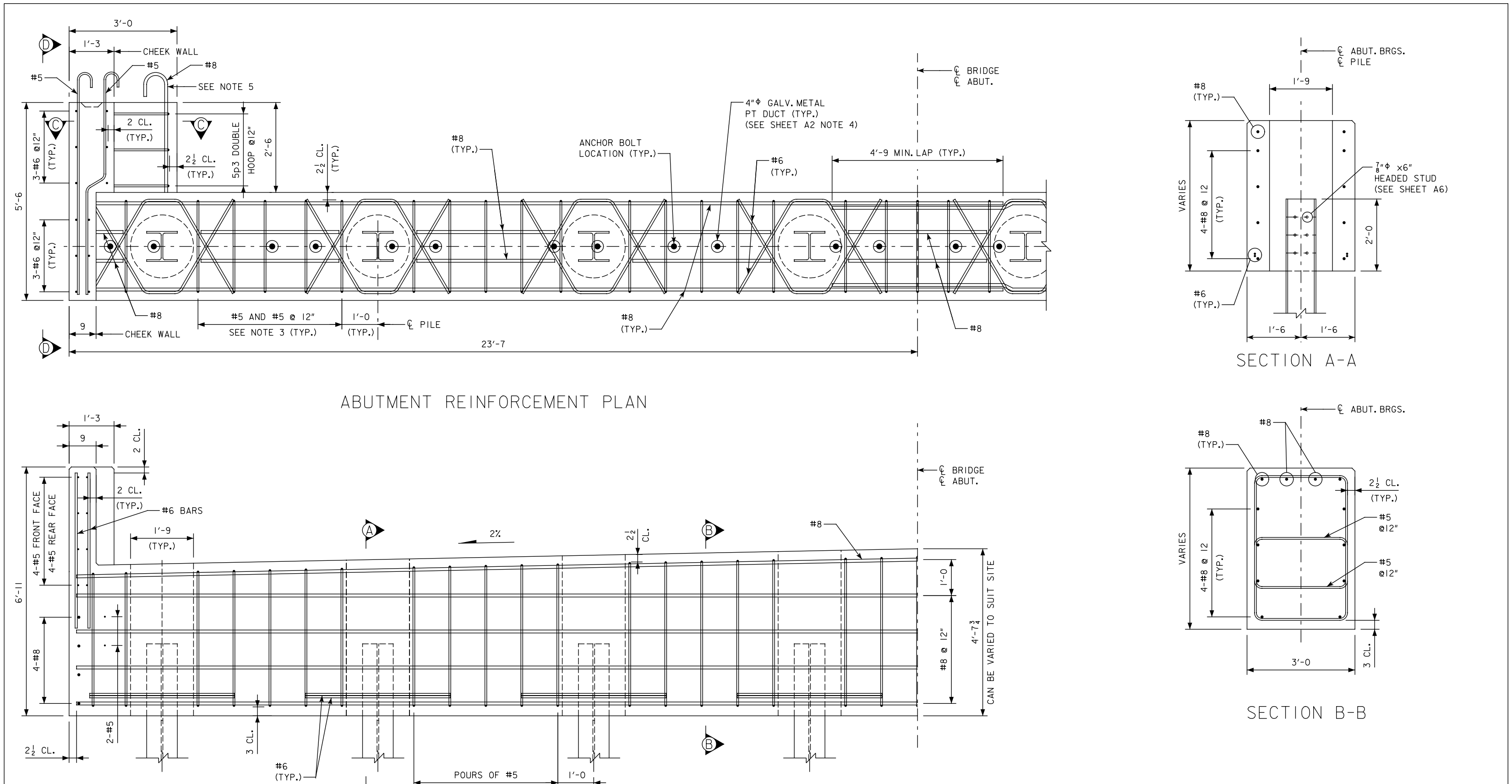

PARTIAL ABUTMENT ELEVATION

NOTE:

1. THE ESTIMATED WEIGHT OF THIS PRECAST ABUTMENT MODULE IS 93 KIPS.
2. FOR ABUTMENT PLAN AND ELEVATION AND PILE LAYOUT, SEE SHEET A2.
3. ABUTMENT BRIDGE SEAT HOOP BARS SHALL BE PLACED TO AVOID INTERFERENCE WITH PT DUCTS FOR BEARING ANCHOR BOLTS.
4. FOR SECTIONS C-C AND D-D, SEE SHEET A4.
5. THESE HOOKS MAY INTERFERE WITH THE HOOKS FROM THE WINGWALL UNIT DURING FIELD ASSEMBLY. USE OF THREADED INSERTS AND THREADED DOWELS MAY BE CONSIDERED.

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ABUT. REINFORCEMENT DETAILS

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DESIGN TEAM

SHEET NUMBER A3

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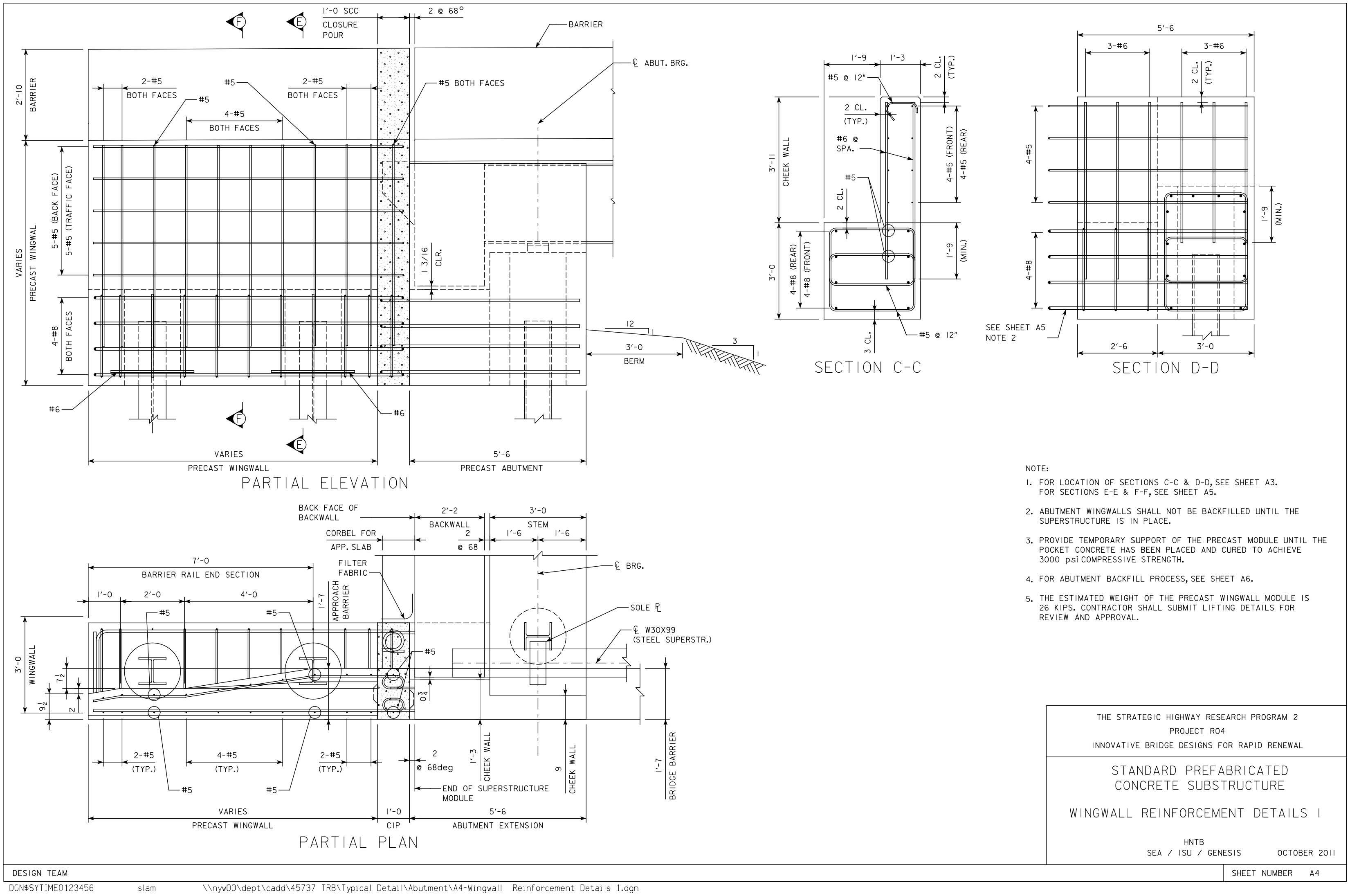
PARTIAL ELEVATION
1'-0 SCC CLOSURE POUR
2 @ 68°
BARRIER
℄ ABUT. BRG.
2-#5 BOTH FACES
#5
4-#5 BOTH FACES
#5 BOTH FACES
2'-10 BARRIER
VARIES PRECAST WINGWALL
5-#5 (BACK FACE) 5-#5 (TRAFFIC FACE)
4-#8 BOTH FACES
1 3/16 CLR.
#6
12
3'-0 BERM
3
1
VARIES PRECAST WINGWALL
5'-6 PRECAST ABUTMENT
SECTION C-C
1'-9
1'-3
2 CL. (TYP.)
#5 @ 12"
#6 @ SPA.
3'-11 CHEEK WALL
4-#5 (FRONT) 4-#5 (REAR)
3'-0
4-#8 (REAR) 4-#8 (FRONT)
1'-9 (MIN.)
3 CL.
#5 @ 12"
SECTION D-D
5'-6
3-#6
3-#6
4-#5
4-#8
1'-9 (MIN.)
SEE SHEET A5 NOTE 2
2'-6
3'-0
PARTIAL PLAN
BACK FACE OF BACKWALL
2'-2 BACKWALL
3'-0 STEM
1'-6
1'-6
CORBEL FOR APP. SLAB
2 @ 68
FILTER FABRIC
℄ BRG.
7'-0 BARRIER RAIL END SECTION
1'-0
2'-0
4'-0
1'-7 APPROACH BARRIER
SOLE ℄
℄ W30X99 (STEEL SUPERSTR.)
3'-0 WINGWALL
7½
9½
2
0¾
2-#5 (TYP.)
4-#5 (TYP.)
2-#5 (TYP.)
2 @ 68deg
1'-3 CHEEK WALL
9 CHEEK WALL
1'-7 BRIDGE BARRIER
END OF SUPERSTRUCTURE MODULE
VARIES PRECAST WINGWALL
1'-0 CIP
5'-6 ABUTMENT EXTENSION
NOTE:
1. FOR LOCATION OF SECTIONS C-C & D-D, SEE SHEET A3. FOR SECTIONS E-E & F-F, SEE SHEET A5.
2. ABUTMENT WINGWALLS SHALL NOT BE BACKFILLED UNTIL THE SUPERSTRUCTURE IS IN PLACE.
3. PROVIDE TEMPORARY SUPPORT OF THE PRECAST MODULE UNTIL THE POCKET CONCRETE HAS BEEN PLACED AND CURED TO ACHIEVE 3000 psi COMPRESSIVE STRENGTH.
4. FOR ABUTMENT BACKFILL PROCESS, SEE SHEET A6.
5. THE ESTIMATED WEIGHT OF THE PRECAST WINGWALL MODULE IS 26 KIPS. CONTRACTOR SHALL SUBMIT LIFTING DETAILS FOR REVIEW AND APPROVAL.
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
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WINGWALL REINFORCEMENT DETAILS I
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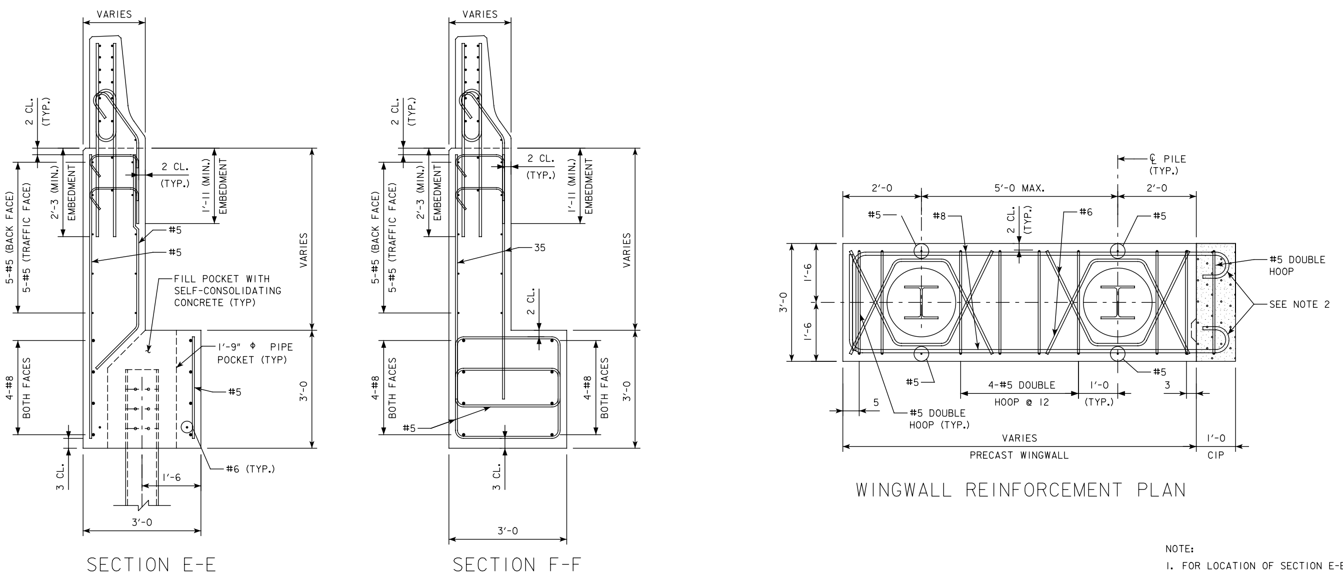

NOTE:

1. FOR LOCATION OF SECTION E-E & F-F, SEE SHEET A4.
2. THESE HOOKS MAY INTERFERE WITH THE HOOKS FROM THE PRECAST ABUTMENT DURING FIELD ASSEMBLY. USE OF THREADED INSERTS AND THREADED DOWELS MAY BE CONSIDERED.

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WINGWALL REINFORCEMENT DETAILS 2

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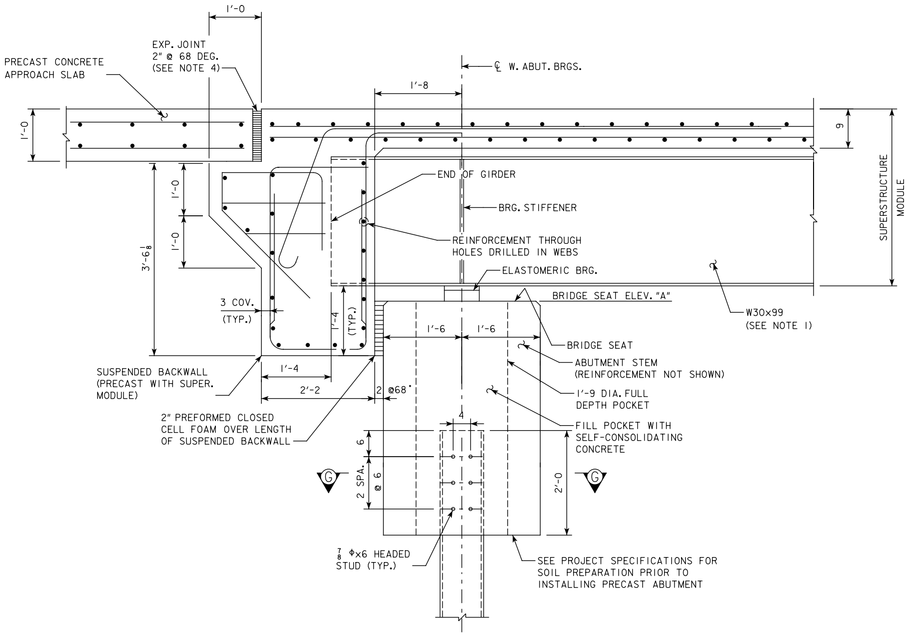
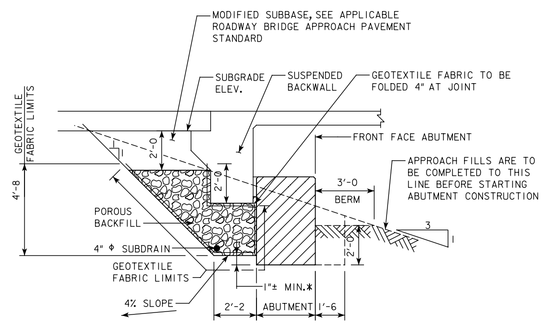


TYPICAL SEMI-INTEGRAL ABUTMENT SECTION WITH STEEL SUPERSTRUCTURE
(SUSPENDED BACKWALL)



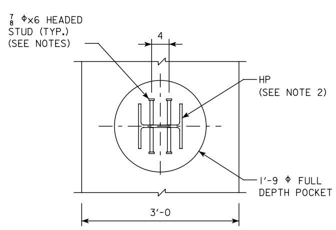
BACKFILL DETAILS

NOTE: GEOTEXTILE FABRIC WILL BE ATTACHED TO FACE OF ABUTMENT STEM AND WINGS

* DIMENISON VARIES DUE TO 2% SUBDRAIN SLOPE

NOTES:

1. SUSPENDED BACKWALL DETAIL SHOWN FOR A STEEL SUPERSTRUCTURE CAN ALSO BE USED WITH CONCRETE SUPERSTRUCTURES, SEE CONCRETE SUPERSTRUCTURE DETAILS.
2. H-PILE SIZE AND SPACING TO BE DESIGNED TO MEET SITE AND PROJECT REQUIREMENTS.
3. OPTION TO HEADED STUDS ON H-PILE SECTION IS 7/8"Φ THREADED ROD WITH NUTS. HOLES FOR ROD COULD BE FIELD DRILLED AFTER FINAL PILE DRIVING.
4. APPROACH SLAB MAY ALSO BE FIXED TO THE BACKWALL SEE SHEET A10. PROVISIONS FOR EXPANSION ARE INCLUDED AT THE SLEEPER SLAB.



SECTION G-G

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PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

STANDARD PREFABRICATED
CONCRETE SUBSTRUCTURE

SEMI-INTEGRAL
ABUTMENT SECTION

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DESIGN TEAM

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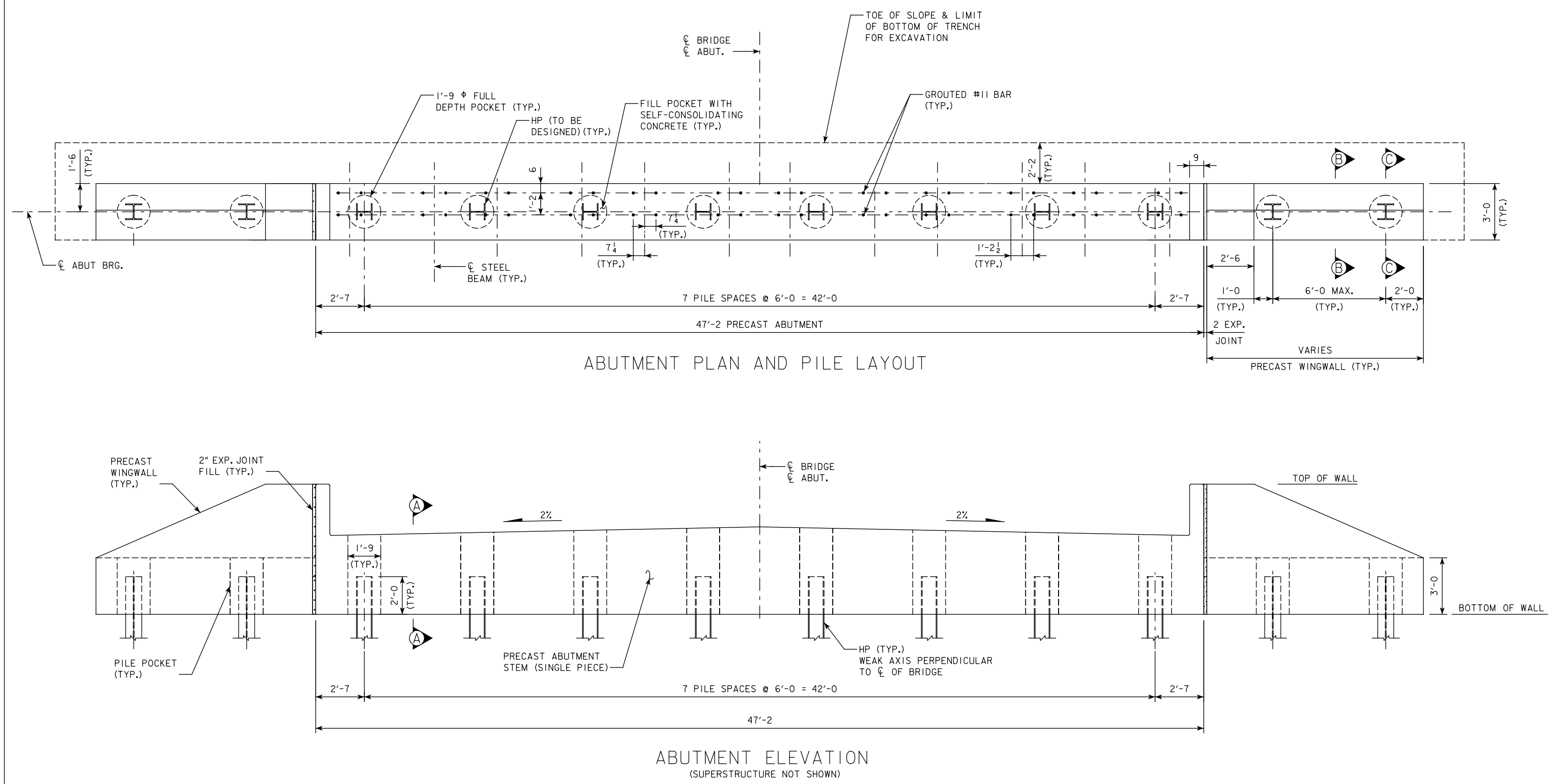

NOTE:

1. IN-LINE WINGWALL SHOWN. WINGWALLS MAY ALSO BE FLARED. LENGTH AND ORIENTATION OF WINGWALL TO BE ESTABLISHED BASED ON SITE REQUIREMENTS.
2. FOR SECTIONS A-A, B-B & C-C, SEE SHEET A8.
3. ABUTMENT AND WINGWALL DIMENSIONS ARE SHOWN FOR EXAMPLE BRIDGE AND SHOULD BE ADJUSTED TO FIT THE SITE AS PART OF THE COMPLETE DESIGN FOR SITE CONDITIONS AND PROJECT REQUIREMENTS.

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STANDARD PREFABRICATED CONCRETE SUBSTRUCTURE

INTEGRAL ABUTMENT PLAN & ELEVATION

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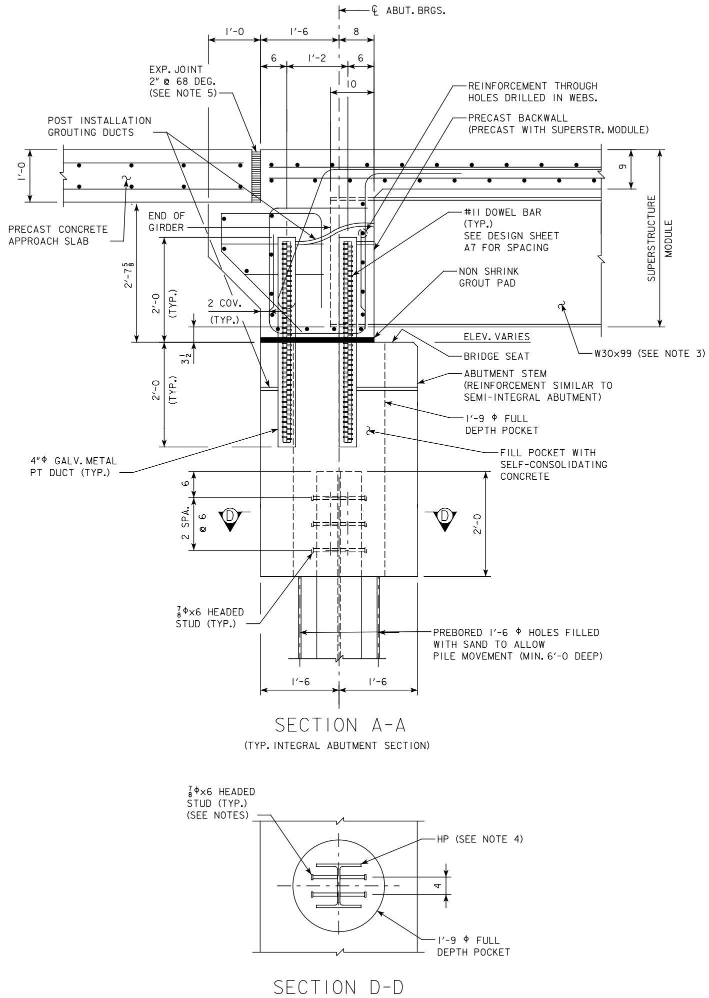
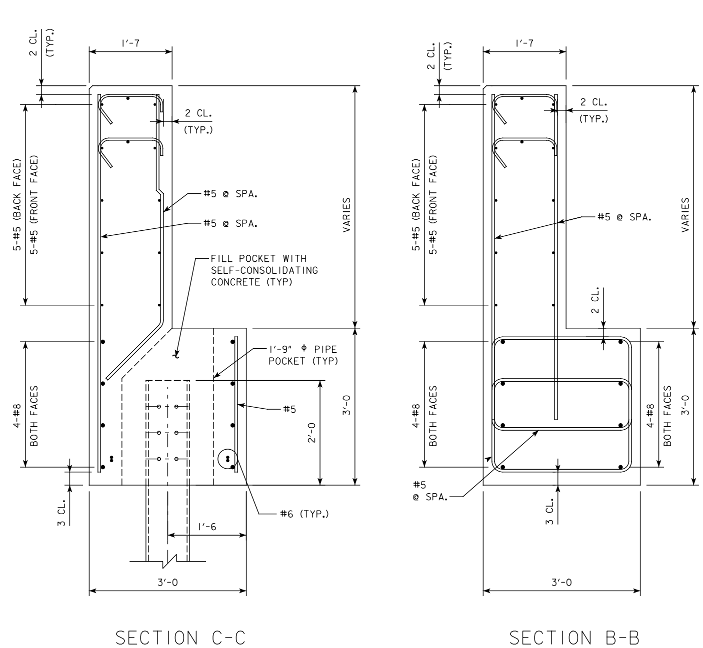


SECTION A-A

(TYP. INTEGRAL ABUTMENT SECTION)

SECTION D-D



SECTION C-C

SECTION B-B

NOTE:

1. FOR LOCATIONS OF SECTIONS B-B & C-C, SEE SHEET A7.
2. WINGWALL HEIGHT TO BE BASED ON SIGHT REQUIREMENTS.
3. SUSPENDED BACKWALL DETAIL SHOWN FOR A STEEL SUPERSTRUCTURE CAN ALSO BE USED WITH CONCRETE SUPERSTRUCTURES, SEE CONCRETE SUPERSTRUCTURE DETAILS.
4. H-PILE SIZE AND SPACING TO BE DESIGNED TO MEET SITE AND PROJECT REQUIREMENTS.
5. APPROACH SLAB MAY ALSO BE FIXED TO THE BACKWALL SEE SHEET A10. PROVISIONS FOR EXPANSION ARE INCLUDED AT THE SLEEPER SLAB.

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INTEGRAL
ABUTMENT SECTION

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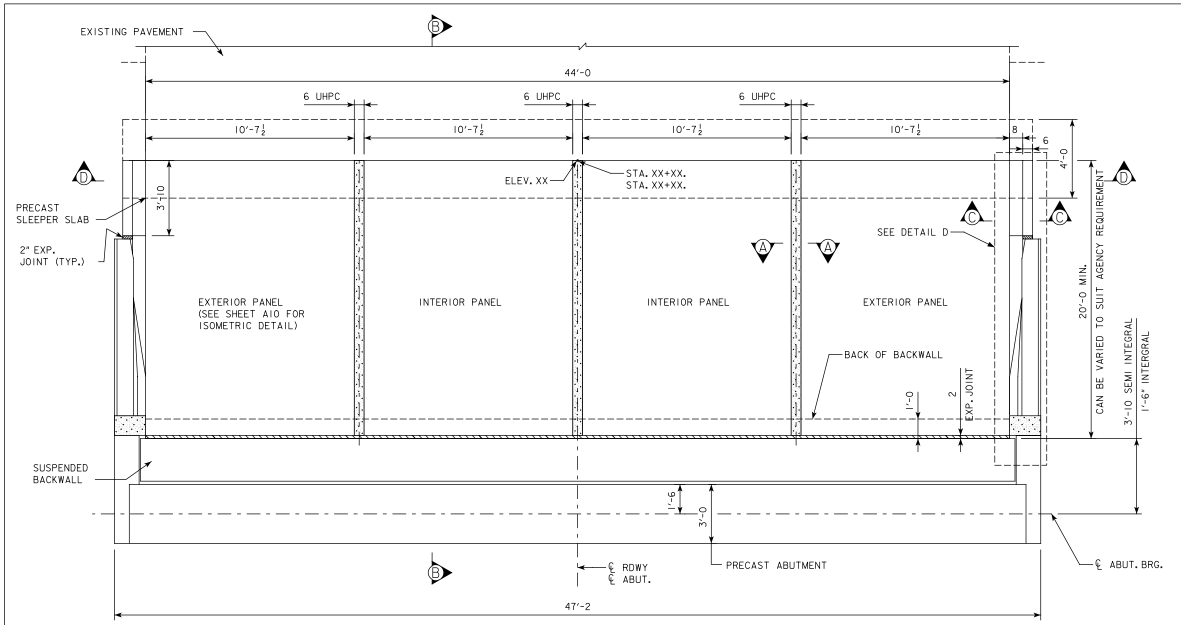
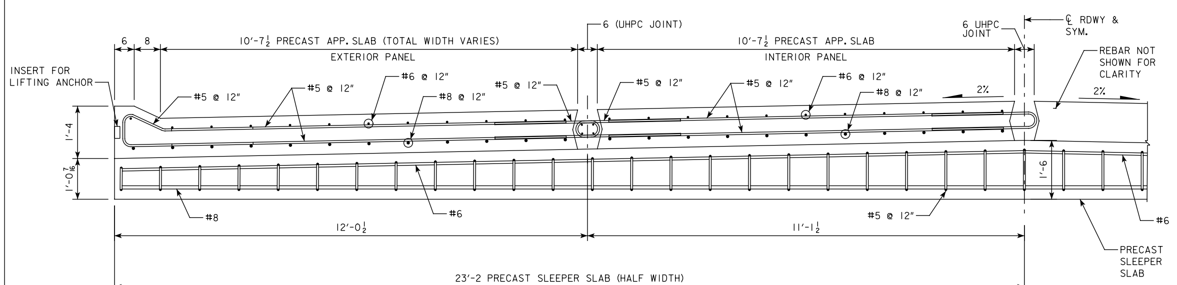


APPROACH SLAB PLAN

(SEMI-INTEGRAL ABUTMENT SHOWN, INTEGRAL ABUTMENT SIMILAR)



HALF-SECTION D-D

TOLERANCES FOR APPROACH SLABS	
LENGTH	+/- ¾"
WIDTH (OVERALL)	+/- ¼"
DEPTH (OVERALL)	+/- ¼"
VARIATION FROM SPECIFIED PLAN END SQUARENESS OR SKEW	+/- ⅛" PER 12 WIDTH +/- ½" MAXIMUM
VARIATION FROM SPECIFIED ELEVATION END SQUARENESS OR SKEW	+/- ⅛" PER 12 WIDTH +/- ½" MAXIMUM
SWEEP, FOR MEMBER LENGTH: UP TO 40 FEET 40 FEET TO 60 FEET	+/- ¼" +/- ½"
LOCATION OF GROUTED SPLICE COUPLER MEASURED FROM A COMMON REFERENCE POINT	+/- ¼"
LOCAL SMOOTHNESS OF ANY SURFACE	+/- ¼" IN 10 FEET
VARIATION FROM SPECIFIED CAMBER	+/- ⅛" PER 10 FEET +/- ¼" MAXIMUM

NOTES:

1. PRECAST APPROACH SLAB DETAILS SHOWN AS AN EXAMPLE ARE FOR THE SEMI-INTEGRAL ABUTMENT WITH U-WINGWALLS. PRECAST APPROACH SLABS CAN ALSO BE USED WITH INTEGRAL ABUTMENTS AND WINGWALLS WITH VARIED ORIENTATIONS.
2. FOR SECTION A-A, B-B, C-C & E-E AND DETAIL D, SEE SHEET A10.
3. USE #5 HAIRPIN BARS AS LIFTING POINTS WHEN AVAILABLE; OTHERWISE, INSERT LIFTING ANCHORS.
4. DETAILS SHOWN FOR U-TYPE WINGWALL SYSTEM. FOR OTHER SYSTEMS 4" SLOPE CURB ON APPROACH SLAB WOULD BE CONTINUED UP TO ABUTMENT.
5. THE CURB PIECE MAY BE DONE AS FIELD CLOSURE POUR USING A RAPID SET CONCRETE MIX.

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APPROACH SLAB 1

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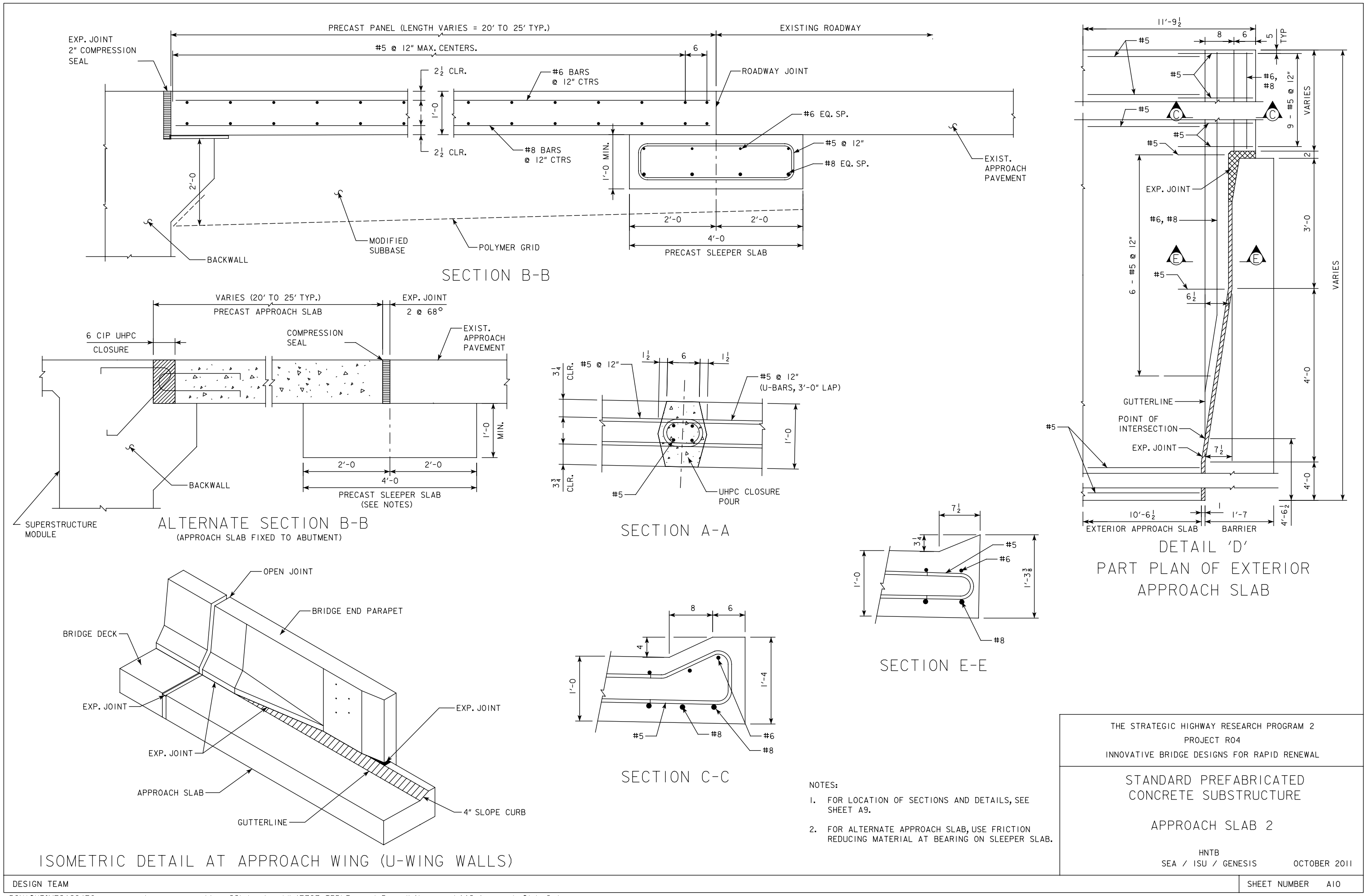

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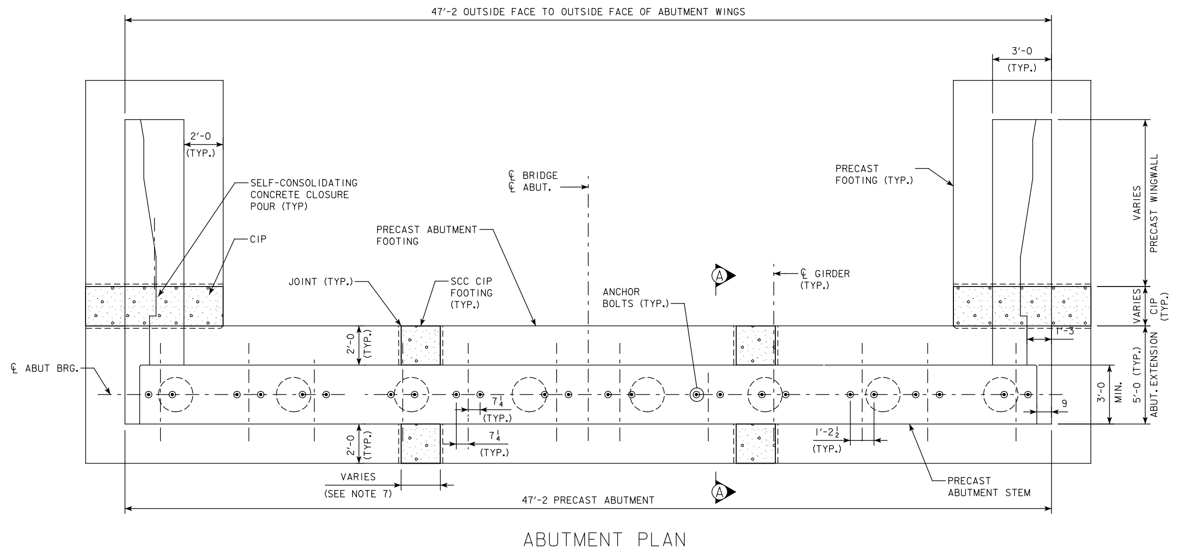
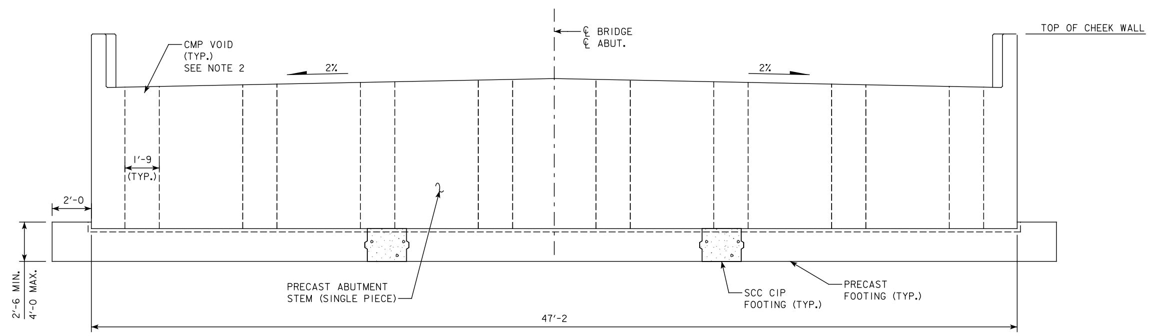


ABUTMENT PLAN



ABUTMENT ELEVATION

ABUTMENT FABRICATION TOLERANCES	
LENGTH	+/- 3/4"
WIDTH (OVERALL)	+/- 1/4"
DEPTH (OVERALL)	+/- 1/4"
VARIATION FROM SPECIFIED PLAN END SQUARENESS OR SKEW	+/- 1/8" PER 12 WIDTH +/- 1/2" MAXIMUM
VARIATION FROM SPECIFIED ELEVATION END SQUARENESS OR SKEW	+/- 1/8" PER 12 WIDTH +/- 1/2" MAXIMUM
SWEEP, FOR MEMBER LENGTH: UP TO 40 FEET 40 FEET TO 60 FEET	 +/- 1/4" +/- 1/2"
LOCATION OF GROUTED SPLICE COUPLER MEASURED FROM A COMMON REFERENCE POINT	+/- 1/4"
LOCAL SMOOTHNESS OF ANY SURFACE	+/- 1/4" IN 10 FEET

NOTES:

1. FOR TYPICAL ABUTMENT SECTION A-A, SEE SHEET A12.
2. USE 1'-9" GALVANIZED CORRUGATED METAL PIPE FOR VOIDS IN ABUTMENT STEM IF A LIGHTER WEIGHT IS DESIRED FOR TRANSPORT AND PICKING. FILL VOIDS WITH SELF CONSOLIDATING CONCRETE AFTER ERECTION.
3. GALVANIZED METAL PT DUCTS SHALL BE USED AT ALL ANCHOR BOLT LOCATIONS EXCEPT THOSE THAT FALL IN VOIDS. THESE LOCATIONS SHALL BE FIELD DRILLED ONCE VOID CONCRETE HAS ACHIEVED 3000 PSI COMPRESSIVE STRENGTH.
4. ABUTMENT AND WINGWALL DIMENSIONS ARE SHOWN FOR EXAMPLE BRIDGE AND SHOULD BE ADJUSTED TO FIT THE SITE AS PART OF THE COMPLETE DESIGN FOR SITE CONDITIONS AND PROJECT REQUIREMENTS.
5. THESE PLANS ILLUSTRATE INDEPENDENTLY SUPPORTED U-WINGWALLS RIGIDLY ATTACHED TO THE ABUTMENT TO MAINTAIN ABUTMENT STABILITY. FLARED WINGWALLS ARE ALSO A FEASIBLE OPTION.
6. BEARING LOCATIONS SHOWN ARE FOR THE STEEL SUPERSTRUCTURE MODULE SYSTEM.
7. LENGTH OF PRECAST FOOTING SECTIONS AND LENGTHS OF CIP SECTIONS TO BE BASED ON SHIPPING WEIGHT AND SIZE REQUIREMENTS. THE PRECAST SECTIONS SHALL BE ADEQUATE TO SUPPORT THE DEAD WEIGHT OF THE ABUTMENTS. THE CIP SECTIONS CAN BE COMPLETED TO SUPPORT THE TOTAL WEIGHT OF STRUCTURE AND LIVE LOAD.

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

STANDARD PREFABRICATED CONCRETE SUBSTRUCTURE SEMI-INTEGRAL ABUTMENT SPREAD FOOTING OPTION PLAN & ELEVATION

HNTB
SEA / ISU / GENESIS   OCTOBER 2011

DESIGN TEAM   SHEET NUMBER A11

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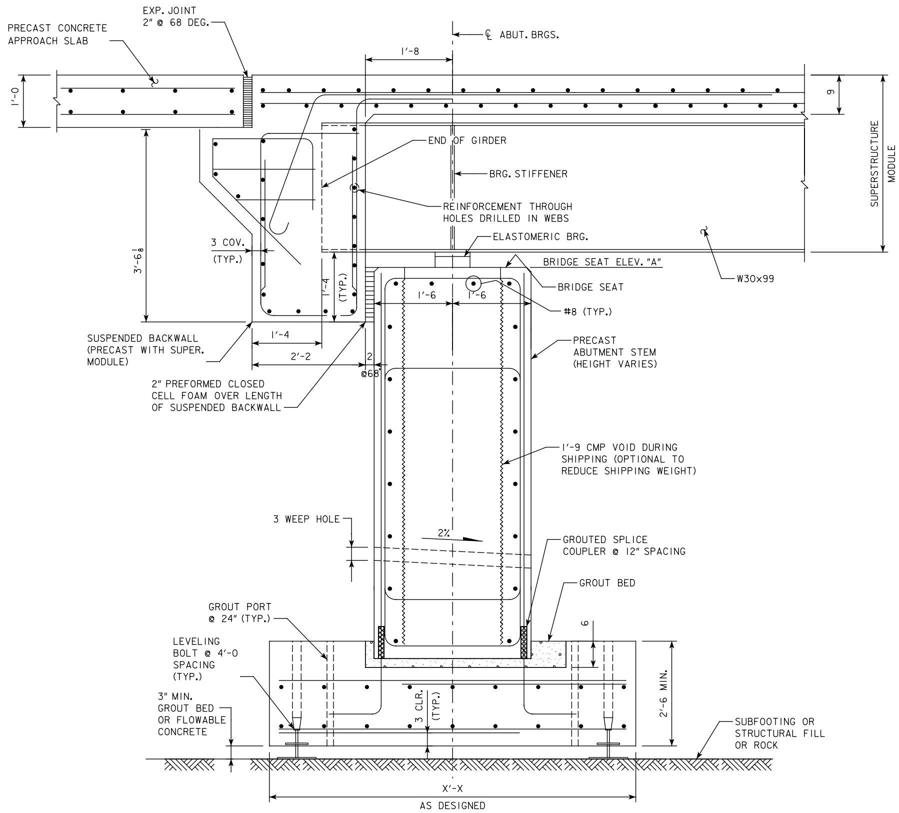
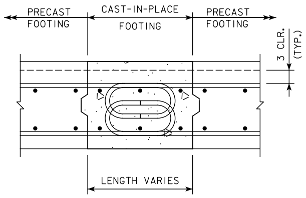


TYPICAL SEMI-INTEGRAL ABUTMENT SECTION WITH STEEL SUPERSTRUCTURE

(SUSPENDED BACKWALL)



CIP FOOTING SECTION DETAIL

NOTES:

1. FOOTING DIMENSIONS AND REINFORCEMENT TO BE DESIGNED BY THE ENGINEER OF RECORD TO MEET GEOTECHNICAL REQUIREMENTS FOR THE SITE.
2. SUSPENDED BACKWALL DETAIL SHOWN FOR A STEEL SUPERSTRUCTURE CAN ALSO BE USED WITH CONCRETE SUPERSTRUCTURES, SEE CONCRETE SUPERSTRUCTURE DETAILS.
3. TOP OF BEARING SURFACE SHALL BE APPROXIMATELY LEVEL. FINAL LEVELING OF PRECAST FOOTING SHALL BE CONTROLLED WITH FLOW THROUGH GROUT PORTS.
4. CIP FOOTING SECTION CAN BE USED TO CONNECT THE PRECAST UNITS AND ALSO REDUCE THE SIZE AND SHIPPING WEIGHTS.

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2

PROJECT R04

INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

STANDARD PREFABRICATED CONCRETE SUBSTRUCTURE

SPREAD FOOTING OPTION SECTION

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DESIGN TEAM

SHEET NUMBER A12

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GENERAL NOTES:

DIMENSIONS SHOWN ARE FOR AN EXAMPLE BRIDGE AND SHOULD BE ADJUSTED TO FIT THE SITE AS PART OF THE COMPLETE DESIGN FOR SITE CONDITIONS AND PROJECT REQUIREMENTS.

CAP CROSS-SLOPE

ENGINEER OF RECORD (EOR) SHALL CLOSELY EVALUATE AND ADJUST SLOPE OF TOP CAP TO ENSURE STABILITY OF SUPER-STRUCTURE DURING ERECTION AND FINAL CONDITION. CURRENT DRAWINGS SHOW ROADWAY SLOPE, BUT SHALL BE ADJUSTED AS PART OF THE COMPLETE DESIGN FOR SITE CONDITIONS AND PROJECT REQUIREMENTS.

PRECAST CONCRETE SUBSTRUCTURE

THE PRECAST FABRICATOR SHALL SUBMIT LIFTING LOCATIONS AND LIFTING ANCHOR DETAILS FOR APPROVAL BY ENGINEER PRIOR TO USE. THE TOP OF THE LIFTING ANCHORS SHALL BE RECESSED ½ INCH MINIMUM FROM THE SURFACE OF THE PRECAST MEMBER. THE LIFTING ANCHORS SHALL BE HOT-DIPPED GALVANIZED.

REMOVAL AND STORAGE

ALL PRECAST ELEMENTS SHALL BE REMOVED FROM THE FORMS IN SUCH A MANNER THAT NO DAMAGE OCCURS TO THE ELEMENT. ANY MATERIALS FORMING BLOCKOUTS IN THE PRECAST ELEMENTS SHALL BE REMOVED SUCH THAT DAMAGE DOES NOT OCCUR TO THE PRECAST ELEMENTS OR THE BLOCKOUT. PRECAST ELEMENTS SHALL BE STORED IN SUCH A MANNER THAT ADEQUATE SUPPORT IS PROVIDED TO PREVENT CRACKING OR CREEP-INDUCED DEFORMATION (SAGGING). DURING STORAGE FOR LONG PERIODS OF TIME (LONGER THAN ONE MONTH), ALL PRECAST ELEMENTS SHALL BE CHECKED AT LEAST ONCE PER MONTH TO ENSURE CREEP-INDUCED DEFORMATION DOES NOT OCCUR.

LIFTING AND HANDLING

ALL PRECAST ELEMENTS SHALL BE HANDLED IN SUCH A MANNER AS NOT TO DAMAGE THE PRECAST ELEMENTS DURING LIFTING OR MOVING. LIFTING ANCHORS CAST INTO THE PRECAST ELEMENTS SHALL BE USED FOR LIFTING AND MOVING THE PRECAST ELEMENTS AT THE FABRICATION PLANT AND IN THE FIELD. THE ANGLE BETWEEN THE TOP SURFACE OF THE PRECAST ELEMENTS AND THE LIFTING LINE SHALL NOT BE LESS THAN SIXTY DEGREES, WHEN MEASURED FROM THE TOP SURFACE OF THE PRECAST ELEMENTS TO THE LIFTING LINE. DAMAGE CAUSED TO ANY PRECAST ELEMENTS SHALL BE REPAIRED AT THE EXPENSE OF THE CONTRACTOR TO THE SATISFACTION OF THE ENGINEER.

TRANSPORTATION

ALL PRECAST ELEMENTS SHALL BE TRANSPORTED IN SUCH A MANNER THAT THE PRECAST ELEMENTS WILL NOT BE DAMAGED DURING TRANSPORTATION. PRECAST ELEMENTS SHALL BE PROPERLY SUPPORTED DURING TRANSPORTATION SUCH THAT CRACKING OR DEFORMATION (SAGGING) DOES NOT OCCUR. IF MORE THAN ONE PRECAST ELEMENTS IS TRANSPORTED PER VEHICLE, PROPER SUPPORT AND SEPARATION MUST BE PROVIDED BETWEEN THE INDIVIDUAL PRECAST ELEMENTS. PRECAST ELEMENTS SHALL LIE HORIZONTAL DURING TRANSPORTATION, UNLESS OTHERWISE APPROVED.

REPAIRS

REPAIRS OF DAMAGE CAUSED TO THE PRECAST ELEMENTS DURING FABRICATION, LIFTING AND HANDLING, OR TRANSPORTATION SHALL BE ADDRESSED ON A CASE-BY-CASE BASIS. DAMAGE WITHIN ACCEPTABLE LIMITS CAUSED TO THE KEYED EDGES OF THE PRECAST ELEMENTS SHALL BE REPAIRED USING MATERIALS AT THE FABRICATION PLANT AT THE EXPENSE OF THE FABRICATOR AND TO THE SATISFACTION OF THE ENGINEER. REPETITIVE DAMAGE TO PRECAST MEMBERS SHALL BE CAUSE FOR STOPPAGE OF FABRICATION OPERATIONS UNTIL THE CAUSE OF THE DAMAGE CAN BE REMEDIED. ALL PROPOSED REPAIRS SHALL BE APPROVED BY ENGINEER IN ADVANCE.

SKEWED STRUCTURES

THESE PLANS PRESENT A CONCEPT WELL-SUITED TO BRIDGES SUPPORTED ON BEARING LINES NORMAL TO THE CENTERLINE OF THE STRUCTURE. LOW TO MODERATE SKEWS CAN BE ACCOMMODATED WITH DUE CONSIDERATION GIVEN TO DESIGN, FABRICATION AND ERECTION.

MATERIAL PROPERTIES

PRECAST CONCRETE IN ACCORDANCE WITH AASHTO LRFD SECTION 5.

CONCRETE : HIGH PERFORMANCE (HPC) WITH A MINIMUM COMPRESSIVE STRENGTH f′c= 5,000 psi (28 DAY)

REINFORCING STEEL : GRADE 60.

CONCRETE COVER : 3″ ON ALL SURFACES IN GROUND CONTACT
2″ ALL OTHER SURFACES

SPECIFICATIONS

DESIGN : AASHTO LRFD 5TH EDITION, 2010.

DESIGN LIVE LOAD : HL-93; 25 psf FUTURE WEARING SURFACE.

CONSTRUCTION : AASHTO LRFD BRIDGE CONSTRUCTION SPECIFICATIONS, 5TH EDITION, WITH INTERIMS.

PRECAST PIER DETAILS SHOWN ARE SUITABLE FOR USE IN NON-SEISMIC AREAS.

DESIGN SPANS

CONVENTIONAL PIER

PIER CAP AND COLUMN DETAILS SHOWN HAVE BEEN DESIGNED FOR 70′, 100′ AND 130′ SPANS. THE ENGINEER OF RECORD COULD ADOPT THESE DESIGNS FOR PROJECT SPECIFIC SPANS.

STRADDLE BENT

BENT CAP AND COLUMN DETAILS DESIGNED FOR 70′ SPAN. THE ENGINEER OF RECORD COULD ADOPT THESE DESIGNS FOR PROJECT SPECIFIC SPANS.

DESIGN STRESSES

PRECAST PIER DETAILS SHOWN ARE SUITABLE FOR USE IN NON-SEISMIC AREAS.

GENERAL INSTALLATION NOTES

1. DRY FIT PRECAST ELEMENTS IN THE YARD PRIOR TO SHIPPING TO THE SITE.
2. DO NOT PLACE MODULES ON FOUNDATION UNTIL THE COMPRESSIVE TEST RESULTS OF THE CYLINDERS FOR THE FOUNDATION CONCRETE HAVE REACHED THE SPECIFIED MINIMUM VALUES.
3. SURVEY THE TOP ELEVATION OF THE FOUNDATION & COLUMNS. ESTABLISH WORKING POINTS, WORKING LINES, AND BENCHMARK ELEVATIONS PRIOR TO PLACEMENT OF ALL MODULES.
4. LIFT AND ERECT MODULES USING LIFTING DEVICES AS SHOWN ON THE SHOP DRAWINGS IN CONFORMANCE WITH THE ASSEMBLY PLANS.
5. SET MODULE IN THE PROPER LOCATION. SURVEY THE TOP ELEVATION OF THE MODULES. CHECK FOR PROPER ALIGNMENT AND GRADE WITHIN SPECIFIED TOLERANCE. APPROVED STEEL SHIMS OR NON-SHRINK GROUT SHALL BE USED BETWEEN THE RESPECTIVE MODULES TO COMPENSATE FOR MINOR DIFFERENCES IN ELEVATION BETWEEN MODULES.
6. TEMPORARILY SUPPORT, ANCHOR, AND BRACE ALL ERECTION MODULES AS NECESSARY FOR STABILITY AND TO RESIST WIND OR OTHER LOADS UNTIL THEY ARE PERMANENTLY SECURED TO THE STRUCTURE. SUPPORT ANCHOR, AND BRACE ALL MODULES AS DETAILED IN THE ASSEMBLY PLAN.

FOUNDATION NOTES

FOUNDATION DETAILS SHOWN ON P8 AND P9 ONLY DEPICT A FEW OF THE VARIOUS FOUNDATION TYPES AVAILABLE. ACTUAL FOUNDATION TYPE, SIZE, AND REINFORCING SHALL BE SELECTED BY ENGINEER OF RECORD (EOR) FOR PROJECT SITE CONDITIONS AND AS DIRECTED BY GEOTECHNICAL SOILS REPORT.

INDEX OF SHEETS

PRECAST PIER CONFIGURATIONS

THESE STANDARDS ILLUSTRATE TWO TYPES OF PIERS. THE FIRST IS A CONVENTIONAL PIER WITH AN OPTIONAL PLACEMENT OF COLUMNS TO ACHIEVE A MORE EFFICIENT DESIGN OF THE PIER CAP. SUCH A PIER CONFIGURATION WOULD USUALLY REQUIRE THE CONSTRUCTION OF THE COLUMN FOUNDATIONS BELOW THE EXISTING BRIDGE. DEPENDING ON THE SITE AND THE TYPE OF FOUNDATIONS REQUIRED THIS MIGHT POSE CERTAIN CHALLENGES, PARTICULARLY WHERE DEEP FOUNDATIONS ARE INVOLVED.

THE SECOND PIER TYPE ILLUSTRATES A PRECAST STRADDLE BENT. IN THIS TYPE THE COLUMNS ARE PLACED AT THE ENDS OF THE PIER CAP, WHICH ALLOW THE FOUNDATIONS TO BE BUILT OUTSIDE THE FOOTPRINT OF AN EXISTING BRIDGE, WHICH COULD BE BENEFICIAL FOR DRILLING DEEP FOUNDATIONS OR DRIVING PILES, WHILE THE EXSITING BRIDGE CONTINUES TO CARRY TRAFFIC.

PRECAST PIER CAP DESIGN

THE PIER CAPS SHOWN UTILIZE A REINFORCED CONCRETE SECTION WITHOUT ANY PRESTRESSING OR POST-TENSIONING. THIS WAS DONE SO THAT THE CONTRACTOR WILL HAVE THE OPTION OF SELF-PERFORMING THE PRECASTING AT A TEMPORARY CASTING YARD NEAR THE BRIDGE SITE USING HIS/HER OWN CREWS. THIS WOULD MINIMIZE TRANSPORTATION COST AND COULD ALSO REALIZE OTHER COST ADVANTAGES

ALTERNATIVELY, THE DESIGNER MAY CHOOSE A PRESTRESSED/POST-TENSIONED DESIGN FOR THE PIER CAP TO ACHIEVE A SECTION OF REDUCED SIZE AND WIEGHT WHERE SUCH CONSIDERATIONS ARE DEEMED CRITICAL FOR CONSTRUCTABILITY.

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
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STANDARD PREFABRICATED
SUPERSTRUCTURE
GENERAL NOTES

HNTB
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DESIGN TEAM

SHEET NUMBER P1

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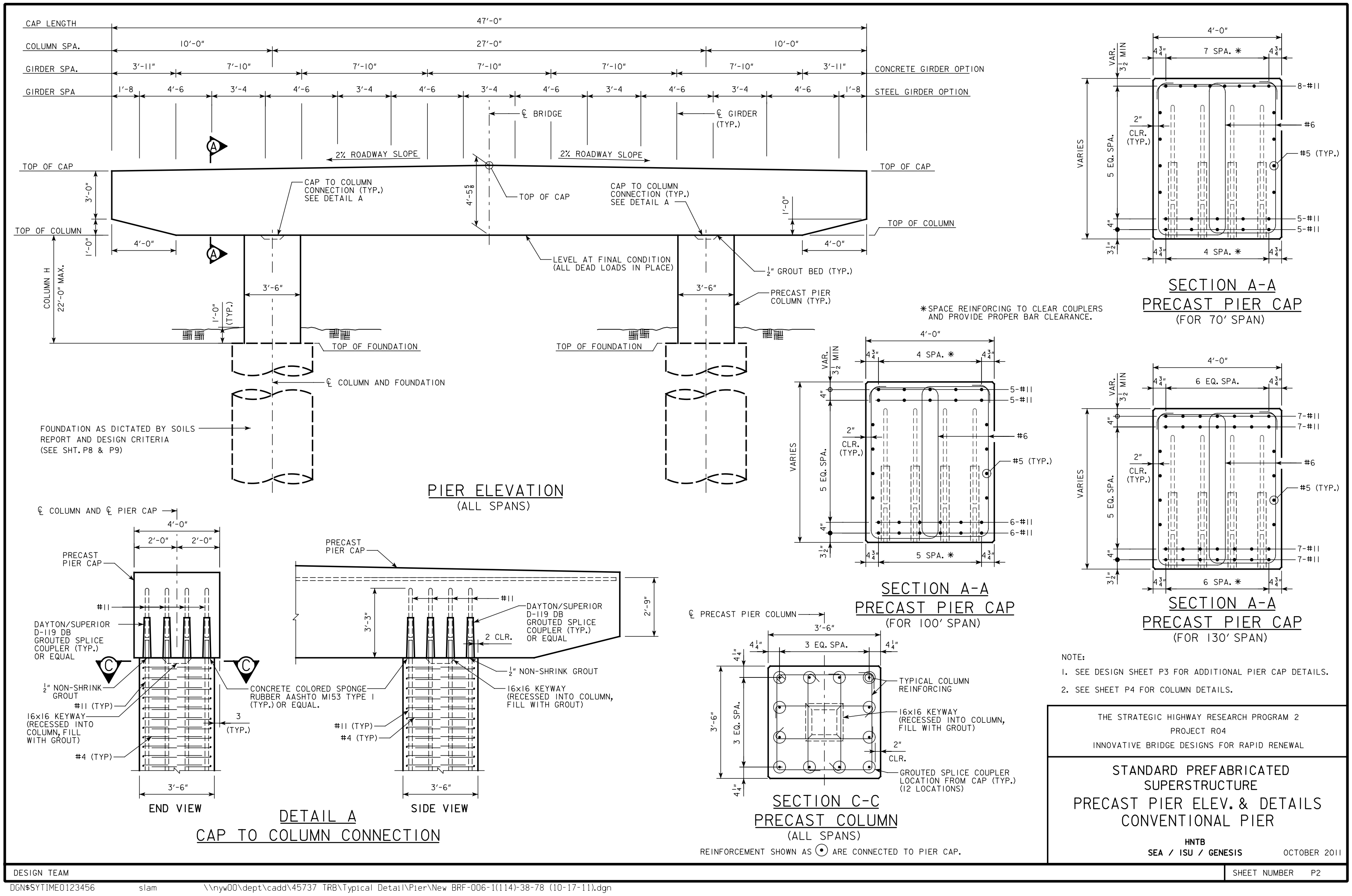
PIER ELEVATION
(ALL SPANS)
CAP LENGTH 47'-0"
COLUMN SPA. 10'-0" 27'-0" 10'-0"
GIRDER SPA. CONCRETE GIRDER OPTION
GIRDER SPA STEEL GIRDER OPTION
℄ BRIDGE
℄ GIRDER (TYP.)
2% ROADWAY SLOPE
TOP OF CAP
TOP OF COLUMN
CAP TO COLUMN CONNECTION (TYP.) SEE DETAIL A
LEVEL AT FINAL CONDITION (ALL DEAD LOADS IN PLACE)
½" GROUT BED (TYP.)
PRECAST PIER COLUMN (TYP.)
COLUMN H 22'-0" MAX.
TOP OF FOUNDATION
℄ COLUMN AND FOUNDATION
FOUNDATION AS DICTATED BY SOILS REPORT AND DESIGN CRITERIA (SEE SHT. P8 & P9)
DETAIL A
CAP TO COLUMN CONNECTION
END VIEW
SIDE VIEW
℄ COLUMN AND ℄ PIER CAP
PRECAST PIER CAP
DAYTON/SUPERIOR D-119 DB GROUTED SPLICE COUPLER (TYP.) OR EQUAL
½" NON-SHRINK GROUT
#11 (TYP)
16x16 KEYWAY (RECESSED INTO COLUMN, FILL WITH GROUT)
#4 (TYP)
CONCRETE COLORED SPONGE RUBBER AASHTO M153 TYPE I (TYP.) OR EQUAL.
SECTION C-C
PRECAST COLUMN
(ALL SPANS)
REINFORCEMENT SHOWN AS ⊙ ARE CONNECTED TO PIER CAP.
TYPICAL COLUMN REINFORCING
GROUTED SPLICE COUPLER LOCATION FROM CAP (TYP.) (12 LOCATIONS)
SECTION A-A
PRECAST PIER CAP
(FOR 70' SPAN)
SECTION A-A
PRECAST PIER CAP
(FOR 100' SPAN)
SECTION A-A
PRECAST PIER CAP
(FOR 130' SPAN)
*SPACE REINFORCING TO CLEAR COUPLERS AND PROVIDE PROPER BAR CLEARANCE.
NOTE:
1. SEE DESIGN SHEET P3 FOR ADDITIONAL PIER CAP DETAILS.
2. SEE SHEET P4 FOR COLUMN DETAILS.
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL
STANDARD PREFABRICATED SUPERSTRUCTURE
PRECAST PIER ELEV. & DETAILS
CONVENTIONAL PIER
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DESIGN TEAM
SHEET NUMBER P2
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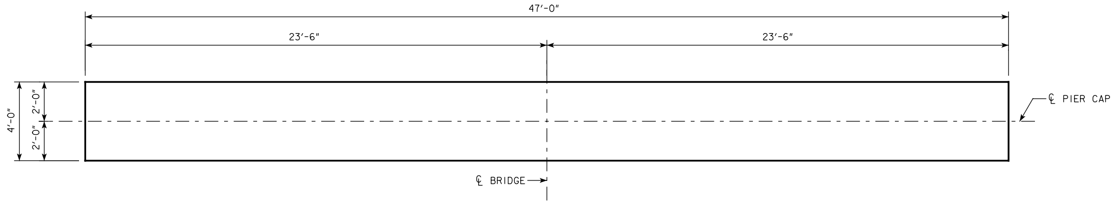
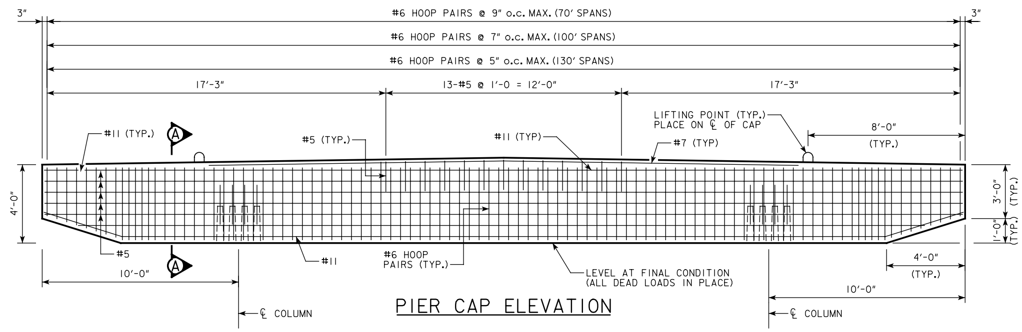
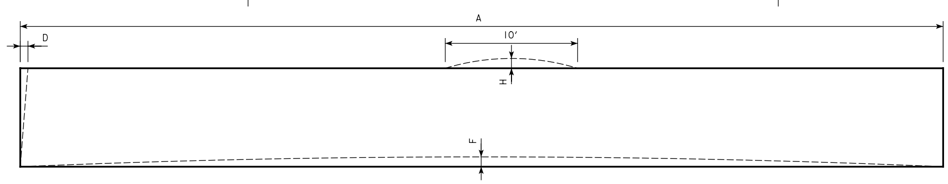


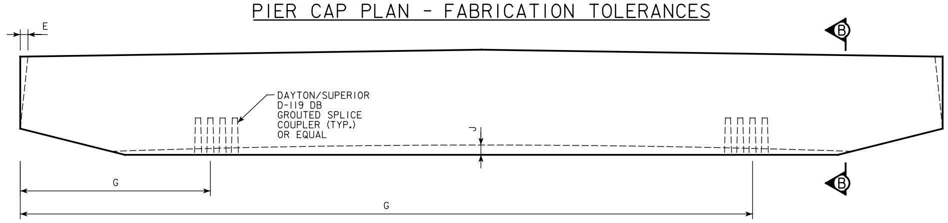
PIER CAP PLAN



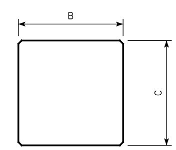
PIER CAP ELEVATION



PIER CAP PLAN - FABRICATION TOLERANCES



PIER CAP ELEVATION - FABRICATION TOLERANCES



SECTION B-B

PIER CAP FABRICATION TOLERANCES		
A	LENGTH	+/- 3/4"
B	WIDTH (OVERALL)	+/- 1/4"
C	DEPTH (OVERALL)	+/- 1/4"
D	VARIATION FROM SPECIFIED PLAN END SQUARENESS OR SKEW	+/- 1/8" PER 12 WIDTH +/- 1/2" MAXIMUM
E	VARIATION FROM SPECIFIED ELEVATION END SQUARENESS OR SKEW	+/- 1/8" PER 12 WIDTH +/- 1/2" MAXIMUM
F	SWEEP, FOR MEMBER LENGTH: UP TO 40 FEET 40 FEET TO 60 FEET	 +/- 1/4" +/- 1/2"
G	LOCATION OF GROUTED SPLICE COUPLER MEASURED FROM A COMMON REFERENCE POINT	+/- 1/4"
H	LOCAL SMOOTHNESS OF ANY SURFACE	+/- 1/4" IN 10 FEET
J	VARIATION FROM SPECIFIED CAMBER	+/- 1/8" PER 10 FEET +/- 1/4" MAXIMUM

CONC. PLACEMENT QUANT. PER CAP	
LOCATION	QUANTITY
HIGH PERFORMANCE STRUCTURAL CONCRETE, CAP	28.9
TOTAL (CU. YDS.)	28.9
TOTAL CAP WEIGHT (TONS)	58.5

NOTE:
SEE DESIGN SHEET P2 FOR SECTIONS A-A.

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
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PRECAST PIER CAP DETAILS
CONVENTIONAL PIER

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DESIGN TEAM

SHEET NUMBER P3

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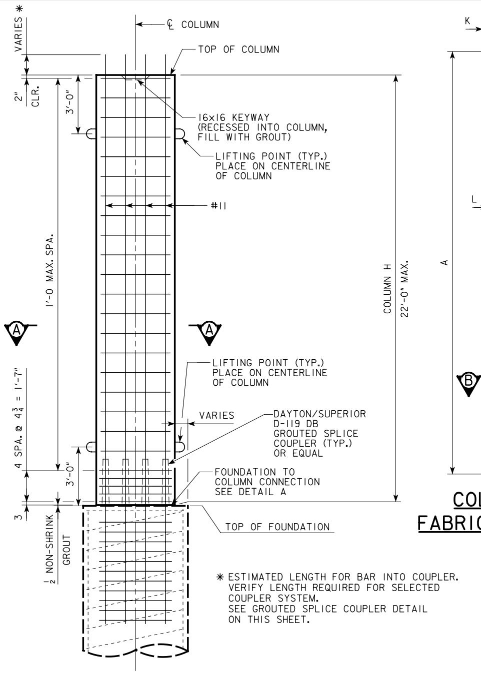
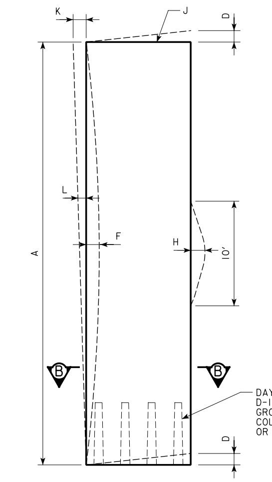
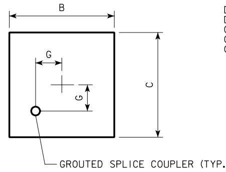


COLUMN ELEVATION
(ALL SPANS)

* ESTIMATED LENGTH FOR BAR INTO COUPLER. VERIFY LENGTH REQUIRED FOR SELECTED COUPLER SYSTEM. SEE GROUTED SPLICE COUPLER DETAIL ON THIS SHEET.



COLUMN ELEVATION
FABRICATION TOLERANCES



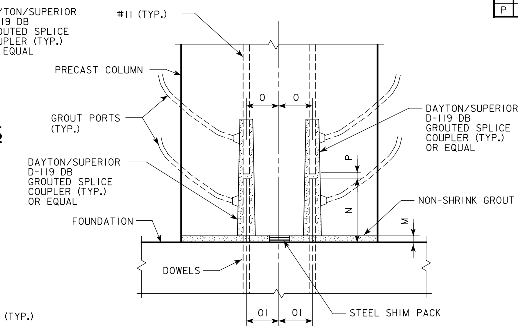
SECTION B-B

CONC. PLACEMENT QUANT. PER COLUMN	
LOCATION	QUANTITY
HIGH PERFORMANCE STRUCTURAL CONCRETE, COLUMN (22')	10.0
TOTAL (CU. YDS.)	10.0
TOTAL COLUMN WEIGHT (TONS)	20.3

COLUMN FABRICATION TOLERANCES		
A	LENGTH	+/- $\frac{1}{2}$"
B	WIDTH (OVERALL)	+/- $\frac{1}{4}$"
C	DEPTH (OVERALL)	+/- $\frac{1}{4}$"
D	VARIATION FROM SPECIFIED END SQUARENESS OR SKEW	+/- $\frac{1}{8}$" PER 12 WIDTH +/- $\frac{3}{8}$" MAXIMUM
F	SWEEP, FOR MEMBER LENGTH:	+/- $\frac{1}{8}$" PER 10 FEET +/- $\frac{1}{2}$" MAXIMUM
G	LOCATION OF GROUTED SPLICE COUPLER MEASURED FROM A COMMON REFERENCE POINT	+/- $\frac{1}{4}$"
H	LOCAL SMOOTHNESS OF ANY SURFACE	+/- $\frac{1}{4}$" IN 10 FEET

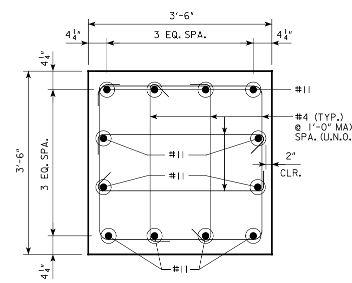
COLUMN ERECTION TOLERANCES		
J	TOP ELEVATION FROM NOMINAL TOP ELEVATION MAXIMUM LOW MAXIMUM HIGH	 $\frac{1}{2}$" $\frac{1}{4}$"
K	MAXIMUM PLUMB VARIATION OVER HEIGHT OF COLUMN	$\frac{1}{2}$"
L	PLUMB IN ANY 10 FEET OF COLUMN HEIGHT	$\frac{1}{4}$"

GROUTED SPLICE COUPLER TOLERANCES		
M	SHIM PACK HEIGHT	$\frac{3}{4}$ +/- $\frac{3}{8}$
N	DOWEL HEIGHT	CONSULT MANUFACTURER
O OI	LOCATION OF COLUMN REINFORCING, GROUTED SPLICE COUPLER, AND FOUNDATION DOWELS MEASURED FROM A COMMON REFERENCE POINT	+/- $\frac{1}{4}$
P	GAP BETWEEN DOWELS AND COLUMN REINFORCING	CONSULT MANUFACTURER



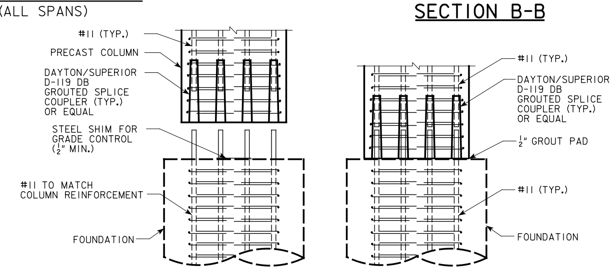
GROUTED SPLICE COUPLER DETAIL

NOTE:
1. USE MATCHING TEMPLATES FOR THE LOCATION OF COLUMN REINFORCEMENT AND GROUTED SPLICE COUPLER PLACEMENT WITHIN THE ELEMENT TO CONTROL CRITICAL DIMENSIONS "O" AND "OI", WHICH WOULD BE IDENTICAL.
2. CONSULT MANUFACTURER OF THE GROUTED SPLICE COUPLER FOR PROPER DIMENSIONS "N" AND "P" AND FOR TOLERANCE ON THESE DIMENSIONS.
3. BEFORE EXECUTING GROUTED SPLICE COUPLER ASSEMBLIES, ALWAYS SEEK INSTALLATION RECOMMENDATIONS FROM THE MANUFACTURER OF THE GROUTED SPLICE COUPLER USED.



SECTION A-A
PRECAST COLUMN
(ALL SPANS)

NOTE:
REINFORCEMENT SHOWN AS ⊙ ARE CONNECTED TO DRILLED SHAFT, OR FOOTING.



DETAIL A
FOUNDATION TO COLUMN CONNECTION

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STANDARD PREFABRICATED SUPERSTRUCTURE
PRECAST COLUMN DETAILS
CONVENTIONAL PIER

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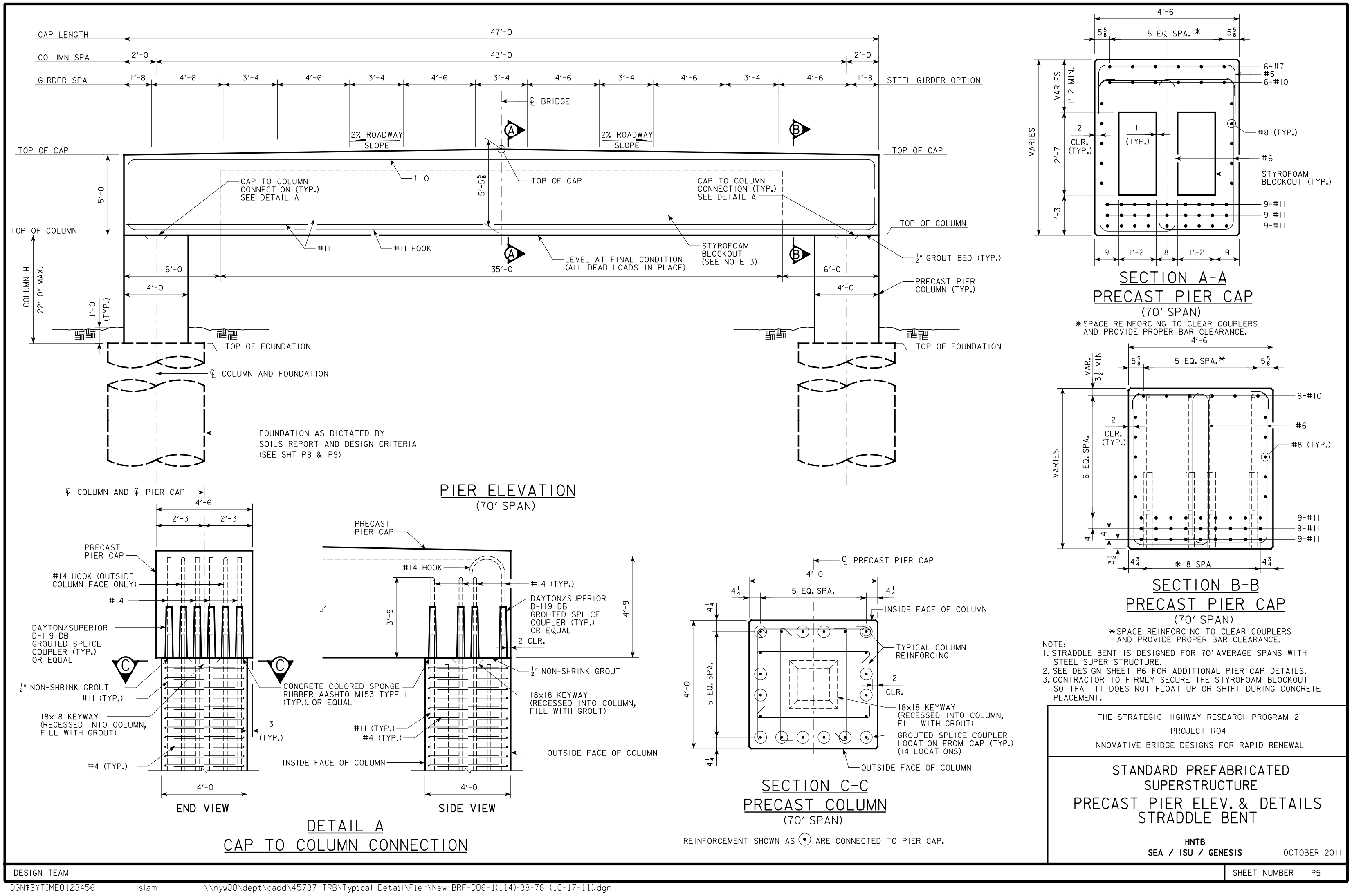
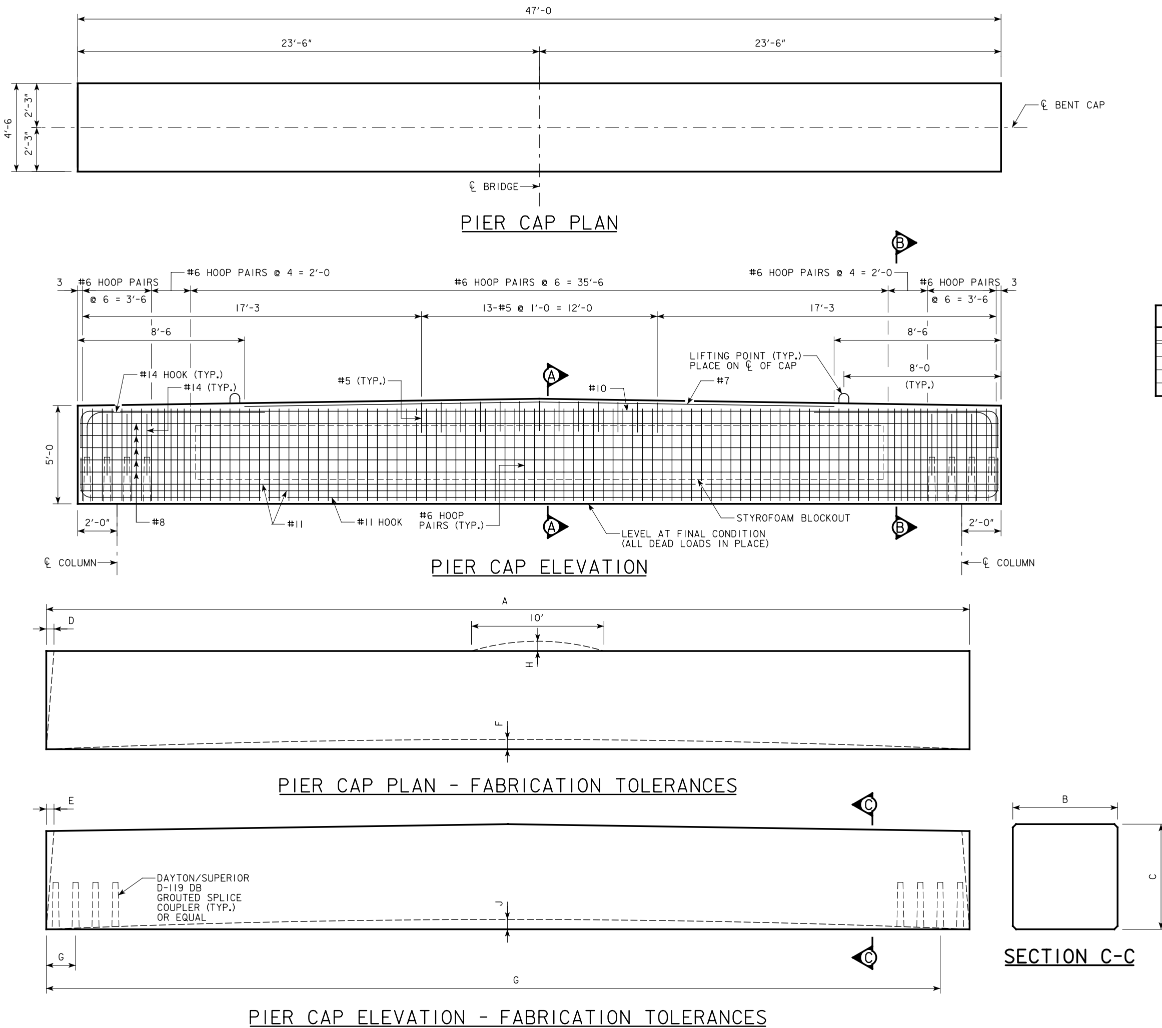

PIER ELEVATION
(70' SPAN)
SECTION A-A
PRECAST PIER CAP
(70' SPAN)
*SPACE REINFORCING TO CLEAR COUPLERS AND PROVIDE PROPER BAR CLEARANCE.
SECTION B-B
PRECAST PIER CAP
(70' SPAN)
*SPACE REINFORCING TO CLEAR COUPLERS AND PROVIDE PROPER BAR CLEARANCE.
SECTION C-C
PRECAST COLUMN
(70' SPAN)
REINFORCEMENT SHOWN AS ⊙ ARE CONNECTED TO PIER CAP.
END VIEW
SIDE VIEW
DETAIL A
CAP TO COLUMN CONNECTION
NOTE:
1. STRADDLE BENT IS DESIGNED FOR 70' AVERAGE SPANS WITH STEEL SUPER STRUCTURE.
2. SEE DESIGN SHEET P6 FOR ADDITIONAL PIER CAP DETAILS.
3. CONTRACTOR TO FIRMLY SECURE THE STYROFOAM BLOCKOUT SO THAT IT DOES NOT FLOAT UP OR SHIFT DURING CONCRETE PLACEMENT.
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL
STANDARD PREFABRICATED SUPERSTRUCTURE
PRECAST PIER ELEV. & DETAILS STRADDLE BENT
HNTB
SEA / ISU / GENESIS
OCTOBER 2011
DESIGN TEAM
SHEET NUMBER P5



PIER CAP FABRICATION TOLERANCES

A	LENGTH	+/- 3/4"
B	WIDTH (OVERALL)	+/- 1/4"
C	DEPTH (OVERALL)	+/- 1/4"
D	VARIATION FROM SPECIFIED PLAN END SQUARENESS OR SKEW	+/- 1/8" PER 12 WIDTH +/- 1/2" MAXIMUM
E	VARIATION FROM SPECIFIED ELEVATION END SQUARENESS OR SKEW	+/- 1/8" PER 12 WIDTH +/- 1/2" MAXIMUM
F	SWEEP, FOR MEMBER LENGTH: UP TO 40 FEET 40 FEET TO 60 FEET	 +/- 1/4" +/- 1/2"
G	LOCATION OF GROUTED SPLICE COUPLER MEASURED FROM A COMMON REFERENCE POINT	+/- 1/4"
H	LOCAL SMOOTHNESS OF ANY SURFACE	+/- 1/4" IN 10 FEET
J	VARIATION FROM SPECIFIED CAMBER	+/- 1/8" PER 10 FEET +/- 1/4" MAXIMUM

CONC. PLACEMENT QUANT. PER CAP

LOCATION	QUANTITY
HIGH PERFORMANCE STRUCTURAL CONCRETE, CAP	33.2
TOTAL (CU. YDS.)	33.2
TOTAL CAP WEIGHT (TONS)	67.2

NOTE:
1. STRADDLE BENT IS DESIGNED FOR 70' AVERAGE SPANS WITH STEEL SUPER STRUCTURE.
2. SEE DESIGN SHEET P5 FOR SECTIONS A-A & B-B.

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
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STANDARD PREFABRICATED SUPERSTRUCTURE
PRECAST PIER CAP DETAILS
STRADDLE BENT

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DESIGN TEAM

SHEET NUMBER P6

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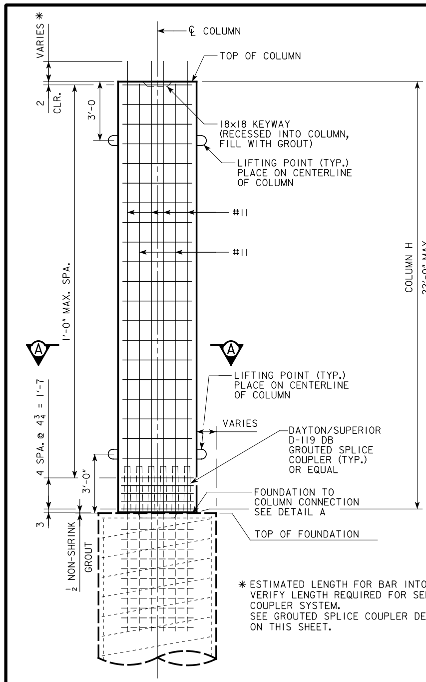
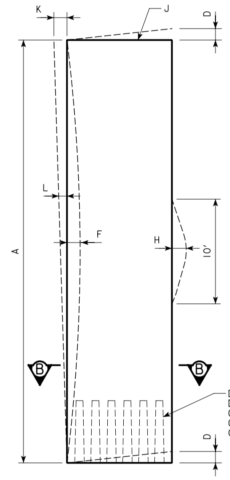
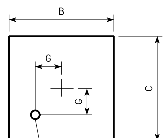


* ESTIMATED LENGTH FOR BAR INTO COUPLER. VERIFY LENGTH REQUIRED FOR SELECTED COUPLER SYSTEM. SEE GROUTED SPLICE COUPLER DETAIL ON THIS SHEET.

COLUMN ELEVATION
(INSIDE FACE OF COLUMN SHOWN)



COLUMN ELEVATION
FABRICATION TOLERANCES



SECTION B-B

CONC. PLACEMENT QUANT. PER COLUMN

LOCATION	QUANTITY
HIGH PERFORMANCE STRUCTURAL CONCRETE, COLUMN (22')	13.0
TOTAL (CU. YDS.)	13.0
TOTAL COLUMN WEIGHT (TONS)	26.4

COLUMN FABRICATION TOLERANCES

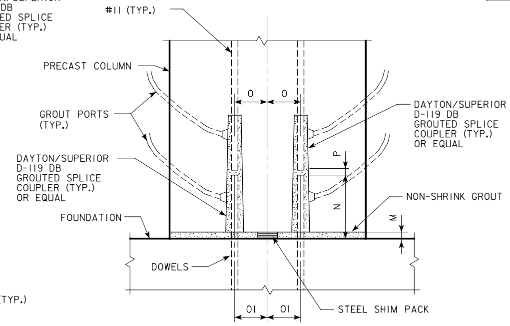
A	LENGTH	+/- ½"
B	WIDTH (OVERALL)	+/- ¼"
C	DEPTH (OVERALL)	+/- ¼"
D	VARIATION FROM SPECIFIED END SQUARENESS OR SKEW	+/- ⅛" PER 12 WIDTH +/- ⅜" MAXIMUM
F	SWEEP, FOR MEMBER LENGTH:	+/- ⅛" PER 10 FEET +/- ½" MAXIMUM
G	LOCATION OF GROUTED SPLICE COUPLER MEASURED FROM A COMMON REFERENCE POINT	+/- ¼"
H	LOCAL SMOOTHNESS OF ANY SURFACE	+/- ¼" IN 10 FEET

COLUMN ERECTION TOLERANCES

J	TOP ELEVATION FROM NOMINAL TOP ELEVATION MAXIMUM LOW MAXIMUM HIGH	 ½" ¼"
K	MAXIMUM PLUMB VARIATION OVER HEIGHT OF COLUMN	½"
L	PLUMB IN ANY 10 FEET OF COLUMN HEIGHT	¼"

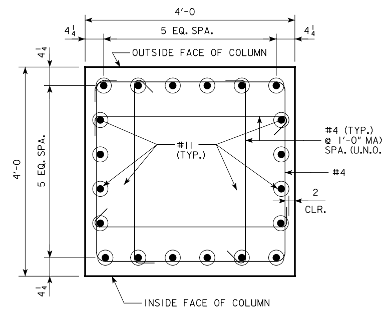
GROUTED SPLICE COUPLER TOLERANCES

M	SHIM PACK HEIGHT	¾ +/- ⅜
N	DOWEL HEIGHT	CONSULT MANUFACTURER
O O1	LOCATION OF COLUMN REINFORCING, GROUTED SPLICE COUPLER, AND FOUNDATION DOWELS MEASURED FROM A COMMON REFERENCE POINT	+/- ¼
P	GAP BETWEEN DOWELS AND COLUMN REINFORCING	CONSULT MANUFACTURER



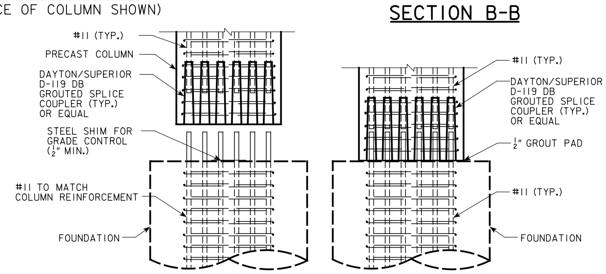
GROUTED SPLICE COUPLER DETAIL

NOTE:
1. USE MATCHING TEMPLATES FOR THE LOCATION OF COLUMN REINFORCEMENT AND GROUTED SPLICE COUPLER PLACEMENT WITHIN THE ELEMENT TO CONTROL CRITICAL DIMENSIONS "O" AND "O1", WHICH WOULD BE IDENTICAL.
2. CONSULT MANUFACTURER OF THE GROUTED SPLICE COUPLER FOR PROPER DIMENSIONS "N" AND "P" AND FOR TOLERANCE ON THESE DIMENSIONS.
3. BEFORE EXECUTING GROUTED SPLICE COUPLER ASSEMBLIES, ALWAYS SEEK INSTALLATION RECOMMENDATIONS FROM THE MANUFACTURER OF THE GROUTED SPLICE COUPLER USED.



SECTION A-A
PRECAST COLUMN

NOTE:
REINFORCEMENT SHOWN AS ⊙ ARE CONNECTED TO DRILLED SHAFT, OR FOOTING.



DETAIL A
FOUNDATION TO COLUMN CONNECTION

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STANDARD PREFABRICATED SUPERSTRUCTURE
PRECAST COLUMN DETAILS STRADDLE BENT

HNTB
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OCTOBER 2011

DESIGN TEAM
SHEET NUMBER P7

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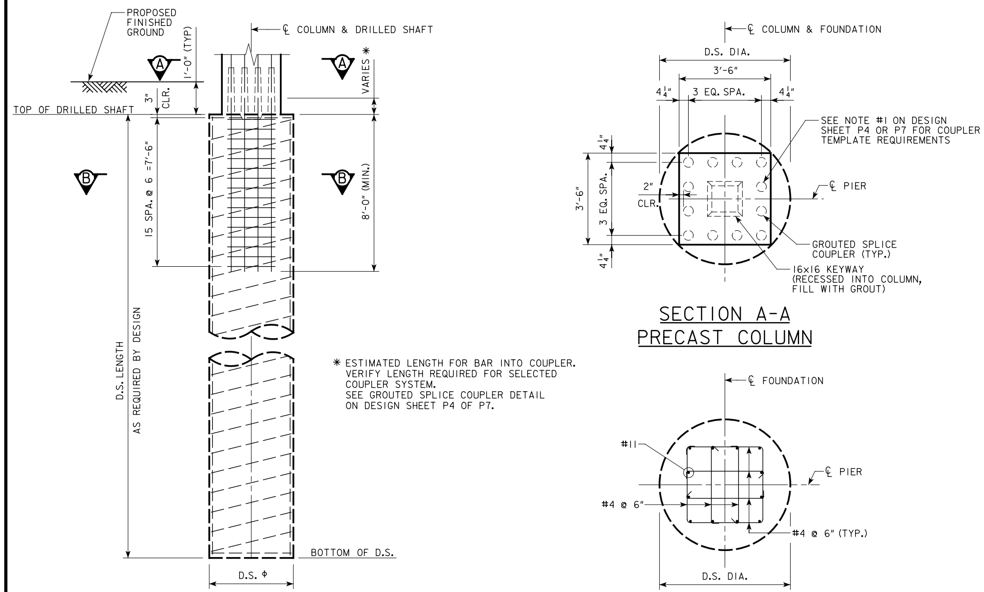

SECTION A-A
PRECAST COLUMN

SECTION B-B

DRILLED SHAFT ELEVATION
(SHOWING CONVENTIONAL PIER COLUMN REINFORCING)

NOTE:

1. THIS SHEET SHOWS A DRILLED SHAFT FOUNDATION AS ONE SAMPLE DEEP FOUNDATION TYPE. OTHER DEEP FOUNDATION TYPES ARE EQUALLY APPLICABLE. THE GEOTECHNICAL DESIGN FOR THE SITE WILL IDENTIFY THE MOST SUITABLE FOUNDATION SYSTEM.

2. FOUNDATION TYPE AND REINFORCING SHALL BE DETERMINED BY ENGINEER OF RECORD FOR SITE CONDITIONS AND AS DIRECTED BY SOILS REPORT.

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STANDARD PREFABRICATED SUPERSTRUCTURE
FOUNDATION DETAILS
DRILLED SHAFT

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DESIGN TEAM | SHEET NUMBER P8

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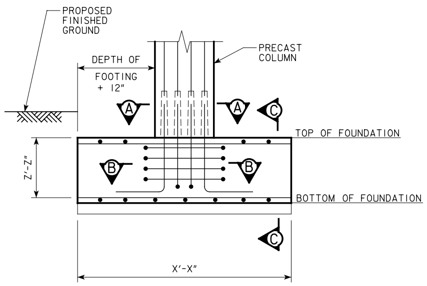
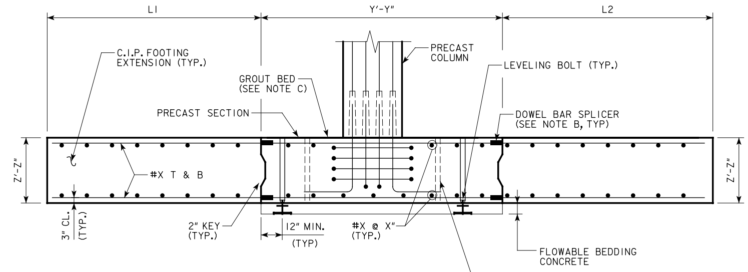


TYPICAL COLUMN PIER FRONT ELEVATION

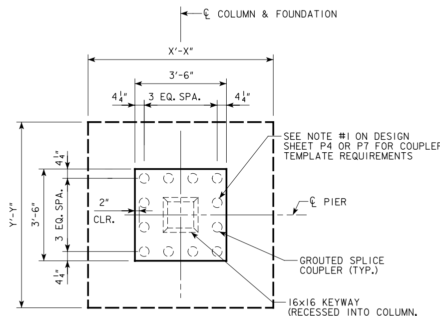
(SHOWING CONVENTIONAL PIER REINFORCING)



SECTION C-C

GENERAL NOTES:

1. A SPREAD FOOTING MAY BE A SUITABLE OPTION WHEN THERE IS COMPETENT SOIL OR ROCK CAPABLE OF SUPPORTING THE COLUMN LOADS. THE GEOTECHNICAL DESIGN FOR THE SITE WILL IDENTIFY THE MOST SUITABLE FOUNDATION SYSTEM.
2. FOUNDATION TYPE AND REINFORCING SHALL BE DETERMINED BY ENGINEER OF RECORD FOR SITE CONDITIONS AND AS DIRECTED BY SOILS REPORT.
3. CAST-IN-PLACE FOOTING EXTENSIONS ARE REQUIRED WHEN PRECAST FOOTING EXCEEDS SHIPPING LIMITS.
4. THE NARROWEST WIDTH OF THE ELEMENT AND ANY PROJECTING REINFORCING SHOULD BE KEPT BELOW 14 FEET FOR SHIPPING LIMITATIONS.
5. USE CONTINUOUS FOOTINGS WHERE FOOTING IS ON SUBSOIL OR PILES. USE INDIVIDUAL FOOTINGS WHERE FOOTING IS ON ROCK.



SECTION A-A
PRECAST COLUMN

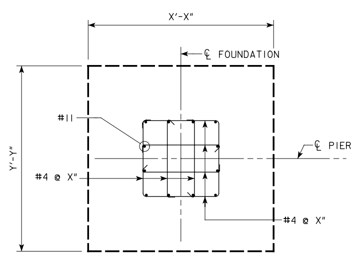
PRECAST FOOTING NOTES:

A. ERECTION TOLERANCE ON ELEVATION ¼" +.

B. DOWEL BAR SPLICES MAY BE SUBSTITUTED WITH LAP SPLICES.

C. PRE-BED SEAT WITH MORTAR THICKNESS SLIGHTLY MORE THAN SHIM STACK.

D. A CAST-IN-PLACE EXTENSION (L1 AND L2) MAY BE USED TO MINIMIZE THE WEIGHT OF THE PRECAST FOOTING. THE CIP EXTENSION CAN BE CONSTRUCTED WHILE THE REST OF THE BRIDGE (SUPPORTED ON THE PRECAST FOOTING) IS BEING ASSEMBLED THUS NOT IMPACTING THE CRITICAL PATH.



SECTION B-B

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

STANDARD PREFABRICATED SUPERSTRUCTURE
FOUNDATION DETAILS
PRECAST FOOTING

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OCTOBER 2011

DESIGN TEAM

SHEET NUMBER P9

GENERAL NOTES:

SUPERSTRUCTURE MODULES:

LIFTING ANCHORS AND LOCATIONS ARE SHOWN ON PLANS. CONTRACTOR MAY PROPOSE ALTERNATE LIFTING DETAILS THAT MUST BE APPROVED BY THE ENGINEER PRIOR TO USE.

PRECASTING:

PRECASTING MATERIALS AND PROCEDURES SHALL CONFORM TO PROVISIONS FOR PREFABRICATED SYSTEMS FOR ACCELERATED BRIDGE CONSTRUCTION.

REMOVAL AND STORAGE:

ALL PRECAST ELEMENTS SHALL BE REMOVED FROM THE FORMS IN SUCH A MANNER THAT NO DAMAGE OCCURS TO THE ELEMENT. FORM REMOVAL SHALL CONFORM TO THE REQUIREMENTS OF PROVISIONS FOR PREFABRICATED SYSTEMS FOR ACCELERATED BRIDGE CONSTRUCTION. ANY MATERIALS FORMING BLOCKOUTS IN THE PRECAST ELEMENTS SHALL BE REMOVED SUCH THAT DAMAGE DOES NOT OCCUR TO THE PRECAST ELEMENTS OR THE BLOCKOUT. PRECAST ELEMENTS SHALL BE STORED IN SUCH A MANNER THAT ADEQUATE SUPPORT IS PROVIDED TO PREVENT CRACKING OR CREEP-INDUCED DEFORMATION (SAGGING). DURING STORAGE FOR LONG PERIODS OF TIME (LONGER THAN ONE MONTH), ALL PRECAST ELEMENTS SHALL BE CHECKED AT LEAST ONCE PER MONTH TO ENSURE CREEP-INDUCED DEFORMATION DOES NOT OCCUR.

LIFTING AND HANDLING:

ALL PRECAST ELEMENTS SHALL BE HANDLED IN SUCH A MANNER AS NOT TO DAMAGE THE PRECAST ELEMENTS DURING LIFTING OR MOVING. LIFTING ANCHORS CAST INTO THE PRECAST ELEMENTS SHALL BE USED FOR LIFTING AND MOVING THE PRECAST ELEMENTS AT THE FABRICATION PLANT AND IN THE FIELD. THE ANGLE BETWEEN THE TOP SURFACE OF THE PRECAST ELEMENTS AND THE LIFTING LINE SHALL NOT BE LESS THAN SIXTY DEGREES, WHEN MEASURED FROM THE TOP SURFACE OF THE PRECAST ELEMENTS TO THE LIFTING LINE. DAMAGE CAUSED TO ANY PRECAST ELEMENTS SHALL BE REPAIRED AT THE EXPENSE OF THE CONTRACTOR TO THE SATISFACTION OF THE ENGINEER.

TRANSPORTATION:

ALL PRECAST ELEMENTS SHALL BE TRANSPORTED IN SUCH A MANNER THAT THE PRECAST ELEMENTS WILL NOT BE DAMAGED DURING TRANSPORTATION. PRECAST ELEMENTS SHALL BE PROPERLY SUPPORTED DURING TRANSPORTATION SUCH THAT CRACKING OR DEFORMATION (SAGGING) DOES NOT OCCUR. IF MORE THAN ONE PRECAST ELEMENTS IS TRANSPORTED PER VEHICLE, PROPER SUPPORT AND SEPARATION MUST BE PROVIDED BETWEEN THE INDIVIDUAL PRECAST ELEMENTS. PRECAST ELEMENTS SHALL LIE HORIZONTAL DURING TRANSPORTATION, UNLESS OTHERWISE APPROVED.

REPAIRS:

REPAIRS OF DAMAGE CAUSED TO THE PRECAST ELEMENTS DURING FABRICATION, LIFTING AND HANDLING, OR TRANSPORTATION SHALL BE ADDRESSED ON A CASE-BY-CASE BASIS. DAMAGE WITHIN ACCEPTABLE LIMITS CAUSED TO THE TOP SURFACE (DRIVING SURFACE) OR TO KEYED EDGES OF THE PRECAST ELEMENTS SHALL BE REPAIRED USING MATERIALS APPROVED BY THE STATE DOT AT THE FABRICATION PLANT AT THE EXPENSE OF THE FABRICATOR. REPETITIVE DAMAGE TO PANELS SHALL BE CAUSE FOR STOPPAGE OF FABRICATION OPERATIONS UNTIL THE CAUSE OF THE DAMAGE CAN BE REMEDIED. ALL PROPOSED REPAIRS SHALL BE APPROVED BY ENGINEER IN ADVANCE.

ULTRA HIGH PERFORMANCE CONCRETE (UHPC):

MOCK POURS OF UHPC JOINTS WILL BE REQUIRED PRIOR TO FIELD ASSEMBLY OF SUPERSTRUCTURE MODULES (AS SPECIFIED BY PROJECT REQUIREMENTS). EACH LONGITUDINAL AND TRANSVERSE CLOSURE POUR SHALL BE CONSTRUCTED IN ONE CONTINUOUS POUR.

DIAMOND GRINDING:

CONTRACTOR TO BID DIAMOND GRINDING BASED ON THE TYPE OF COARSE AGGREGATE IN THE CONCRETE MIX FOR BRIDGE DECKS. FOR PLANT PRECASTING OF ABC COMPONENTS, COARSE AGGREGATE SHALL BE IN ACCORDANCE WITH DOT STANDARD SPECIFICATIONS. DIAMOND GRINDING OF THE BRIDGE DECK SHOULD BE IN ACCORDANCE WITH DOT STANDARD SPECIFICATIONS.

SPECIFICATIONS:

DESIGN: AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS 5TH EDITION.
DESIGN LIVE LOAD: HL-93

LIVE LOAD DEFLECTION LIMIT: L/1000

WELDING: AASHTO/AWS D1.5

CONCRETE: HIGH PERFORMANCE CONCRETE (HPC) SHALL BE USED FOR ALL PRECAST ELEMENTS, INCLUDING BRIDGE DECKS, AND BARRIERS. HPC SHALL BE IN ACCORDANCE WITH STATE DOT DESIGN SPECIFICATIONS AND SPECIAL PROVISIONS.

TARGET PERMEABILITY: 1500 COLOUMBS FOR THE DECK

ULTRA HIGH PERFORMANCE CONCRETE (UHPC) SHALL BE USED FOR CAST-IN-PLACE JOINTS IN SUPERSTRUCTURE AND APPROACH SLABS. UHPC SHALL BE IN ACCORDANCE WITH STATE DOT DESIGN SPECIFICATIONS AND SPECIAL PROVISIONS.

HIGH-STRENGTH BOLTS: ALL BOLTS SHALL BE HIGH-STRENGTH ASTM A325 TYPE 1 BOLTS IN HOLES $\frac{1}{16}$ IN LARGER THAN THE DIAMETER OF THE BOLT. ALL NUTS SHALL BE ASTM A563 HEAVY HEX NUT GRADE DH. ALL WASHERS SHALL BE ASTM F436 GRADE 1. ALL BOLTS, NUTS, AND WASHERS SHALL BE HOT-DIPPED GALVANIZED IN ACCORDANCE WITH ASTM A153.

DESIGN STRESSES:

DESIGN STRESSES FOR THE FOLLOWING MATERIALS ARE IN ACCORDANCE WITH THE AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, 5TH EDITION, SERIES 2010:

REINFORCING STEEL IN ACCORDANCE WITH AASHTO LRFD SECTION 5, GRADE 60, EPOXY-COATED.

DECK CONCRETE IN ACCORDANCE WITH AASHTO LRFD SECTION 5, $f'c=5000$ PSI, EXCEPT CAST-IN-PLACE JOINTS AS NOTED.

STRUCTURAL STEEL IN ACCORDANCE WITH AASHTO LRFD SECTION 6, GRADE 50W.

INDEX OF DRAWINGS

SHEET NO.	DESCRIPTION
S1	GENERAL NOTES AND INDEX OF DRAWINGS
S2	TYPICAL SECTION DETAILS
S3	INTERIOR MODULE
S4	INTERIOR MODULE REINF.
S5	EXTERIOR MODULE
S6	EXTERIOR MODULE REINF.
S7	BEARING DETAILS
S8	MISCELLANEOUS DETAILS

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STANDARD PREFABRICATED
STEEL GIRDER SUPERSTRUCTURE

GENERAL NOTES AND
INDEX OF DRAWINGS

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DESIGN TEAM

SHEET NUMBER S1

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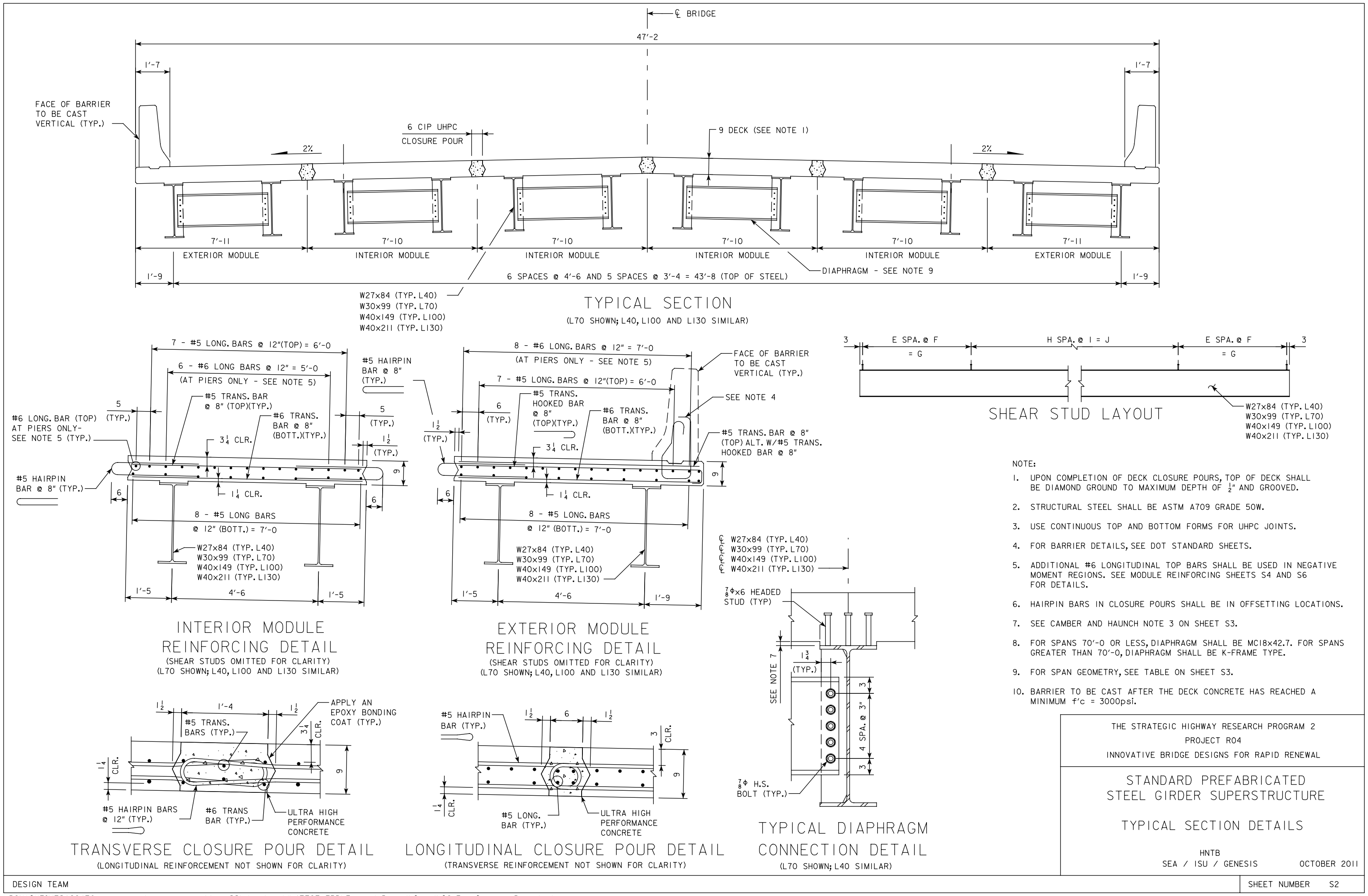
℄ BRIDGE
47'-2
1'-7
1'-7
FACE OF BARRIER TO BE CAST VERTICAL (TYP.)
6 CIP UHPC CLOSURE POUR
9 DECK (SEE NOTE 1)
2%
2%
7'-11
7'-10
7'-10
7'-10
7'-10
7'-11
EXTERIOR MODULE
INTERIOR MODULE
INTERIOR MODULE
INTERIOR MODULE
INTERIOR MODULE
EXTERIOR MODULE
1'-9
6 SPACES @ 4'-6 AND 5 SPACES @ 3'-4 = 43'-8 (TOP OF STEEL)
1'-9
DIAPHRAGM - SEE NOTE 9
W27x84 (TYP. L40)
W30x99 (TYP. L70)
W40x149 (TYP. L100)
W40x211 (TYP. L130)
TYPICAL SECTION
(L70 SHOWN; L40, L100 AND L130 SIMILAR)
7 - #5 LONG. BARS @ 12"(TOP) = 6'-0
6 - #6 LONG BARS @ 12" = 5'-0
(AT PIERS ONLY - SEE NOTE 5)
#5 HAIRPIN BAR @ 8" (TYP.)
#5 TRANS. BAR @ 8" (TOP)(TYP.)
#6 TRANS. BAR @ 8" (BOTT.)(TYP.)
#6 LONG. BAR (TOP) AT PIERS ONLY- SEE NOTE 5 (TYP.)
5 (TYP.)
5 (TYP.)
3¼ CLR.
1½ (TYP.)
#5 HAIRPIN BAR @ 8" (TYP.)
6
1¼ CLR.
6
8 - #5 LONG BARS @ 12" (BOTT.) = 7'-0
W27x84 (TYP. L40)
W30x99 (TYP. L70)
W40x149 (TYP. L100)
W40x211 (TYP. L130)
1'-5
4'-6
1'-5
INTERIOR MODULE REINFORCING DETAIL
(SHEAR STUDS OMITTED FOR CLARITY)
(L70 SHOWN; L40, L100 AND L130 SIMILAR)
8 - #6 LONG. BARS @ 12" = 7'-0
(AT PIERS ONLY - SEE NOTE 5)
7 - #5 LONG. BARS @ 12"(TOP) = 6'-0
#5 TRANS. HOOKED BAR @ 8" (TOP)(TYP.)
#6 TRANS. BAR @ 8" (BOTT.)(TYP.)
1½ (TYP.)
6 (TYP.)
3¼ CLR.
FACE OF BARRIER TO BE CAST VERTICAL (TYP.)
SEE NOTE 4
#5 TRANS. BAR @ 8" (TOP) ALT. W/#5 TRANS. HOOKED BAR @ 8"
9
6
1¼ CLR.
8 - #5 LONG. BARS @ 12" (BOTT.) = 7'-0
W27x84 (TYP. L40)
W30x99 (TYP. L70)
W40x149 (TYP. L100)
W40x211 (TYP. L130)
1'-5
4'-6
1'-9
EXTERIOR MODULE REINFORCING DETAIL
(SHEAR STUDS OMITTED FOR CLARITY)
(L70 SHOWN; L40, L100 AND L130 SIMILAR)
3
E SPA. @ F = G
H SPA. @ I = J
E SPA. @ F = G
3
SHEAR STUD LAYOUT
W27x84 (TYP. L40)
W30x99 (TYP. L70)
W40x149 (TYP. L100)
W40x211 (TYP. L130)
1½
1'-4
1½
#5 TRANS. BARS (TYP.)
APPLY AN EPOXY BONDING COAT (TYP.)
3¼ CLR.
1¼ CLR.
9
#5 HAIRPIN BARS @ 12" (TYP.)
#6 TRANS BAR (TYP.)
ULTRA HIGH PERFORMANCE CONCRETE
TRANSVERSE CLOSURE POUR DETAIL
(LONGITUDINAL REINFORCEMENT NOT SHOWN FOR CLARITY)
#5 HAIRPIN BAR (TYP.)
1½
6
1½
3 CLR.
9
1¼ CLR.
#5 LONG. BAR (TYP.)
ULTRA HIGH PERFORMANCE CONCRETE
LONGITUDINAL CLOSURE POUR DETAIL
(TRANSVERSE REINFORCEMENT NOT SHOWN FOR CLARITY)
℄ W27x84 (TYP. L40)
W30x99 (TYP. L70)
W40x149 (TYP. L100)
W40x211 (TYP. L130)
⅞Φx6 HEADED STUD (TYP)
SEE NOTE 7
1¼ (TYP.)
3
4 SPA. @ 3"
3
⅞Φ H.S. BOLT (TYP.)
TYPICAL DIAPHRAGM CONNECTION DETAIL
(L70 SHOWN; L40 SIMILAR)
NOTE:
1. UPON COMPLETION OF DECK CLOSURE POURS, TOP OF DECK SHALL BE DIAMOND GROUND TO MAXIMUM DEPTH OF ½" AND GROOVED.
2. STRUCTURAL STEEL SHALL BE ASTM A709 GRADE 50W.
3. USE CONTINUOUS TOP AND BOTTOM FORMS FOR UHPC JOINTS.
4. FOR BARRIER DETAILS, SEE DOT STANDARD SHEETS.
5. ADDITIONAL #6 LONGITUDINAL TOP BARS SHALL BE USED IN NEGATIVE MOMENT REGIONS. SEE MODULE REINFORCING SHEETS S4 AND S6 FOR DETAILS.
6. HAIRPIN BARS IN CLOSURE POURS SHALL BE IN OFFSETTING LOCATIONS.
7. SEE CAMBER AND HAUNCH NOTE 3 ON SHEET S3.
8. FOR SPANS 70'-0 OR LESS, DIAPHRAGM SHALL BE MC18x42.7. FOR SPANS GREATER THAN 70'-0, DIAPHRAGM SHALL BE K-FRAME TYPE.
9. FOR SPAN GEOMETRY, SEE TABLE ON SHEET S3.
10. BARRIER TO BE CAST AFTER THE DECK CONCRETE HAS REACHED A MINIMUM f'c = 3000psi.
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DESIGN TEAM
SHEET NUMBER S2
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slam
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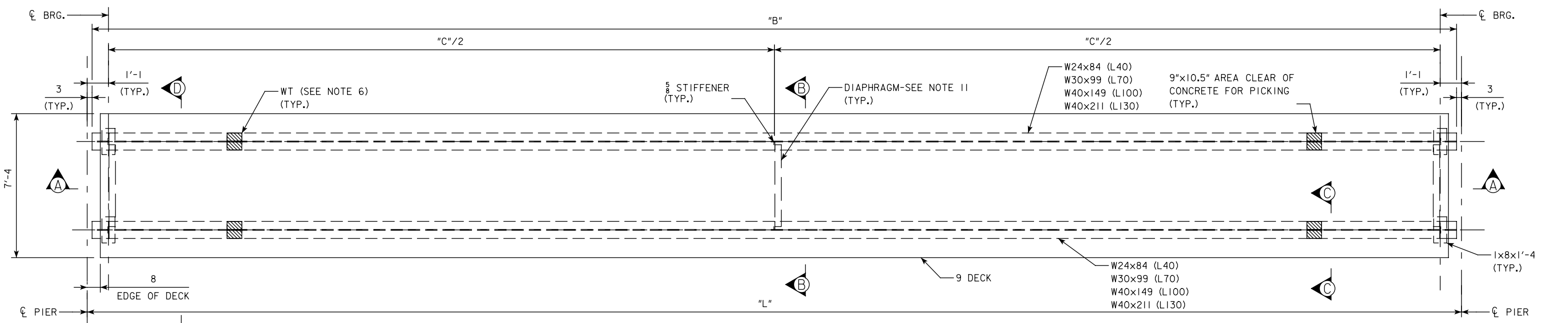
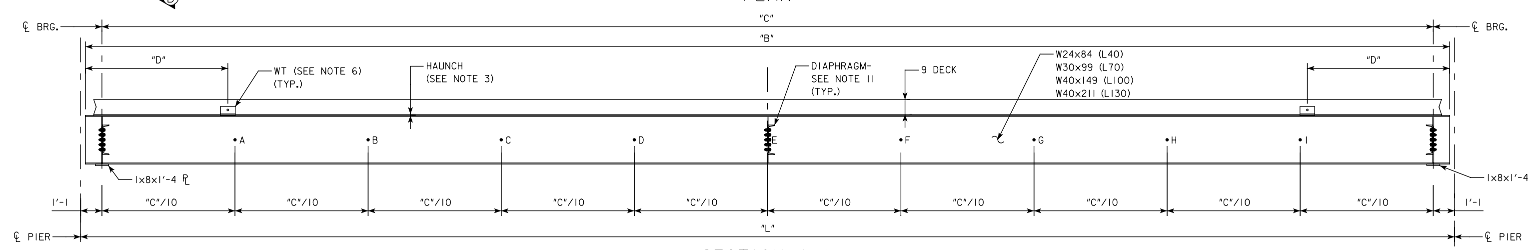


PLAN



SECTION A-A

NOTES:

1. STRUCTURAL STEEL SHALL BE ASTM A709 GRADE 50W.
2. FOR BEARING DETAILS, SEE SHEET S7.
3. FOR SPANS 40'-0 OR LESS STEEL BEAMS NEED NOT BE CAMBERED FOR DEAD LOADS. MODULES ARE DESIGNED TO ACCEPT 2" MAXIMUM HAUNCH. SPANS GREATER THAN 40'-0 SHOULD BE DESIGNED WITH BEAM CAMBER.
4. FOR DIAPHRAGM CONNECTION AND SHEAR STUD DETAILS, SEE SHEET S2.
5. MODULE SHALL BE SUPPORTED AT BEARING POINTS DURING CASTING OPERATIONS AND STORAGE.
6. FOR WT CONNECTION DETAILS, SEE SHEET S5.
7. ALTERNATE LIFT CONFIGURATION MAY BE DEVELOPED BY THE DESIGNER.
8. POCKETS FOR LIFTING CONNECTIONS SHALL BE FILLED WITH U.H.P.C. UPON COMPLETION OF LIFTING OPERATIONS AND PRIOR TO DIAMOND GRINDING.
9. FOR SECTIONS B-B AND D-D, SEE SHEET S4.
10. LONGITUDINAL BARS THAT INTERFERE WITH PICK LOCATIONS SHALL BE CUT. TRANSVERSE BARS THAT INTERFERE WITH PICK LOCATIONS SHALL HAVE SPACING ADJUSTED TO PREVENT CONFLICT.
11. FOR SPANS 70'-0 OR LESS, DIAPHRAGM SHALL BE MC18x42.7. FOR SPANS GREATER THAN 70'-0, DIAPHRAGM SUCH AS K-FRAME MAY BE USED. FOR SPANS GREATER THAN 70'-0" ADDITIONAL INTERMEDIATE DIAPHRAGMS SHOULD BE PROVIDED TO SATISFY MAXIMUM DIAPHRAGM SPACING REQUIREMENTS.
12. FOR SECTION C-C AND WT DETAILS, SEE SHEET S5.

SAMPLE DEAD LOAD DEFLECTIONS - INTERIOR MODULE

SPAN LENGTH		A (IN.)	B (IN.)	C (IN.)	D (IN.)	E (IN.)	F (IN.)	G (IN.)	H (IN.)	I (IN.)
L = 40'-0	STEEL	-0.013	-0.025	-0.034	-0.040	-0.042	-0.040	-0.034	-0.025	-0.013
	DECK	-0.055	-0.103	-0.142	-0.166	-0.174	-0.166	-0.142	-0.103	-0.055
L = 70'-0	STEEL	-0.136	-0.257	-0.352	-0.412	-0.433	-0.412	-0.352	-0.257	-0.136
	DECK	-0.537	-1.017	-1.392	-1.630	-1.712	-1.630	-1.392	-1.017	-0.537
L = 100'-0	STEEL	-0.344	-0.652	-0.892	-1.045	-1.098	-1.045	-0.892	-0.652	-0.344
	DECK	-0.997	-1.886	-2.582	-3.024	-3.176	-3.024	-2.582	-1.886	-0.997
L = 130'-0	STEEL	-0.892	-1.689	-2.312	-2.709	-2.845	-2.709	-2.312	-1.689	-0.892
	DECK	-1.838	-3.477	-4.760	-5.575	-5.854	-5.575	-4.760	-3.477	-1.838

SPAN GEOMETRY

SPAN LENGTH ("L")	BEAM DESIGNATION	MAX. "L"	MAX. "B"	MAX. "C"	MAX. "D"	E	F	G	H (MAX.)	I	J (MAX.)	S
20'-0 < L ⩽40'-0	L40	40'-0	39'-6	37'-10	5'-0	9	15	11'-3	11	18	16'-6	3.5
40'-0 < L ⩽70'-0	L70	70'-0	69'-6	67'-10	8'-9	21	12	21'-0	18	18	27'-0	3.75
70'-0 < L ⩽100'-0	L100	100'-0	99'-6	97'-10	12'-6	32	12	32'-0	20	21	35'-0	4.25
100'-0 < L ⩽ 130'-0	L130	130'-0	129'-6	127'-10	16'-3	32	15	40'-0	28	21	49'-0	4.25

WHERE:
"B" = "L" - 6
"C" = "L" -2(1'-1)
"D" = "L"/8

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STANDARD PREFABRICATED STEEL GIRDER SUPERSTRUCTURE

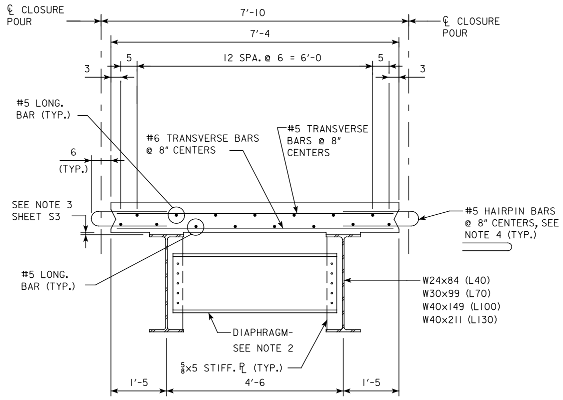
INTERIOR MODULE (INTERIOR SPAN)

HNTB
SEA / ISU / GENESIS OCTOBER 2011

DESIGN TEAM

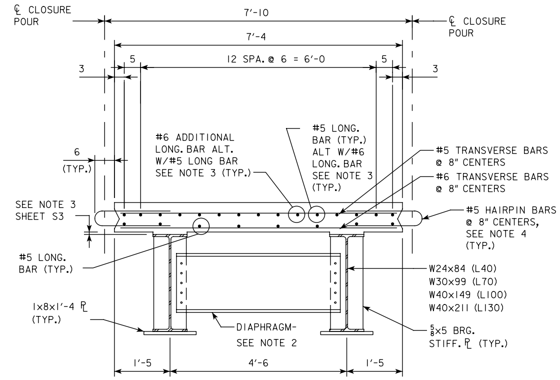
SHEET NUMBER S3

DGN$SYTIME0123456 slam \\nyw00\dept\cadd\45737 TRB\Typical Detail\Steel\S3-Interior Module.dgn



SECTION B-B

(L40 & L70 SHOWN; L100 & L130 SIMILAR)



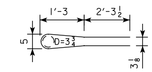
SECTION D-D

(L40 & L70 SHOWN; L100 & L130 SIMILAR)

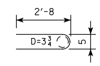
NOTES:

1. FOR LOCATION OF SECTIONS B-B AND D-D, SEE SHEET S3.
2. FOR SPANS 70'-0 OR LESS, DIAPHRAGM SHALL BE MC18x42.7. FOR SPANS OVER 70'-0, DIAPHRAGM SUCH AS K-FRAME MAY BE USED.
3. LENGTH OF ADDITIONAL #6 TOP REINFORCEMENT BAR FOR NEGATIVE MOMENT REGIONS TO BE DETERMINED BY ANALYSIS.
4. PLACEMENT OF HAIRPIN BARS TO AVOID INTERFERENCE WITH BARS FROM ADJACENT MODULES. THIS NEEDS TO BE VERIFIED DURING THE PRECASTING OF DECK.

BENT BAR DETAILS



#5 TRANSVERSE CLOSURE POUR HAIRPIN BAR



#5 LONG. CLOSURE POUR HAIRPIN BAR

NOTES:
ALL DIMENSION ARE OUT TO OUT

D = PIN DIAMETER

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2

PROJECT R04

INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

STANDARD PREFABRICATED STEEL GIRDER SUPERSTRUCTURE

INTERIOR MODULE REINF.

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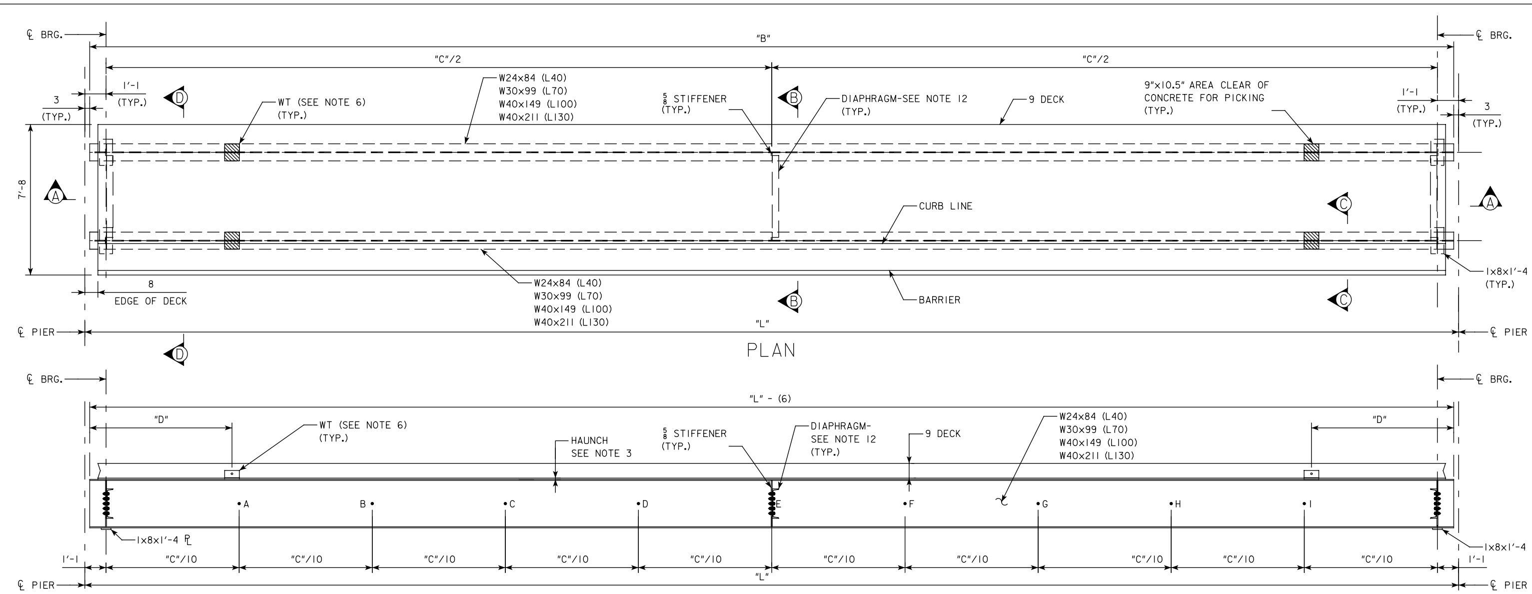
DESIGN TEAM

SHEET NUMBER S4

DGN$SYTIME0123456   slam   \\nyw00\dept\cadd\45737 TRB\Typical Detail\Steel\S4-Interior Module Reinf.dgn



PLAN

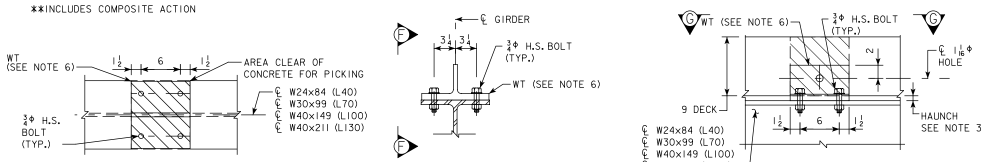
SECTION A-A

SAMPLE DEAD LOAD DEFLECTIONS - EXTERIOR MODULE										
SPAN LENGTH		A (IN.)	B (IN.)	C (IN.)	D (IN.)	E (IN.)	F (IN.)	G (IN.)	H (IN.)	I (IN.)
L = 40'-0	STEEL	-0.013	-0.025	-0.034	-0.040	-0.042	-0.040	-0.034	-0.025	-0.013
	DECK	-0.055	-0.103	-0.142	-0.166	-0.174	-0.166	-0.142	-0.103	-0.055
	BARRIER**	-0.020	-0.037	-0.051	-0.060	-0.063	-0.060	-0.051	-0.037	-0.020
L = 70'-0	STEEL	-0.136	-0.257	-0.352	-0.412	-0.433	-0.412	-0.352	-0.257	-0.136
	DECK	-0.537	-1.017	-1.392	-1.630	-1.712	-1.630	-1.392	-1.017	-0.537
	BARRIER**	-0.212	-0.401	-0.549	-0.643	-0.675	-0.643	-0.549	-0.401	-0.212
L = 100'-0	STEEL	-0.344	-0.652	-0.892	-1.045	-1.098	-1.045	-0.892	-0.652	-0.344
	DECK	-0.997	-1.886	-2.582	-3.024	-3.176	-3.024	-2.582	-1.886	-0.997
	BARRIER**	-0.425	-0.804	-1.101	-1.289	-1.354	-1.289	-1.101	-0.804	-0.425
L = 130'-0	STEEL	-0.892	-1.689	-2.312	-2.709	-2.845	-2.709	-2.312	-1.689	-0.892
	DECK	-1.838	-3.477	-4.760	-5.575	-5.854	-5.575	-4.760	-3.477	-1.838
	BARRIER**	-0.879	-1.664	-2.278	-2.668	-2.801	-2.668	-2.278	-1.664	-0.879

**INCLUDES COMPOSITE ACTION

NOTES:

1. STRUCTURAL STEEL SHALL BE ASTM A709 GRADE 50W.
2. FOR BEARING DETAILS, SEE STANDARD DESIGN SHEETS.
3. FOR SPANS 40'-0 OR LESS STEEL BEAMS NEED NOT BE CAMBERED FOR DEAD LOADS. MODULES ARE DESIGNED TO ACCEPT 2" MAXIMUM HAUNCH. SPANS GREATER THAN 40'-0 SHOULD BE DESIGNED WITH BEAM CAMBER.
4. FOR DIAPHRAGM CONNECTION AND SHEAR STUD DETAILS, SEE DESIGN SHEET S2.
5. MODULE SHALL BE SUPPORTED AT BEARING POINTS DURING CASTING OPERATIONS AND STORAGE.
6. SIZE OF WT FOR LIFTING TO BE DETERMINED BY ENGINEER OF RECORD. ALL LIFTING INSERT POCKETS TO BE FILLED WITH UHPC OR NON-SHRINK GROUT.
7. ALTERNATE LIFT CONFIGURATION MAY BE DEVELOPED BY THE DESIGNER.
8. POCKETS FOR LIFTING CONNECTIONS SHALL BE FILLED WITH U.H.P.C. UPON COMPLETION OF LIFTING OPERATIONS AND PRIOR TO DIAMOND GRINDING.
9. FOR SECTIONS B-B AND D-D, SEE SHEET S6.
10. LONGITUDINAL BARS THAT INTERFERE WITH PICK LOCATIONS SHALL BE CUT. TRANSVERSE BARS THAT INTERFERE WITH PICK LOCATIONS SHALL HAVE SPACING ADJUSTED TO PREVENT CONFLICT.
11. FOR SPANS 70'-0 OR LESS, DIAPHRAGM CAN BE MC18x42.7. FOR SPANS GREATER THAN 70'-0, DIAPHRAGM SUCH AS K-FRAME MAY BE USED. FOR SPANS GREATER THAN 70'-0" ADDITIONAL INTERMEDIATE DIAPHRAGMS SHOULD BE PROVIDED TO SATISFY MAXIMUM DIAPHRAGM SPACING REQUIREMENTS.
12. FOR SPAN GEOMETRY, SEE TABLE ON SHEET S3.



SECTION G-G

SECTION C-C
(DECK NOT SHOWN FOR CLARITY)

SECTION F-F

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STANDARD PREFABRICATED STEEL GIRDER SUPERSTRUCTURE

EXTERIOR MODULE (INTERIOR SPAN)

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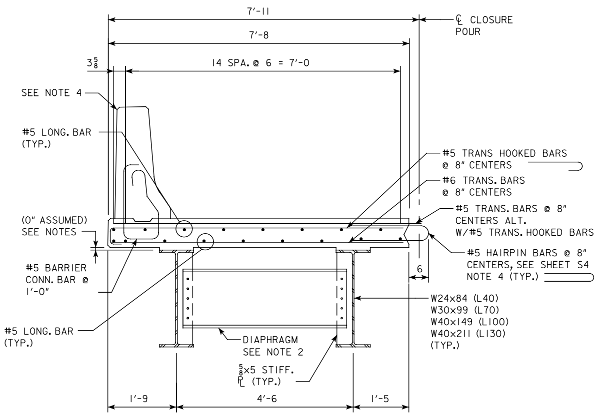
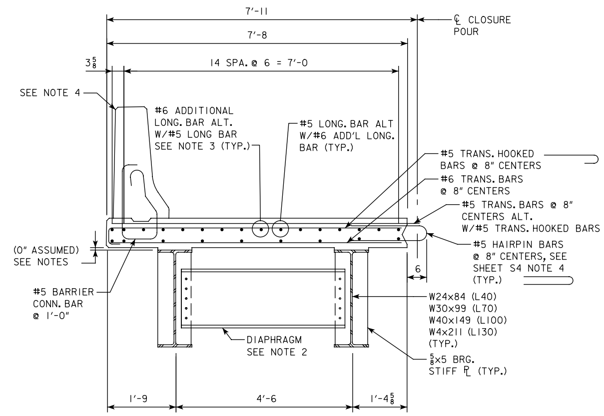
DESIGN TEAM

SHEET NUMBER S5

DGN$SYTIME0123456 slam \\nyw00\dept\cadd\45737 TRB\Typical Detail\Steel\S5-Exterior Module.dgn



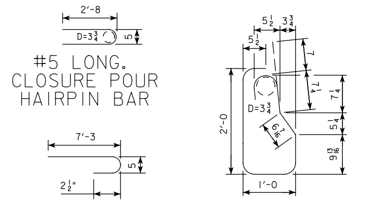
SECTION B-B
(L40 & L70 SHOWN; L100 & L130 SIMILAR)



SECTION D-D
(L40 & L70 SHOWN; L100 & L130 SIMILAR)

NOTES:
1. FOR LOCATION OF SECTIONS B-B AND D-D, SEE SHEET S5.
2. FOR SPANS 70'-0" OR LESS, DIAPHRAGM SHALL BE MC18x42.7. FOR SPANS OVER 70'-0", DIAPHRAGM SUCH AS K-FRAME MAY BE USED.
3. LENGTH OF ADDITIONAL #6 TOP REINFORCEMENT BAR FOR NEGATIVE MOMENT REGIONS TO BE DETERMINED BY ANALYSIS.
4. FACE OF BARRIER TO BE CAST ALIGNED WITH VERTICAL.
5. FOR BARRIER DETAILS, SEE DOT STANDARD SHEETS.
6. CAMBERS FOR EXTERIOR MODULE NEED TO ACCOUNT FOR THE SUPERIMPOSED WEIGHT OF THE BARRIER

BENT BAR DETAILS



#5 LONG. CLOSURE POUR HAIRPIN BAR

#5 TRANS. HOOKED BAR

#5 BARRIER CONNECTION BAR (SEE NOTE 5)

NOTES:
ALL DIMENSION ARE OUT TO OUT

D = PIN DIAMETER

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
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STANDARD PREFABRICATED STEEL GIRDER SUPERSTRUCTURE

EXT. MODULE 2 REINF.

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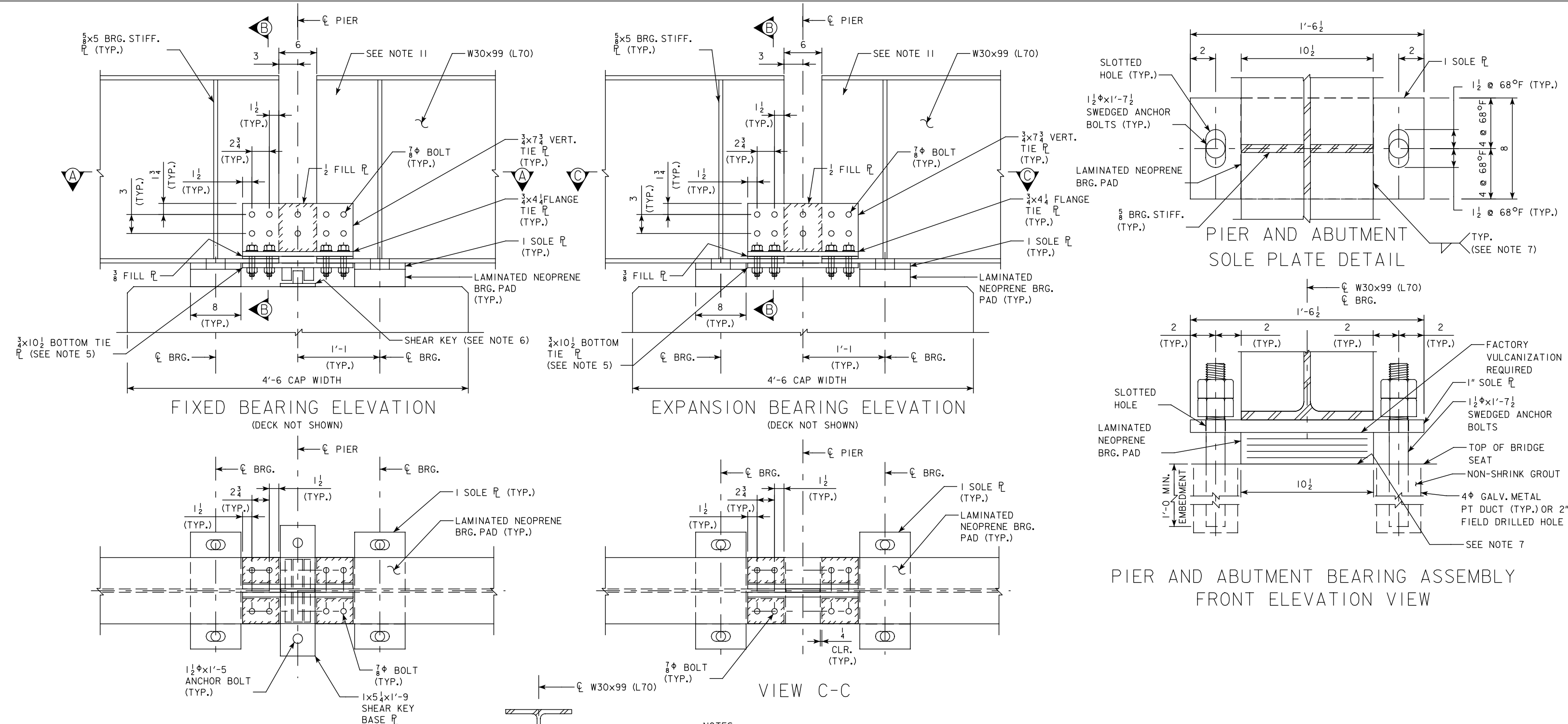
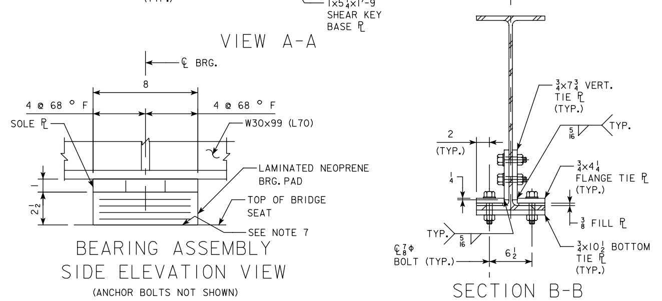
OCTOBER 2011

DESIGN TEAM

SHEET NUMBER S6

DGN$SYTIME0123456 slam \\nyw00\dept\cadd\45737 TRB\Typical Detail\Steel\S6-Exterior Module 2R.dgn





NOTES:

1. FOR GENERAL NOTES, SEE SHEET S1.
2. DETAILS SHOWN ARE FOR L70 SPAN CONFIGURATION. OTHER SPAN LENGTHS REQUIRE CHANGES IN SIZING BY THE ENGINEER OF RECORD.
3. CAUTION SHALL BE EXERCISED WHERE A FIELD WELD OR SHOP WELD WILL BE MADE WHILE ELASTOMERIC BEARING PADS ARE IN CONTACT WITH STEEL. IN NO CASE SHALL THE ELASTOMER BE EXPOSED TO INSTANTANEOUS TEMPS GREATER THAN 400°F.
4. PROVIDE GRADE 60 DUROMETER ELASTOMER WITH A SHEAR MODULUS OF 175 PSI, IN ACCORDANCE WITH AASHTO LRFD BRIDGE CONSTRUCTION SPECIFICATIONS.
5. FOR BOTTOM TIE PLATE DETAILS, SEE SHEET S8.
6. FOR SHEAR KEY DETAILS, SEE SHEET S8.
7. CONTRACTOR TO HAVE AVAILABLE $\frac{1}{8}$" NEOPRENE PADS TO BE PLACED ON THE PIER CAP TO PROVIDE VERTICAL ADJUSTMENT. PLACE BEARING ON THE NEOPRENE PAD, WITH MAXIMUM OF 3 SUCH PADS PER BEARING.
8. TIE PLATES, SHEAR KEYS, ANCHOR BOLTS, NON-SHRINK GROUT, AND ELASTOMERIC BEARINGS TO BE INCLUDED IN PRICE BID FOR SUPERSTRUCTURE MODULES.
9. DRILL ALL BOTTOM TIE PLATE HOLES ONLY AFTER CASTING OF DECK AND FIT UP OF ADJACENT MODULES. BOLT TIE PLATES PRIOR TO CASTING UHPC JOINTS.
10. FOR ABUTMENT BEARING DETAILS, SEE SHEETS A6 AND A8.
11. FOR POST-TENSIONING BARS OF THE TOP OF WEB, SEE SHEET S8.

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STANDARD PREFABRICATED STEEL GIRDER SUPERSTRUCTURE

BEARING DETAILS

HNTB
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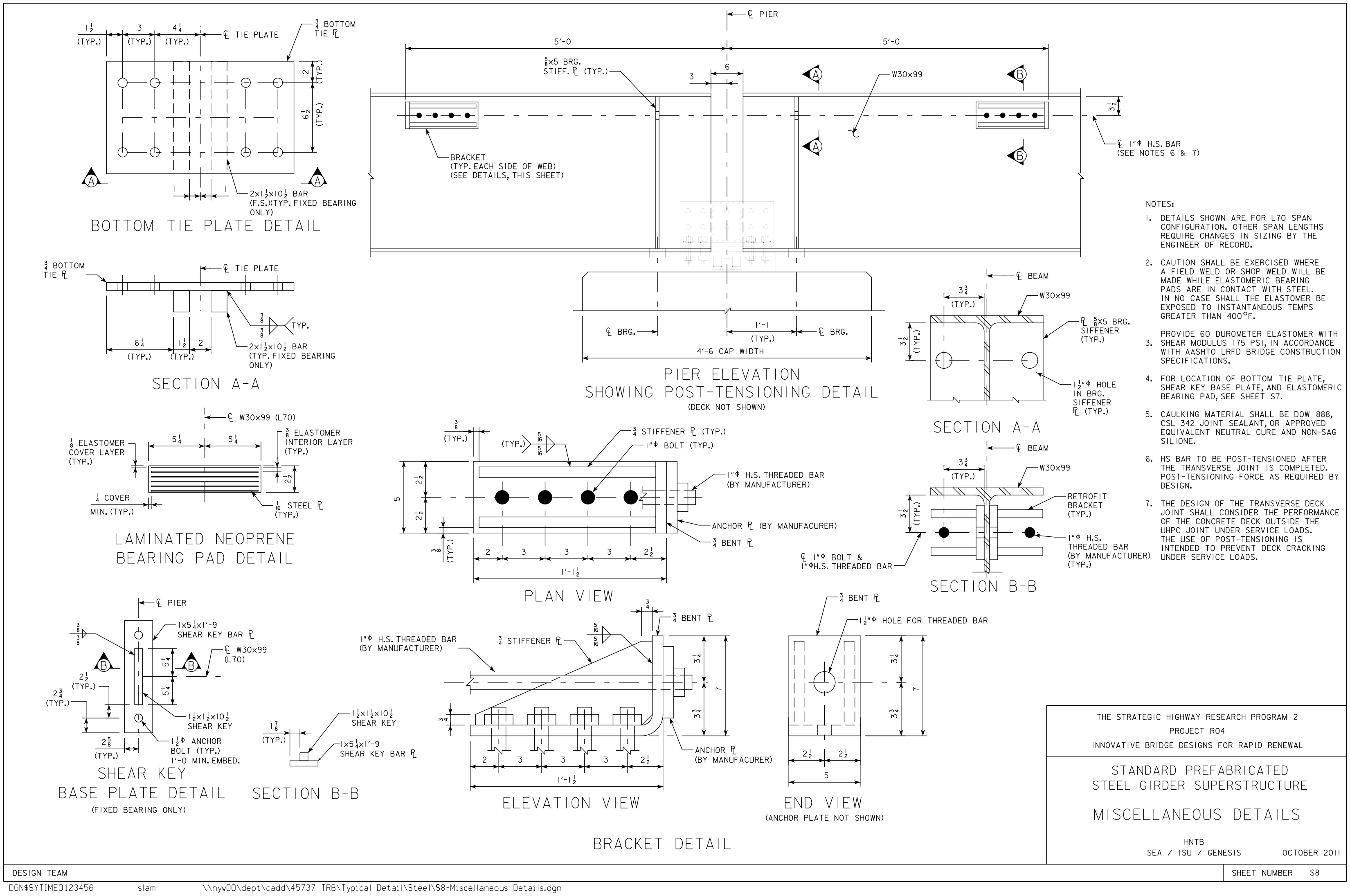
DESIGN TEAM SHEET NUMBER S7

DGN$SYTIME0123456 slam \\nyw00\dept\cadd\45737 TRB\Typical Detail\Steel\S7-Bearing Details.dgn



BOTTOM TIE PLATE DETAIL
℄ TIE PLATE
¾ BOTTOM TIE ℙ
2×1½×10½ BAR (F.S.)(TYP. FIXED BEARING ONLY)
SECTION A-A
PIER ELEVATION
SHOWING POST-TENSIONING DETAIL
(DECK NOT SHOWN)
℄ PIER
⅝×5 BRG. STIFF. ℙ (TYP.)
W30×99
BRACKET (TYP. EACH SIDE OF WEB) (SEE DETAILS, THIS SHEET)
℄ 1"Φ H.S. BAR (SEE NOTES 6 & 7)
℄ BRG.
4'-6 CAP WIDTH
LAMINATED NEOPRENE
BEARING PAD DETAIL
⅛ ELASTOMER COVER LAYER (TYP.)
⅜ ELASTOMER INTERIOR LAYER (TYP.)
¼ COVER MIN. (TYP.)
1/16 STEEL ℙ (TYP.)
℄ W30×99 (L70)
PLAN VIEW
¾ STIFFENER ℙ (TYP.)
1"Φ BOLT (TYP.)
1"Φ H.S. THREADED BAR (BY MANUFACTURER)
ANCHOR ℙ (BY MANUFACURER)
¾ BENT ℙ
SECTION A-A
℄ BEAM
⅝×5 BRG. SIFFENER (TYP.)
1½"Φ HOLE IN BRG. SIFFENER ℙ (TYP.)
SECTION B-B
RETROFIT BRACKET (TYP.)
1"Φ H.S. THREADED BAR (BY MANUFACTURER) (TYP.)
℄ 1"Φ BOLT & 1"ΦH.S. THREADED BAR
NOTES:
1. DETAILS SHOWN ARE FOR L70 SPAN CONFIGURATION. OTHER SPAN LENGTHS REQUIRE CHANGES IN SIZING BY THE ENGINEER OF RECORD.
2. CAUTION SHALL BE EXERCISED WHERE A FIELD WELD OR SHOP WELD WILL BE MADE WHILE ELASTOMERIC BEARING PADS ARE IN CONTACT WITH STEEL. IN NO CASE SHALL THE ELASTOMER BE EXPOSED TO INSTANTANEOUS TEMPS GREATER THAN 400°F.
PROVIDE 60 DUROMETER ELASTOMER WITH
3. SHEAR MODULUS 175 PSI, IN ACCORDANCE WITH AASHTO LRFD BRIDGE CONSTRUCTION SPECIFICATIONS.
4. FOR LOCATION OF BOTTOM TIE PLATE, SHEAR KEY BASE PLATE, AND ELASTOMERIC BEARING PAD, SEE SHEET S7.
5. CAULKING MATERIAL SHALL BE DOW 888, CSL 342 JOINT SEALANT, OR APPROVED EQUIVALENT NEUTRAL CURE AND NON-SAG SILIONE.
6. HS BAR TO BE POST-TENSIONED AFTER THE TRANSVERSE JOINT IS COMPLETED. POST-TENSIONING FORCE AS REQUIRED BY DESIGN.
7. THE DESIGN OF THE TRANSVERSE DECK JOINT SHALL CONSIDER THE PERFORMANCE OF THE CONCRETE DECK OUTSIDE THE UHPC JOINT UNDER SERVICE LOADS. THE USE OF POST-TENSIONING IS INTENDED TO PREVENT DECK CRACKING UNDER SERVICE LOADS.
SHEAR KEY
BASE PLATE DETAIL
(FIXED BEARING ONLY)
1×5¼×1'-9 SHEAR KEY BAR ℙ
℄ W30×99 (L70)
1½×1½×10½ SHEAR KEY
1½Φ ANCHOR BOLT (TYP.) 1'-0 MIN. EMBED.
SECTION B-B
ELEVATION VIEW
¾ STIFFENER ℙ
¾ BENT ℙ
ANCHOR ℙ (BY MANUFACURER)
END VIEW
(ANCHOR PLATE NOT SHOWN)
1½"Φ HOLE FOR THREADED BAR
BRACKET DETAIL
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STANDARD PREFABRICATED
STEEL GIRDER SUPERSTRUCTURE
MISCELLANEOUS DETAILS
HNTB
SEA / ISU / GENESIS
OCTOBER 2011
DESIGN TEAM
SHEET NUMBER S8
DGN$SYTIME0123456 slam \\nyw00\dept\cadd\45737 TRB\Typical Detail\Steel\S8-Miscellaneous Details.dgn

GENERAL NOTES:

THESE PLANS PRESENT AN ACCELERATED BRIDGE CONSTRUCTION CONCEPT FOR ERECTION OF A PRECAST CONCRETE SUPERSTRUCTURE.

THE SYSTEM PRESENTED IN THESE CONCEPT PLANS CONSISTS OF PRECAST PRESTRESSED CONCRETE GIRDERS WITH A FULL WIDTH TOP FLANGE INTENDED TO SERVE AS THE RIDING SURFACE, ELIMINATING THE NEED FOR A CAST-IN-PLACE DECK.

TRANSVERSE CONTINUITY BETWEEN THE GIRDERS IS ESTABLISHED BY REMOVING ELEVATION DIFFERENCES BETWEEN ADJACENT FLANGE TIPS AND CASTING A LONGITUDINAL JOINT FILLED WITH UHPC ALONG THE LENGTH OF THE BRIDGE. THE LONGITUDINAL JOINT IS DETAILED TO PROVIDE A FULL MOMENT CONNECTION BETWEEN THE BEAMS.

LONGITUDINAL CONTINUITY AT INTERMEDIATE PIER LINES IN MULTIPLE-SPAN BRIDGES IS ESTABLISHED WITH CONTINUITY POURS BETWEEN THE OPPOSING GIRDER ENDS AND FULL-WIDTH TRANSVERSE JOINTS BETWEEN THE FLANGES USING UHPC.

FUTURE DECK REPLACEMENT IS NOT CONSIDERED IN THE DESIGN OF THESE BEAMS. PROVISIONS FOR DECK REPLACEMENT TEND TO RESULT IN LESS EFFICIENT BEAMS BECAUSE THE REMOVAL STAGE WHERE THE COMPOSITE TOP FLANGE IS NOT PRESENT GENERALLY GOVERNS THE DESIGN OF THESE BEAMS.

EXTENSIVE DECK REPAIR AND/OR REPLACEMENT CONSISTENT WITH THE OBJECTIVES OF ACCELERATED BRIDGE CONSTRUCTION CAN BE ACCOMPLISHED BY REMOVING AND REPLACING AN ENTIRE BEAM.

RAPID DECK REPAIR OR REPLACMENT CAN BE ACCOMPLISHED BY REMOVAL AND REPLACEMENT OF AN ENTIRE BEAM, ADHERING TO OBJECTIVES OF ACCELERATED BRIDGE CONSTRUCTION WITH RESPECT TO MINIMIZING DISRUPTIONS.

DURABILITY CONCERNS THAT CONTRIBUTE TO DECK DETERIORATION AND THE NEED TO REPLACE THE DECK ARE ADDRESSED IN THESE CONCEPT PLANS BY INCLUDING ADDITIONAL COVER ON THE RIDING SURFACE OF THE BEAM FLANGES AND DESIGNING WITH AN ALLOWANCE FOR FUTURE INSTALLATION OF AN OVERLAY THAT CAN INCLUDE A WATERPROOFING MEMBRANE.

FURTHER ENHANCED DURABILITY CAN BE ACHIEVED BY USING EPOXY-COATED REINFORCING BARS, ADMIXTURES THAT INCREASE WORKABILITY OR REDUCE PERMEABILITY OF THE CONCRETE MIX, AND CONTROLLED CURING CONDITIONS.

LONGER SPANS ARE SUSCEPTIBLE TO HIGH END ZONE STRESSES DUE TO THE SIGNIFICANT SELF-WEIGHT OF THESE BEAMS AT RELEASE AND THE AMOUNT OF PRESTRESS REQUIRED TO SATISFY BOTTOM FLANGE ALLOWABLE TENSION AT THE SERVICE LIMIT STATE. USE OF LIGHTWEIGHT CONCRETE FOR A PORTION, OR ALL, OF THE BEAM SECTION MAY PRESENT OPPORTUNITIES FOR COST SAVINGS BY REDUCING THE REQUIRED PRESTRESS FORCE.

PERMANENT LOADS:

THE FOLLOWING PERMANENT LOADS WERE CONSIDERED IN THE DESIGN OF THE BEAMS PRESENTED IN THESE CONCEPT PLANS:

1. GIRDER SELF-WEIGHT: NOTED IN PLANS
2. CIP LONGITUDINAL JOINT: 60 PLF
3. TRAFFIC BARRIERS: 430 PLF
4. FUTURE WEARING SURFACE: 25 PSF

VEHICULAR LOADS:

DESIGN LIVE LOAD FOR THE BEAMS PRESENTED IN THESE CONCEPT PLANS IS THE HL-93 LOADING, AS DEFINED BY AASHTO.

LIVE LOAD DISTRIBUTION FACTORS ARE COMPUTED IN ACCORDANCE WITH AASHTO LRFD 4.6.2.2.2 AND 4.6.2.2.3, USING CROSS-SECTION TYPE "J" SUFFICIENTLY CONNECTED TO ACT AS A UNIT.

CONSTRUCTION LOADS:

GIRDER STRESSES DURING HANDLING SHALL NOT EXCEED THE ALLOWABLE STRESSES SPECIFIED. LIFT POINTS AND TEMPORARY SUPPORTS SHALL BE LOCATED WITHIN THE DIMENSIONS SHOWN ON THE PLANS, RELATIVE TO THE FINAL BEARING LOCATIONS.

EFFECTS OF DEAD LOAD STRESSES AT THE HANDLING STAGE SHALL BE INCREASED 30 PERCENT TO ACCOUNT FOR DYNAMIC EFFECTS DURING TRANSPORTATION.

SPECIFIED ALLOWABLE TENSION AT THE SERVICE LIMIT STATE IS REDUCED TO PROVIDE AN ALLOWANCE FOR TENSION IN THE BEAMS DUE TO CAMBER LEVELING FORCES. AS A RESULT, CAMBER LEVELING FORCES DO NOT NEED TO BE CONSIDERED IN THE DESIGN OF THE BEAMS. IF CAMBER LEVELING FORCES ARE CONSIDERED, ALLOWABLE TENSION AT THE SERVICE LIMIT STATE AFTER LOSSES, AS PRESCRIBED BY AASHTO LRFD, MAY BE USED.

TOLERANCES:

TOLERANCES FOR THE FABRICATION OF PRECAST PRESTRESSED COMPONENTS ARE GENERALLY IN ACCORDANCE WITH APPENDIX B OF PCI MANUAL MNL-116. TOLERANCES FOR CAMBER AND LATERAL SWEEP ARE SPECIFIED ON THESE PLANS. TOLERANCE FOR TRANSVERSE SMOOTHNESS OF THE RIDING SURFACE IN THE FINAL CONDITION SHALL BE LESS THAN $\frac{3}{8}$" IN 10 FEET, AS MEASURED WITH A STRAIGHTEDGE PLACED NORMAL TO THE CENTERLINE OF THE BRIDGE.

REMOVAL AND STORAGE:

PRECAST ELEMENTS SHALL BE REMOVED FROM THE FORMS IN SUCH A MANNER THAT NO DAMAGE OCCURS. ANY MATERIALS FORMING BLOCKOUTS IN THE PRECAST ELEMENTS SHALL BE REMOVED SUCH THAT DAMAGE DOES NOT OCCUR TO THE PRECAST ELEMENTS OR THE BLOCKOUT. PRECAST ELEMENTS SHALL BE STORED IN WITH ADEQUATE SUPPORT PROVIDED IN LOCATIONS AS CLOSE AS PRACTICAL TO THE FINAL BEARING LOCATIONS.

LIFTING AND HANDLING:

PRECAST ELEMENTS SHALL BE HANDLED IN SUCH A MANNER AS NOT TO DAMAGE THE PRECAST ELEMENTS DURING LIFTING OR MOVING. LIFTING ANCHORS, AS DETAILED ON THE PLANS, SHALL BE USED FOR LIFTING AND MOVING THE PRECAST ELEMENTS AT THE FABRICATION PLANT AND IN THE FIELD. THE CONTRACTOR SHALL BE RESPONSIBLE FOR ASSURING THAT GIRDERS ARE ADEQUATELY BRACED TO PREVENT TIPPING AND TO CONTROL LATERAL BENDING DURING ALL PHASES OF CONSTRUCTION. DAMAGE CAUSED TO ANY PRECAST ELEMENT SHALL BE REPAIRED WITH APPROVED MATERIALS AND PROCEDURES, TO THE SATISFACTION OF THE ENGINEER, AT THE EXPENSE OF THE CONTRACTOR. REPETITIVE DAMAGE SHALL BE CAUSE FOR STOPPAGE OF FABRICATION OPERATIONS UNTIL THE CAUSE OF THE DAMAGE CAN BE REMEDIED.

TRANSPORTATION:

PRECAST ELEMENTS SHALL BE TRANSPORTED IN SUCH A MANNER THAT THEY WILL NOT BE DAMAGED DURING TRANSPORTATION. THE GIRDERS SHALL BE PROPERLY SUPPORTED SUCH THAT CRACKING OR DEFORMATION DOES NOT OCCUR AND THEY SHALL BE PROPERLY BRACED TO MAINTAIN STABILITY AT ALL TIMES.

LIMITATIONS:

THESE GUIDELINES ARE BASED ON THE GENERAL INFORMATION (DIMENSIONS, MATERIALS, LOADS, STRESSES, ETC.) PRESENTED ON THESE CONCEPT PLANS AND ARE INTENDED TO ASSIST THE DESIGN ENGINEER IN THE DEVELOPMENT OF A SET OF CONTRACT PLANS. THESE GUIDELINES SHALL NOT BE INTERPRETED AS UNIVERSALLY APPLICABLE TO ANY DESIGN PROBLEM, NOR DO THEY RELIEVE THE ENGINEER OF RECORD OF ANY DUTIES PERTAINING TO THE RESPONSIBLE DESIGN OF THE TYPE OF BRIDGE FOR WHICH THESE GUIDELINES HAVE BEEN PREPARED. THE ENGINEER OF RECORD SHALL BE RESPONSIBLE FOR CONFORMANCE WITH STANDARDS AND POLICIES OF THE GOVERNING AGENCY.

SPECIFICATIONS:

AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, 5TH EDITION

SUPPLEMENTAL DESIGN SPECIFICATIONS AS REQUIRED BY THE GOVERNING AGENCY

THESE CONCEPT DESIGNS DO NOT CONSIDER PERMIT OR OVERLOAD VEHICLES AT THE STRENGTH LIMIT STATE THAT MAY BE REQUIRED BY THE GOVERNING AGENCY.

MATERIAL PROPERTIES:

CONCRETE: HIGH PERFORMANCE CONCRETE (HPC) WITH A MINIMUM 28-DAY COMPRESSIVE STRENGTH OF 8,000 PSI.

PRIOR TO RELEASE OF PRESTRESS, CONCRETE SHALL HAVE ATTAINED STRENGTH AT LEAST EQUAL TO 80% OF THE SPECIFIED MINIMUM 28-DAY COMPRESSIVE STRENGTH.

RELATIVE HUMIDITY, H, EQUAL TO 70 PERCENT

PRESTRESSING STEEL: GRADE 270 LOW-RELAXATION STRANDS

REINFORCING STEEL: GRADE 60 DEFORMED BARS

CONCRETE COVER:

EDGES OF GIRDER FLANGES:	2"
BOTTOM AND SIDE FACES OF GIRDER WEB AND FLANGES:	1"
TOP SURFACE OF DECK SLAB	3"

COVER ON TOP OF DECK SLAB INCLUDES $\frac{1}{2}$" ALLOWANCE FOR WEAR AND $\frac{1}{2}$" ALLOWANCE FOR GRINDING.

GIRDERS ARE SHOWN TO HAVE CONSTANT FLANGE THICKNESS. WHERE AN ODD NUMBER OF GIRDERS ARE USED, THE TOP FLANGE THICKNESS OF THE CENTER GIRDER SHALL BE INCREASED TO ACCOUNT FOR THE CROSS-SLOPE.

DESIGN STRESSES:

AT A MINIMUM, BEAM DESIGNS SHALL CONSIDER DESIGN STRESSES AT THE FOLLOWING STAGES DURING FABRICATION, ERECTION AND SERVICE:

1. TRANSFER OF PRESTRESS (RELEASE)
2. STORAGE, LIFTING, AND HAULING (HANDLING)
3. IN SERVICE (FINAL)

THE ENGINEER OF RECORD SHALL BE RESPONSIBLE FOR DETERMINING IF ADDITIONAL INTERMEDIATE STAGES INTRODUCE CRITICAL STRESSES IN THE BEAMS DUE TO TEMPORARY SUPPORT CONDITIONS AND/OR APPLIED LOADINGS. ALLOWABLE STRESS AT INTERMEDIATE STAGES SHALL BE BASED ON A REASONABLE ESTIMATE OF THE AGE-ADJUSTED CONCRETE STRENGTH.

CONCRETE STRESSES AT THE STAGES NOTED SHALL NOT EXCEED THE FOLLOWING ALLOWABLE VALUES AT THE SERVICE LIMIT STATE:

STAGE	CONCRETE STRENGTH (KSI)		LIMIT STATE	LOADS	ALLOWABLE STRESS (KSI)	
RELEASE	f'ci=0.8f'c	6.4	SERVICE I	DL, PS	COMPRESSION	3.84
					TENSION	-0.20
HANDLING	f'cm=0.9f'c	7.2	SERVICE I	DL, DIM, PS	COMPRESSION	4.32
					TENSION	-0.20
FINAL	f'c	8.0	SERVICE I	DL, PS	COMPRESSION	3.60
				DL, PS, LL+IM	COMPRESSION	4.80
			SERVICE III	DL, PS, LL+IM	TENSION	0

DIM REPRESENTS DYNAMIC ALLOWANCE FOR DEAD LOAD DURING SHIPPING, DEFINED HEREIN AS 30 PERCENT.

PRESTRESSING STEEL DESIGN STRESSES AT THE SERVICE LIMIT STATE ARE AS FOLLOWS:

SERVICE I, IMMEDIATELY PRIOR TO TRANSFER 202.5 KSI
SERVICE III, AFTER ALL LOSSES 194.4 KSI

INDEX OF DRAWINGS

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GENERAL NOTES AND INDEX OF DRAWINGS

HNTB
SEA / ISU / GENESIS OCTOBER 2011

DESIGN TEAM

SHEET NUMBER C1

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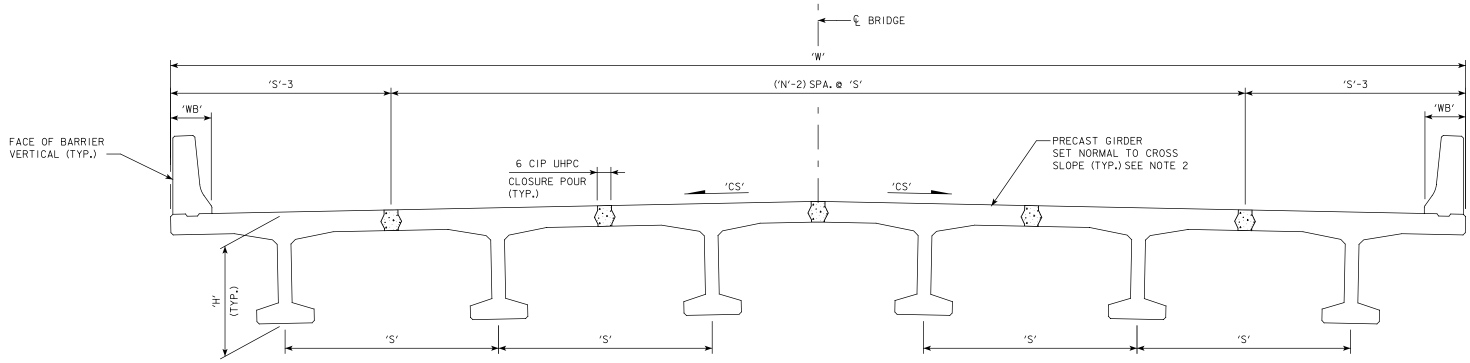
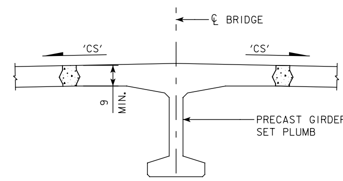


TYPICAL SECTION

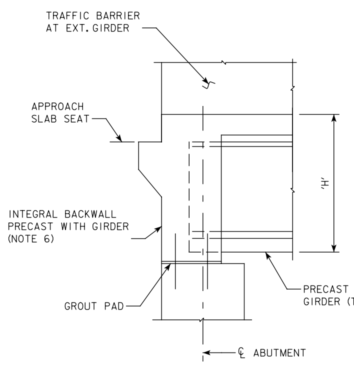
TYPICAL SECTION DIMENSIONS USED FOR EXAMPLE BRIDGE		
DESCRIPTION	VARIABLE	VALUE
OUT-OUT WIDTH	'W'	47'-2
BARRIER WIDTH	'WB'	1'-7
NO. OF GIRDERS	'N'	7
GIRDER SPACING	'S'	7'-11¼
GIRDER HEIGHT	'H'	3'-6
CROSS SLOPE	'CS'	2 %



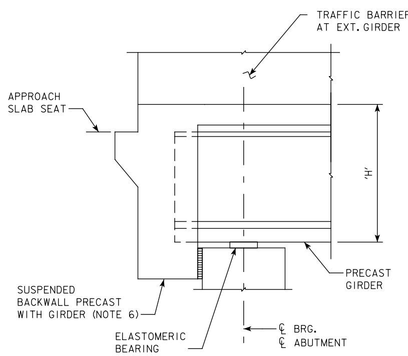
CENTRAL GIRDER
(SEE NOTE 2)

NOTES:

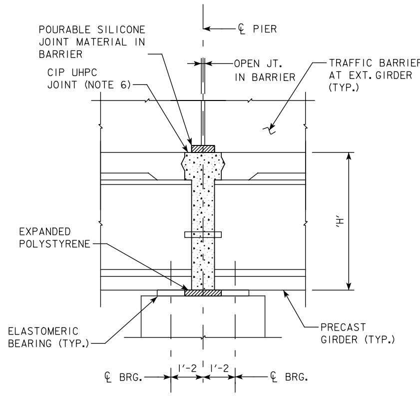
1. DETAILS PRESENTED HERE ARE SUITABLE FOR BRIDGES WITH CONSTANT CROSS-SLOPE AND NO SKEW. THEY CAN BE READILY ADAPTED TO ACCOMMODATE LOW TO MODERATE SKEW ANGLES WITH PARALLEL LINES OF SUPPORT.
2. TYPICAL SECTION WITH EVEN NUMBER OF GIRDERS SHOWN. WHERE AN ODD NUMBER OF GIRDERS IS REQUIRED, THE CENTER GIRDER SHALL BE SET PLUMB.
3. UPON COMPLETION OF DECK CLOSURE POURS, TOP OF DECK SHALL BE DIAMOND GROUND TO MAXIMUM DEPTH OF ½" AND GROOVED FOR SKID RESISTANCE.
4. FOR UHPC JOINT REQUIREMENTS, SEE GENERAL INFORMATION SHEET.
5. FOR GIRDER DETAILS, SEE SHEET NOS. C4 AND C5.
6. FOR ABUTMENT AND PIER CONTINUITY DETAILS, SEE SHEET NOS. C6 AND C7.
7. FOR CLOSURE POUR DETAILS, SEE SHEET NO. C9.
8. FOR ALTERNATE SECTION TO BE USED FOR SHORTER SPAN LENGTHS AND/OR CRITICAL VERTICAL CLEARANCE CONDITIONS, SEE SHEET NOS. C10 AND C11.
9. ALL CONCRETE FACES INCLUDED IN CAST-IN-PLACE CLOSURE POURS AND JOINTS SHALL BE CLEANED AND COATED WITH AN EPOXY BONDING AGENT APPROVED BY THE AGENCY OR PROJECT SPECIFICATIONS AND INSTALLED IN ACCORDANCE WITH THE MANUFACTURER'S INSTRUCTIONS PRIOR TO POURING THE CONCRETE.



ELEVATION AT INTEGRAL ABUTMENT



ELEVATION AT SEMI-INTEGRAL ABUTMENT



ELEVATION AT INTERMEDIATE PIER

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
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STANDARD PRECAST PRESTRESSED DECKED GIRDER SUPERSTRUCTURE

TYPICAL SECTION

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SHEET NUMBER C2

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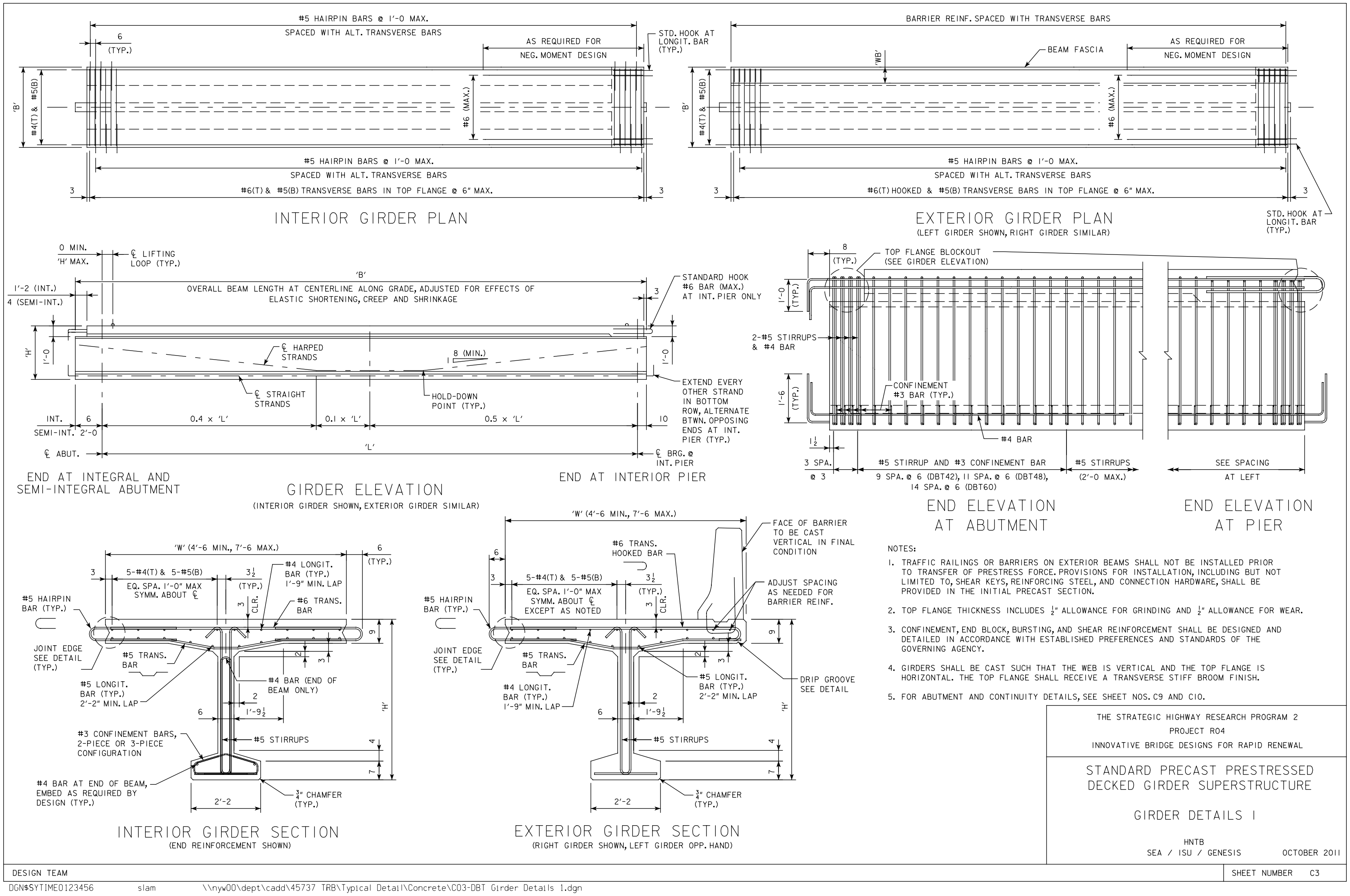
INTERIOR GIRDER PLAN
EXTERIOR GIRDER PLAN
(LEFT GIRDER SHOWN, RIGHT GIRDER SIMILAR)
END AT INTEGRAL AND SEMI-INTEGRAL ABUTMENT
GIRDER ELEVATION
(INTERIOR GIRDER SHOWN, EXTERIOR GIRDER SIMILAR)
END AT INTERIOR PIER
END ELEVATION AT ABUTMENT
END ELEVATION AT PIER
INTERIOR GIRDER SECTION
(END REINFORCEMENT SHOWN)
EXTERIOR GIRDER SECTION
(RIGHT GIRDER SHOWN, LEFT GIRDER OPP. HAND)
NOTES:
1. TRAFFIC RAILINGS OR BARRIERS ON EXTERIOR BEAMS SHALL NOT BE INSTALLED PRIOR TO TRANSFER OF PRESTRESS FORCE. PROVISIONS FOR INSTALLATION, INCLUDING BUT NOT LIMITED TO, SHEAR KEYS, REINFORCING STEEL, AND CONNECTION HARDWARE, SHALL BE PROVIDED IN THE INITIAL PRECAST SECTION.
2. TOP FLANGE THICKNESS INCLUDES ½" ALLOWANCE FOR GRINDING AND ½" ALLOWANCE FOR WEAR.
3. CONFINEMENT, END BLOCK, BURSTING, AND SHEAR REINFORCEMENT SHALL BE DESIGNED AND DETAILED IN ACCORDANCE WITH ESTABLISHED PREFERENCES AND STANDARDS OF THE GOVERNING AGENCY.
4. GIRDERS SHALL BE CAST SUCH THAT THE WEB IS VERTICAL AND THE TOP FLANGE IS HORIZONTAL. THE TOP FLANGE SHALL RECEIVE A TRANSVERSE STIFF BROOM FINISH.
5. FOR ABUTMENT AND CONTINUITY DETAILS, SEE SHEET NOS. C9 AND C10.
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
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STANDARD PRECAST PRESTRESSED DECKED GIRDER SUPERSTRUCTURE
GIRDER DETAILS I
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DESIGN TEAM
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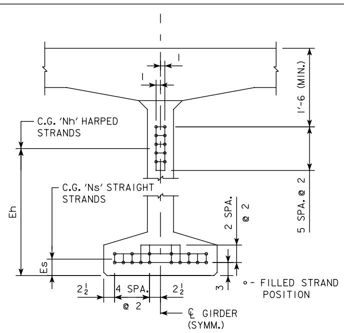
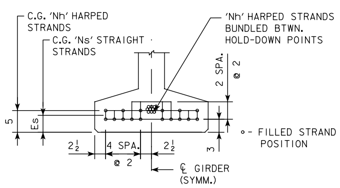
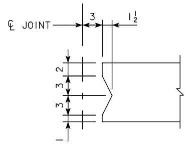


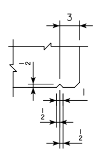
STRAND PATTERN AT ℄ BEARING
(EXAMPLE PATTERN SHOWN, SEE TABLE)



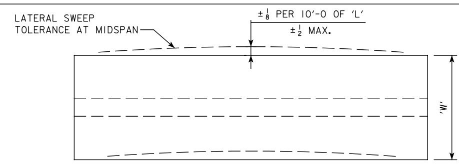
STRAND PATTERN AT MIDSPAN
(EXAMPLE PATTERN SHOWN, SEE TABLE)



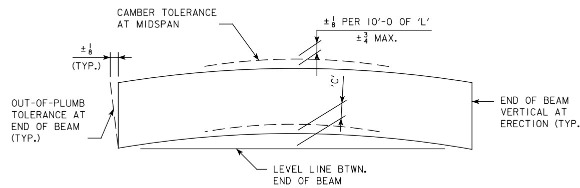
JOINT EDGE DETAIL



DRIP GROOVE DETAIL



FABRICATION TOLERANCE - PLAN



FABRICATION TOLERANCE - ELEVATION

EXAMPLE STRAND LAYOUT - EXTERIOR GIRDER

BEAM TYPE	'L' (FT)	'S' (FT)	f'c (KSI)	STRAND SIZE (IN)	STRAIGHT 'Ns'	STRAIGHT 'Es' (IN)	HARPED 'Nh'	HARPED 'Eh' (IN)	'N'	'E' (BRG.) (IN)	'E' (MID.) (IN)
DBT42	40	5	8	0.5	6	3.00	6	22.00	12	12.50	4.00
		8	8	0.5	8	3.00	8	21.00	16	12.00	4.00
	70	5	8	0.5	14	3.57	14	18.00	28	10.79	4.29
		8	8	0.5	20	4.00	16	17.00	36	9.78	4.44
DBT48	70	5	8	0.5	12	3.33	12	25.00	24	14.17	4.17
		8	8	0.5	16	3.75	14	24.00	30	13.20	4.33
	100	5	8	0.6	18	3.89	16	23.00	34	12.88	4.41
		7	8	0.6	24	4.50	16	23.00	40	11.90	4.70
DBT60	100	5	8	0.6	14	3.57	12	37.00	26	19.00	4.23
		8	8	0.6	18	3.89	16	35.00	34	18.53	4.41
	130	5	10	0.6	24	4.50	16	35.00	40	16.70	4.70
		SPACING LIMITED TO 5'-0 FOR 10 KSI CONCRETE									

EXAMPLE STRAND LAYOUT - INTERIOR GIRDER

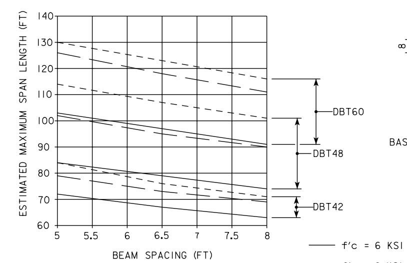
BEAM TYPE	'L' (FT)	'S' (FT)	f'c (KSI)	STRAND SIZE (IN)	STRAIGHT 'Ns'	STRAIGHT 'Es' (IN)	HARPED 'Nh'	HARPED 'Eh' (IN)	TOTAL 'N'	TOTAL 'E' (BRG.) (IN)	TOTAL 'E' (MID.) (IN)
DBT42	40	5	8	0.5	6	3.00	4	23.00	10	11.00	3.80
		8	8	0.5	6	3.00	6	22.00	12	12.50	4.00
	70	5	8	0.5	12	3.33	10	20.00	22	10.91	4.09
		8	8	0.5	16	3.75	14	18.00	30	10.40	4.33 *
DBT48	70	5	8	0.5	10	3.00	10	26.00	20	14.50	4.00
		8	8	0.5	14	3.57	14	24.00	28	13.79	4.29
	100	5	8	0.6	14	3.57	14	24.00	28	13.79	4.29
		7	8	0.6	16	3.75	16	23.00	32	13.38	4.38
DBT60	100	5	8	0.6	12	3.33	10	38.00	22	19.09	4.09
		8	8	0.6	14	3.57	14	36.00	28	19.79	4.29
	130	5	10	0.6	16	3.75	16	35.00	32	19.38	4.38
		7	10	0.6	24	4.50	16	35.00	40	16.70	4.70

* EXAMPLE PATTERN SHOWN ON THIS SHEET

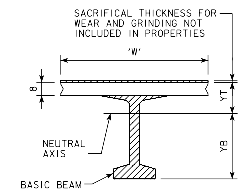
SECTION PROPERITES FOR BEAM DESIGN

BEAM TYPE	SECTION DEPTH 'H' (IN)	BEAM SPACING 'S' (FT)	FLANGE WIDTH 'B' (IN)	UNIT WEIGHT (NOTE 3) (KLF)	A (IN^2)	I (IN^4)	YT (IN)	YB (IN)	LIVE LOAD DISTRIBUTION BASIC BEAM A (IN^2)	LIVE LOAD DISTRIBUTION BASIC BEAM I (IN^4)	eg (IN)	Kg (n=1.0) (IN^4)
DBT42	42	5	54	0.978	874.5	173976	15.44	25.56	442.5	59851	22.62	286209
		6	66	1.094	970.5	185817	14.31	26.69				
		7	78	1.210	1066.5	195620	13.38	27.62				
		8	90	1.327	1162.5	203887	12.61	28.39				
DBT48	48	5	54	1.017	910.5	248325	17.69	29.31	478.5	91877	26.06	416757
		6	66	1.133	1006.5	265122	16.39	30.61				
		7	78	1.249	1102.5	279083	15.31	31.69				
		8	90	1.365	1198.5	290889	14.40	32.60				
DBT60	60	5	54	1.094	982.5	444100	22.36	36.64	550.5	181970	32.76	772893
		6	66	1.210	1078.5	474084	20.72	38.28				
		7	78	1.327	1174.5	499250	19.36	39.64				
		8	90	1.443	1270.5	520690	18.20	40.80				

NOTE: LIVE LOAD DISTRIBUTION PROPERTIES USE THE SECTION BELOW THE UNIFORM-DEPTH TOP FLANGE AS THE BASIC BEAM.



ESTIMATED MAXIMUM SPAN LENGTHS



SECTION PROPERTY DEFINITION

NOTES:

1. TOLERANCES ON THIS SHEET ARE INTENDED TO SUPPLEMENT THOSE PROVIDED IN APPENDIX B OF PCI MANUAL MNL-116.
2. PRESTRESSING STRANDS SHALL BE ½" OR 0.6" DIAM. GR. 270 LOW-RELAXATION SEVEN-WIRE STRANDS.
3. FOR EXAMPLE SHIPPING WEIGHTS, SEE SHEET NO. C9.
4. FOR EXAMPLE CAMBER AND DEFLECTIONS, SEE SHEET NO. C8.
5. GIRDERS SHALL BE SUPPORTED AGAINST TIPPING DURING RELEASE OF THE STRANDS AND STORAGE. GIRDERS SHALL BE STORED WITH WEBS VERTICAL.

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

STANDARD PRECAST PRESTRESSED
DECKED GIRDER SUPERSTRUCTURE

GIRDER DETAILS 2

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DESIGN TEAM

SHEET NUMBER C4

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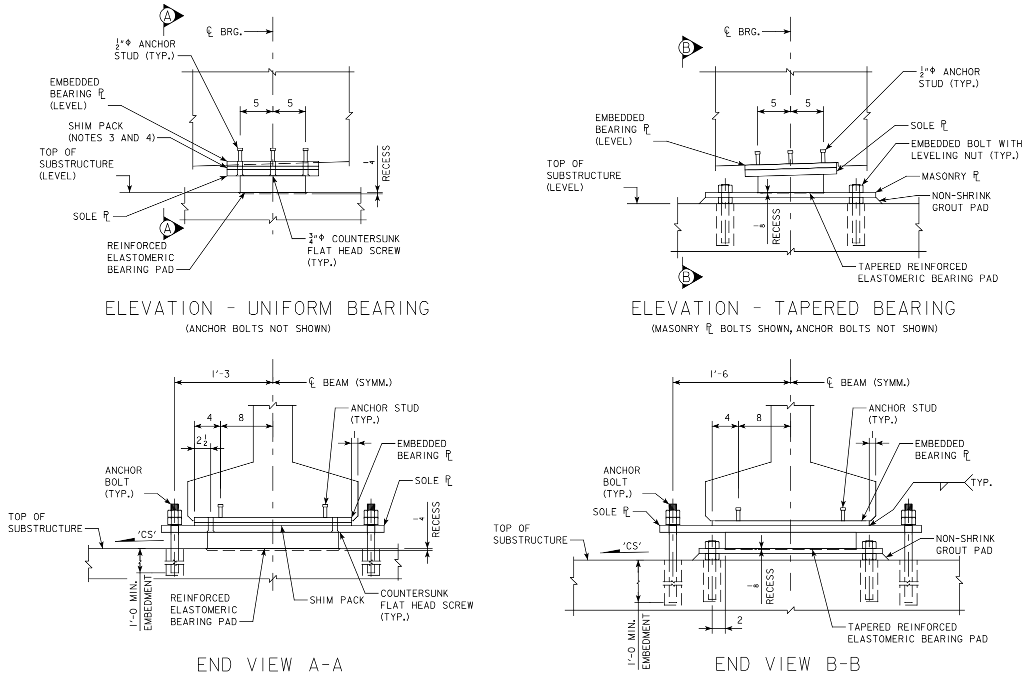

NOTES:

1. THIS BEARING DETAIL APPLIES TO SUPPORTS AT SEMI-INTEGRAL ABUTMENTS AND INTERMEDIATE PIERS.
2. BEARING DETAILS PROVIDED ON THIS SHEET ARE SIMILAR TO A CONVENTIONAL RECTANGULAR REINFORCED ELASTOMERIC BEARING PAD DESIGN WITH ANCHOR BOLTS PROVIDING POSITIVE LATERAL AND LONGITUDINAL CONNECTIONS TO THE SUBSTRUCTURE.
3. SHIM PACK CONSISTS OF TWO ¼" PLATES AS DETAILED. PLATES ARE TO BE ADDED OR REMOVED TO ADJUST TOP OF BEAM ELEVATION AT BEARING TO MATCH GRADE.
4. ALTERNATE DETAILS THAT INCORPORATE A VERTICAL ADJUSTMENT ALLOWANCE ARE PERMISSIBLE.
5. DESIGN OF THE BEARING PAD, ANCHOR BOLTS, AND ALL OTHER DETAILS SHALL CONFORM TO AGENCY STANDARDS AND ARE THE RESPONSIBILITY OF THE ENGINEER OF RECORD.
6. FOR VARIATIONS OF END VIEWS A-A AND B-B CONSISTENT WITH GIRDERS USED FOR SHORT SPAN LENGTHS AND/OR CRITICAL VERTICAL CLEARANCE CONDITIONS, SEE SHEET NO. C11.

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

STANDARD PRECAST PRESTRESSED DECKED GIRDER SUPERSTRUCTURE

BEARING DETAILS

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DESIGN TEAM SHEET NUMBER C5

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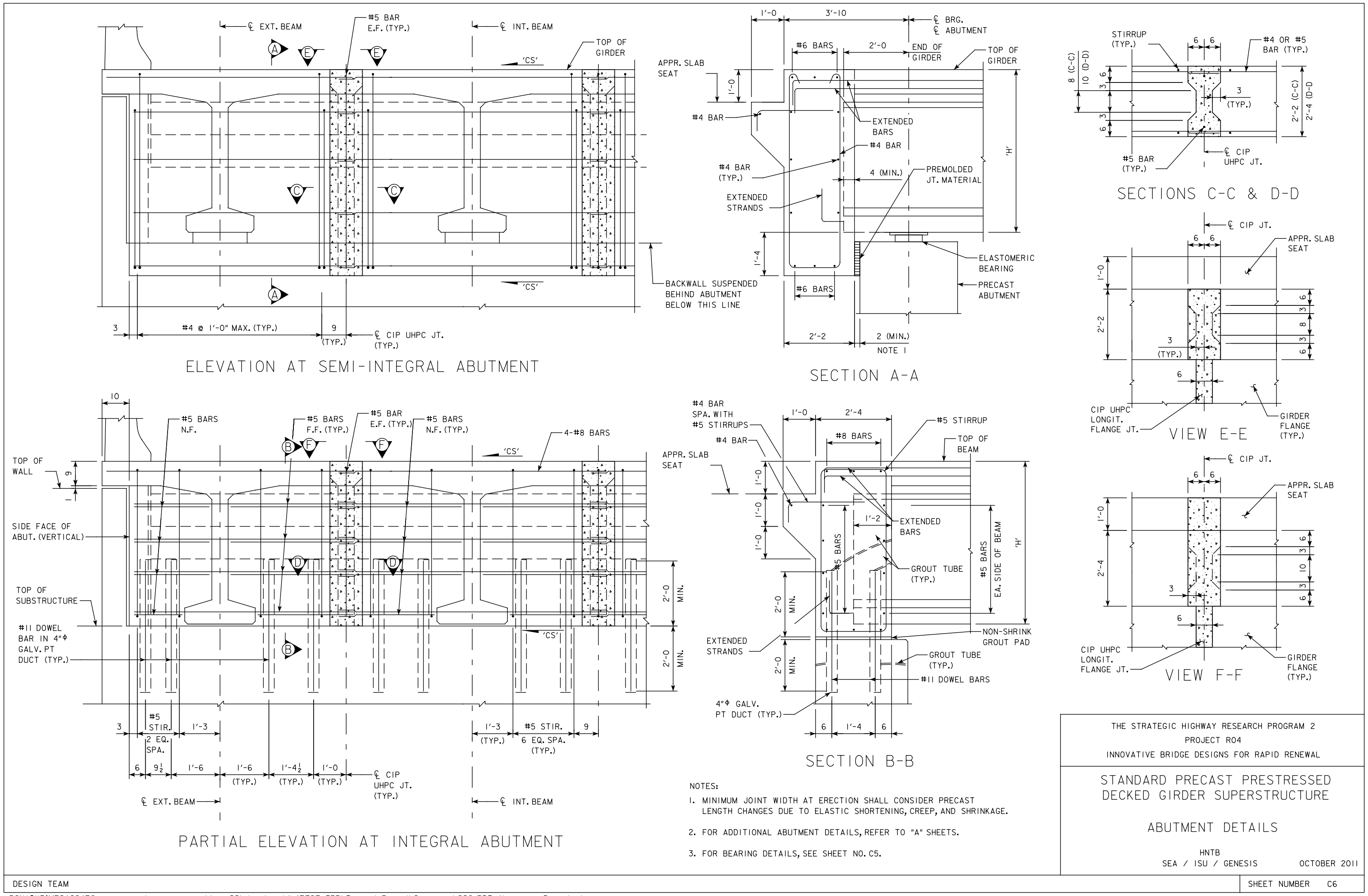

NOTES:

1. MINIMUM JOINT WIDTH AT ERECTION SHALL CONSIDER PRECAST LENGTH CHANGES DUE TO ELASTIC SHORTENING, CREEP, AND SHRINKAGE.
2. FOR ADDITIONAL ABUTMENT DETAILS, REFER TO "A" SHEETS.
3. FOR BEARING DETAILS, SEE SHEET NO. C5.

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

STANDARD PRECAST PRESTRESSED DECKED GIRDER SUPERSTRUCTURE

ABUTMENT DETAILS

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OCTOBER 2011

DESIGN TEAM
SHEET NUMBER C6

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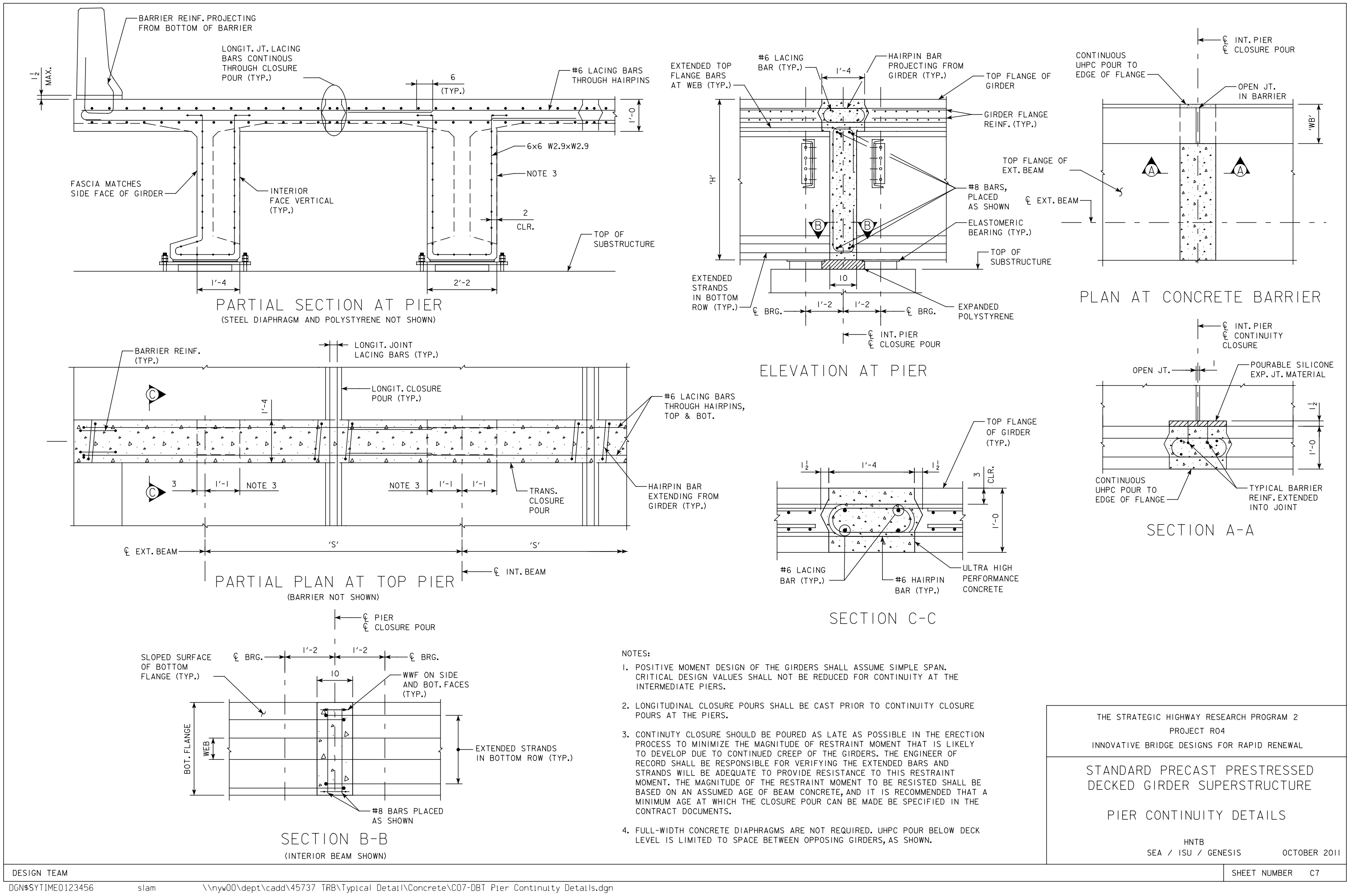
BARRIER REINF. PROJECTING FROM BOTTOM OF BARRIER
LONGIT. JT. LACING BARS CONTINOUS THROUGH CLOSURE POUR (TYP.)
#6 LACING BARS THROUGH HAIRPINS
6 (TYP.)
1½ MAX.
1'-0
6x6 W2.9xW2.9
NOTE 3
FASCIA MATCHES SIDE FACE OF GIRDER
INTERIOR FACE VERTICAL (TYP.)
2 CLR.
TOP OF SUBSTRUCTURE
1'-4
2'-2
PARTIAL SECTION AT PIER
(STEEL DIAPHRAGM AND POLYSTYRENE NOT SHOWN)
EXTENDED TOP FLANGE BARS AT WEB (TYP.)
#6 LACING BAR (TYP.)
HAIRPIN BAR PROJECTING FROM GIRDER (TYP.)
TOP FLANGE OF GIRDER
GIRDER FLANGE REINF. (TYP.)
'H'
#8 BARS, PLACED AS SHOWN
ELASTOMERIC BEARING (TYP.)
TOP OF SUBSTRUCTURE
EXTENDED STRANDS IN BOTTOM ROW (TYP.)
10
℄ BRG.
1'-2
1'-2
℄ BRG.
EXPANDED POLYSTYRENE
℄ INT. PIER
℄ CLOSURE POUR
ELEVATION AT PIER
℄ INT. PIER
℄ CLOSURE POUR
CONTINUOUS UHPC POUR TO EDGE OF FLANGE
OPEN JT. IN BARRIER
'WB'
TOP FLANGE OF EXT. BEAM
℄ EXT. BEAM
PLAN AT CONCRETE BARRIER
BARRIER REINF. (TYP.)
LONGIT. JOINT LACING BARS (TYP.)
LONGIT. CLOSURE POUR (TYP.)
#6 LACING BARS THROUGH HAIRPINS, TOP & BOT.
1'-4
3
1'-1
NOTE 3
NOTE 3
1'-1
1'-1
TRANS. CLOSURE POUR
HAIRPIN BAR EXTENDING FROM GIRDER (TYP.)
℄ EXT. BEAM
'S'
'S'
℄ INT. BEAM
PARTIAL PLAN AT TOP PIER
(BARRIER NOT SHOWN)
℄ INT. PIER
℄ CONTINUITY CLOSURE
OPEN JT.
1
POURABLE SILICONE EXP. JT. MATERIAL
1½
1'-0
CONTINUOUS UHPC POUR TO EDGE OF FLANGE
TYPICAL BARRIER REINF. EXTENDED INTO JOINT
SECTION A-A
TOP FLANGE OF GIRDER (TYP.)
1½
1'-4
1½
3 CLR.
1'-0
#6 LACING BAR (TYP.)
#6 HAIRPIN BAR (TYP.)
ULTRA HIGH PERFORMANCE CONCRETE
SECTION C-C
℄ PIER
℄ CLOSURE POUR
SLOPED SURFACE OF BOTTOM FLANGE (TYP.)
℄ BRG.
1'-2
1'-2
℄ BRG.
10
WWF ON SIDE AND BOT. FACES (TYP.)
BOT. FLANGE
WEB
EXTENDED STRANDS IN BOTTOM ROW (TYP.)
#8 BARS PLACED AS SHOWN
SECTION B-B
(INTERIOR BEAM SHOWN)
NOTES:
1. POSITIVE MOMENT DESIGN OF THE GIRDERS SHALL ASSUME SIMPLE SPAN. CRITICAL DESIGN VALUES SHALL NOT BE REDUCED FOR CONTINUITY AT THE INTERMEDIATE PIERS.
2. LONGITUDINAL CLOSURE POURS SHALL BE CAST PRIOR TO CONTINUITY CLOSURE POURS AT THE PIERS.
3. CONTINUITY CLOSURE SHOULD BE POURED AS LATE AS POSSIBLE IN THE ERECTION PROCESS TO MINIMIZE THE MAGNITUDE OF RESTRAINT MOMENT THAT IS LIKELY TO DEVELOP DUE TO CONTINUED CREEP OF THE GIRDERS. THE ENGINEER OF RECORD SHALL BE RESPONSIBLE FOR VERIFYING THE EXTENDED BARS AND STRANDS WILL BE ADEQUATE TO PROVIDE RESISTANCE TO THIS RESTRAINT MOMENT. THE MAGNITUDE OF THE RESTRAINT MOMENT TO BE RESISTED SHALL BE BASED ON AN ASSUMED AGE OF BEAM CONCRETE, AND IT IS RECOMMENDED THAT A MINIMUM AGE AT WHICH THE CLOSURE POUR CAN BE MADE BE SPECIFIED IN THE CONTRACT DOCUMENTS.
4. FULL-WIDTH CONCRETE DIAPHRAGMS ARE NOT REQUIRED. UHPC POUR BELOW DECK LEVEL IS LIMITED TO SPACE BETWEEN OPPOSING GIRDERS, AS SHOWN.
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PIER CONTINUITY DETAILS
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OCTOBER 2011
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SHEET NUMBER C7
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CAMBER NOTES:

1. TO THE EXTENT PRACTICAL, FABRICATION OF BEAMS WITHIN A GIVEN SPAN SHALL BE SCHEDULED SUCH THAT CURING CONDITIONS, CURING PROCEDURES, SUPPORT POINTS IN STORAGE, TIME TO SHIPPING, AND TIME TO ERECTION ARE SIMILAR TO MINIMIZED DIFFERENCES IN CAMBER BETWEEN ADJACENT BEAMS IN THE FIELD.

2. TO INCREASE RELIABILITY OF CAMBER PREDICITION, CONSIDERATION SHOULD BE GIVEN TO MAXIMIZING TIME IN THE CASTING BED PRIOR TO DETENSIONING

3. THE CONTRACTOR SHALL MEASURE AND RECORD VERTICAL DEFLECTION (CAMBER) OF EACH GIRDER AT RELEASE OF PRESTRESS, AND DURING THE PERIOD OF TIME BETWEEN RELEASE OF PRESTRESS AND SHIPPING AT AN INTERVAL NOT TO EXCEED 7 DAYS. FINAL MEASUREMENT IN STORAGE SHALL OCCUR NO MORE THAN 72 HOURS PRIOR TO SHIPPING. MEASUREMENTS FOR ALL GIRDERS WITHIN A SPAN SHALL BE TAKEN AT THE SAME TIME.

4. CONTRACTOR SHALL ENSURE THAT THE PREFABRICATED COMPONENTS WILL FIT UP AND ALIGN PROPERLY BEFORE SHIPPING FROM THE PRECAST FACILITY. CONSIDER ASSEMBLING EACH SUPERSTRUCTURE AND SUBSTRUCTURE COMPOSED OF PREFABRICATED COMPONENTS IN THE YARD PRIOR TO SHIPPING THE COMPONENTS TO THE PROJECT SITE. IF ASSEMBLED IN THE YARD, USE BLOCKING TO SIMULATE THE SUPPORT OF THE COMPONENTS, AND THE SPACING BETWEEN THE COMPONENTS. VERIFY THE CONSTRUCTION OF ALL COMPONENTS UNITS IN COMPLIANCE WITH ALL PLAN REQUIREMENTS.

5. AFTER ALL BEAMS IN A SPAN HAVE BEEN PLACED, PRIOR TO POURING LONGITUDINAL OR TRANSVERSE CLOSURE JOINTS, MEASURE AND RECORD THE RELATIVE VERTICAL DISTANCE BETWEEN FLANGE TIPS OF ADJACENT BEAMS.

6. DIFFERENTIAL CAMBER SHALL BE EQUALIZED WHEN THE RELATIVE VERTICAL DISTANCE BETWEEN FLANGE TIPS OF ADJACENT BEAMS EXCEEDS $\frac{1}{4}$".

7. THE MAXIMUM PERMISSIBLE RELATIVE VERTICAL DISTANCE BETWEEN FLANGE TIPS OF ADJACENT BEAMS IS $\frac{3}{4}$". WHERE THIS MAXIMUM VALUE IS EXCEEDED, VERTICALLY ADJUST ONE OR BOTH BEARINGS TO BRING THE MAXIMUM MEASURED VALUE WITHIN THIS LIMIT PRIOR TO INITIATING CAMBER LEVELING PROCEDURES.

8. EXTERIOR BEAMS WITH PRECAST BARRIERS HAVE HIGHER DEAD LOAD AND LEVELS OF PRESTRESSING THAN TYPICAL INTERIOR BEAMS. AS A RESULT, CRITICAL DIFFERENTIAL CAMBER MEASUREMENTS ARE EXPECTED BETWEEN EXTERIOR BEAMS AND THE ADJACENT INTERIOR BEAM. SEE TABLE ON THIS SHEET. ADDITIONAL PRESTRESS IN THE INTERIOR BEAMS MAY BE USED TO MINIMIZE THIS DIFFERENCE, PROVIDED ALL DESIGN REQUIREMENTS ARE SATISFIED.

9. SHOP DRAWING SUBMITTALS FOR THE PRESTRESSED BEAMS SHALL INCLUDE A PROCEDURE FOR CONTROLLING CAMBER AND REMOVING DIFFERENTIAL CAMBER (CAMBER EQUALIZATION) DURING FABRICATION, STORAGE, TRANSPORTATION, AND ERECTION.

10. HARDWARE REQUIRED FOR THE APPROVED CAMBER EQUALIZATION METHOD SHALL BE FULLY DETAILED ON THE SHOP DRAWINGS AND EMBEDDED IN THE INITIAL PRECAST SECTION. POST-INSTALLED SLEEVES, THREADED INSERTS, ANCHORS, DOWELS, AND OTHER HARDWARE ARE NOT PERMITTED.

CAMBER LEVELING NOTES

1. ERECT ALL BEAMS WITHIN A SPAN PRIOR TO CAMBER EQUALIZATION.

2. ONCE ALL BEAMS WITHIN A SPAN ARE ERECTED, IDENTIFY THE BEAM WITH THE MOST CAMBER (HIGH BEAM) AND THE BEAM WITH THE LEAST CAMBER (LOW BEAM).

3. WHERE THE FLANGE TIP OF THE HIGH BEAM IS MORE THAN $\frac{1}{4}$" ABOVE THE FLANGE TIP OF AN ADJACENT BEAM, THE HIGH BEAM IS CONSIDERED TO HAVE EXCESSIVE CAMBER THAT SHALL BE REDUCED. TYPICAL METHODS FOR REDUCING EXCESSIVE CAMBER INCLUDE SURCHARGE LOADING AND HYDRAULIC JACKING.

4. WHERE THE FLANGE TIP OF THE LOW BEAM IS MORE THAN $\frac{1}{4}$" BELOW THE FLANGE TIP OF AN ADJACENT BEAM, THE LOW BEAM IS CONSIDERED TO HAVE DEFICIENT CAMBER THAT SHALL BE INCREASED. TYPICAL METHODS FOR INCREASING DEFICIENT CAMBER INCLUDE HYDRAULIC JACKING AND BEARING ELEVATION ADJUSTMENT.

5. A CLAMPING SYSTEM COMPRISED OF UPPER AND LOWER STRONGBACKS SPANNING THE JOINT TRANSVERSELY AT REGULARLY SPACED INTERVALS ALONG THE LENGTH OF THE SPAN CAN ALSO BE EFFECTIVE FOR ALIGNING ADJACENT BEAMS. THIS METHOD CAN BE COMBINED WITH CRANE ASSISTED LEVELING WHERE ONE END OF THE GIRDER WITH THE LEAST CAMBER OF THE PAIR BEING ALIGNED IS INCREMENTALLY LOWERED TO BEARING AS CLAMPS ARE INSTALLED ALONG THE LENGTH OF THE JOINT.

6. THE METHOD TO BE EMPLOYED BY THE CONTRACTOR SHALL BE APPROVED BY THE OWNER OR OWNER'S REPRESENTATIVE PRIOR TO FABRICATION OF THE BEAMS.

7. REGARDLESS OF THE METHOD USED, THE LOAD APPLIED FOR PURPOSES OF CAMBER LEVELING SHALL NOT BE REMOVED UNTIL THE UHPC CONCRETE IN THE LONGITUDINAL JOINT HAS ATTAINED A STRENGTH OF 10 KSI.

8. ADDITIONAL STIFFNESS OF EXTERIOR BEAMS DUE TO PRESENCE OF BARRIER SHALL BE CONSIDERED.

BEAM PLACEMENT NOTES FOR NORMAL BRIDGES:

1. SET BEAMS WITH WEBS ORIENTED NORMAL TO CROSS-SLOPE, UP TO A MAXIMUM SLOPE OF 4 PERCENT.

2. FINAL ROADWAY PROFILE, SPECIFICALLY INCOMING AND OUTGOING LONGITUDINAL GRADE, SHALL CONSIDER THE SHAPE OF THE CAMBERED PRESTRESSED GIRDER TO MINIMIZE DIAMOND-GRINDING OF THE RIDING SURFACE.

3. FOR SINGLE-SPAN BRIDGES, THE FINAL PROFILE GRADE IS ESTABLISHED BY THE EQUALIZED CAMBER OF THE SPAN. APPROACH SLAB GRADE SHOULD BE SET BASED ON THE ANTICIPATED END SLOPE OF THE SPAN AT AN AGE ESTABLISHED BY THE OWNER OR OWNER'S REPRESENTATIVE, PREFERABLY NOT LESS THAN 240 DAYS, WHICH CORRESPONDS TO THE AGE AT WHICH 85 TO 90 PERCENT OF THEORETICAL CAMBER IS ASSUMED TO HAVE TAKEN PLACE.

4. WHERE THE ACTUAL CAMBERS OF ALL BEAMS IN A SPAN DIFFERS UNIFORMLY FROM THE THEORETICAL VALUES DURING THE MONITORING PHASE, FINAL CAMBER AND BEAM ROTATION PREDICTIONS MAY BE REVISED AND APPROACH GRADES BASED ON END SLOPES ADJUSTED ACCORDINGLY.

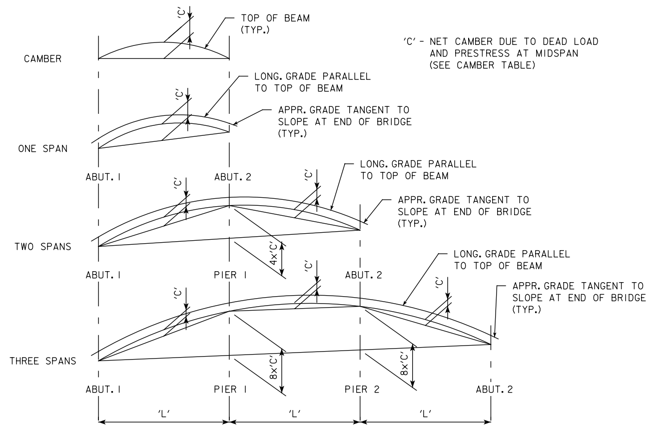
5. FOR MULTIPLE-SPAN BRIDGES, THE FINAL PROFILE GRADE IS ESTABLISHED BY FITTING A PARABOLA TO THE THEORETICAL CAMBER PROFILES OF ALL SPANS IN SEQUENCE, ASSUMING NO BREAK IN SLOPE ACROSS INTERMEDIATE PIER LINES. THIS IS ACCOMPLISHED BY ADJUSTING THE SUBSTRUCTURE ELEVATIONS AT BEARING LOCATIONS BASED ON PREDICTED CAMBER VALUES. SEE PLACEMENT DETAIL ILLUSTRATING THIS CONCEPT FOR TWO-SPAN AND THREE-SPAN BRIDGES WITH UNIFORM SPAN LENGTH.

6. WHERE CREST PROFILES ARE UNDESIRABLE AND/OR SAG PROFILES ARE UNAVOIDABLE, CONSIDERATION SHOULD BE GIVEN TO VARYING THE DEPTH OF THE GIRDERS ALONG THE LENGTH TO ESTABLISH THE PROFILE GRADE.

BEAM PLACEMENT NOTES FOR SKEWED BRIDGES:

1. FOR SINGLE-SPAN AND MULTIPLE-SPAN SKEWED BRIDGES, NOTES FOR NORMAL BRIDGES APPLY.

2. IN ADDITION TO SUBSTRUCTURE ELEVATION ADJUSTMENT FOR PROFILE GRADE. SKEWED BRIDGES MAY REQUIRE ADDITIONAL VERTICAL ADJUSTMENT OF BEARINGS ALONG THE CAP TO ELIMINATE A SAWTOOTH EFFECT BETWEEN FLANGE TIPS AT THE ENDS OF EACH SPAN.



PLACEMENT DETAIL FOR UNIFORM SPAN LENGTHS

EXAMPLE DEFLECTION AND CAMBER TABLE

GIRDER	DEFLECTION					NET CAMBER								
	BEAM WEIGHT	BARRIER	LONGIT. JOINT	WEARING SURFACE	INITIAL PRETRESS	RELEASE	DAY 7	DAY 14	DAY 21	DAY 28	DAY 60	DAY 120	DAY 240	FINAL
INTERIOR	-0.917"	0"	-0.025"	-0.097"	1.915"	0.834"	0.989"	1.121"	1.220"	1.297"	1.467"	1.618"	1.722"	1.857"
EXTERIOR	-0.917"	-0.263"	-0.013"	-0.079"	2.122"	1.205"	1.429"	1.621"	1.494"	1.568"	1.766"	1.939"	2.063"	2.229"
CAMBER DIFF. = EXTERIOR - INTERIOR						0.371"	0.440"	0.400"	0.474"	0.271"	0.299"	0.321"	0.341"	0.372"

NOTES:

1. TABULATED DEFLECTION AND CAMBER VALUES WERE COMPUTED FOR INTERIOR AND EXTERIOR DBT42 BEAMS SPACED AT 8'-0 WITH A SPAN LENGTH OF 70'-0.

2. PRESTRESS LAYOUTS FOR INTERIOR AND EXTERIOR BEAMS ARE SHOWN ON SHEET NO. C5.

3. CAMBER COMPUTED USING THE FOLLOWING ASSUMPTIONS:

   A. BARRIER INSTALLED ON EXTERIOR GIRDER ON DAY 20.

   B. LONGITUDINAL JOINT IS INSTALLED ON DAY 30.

   C. WEARING SURFACE IS APPLIED AT FINAL.

4. POSTIVE VALUES REPRESENT UPWARD DEFLECTION.

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CAMBER AND PLACEMENT NOTES

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SHEET NUMBER C8

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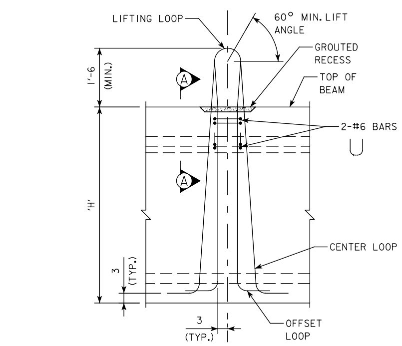
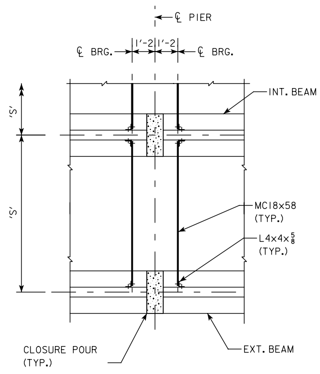


LIFTING LOOP DETAIL



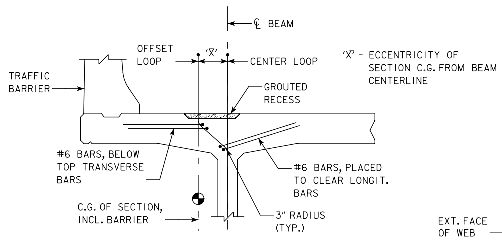
PARTIAL FRAMING PLAN AT INT. PIER

EXAMPLE PRECAST WEIGHTS

BEAM TYPE	'S' (FT)	'B' (IN)	ABUTMENT WEIGHT (TON) INT.	ABUTMENT WEIGHT (TON) SEMI-INT.	'L' (FT)	BEAM WEIGHT (TON) INT.	BEAM WEIGHT (TON) EXT.
DBT42	5	54	3	4	40	22	32
					70	37	53
	8	90	5	6.5	40	30	40
					70	50	66
DBT48	5	54	3.5	4.5	70	39	55
					100	54	77
	8	90	6.5	7.5	70	52	68
					100	73	95
DBT60	5	54	4	5	100	58	81
					130	75	104
	8	90	7	8.5	100	77	99
					130	99	127

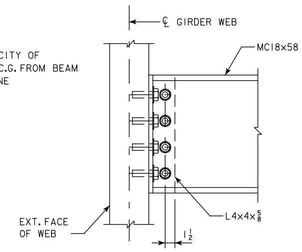
TABLE NOTES:

1. THIS TABLE PRESENTS SAMPLE LIFTING WEIGHTS FOR LIMITING DIMENSIONS OF EACH BEAM TYPE.
2. EXTERIOR BEAM WEIGHT INCLUDES THE WEIGHT OF THE TRAFFIC BARRIER.
3. BEAM WEIGHT AT THE UPPER END OF THE SPAN RANGE FOR DBT60 SECTIONS CAN EXCEED 100 TONS FOR AN EXTERIOR BEAM DUE TO THE WEIGHT OF THE PRECAST TRAFFIC BARRIER.

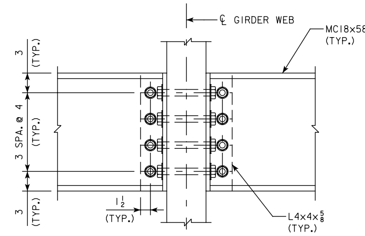


SECTION A-A

(EXT. BEAM SHOWN, OFFSET LOOP MAY BE OMITTED FOR INT. BEAM)



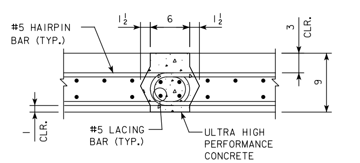
DIAPHRAGM CONNECTION AT EXTERIOR GIRDER



DIAPHRAGM CONNECTION AT INTERIOR GIRDER

NOTES:

1. LIFTING LOOPS SHALL BE DESIGNED BY THE CONTRACTOR'S ENGINEER AND SUBMITTED FOR APPROVAL. MINIMUM SUBMITTAL REQUIREMENTS INCLUDE LOOP CAPACITY, EMBEDMENT SUFFICIENT TO DEVELOP THE LIFTING FORCE, AND BEAM STABILITY VERIFICATION. ALL SUBMITTALS SHALL BE SIGNED AND SEALED BY A LICENSED PROFESSIONAL ENGINEER.
2. LIFTING LOOPS SHALL BE 0.5" OR 0.6" DIAMETER GRADE 270 LOW-RELAXATION STRANDS. NUMBER OF STRANDS PER LOOP TO BE DETERMINED BY CONTRACTOR. EACH LOOP SHALL BE FITTED WITH A GALVANIZED PIPE SLEEVE EXTENDING BEYOND THE CURVED REGION.
3. FIELD CUT LIFTING LOOP AND BURN OFF TO 1" MIN. BELOW TOP OF BEAM. FILL RECESS WITH NON-SHRINK GROUT AND FINISH FLUSH WITH SURFACE PRIOR TO GROOVING OR GRINDING THE TOP FLANGE.
4. TILT U-SHAPED BARS AS NEED TO SATISFY COVER REQUIREMENTS.
5. STEEL DIAPHRAGMS AT INTERMEDIATE PIERS MAY NOT BE NECESSARY. THE ENGINEER SHALL BE RESPONSIBLE FOR EVALUATING THE NEED FOR DIAPHRAGMS BASED ON STABILITY OR GOVERNING AGENCY DESIGN REQUIREMENTS.



LONGITUDINAL CLOSURE POUR DETAIL

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MISCELLANEOUS DETAILS

HNTB
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DESIGN TEAM   SHEET NUMBER C9

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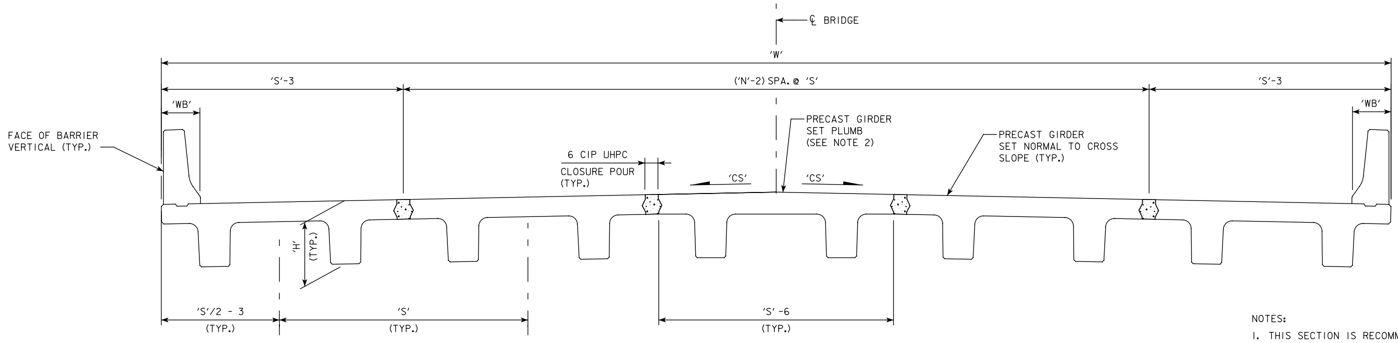
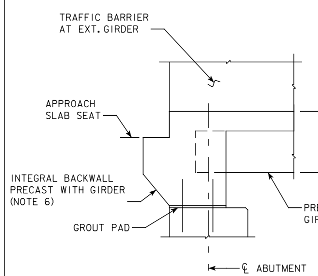


TYPICAL SECTION

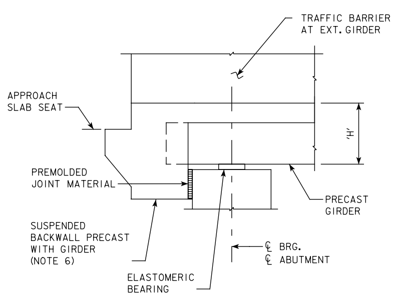
TYPICAL SECTION DIMENSIONS		
DESCRIPTION	VARIABLE	VALUE
OUT-OUT WIDTH	'W'	47'-2
BARRIER WIDTH	'WB'	1'-7
NO. OF GIRDERS	'N'	5
GIRDER SPACING	'S'	9'-6$\frac{3}{8}$
GIRDER HEIGHT	'H'	2'-4
CROSS SLOPE	'CS'	2 %

NOTES:

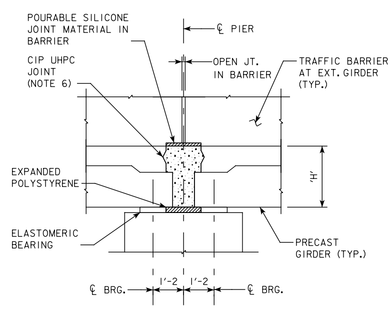
1. THIS SECTION IS RECOMMENDED FOR SHORTER SPANS AND WHERE VERTICAL CLEARANCE IS CRITICAL. STANDARD DEPTHS FOR THIS SECTION ARE 28", 32", 36" AND 40" WITH APPROXIMATE MAXIMUM SPANS OF 70' TO 90' OVER THE RANGE OF SIZES.
2. DETAILS PRESENTED HERE ARE SUITABLE FOR APPLICATIONS WITH CONSTANT CROSS-SLOPE AND NO SKEW. THEY CAN BE READILY ADAPTED TO ACCOMMODATE LOW TO MODERATE SKEW ANGLES WITH PARALLED LINES OF SUPPORT.
3. SECTION WITH ODD NUMBER OF GIRDERS SHOWN. WHERE EVEN NUMBER IS APPROPRIATE, ALL GIRDERS ARE SET NORMAL TO THE CROSS-SLOPE.
4. UPON COMPLETION OF DECK CLOSURE POURS, TOP OF DECK SHALL BE DIAMOND GROUND TO MAXIMUM DEPTH OF $\frac{1}{2}$" AND GROOVED FOR SKID RESISTANCE.
5. FOR UHPC JOINT REQUIREMENTS, SEE GENERAL INFORMATION SHEET.
6. ABUTMENT AND PIER CONTINUITY CONCEPTS PRESENTED ON SHEET NOS. C6 AND C7 FOR SINGLE WEB BEAMS APPLY.
7. FOR CLOSURE POUR DETAIL, SEE SHEET NO. C9.
8. ALL CONCRETE FACES INCLUDED IN CAST-IN-PLACE CLOSURE POURS AND JOINTS SHALL BE CLEANED AND COATED WITH AN EPOXY BONDING AGENT APPROVED BY THE AGENCY OR PROJECT SPECIFICATIONS AND INSTALLED IN ACCORDANCE WITH THE MANUFACTURER'S INSTRUCTIONS PRIOR TO POURING THE CONCRETE.



ELEVATION AT INTEGRAL ABUTMENT



ELEVATION AT SEMI-INTEGRAL ABUTMENT



ELEVATION AT INTERMEDIATE PIER

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STANDARD PRECAST PRESTRESSED DECKED GIRDER SUPERSTRUCTURE

ALTERNATE TYPICAL SECTION

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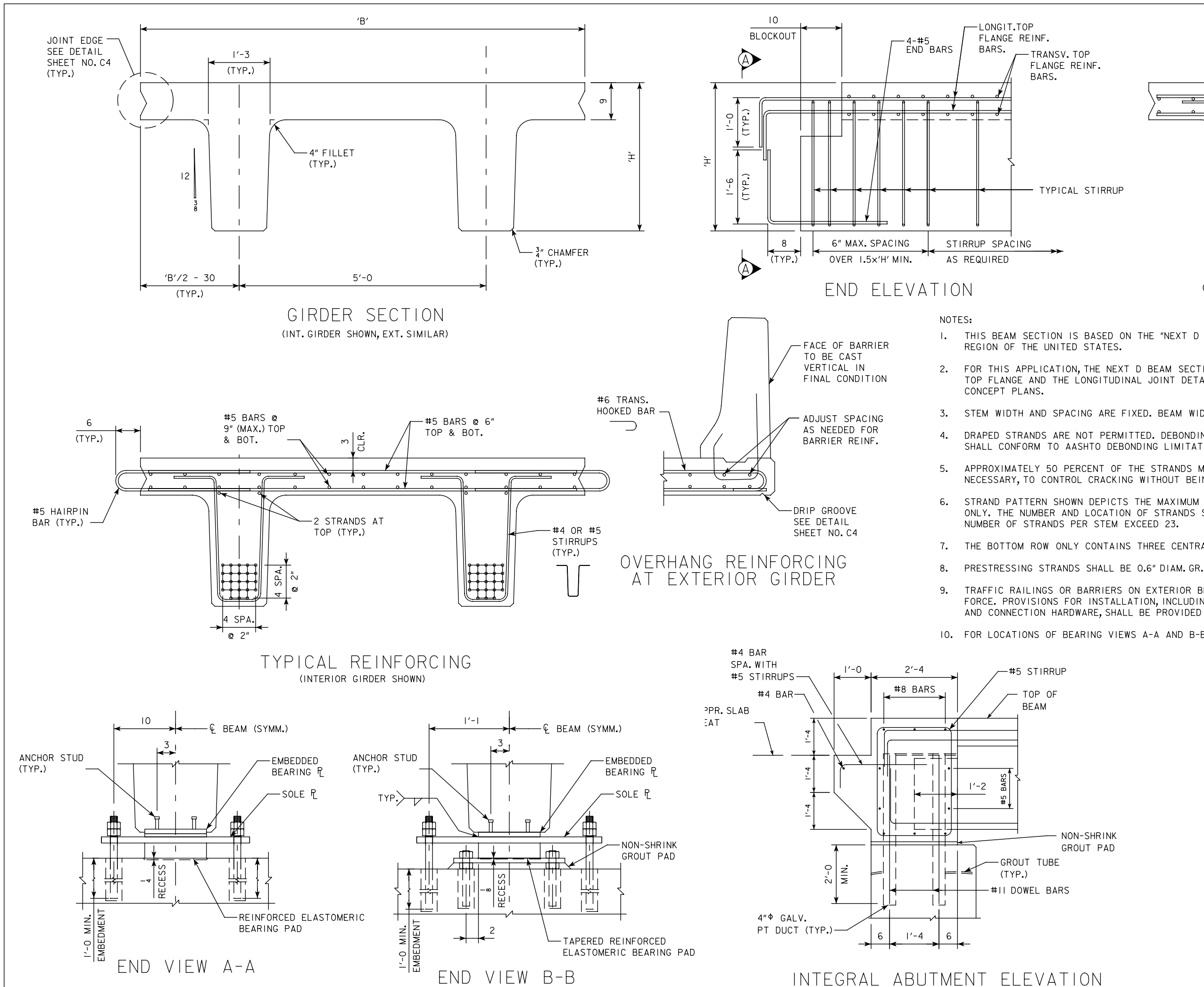

NOTES:

1. THIS BEAM SECTION IS BASED ON THE "NEXT D BEAM" STANDARDS DEVELOPED BY PCI FOR USE IN THE NORTHEAST REGION OF THE UNITED STATES.
2. FOR THIS APPLICATION, THE NEXT D BEAM SECTION IS MODIFIED TO ADD 1" OF SACRIFICIAL THICKNESS TO THE TOP FLANGE AND THE LONGITUDINAL JOINT DETAILS ARE MADE CONSISTENT WITH THE DETAILS PROVIDED IN THESE CONCEPT PLANS.
3. STEM WIDTH AND SPACING ARE FIXED. BEAM WIDTH IS MODIFIED BY ADJUSTING THE OVERHANG LENGTH.
4. DRAPED STRANDS ARE NOT PERMITTED. DEBONDING OF STRANDS TO LIMIT END ZONE STRESSES IS PERMITTED AND SHALL CONFORM TO AASHTO DEBONDING LIMITATIONS, AT A MINIMUM.
5. APPROXIMATELY 50 PERCENT OF THE STRANDS MAY BE DEBONDED WITHIN 6 INCHES OF THE END OF THE BEAM, IF NECESSARY, TO CONTROL CRACKING WITHOUT BEING SUBJECT TO REQUIREMENTS OF NOTE 2.
6. STRAND PATTERN SHOWN DEPICTS THE MAXIMUM NUMBER OF STRANDS PERMITTED AND IS FOR EXAMPLE PURPOSES ONLY. THE NUMBER AND LOCATION OF STRANDS SHALL BE DETERMINED BY DESIGN, BUT IN NO CASE SHALL THE NUMBER OF STRANDS PER STEM EXCEED 23.
7. THE BOTTOM ROW ONLY CONTAINS THREE CENTRALLY LOCATED STRANDS TO PROVIDE FOR STIRRUP BAR BENDS.
8. PRESTRESSING STRANDS SHALL BE 0.6" DIAM. GR. 270 LOW-RELAXATION SEVEN-WIRE STRANDS.
9. TRAFFIC RAILINGS OR BARRIERS ON EXTERIOR BEAMS SHALL NOT BE INSTALLED PRIOR TO TRANSFER OF PRESTRESS FORCE. PROVISIONS FOR INSTALLATION, INCLUDING BUT NOT LIMITED TO, SHEAR KEYS, REINFORCING STEEL, AND CONNECTION HARDWARE, SHALL BE PROVIDED IN THE INITIAL PRECAST SECTION.
10. FOR LOCATIONS OF BEARING VIEWS A-A AND B-B, SEE SHEET NO. C5.

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STANDARD PRECAST PRESTRESSED DECKED GIRDER SUPERSTRUCTURE

ALTERNATE GIRDER DETAILS

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DESIGN TEAM

SHEET NUMBER C11

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GENERAL INFORMATION

PREFABRICATED COMPONENTS PRODUCED OFF–SITE CAN BE QUICKLY ASSEMBLED, AND CAN REDUCE CONSTRUCTION TIME, COST, MINIMIZE LANE CLOSURE TIME AND/THE NEED FOR A TEMPORARY BRIDGE. TYPICAL DESIGNS FOR SUPERSTRUCTURE AND SUBSTRUCTURE MODULES HAVE BEEN GROUPED INTO THE FOLLOWING SPAN RANGES:

- 40 FT ≤ SPAN ≤ 70 FT
- 70 FT ≤ SPAN ≤ 100 FT
- 100 FT ≤ SPAN ≤ 130 FT

THE INTENT OF THESE DESIGN STANDARDS IS TO PROVIDE INFORMATION THAT APPLIES TO THE DESIGN, DETAILING, FABRICATION, HANDLING AND ASSEMBLY OF PREFABRICATED COMPONENTS USED IN ACCELERATED BRIDGE CONSTRUCTION.

THE ERECTION CONCEPTS PRESENTED IN THESE DRAWINGS ARE INTENDED TO ASSIST THE OWNER, DESIGNER, AND THE CONTRACTOR IN SELECTING THE SUITABLE ERECTION EQUIPMENT FOR THE HANDLING AND ASSEMBLY OF THESE PREFABRICATED MODULAR SYSTEMS.

GENERAL NOTES:

DESIGN SPECIFICATIONS

DESIGN SPECIFICATIONS FOR TEMPORARY STRUCTURES USED IN BRIDGE CONSTRUCTION ARE DESIGNED TO A VARIETY OF STANDARDS DEPENDENT UPON THE SPECIFIC PROJECT REQUIREMENTS. APPLICABLE SPECIFICATIONS MAY INCLUDE:

1. AASHTO "GUIDE DESIGN SPECIFICATIONS FOR BRIDGE TEMPORARY WORKS", 1ST EDITION, 2008 INTERIMS.
2. AASHTO "LRFD BRIDGE DESIGN SPECIFICATIONS", 5TH EDITION, 2010 INTERIM REVISIONS.
3. AASHTO "LRFD BRIDGE CONSTRUCTION SPECIFICATIONS", 3RD EDITION, 2010.
4. AISC "STEEL CONSTRUCTION MANUAL", 13TH EDITION.
5. PROJECT–SPECIFIC AND STATE–SPECIFIC DESIGN REQUIREMENTS.

BRIDGE ERECTION REPONSIBILITIES

SAFE ERECTION OF THE BRIDGE IS ALWAYS THE RESPONSIBILITY OF THE CONTRACTOR.

ENGINEERING FOR UNIQUE CONSTRUCTION METHODS IS OFTEN CRITICAL FOR SAFE AND EFFECTIVE BRIDGE ERECTION OPERATIONS. PLANNING AND ENGINEERING SPECIFIC CONSTRUCTION OPERATIONS IS PERFORMED BY THE CONTRACTOR AND HIS OR HER ENGINEER, BUT ANTICIPATING THE CONSTRUCTION OPERATIONS EARLY IN THE PROJECT DESIGN PHASE CAN HAVE SIGNIFICANT BENEFITS.

DESIGNER – CONTRACTOR COMMUNICATIONS

THE BRIDGE DESIGNER CAN INFLUENCE THE ERECTION TECHNIQUE AND POTENTIALLY REDUCE CONSTRUCTION COSTS BY CONSIDERING THE LIKELY ERECTION METHODS DURING THE DESIGN OF A NEW STRUCTURE. FOR INSTANCE, IF THE ANTICIPATED ERECTION TECHNIQUE REQUIRES THAT THE NEW STRUCTURE SUPPORT A GANTRY SYSTEM OR CRAWLER CRANE TEMPORARILY, THESE TEMPORARY LOADING CONDITIONS (WITH APPROPRIATE LOAD AND IMPACT FACTORS) COULD BE CONSIDERED IN THE DESIGN OF THE NEW STRUCTURE. DESIGN DETAILS TO ACCOMMODATE THE ANTICIPATED ERECTION SEQUENCE CAN OFTEN BE IMPLEMENTED EARLY IN THE DESIGN PROCESS AT A LOW COST TO THE PROJECT RESULTING IN POTENTIALLY SIGNIFICANT OVERALL PROJECT COST SAVINGS DUE TO DECREASED CONSTRUCTION COSTS.

COMMUNICATION BETWEEN THE DESIGNER AND POTENTIAL CONTRACTORS EARLY IN THE DESIGN PHASE IS CRITICAL TO IDENTIFYING THE ANTICIPATED ERECTION TECHNIQUE FOR A SPECIFIC PROJECT. DISCUSSING POTENTIAL CONSTRUCTION METHODS WITH SEVERAL CONTRACTORS IS BENEFICIAL BECAUSE OFTEN EACH CONTRACTOR CAN OFFER A UNIQUE OPINION ON POTENTIAL CONSTRUCTION TECHNIQUES THAT CAN ALL BE TAKEN INTO CONSIDERATION FOR PLANNING PURPOSES. DISCUSSION WITH MULTIPLE POTENTIAL CONTRACTORS ALSO GROWS COMMUNITY AWARENESS OF THE PROJECT AND CAN INDUCE COMPETITION BETWEEN POTENTIAL BUILDERS, LIKELY RESULTING IN REDUCED OVERALL PROJECT COSTS.

THE ERECTION SEQUENCES SHOWN ARE NOT REPRESENTED AS BEING THE COMPLETE STEP–BY–STEP PROCEDURE AND ARE INCLUDED FOR INFORMATION ONLY. THE CONTRACTOR SHALL DETERMINE THE COMPLETE ERECTION SEQUENCE AND EQUIPMENT REQUIRED FOR ERECTION OF THE BRIDGE.

THE CONTRACTOR SHALL BE RESPONSIBLE FOR THE ERECTION SEQUENCE IN ITS ENTIRETY. THE CONTRACTOR'S BID SHALL BE BASED SOLELY UPON THE ERECTION SEQUENCE PROPOSED BY THE CONTRACTOR.

DEFINITIONS

ABOVE DECK DRIVEN CARRIER (ADDC)

ERECTION DEVICES WHICH TRAVEL ON AND ARE SUPPORTED BY THE EXISTING BRIDGE STRUCTURE. FOLLOWING POSITIONING AND BLOCKING, NEW BRIDGE COMPONENTS ARE DELIVERED FOR PLACEMENT USING HOISTS MOUNTED TO OVERHEAD GANTRIES WITH TRAVELING BOGIES.

CONVENTIONAL ERECTION:

THE TYPICAL CONSTRUCTION METHODS THAT ARE EMPLOYED IN MOST BRIDGE CONSTRUCTION APPLICATIONS. BRIDGE COMPONENT ERECTION IS DONE USING A LAND–BASED CRANE (RUBBER–TIRE OR CRAWLER).

CRAWLER CRANE:

A LATTICE–BOOM CRANE SUPPORTED ON AN UNDERCARRIAGE WITH A SET OF TRACKS (ALSO CALLED CRAWLERS) THAT PROVIDE STABILITY AND MOBILITY.

ERECTION TRUSS

SPECIALLY–DESIGNED MODULAR STEEL TRUSS INTENDED FOR USE IN ACCELERATED BRIDGE CONSTRUCTION.

LAUNCHED TEMPORARY TRUSS BRIDGE (LTTB):

ERECTION TRUSSES WHICH ARE LAUNCHED ACROSS OR LIFTED OVER A SPAN OR SET OF SPANS. PLACEMENT OF NEW BRIDGE COMPONENTS IS FACILITATED THROUGH USE OF THESE ERECTION DEVICES AS TEMPORARY BRIDGES.

LONG SPAN BRIDGE:

BRIDGE WITH SPAN LENGTH 71'–130'.
MAXIMUM PREFABRICATED BRIDGE MODULE WEIGHT = 250,000LB.

SAND ISLAND/CAUSEWAY:

CONSTRUCTION TECHNIQUE FOR PROVIDING CRANE SUPPORT IN WHICH NATIVE RIVER SAND IS DREDGED AND COLLECTED AT A SPECIFIC LOCATION INTENDED TO SUPPORT CRANE OPERATIONS. ENOUGH SAND IS DREDGED AND RELOCATED TO BUILD–UP AN EXTENSION OF LAND INTO THE CREEK AND TO TEMPORARILY MODIFY FLOW. CRAWLER CRANES CAN BE SUPPORTED ON THE SAND USING STEEL PLATES OR TIMBER CRANE MATS TO SPREAD THE BEARING PRESSURE. RISKS INCLUDE HIGH RIVER FLOWS WASHING THE SAND AWAY. BENEFITS INCLUDE COST SAVINGS THROUGH USE OF NATIVE MATERIAL INSTEAD OF BUILDING A CRANE TRESTLE.

A MODIFICATION OF THE SAND ISLAND CONCEPT IS TO INSTALL CULVERT PIPES IN THE SAND TO ALLOW WATER FLOW THROUGH THE SAND ISLAND.

SHORT SPAN BRIDGE:

BRIDGE WITH SPAN UP TO 70'.
MAXIMUM PREFABRICATED BRIDGE MODULE WEIGHT = 90,000LB.

STRADDLE CARRIER:

A SELF–PROPELLED FRAME SYSTEM IN WHICH THE SUPPORTED LOAD IS LOCATED WITHIN THE CENTRAL PORTION OF THE FRAME. COMMONLY USED IN THE PRECAST CONCRETE INDUSTRY TO TRANSPORT LONG & HEAVY PRECAST BEAMS, THESE COMMERCIALLY–AVAILABLE ROLLING GANTRY CRANES CAN BE USED IN BRIDGE CONSTRUCTION IN CERTAIN SITUATIONS.

TRESTLE BRIDGE:

A TEMPORARY BRIDGE SUPPORTING CRANE OPERATIONS DURING PERMANENT BRIDGE CONSTRUCTION. STEEL PIPE PILES ARE TYPICALLY USED AS VERTICAL COLUMNS AND STEEL ROLLED OR BOX–SHAPED MEMBER WITH TIMBER CRANE MATS ARE USED AS THE SUPERSTRUCTURE. TYPICALLY CONSTRUCTED USING SINGLE–UNIT SIMPLE–SPAN SUPERSTRUCTURE UNITS.

INDEX OF SHEETS

SPAN LENGTH	INTERSECTION FEATURE	CONVENTIONAL SHEET NO.	ABC SHEET NO.
SHORT	ROADWAY	CC3–CC7	CC18–CC23
SHORT	WATERWAY	CC3, CC8–CC11	CC18–CC23
LONG	ROADWAY	CC12–CC17	CC24–CC31

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

GENERAL NOTES I

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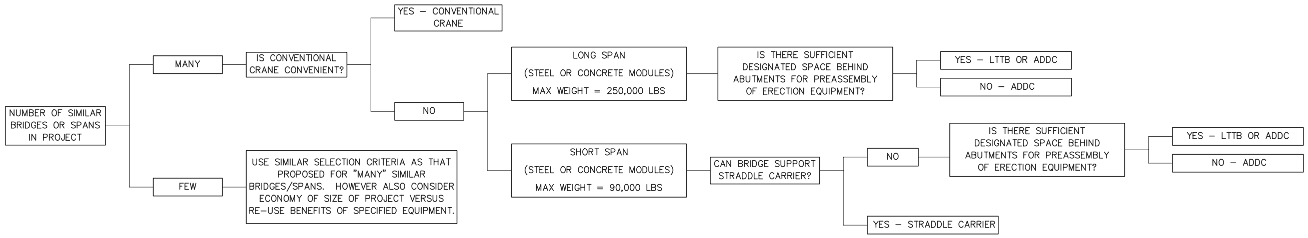
SECTION OF ERECTION TECHNOLOGY:

ERECTION TECHNOLOGY SELECTION IS A COMPLEX PROCESS AND IS DEPENDENT UPON A NUMBER OF FACTORS INCLUDING THE NUMBER OF BRIDGES TO BE BUILT, CONVENIENCE OF CRANE SUPPORT ON GROUND OR BY OTHER MEANS, SPAN LENGTHS, CONDITION OF EXISTING BRIDGE STRUCTURE AND SITE RESTRICTIONS. ALTHOUGH THERE ARE MANY FACTORS TO CONSIDER WHEN SELECTING AN ERECTION TECHNOLOGY, GENERAL SELECTION GUIDELINES ARE PROVIDED [BELOW].



RAPID BRIDGE DEMOLITION:

FOR A REPLACEMENT STRUCTURE, DEMOLISHING THE EXISTING BRIDGE MUST BE COMPLETED PRIOR TO ERECTING THE REPLACEMENT STRUCTURE. BECAUSE THE DEMOLITION OPERATIONS REQUIRE ROADWAY CLOSURES AND OTHER TRAFFIC DISRUPTIONS, COMPLETING THE DEMOLITION PROCESS QUICKLY AND EFFICIENTLY IS OFTEN AS CRITICAL AS THE REPLACEMENT BRIDGE ERECTION OPERATIONS.

TYPICALLY, THE MOST EFFECTIVE USE OF FIELD RESOURCES FOR ACCELERATED BRIDGE CONSTRUCTION IS TO USE THE SAME EQUIPMENT FOR THE DEMOLITION OPERATIONS AND FOR THE REPLACEMENT STRUCTURE ERECTION OPERATIONS. RE-USE OF THE EQUIPMENT AVOIDS DUPLICATION OF TEMPORARY SUPPORT CONDITIONS (CRANE MATS, TRESTLE BRIDGES, CAUSEWAY, FALSEWORK SUPPORTS, ETC.) AND EQUIPMENT ASSEMBLY / DIS-ASSEMBLY TIME REQUIREMENTS.

CONTROLLED DEMOLITION OPERATIONS CAN BE COMPLETED USING VARIOUS METHODS DEPENDENT UPON THE STRUCTURE TYPE. TYPICALLY, EXISTING BRIDGE SUPERSTRUCTURE COMPONENTS (DECK AND GIRDERS) ARE CUT INTO INDIVIDUAL SEGMENTS FOR REMOVAL. CAREFUL CONSIDERATION OF THE TEMPORARY SUPPORT CONDITIONS FOR THE PIECES TO BE REMOVED IS CRITICAL TO MAINTAINING STABILITY AND ADEQUACY DURING DEMOLITION OPERATIONS. SUBSTRUCTURE ELEMENTS CAN BE REMOVED IN PIECES AS WELL BUT MAY ALSO BE REMOVED USING MORE DESTRUCTIVE TECHNIQUES FOLLOWING SUPERSTRUCTURE REMOVAL.

CONVENTIONAL CRANES CAN BE USED TO REMOVE THE EXISTING BRIDGE SEGMENTS. AGAIN, CONSIDERATION SHOULD BE GIVEN TO THE STABILITY OF THE SEGMENTS PROPOSED FOR REMOVAL, AS WELL AS THE WEIGHT OF THE SEGMENTS COMPARED TO THE REPLACEMENT PREFABRICATED ELEMENTS.

THE INNOVATIVE CONSTRUCTION METHODS PRESENTED CAN ALSO BE UTILIZED FOR DEMOLITION OPERATIONS IF PROPER ENGINEERING IS PERFORMED PRIOR TO FIELD USE. DETAILS PROVIDED IN THE PLANS ILLUSTRATE BRIDGE DEMOLITION CONCEPTS. PROJECT-SPECIFIC CONDITIONS AND VARIABLES SHOULD BE CONSIDERED FOR THE USE OF ANY INNOVATIVE DEMOLITION AND/OR CONSTRUCTION METHODS.

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GENERAL NOTES II

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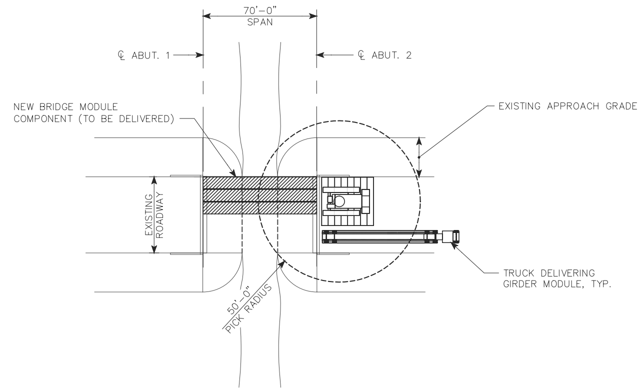
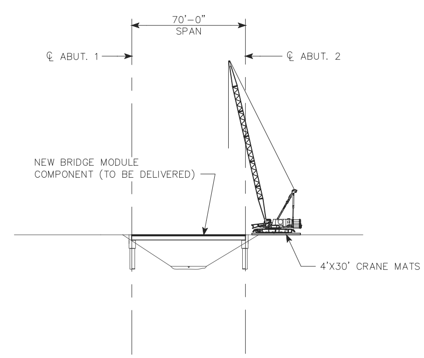
TRAFFIC DISRUPTION

EXISTING ELEVATED ROADWAY CLOSED DURING BRIDGE REPLACEMENT CONSTRUCTION OPERATIONS.
FOR ROADWAY CROSSING, LOWER ROADWAY CLOSED DURING DEMOLITION AND ERECTION OPERATIONS.



REPLACEMENT SINGLE SHORT SPAN BRIDGE – PLAN VIEW



REPLACEMENT SINGLE SHORT SPAN BRIDGE – ELEVATION VIEW

NOTES:

1. SCENARIO SHOWN IS SIMPLE SPAN STREAM CROSSING FOR LOW VOLUME LOCAL ROADWAY. REPLACEMENT BRIDGE WIDTH APPROXIMATELY EQUAL TO EXISTING ROADWAY WIDTH.
2. DETAILS SIMILAR FOR CROSSING OVER ROADWAY.
3. CRANES SELECTED FOR 90,000LB PICKS (PREFABRICATED STEEL OR CONCRETE GIRDER SUPERSTRUCTURE MODULES)
4. DEMO EXISTING BRIDGE PRIOR TO ERECTING REPLACEMENT STRUCTURE.

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
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ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

CONVENTIONAL ERECTION REPLACEMENT SINGLE SHORT SPAN BRIDGE

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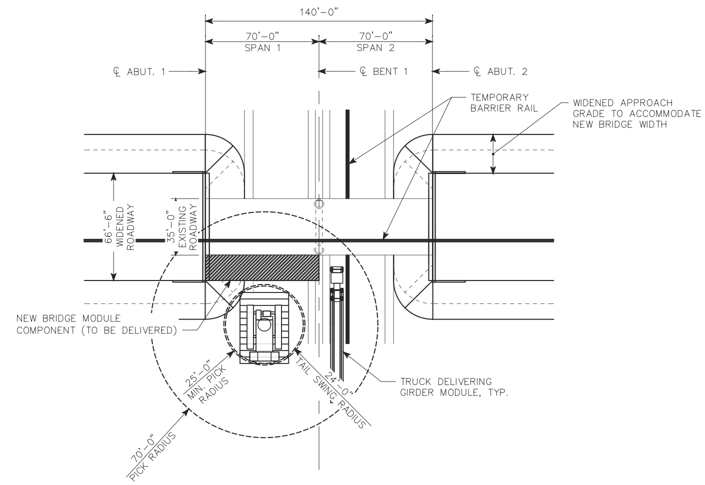
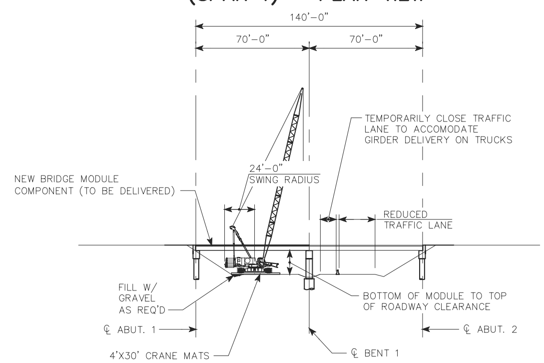


EXISTING SHORT SPAN BRIDGE WIDENING OVER ROADWAY USING CONVENTIONAL CRANES
(SPAN 1) – PLAN VIEW



EXISTING SHORT SPAN BRIDGE WIDENING OVER ROADWAY USING CONVENTIONAL CRANES
(SPAN 1) – ELEVATION VIEW

TRAFFIC DISRUPTION

DURING SUPERSTRUCTURE ERECTION OPERATION, MAINLINE ROADWAY (BELOW) WILL REQUIRE TEMPORARY CLOSURES AND REDUCED TRAFFIC LANES TO ACCOMMODATE CRANE SWINGS & GIRDER DELIVERY AND EXISTING ROADWAY FOR WIDENING REMAINS OPEN WITH REDUCED LANES.

NOTES:

1. CRANES SELECTED FOR 90,000LB PICKS EACH (PREFABRICATED STEEL OR CONCRETE GIRDER SUPERSTRUCTURE MODULES)
2. DUE TO CRITICAL PICK RADIUS OF CRANES, NEW STRUCTURES CAN ONLY BE ERECTED FROM ONE SIDE OF BRIDGE. ONCE CRANES HAVE ERECTED ONE SIDE OF BRIDGE, THEY WILL REQUIRE TEAR-DOWN AND NEW SETUP TO SERVICE OTHER SIDE OF BRIDGE.
3. ERECTION OPERATIONS SHOWN FOR ONE SIDE OF BRIDGE (OTHER SIDE SIMILAR).
4. ASSUMES MINIMAL REHAB TO EXISTING STRUCTURE. IF COMPLETE REPLACEMENT IS REQUIRED, DEMO HALF OF THE EXISTING BRIDGE PRIOR TO ERECTING NEW STRUCTURE WHILE SHIFTING TRAFFIC TO REMAINING HALF OF EXISTING BRIDGE.

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ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

CONVENTIONAL ERECTION WIDEN SHORT SPAN BRIDGE OVER ROADWAY

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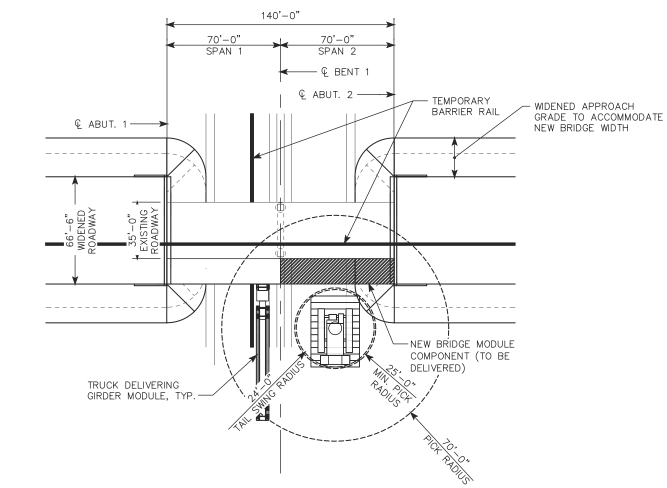
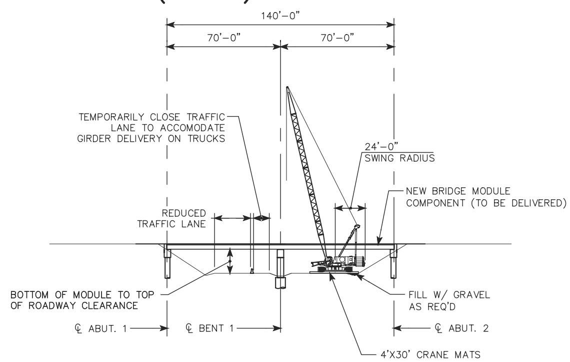


EXISTING SHORT SPAN BRIDGE WIDENING OVER ROADWAY USING CONVENTIONAL CRANES
(SPAN 2) – PLAN VIEW



EXISTING SHORT SPAN BRIDGE WIDENING OVER ROADWAY USING CONVENTIONAL CRANES
(SPAN 2) – ELEVATION VIEW

TRAFFIC DISRUPTION

DURING SUPERSTRUCTURE ERECTION OPERATION, MAINLINE ROADWAY (BELOW) WILL REQUIRE TEMPORARY CLOSURES AND REDUCED TRAFFIC LANES TO ACCOMMODATE CRANE SWINGS & GIRDER DELIVERY AND EXISTING ROADWAY FOR WIDENING REMAINS OPEN WITH REDUCED LANES.

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
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ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

CONVENTIONAL ERECTION WIDEN SHORT SPAN BRIDGE OVER ROADWAY

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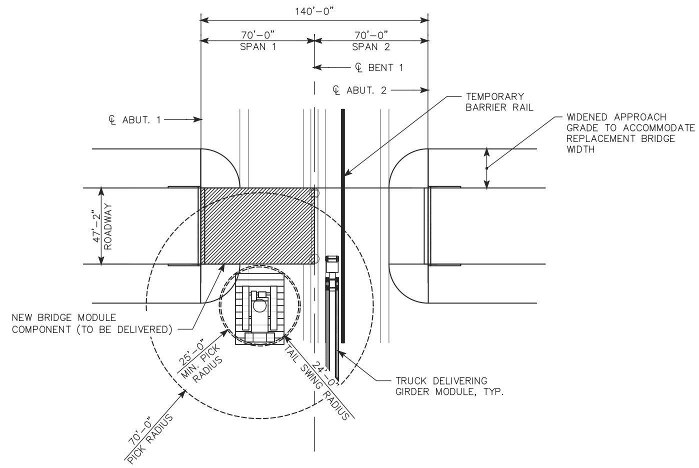
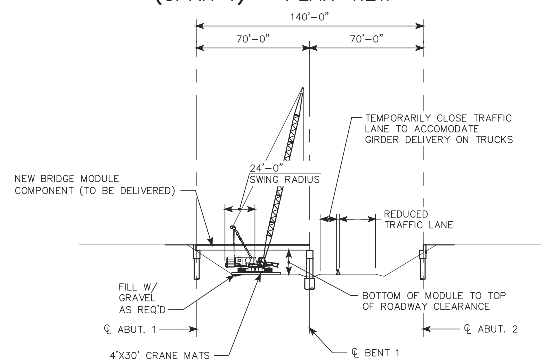


REPLACEMENT SHORT SPAN BRIDGE OVER ROADWAY USING CONVENTIONAL CRANES
(SPAN 1) – PLAN VIEW



REPLACEMENT SHORT SPAN BRIDGE OVER ROADWAY USING CONVENTIONAL CRANES
(SPAN 1) – ELEVATION VIEW

TRAFFIC DISRUPTION

DURING SUPERSTRUCTURE ERECTION OPERATION, MAINLINE ROADWAY (BELOW) WILL REQUIRE TEMPORARY CLOSURES AND REDUCED TRAFFIC LANES TO ACCOMMODATE CRANE SWINGS & GIRDER DELIVERY AND NO TRAFFIC ON REPLACEMENT BRIDGE.

NOTES:

1. CRANES SELECTED FOR 90,000LB PICKS EACH (PREFABRICATED STEEL OR CONCRETE GIRDER SUPERSTRUCTURE MODULES)
2. DEMO EXISTING PRIOR TO ERECTING REPLACEMENT STRUCTURE.

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
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ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

CONVENTIONAL ERECTION REPLACEMENT BRIDGE SHORT SPAN BRIDGE OVER ROADWAY

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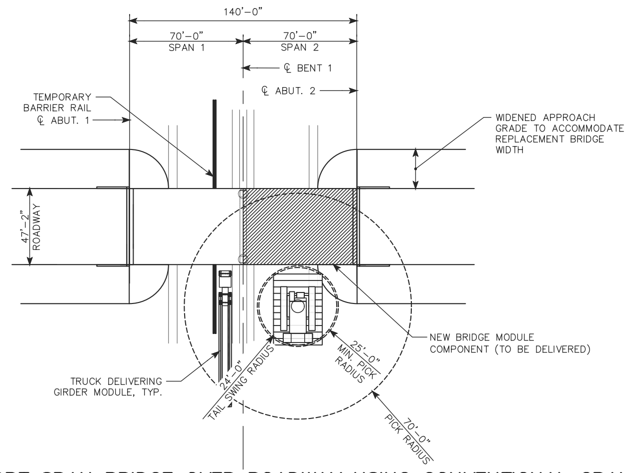
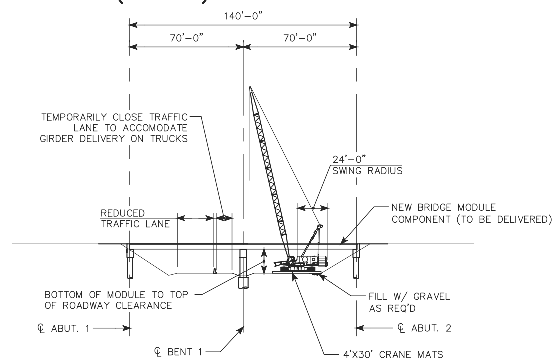


REPLACEMENT SHORT SPAN BRIDGE OVER ROADWAY USING CONVENTIONAL CRANES
(SPAN 2) – PLAN VIEW



REPLACEMENT SHORT SPAN BRIDGE OVER ROADWAY USING CONVENTIONAL CRANES
(SPAN 2) – ELEVATION VIEW

TRAFFIC DISRUPTION

DURING SUPERSTRUCTURE ERECTION OPERATION, MAINLINE ROADWAY (BELOW) WILL REQUIRE TEMPORARY CLOSURES AND REDUCED TRAFFIC LANES TO ACCOMMODATE CRANE SWINGS & GIRDER DELIVERY AND NO TRAFFIC ON REPLACEMENT BRIDGE.

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INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

CONVENTIONAL ERECTION REPLACEMENT BRIDGE SHORT SPAN BRIDGE OVER ROADWAY

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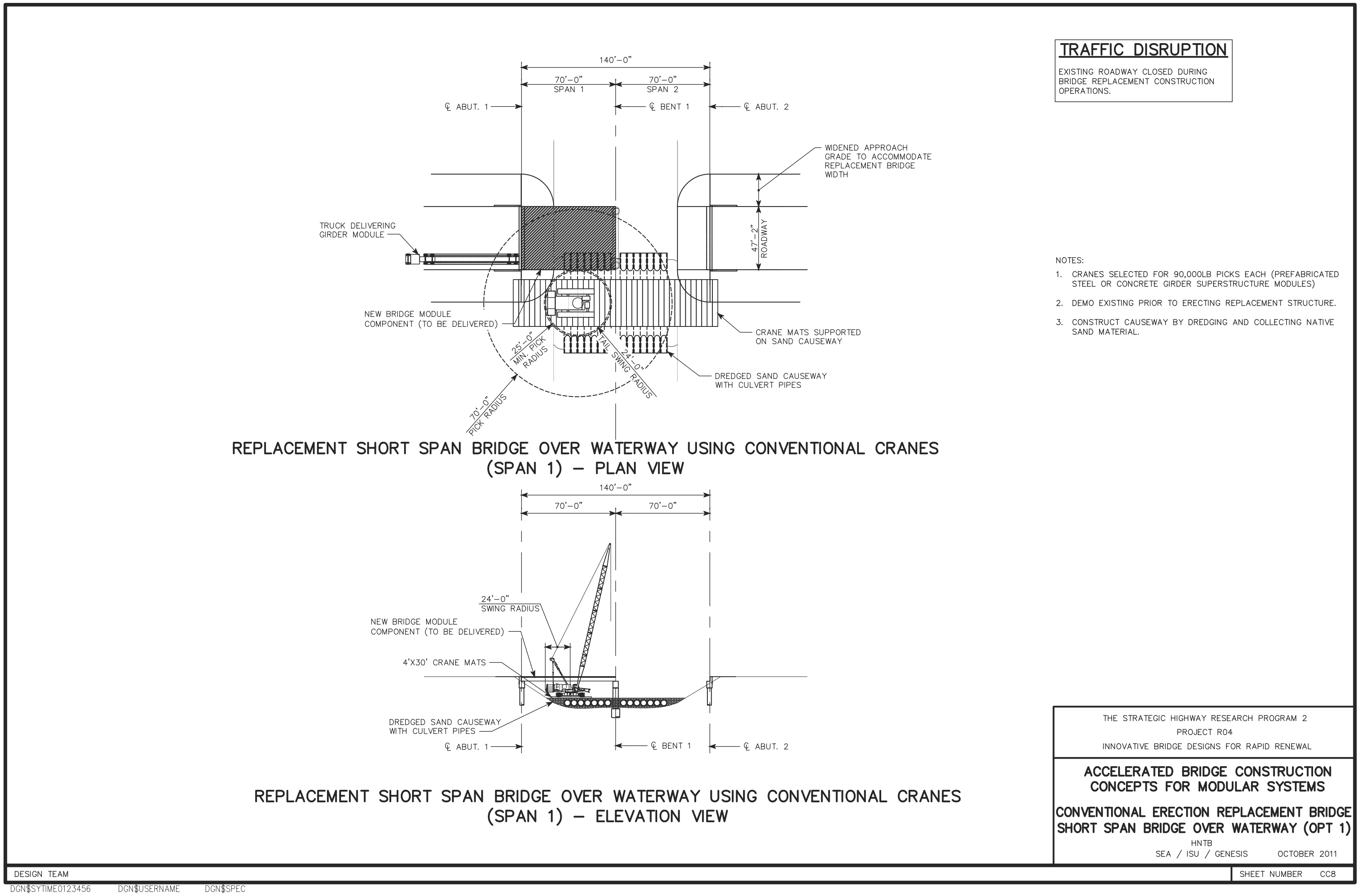

TRAFFIC DISRUPTION
EXISTING ROADWAY CLOSED DURING BRIDGE REPLACEMENT CONSTRUCTION OPERATIONS.
140'-0"
70'-0" SPAN 1
70'-0" SPAN 2
℄ ABUT. 1
℄ BENT 1
℄ ABUT. 2
WIDENED APPROACH GRADE TO ACCOMMODATE REPLACEMENT BRIDGE WIDTH
47'-2" ROADWAY
TRUCK DELIVERING GIRDER MODULE
NEW BRIDGE MODULE COMPONENT (TO BE DELIVERED)
25'-0" MIN. PICK RADIUS
24'-0" TAIL SWING RADIUS
70'-0" PICK RADIUS
CRANE MATS SUPPORTED ON SAND CAUSEWAY
DREDGED SAND CAUSEWAY WITH CULVERT PIPES
NOTES:
1. CRANES SELECTED FOR 90,000LB PICKS EACH (PREFABRICATED STEEL OR CONCRETE GIRDER SUPERSTRUCTURE MODULES)
2. DEMO EXISTING PRIOR TO ERECTING REPLACEMENT STRUCTURE.
3. CONSTRUCT CAUSEWAY BY DREDGING AND COLLECTING NATIVE SAND MATERIAL.
REPLACEMENT SHORT SPAN BRIDGE OVER WATERWAY USING CONVENTIONAL CRANES (SPAN 1) – PLAN VIEW
140'-0"
70'-0"
70'-0"
24'-0" SWING RADIUS
NEW BRIDGE MODULE COMPONENT (TO BE DELIVERED)
4"X30' CRANE MATS
DREDGED SAND CAUSEWAY WITH CULVERT PIPES
℄ ABUT. 1
℄ BENT 1
℄ ABUT. 2
REPLACEMENT SHORT SPAN BRIDGE OVER WATERWAY USING CONVENTIONAL CRANES (SPAN 1) – ELEVATION VIEW
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CONVENTIONAL ERECTION REPLACEMENT BRIDGE SHORT SPAN BRIDGE OVER WATERWAY (OPT 1)
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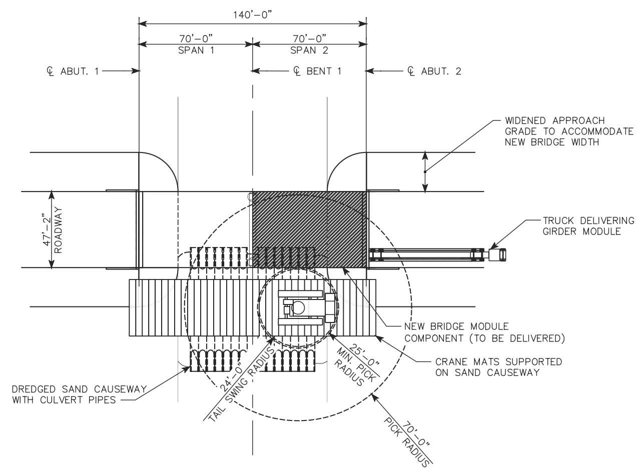
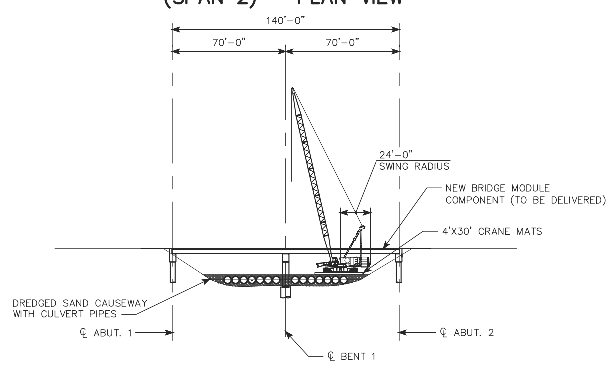
TRAFFIC DISRUPTION

EXISTING ROADWAY CLOSED DURING BRIDGE REPLACEMENT CONSTRUCTION OPERATIONS.



REPLACEMENT SHORT SPAN BRIDGE OVER WATERWAY USING CONVENTIONAL CRANES
(SPAN 2) – PLAN VIEW



REPLACEMENT SHORT SPAN BRIDGE OVER WATERWAY USING CONVENTIONAL CRANES
(SPAN 2) – ELEVATION VIEW

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
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INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

CONVENTIONAL ERECTION REPLACEMENT BRIDGE SHORT SPAN BRIDGE OVER WATERWAY (OPT 1)

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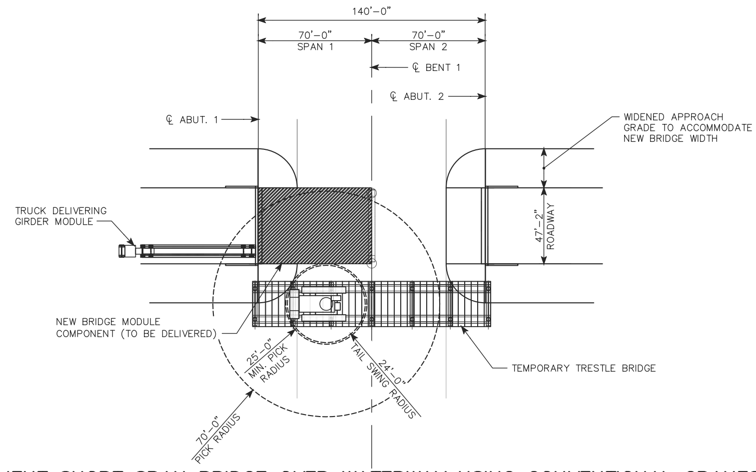
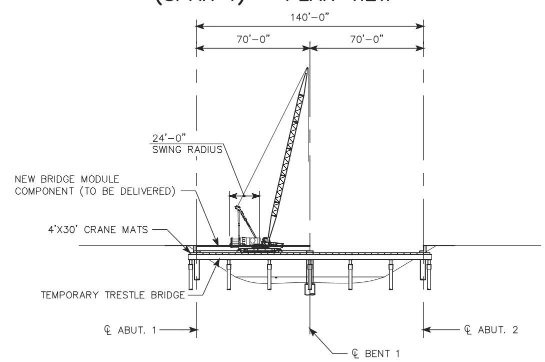
TRAFFIC DISRUPTION

EXISTING ROADWAY CLOSED DURING BRIDGE REPLACEMENT CONSTRUCTION OPERATIONS.



REPLACEMENT SHORT SPAN BRIDGE OVER WATERWAY USING CONVENTIONAL CRANES
(SPAN 1) – PLAN VIEW



REPLACEMENT SHORT SPAN BRIDGE OVER WATERWAY USING CONVENTIONAL CRANES
(SPAN 1) – ELEVATION VIEW

NOTES:

1. CRANES SELECTED FOR 90,000LB PICKS EACH (PREFABRICATED STEEL OR CONCRETE GIRDER SUPERSTRUCTURE MODULES).
2. DEMO EXISTING PRIOR TO ERECTING REPLACEMENT STRUCTURE.
3. CONSTRUCT TEMPORARY TRESTLE BRIDGE TO SUPPORT CRANE OPERATION. REMOVE TRESTLE BRIDGE FOLLOWING COMPLETION OF CRANE ACTIVITIES.

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

CONVENTIONAL ERECTION REPLACEMENT BRIDGE SHORT SPAN BRIDGE OVER WATERWAY (OPT 2)

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DESIGN TEAM

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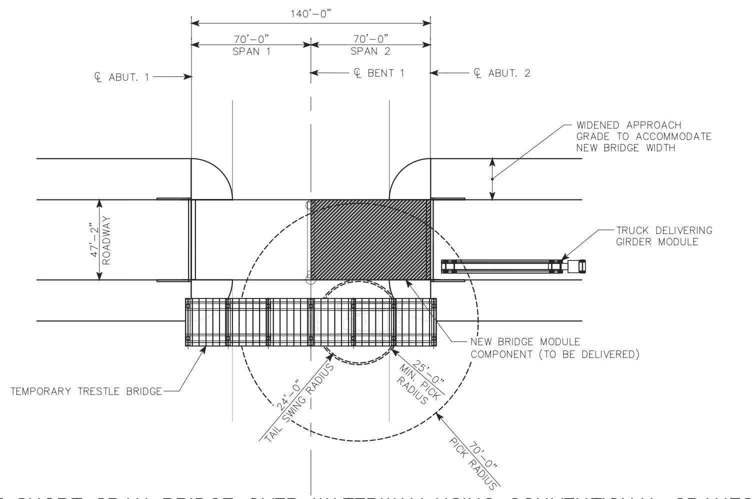
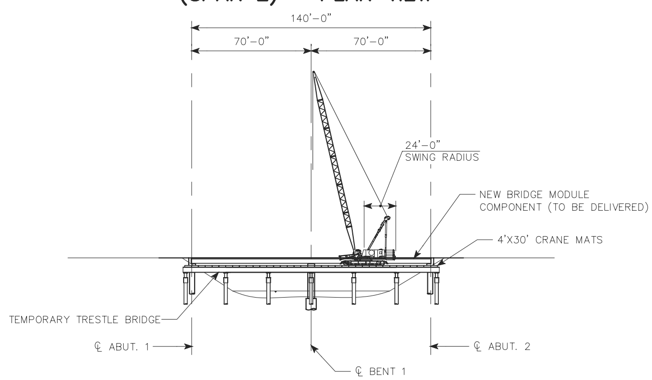
TRAFFIC DISRUPTION

EXISTING ROADWAY CLOSED DURING BRIDGE REPLACEMENT CONSTRUCTION OPERATIONS.



REPLACEMENT SHORT SPAN BRIDGE OVER WATERWAY USING CONVENTIONAL CRANES (SPAN 2) – PLAN VIEW



REPLACEMENT SHORT SPAN BRIDGE OVER WATERWAY USING CONVENTIONAL CRANES (SPAN 2) – ELEVATION VIEW

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
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INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

CONVENTIONAL ERECTION REPLACEMENT BRIDGE SHORT SPAN BRIDGE OVER WATERWAY (OPT 2)

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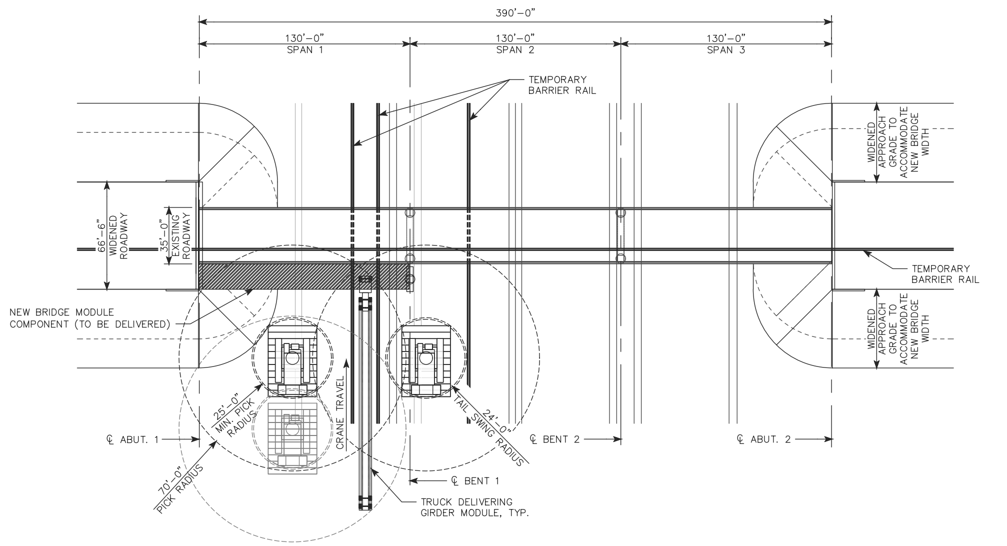
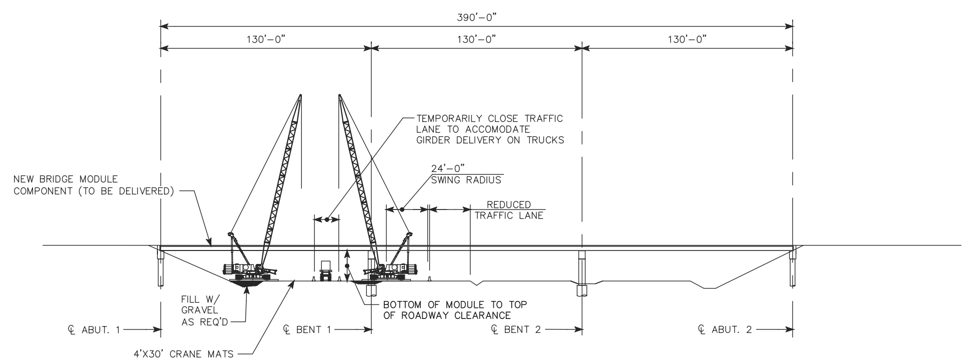


EXISTING LONG SPAN BRIDGE WIDENING OVER ROADWAY USING CONVENTIONAL CRANES (SPAN 1) – PLAN VIEW



EXISTING LONG SPAN BRIDGE WIDENING OVER ROADWAY USING CONVENTIONAL CRANES (SPAN 1) – ELEVATION VIEW

TRAFFIC DISRUPTION

DURING SUPERSTRUCTURE ERECTION OPERATION, MAINLINE ROADWAY (BELOW) WILL REQUIRE TEMPORARY CLOSURES AND REDUCED TRAFFIC LANES TO ACCOMMODATE CRANE SWINGS & GIRDER DELIVERY AND EXISTING ROADWAY FOR WIDENING REMAINS OPEN WITH REDUCED LANES.

NOTES:

1. CRANES SELECTED FOR 125,000LB PICKS EACH (PREFABRICATED STEEL OR CONCRETE GIRDER SUPERSTRUCTURE MODULES)
2. DUE TO CRITICAL PICK RADIUS OF CRANES, NEW STRUCTURES CAN ONLY BE ERECTED FROM ONE SIDE OF BRIDGE. ONCE CRANES HAVE ERECTED ONE SIDE OF BRIDGE, THEY WILL REQUIRE TEAR–DOWN AND NEW SETUP TO SERVICE OTHER SIDE OF BRIDGE.
3. ERECTION OPERATIONS SHOWN FOR ONE SIDE OF BRIDGE (OTHER SIDE SIMILAR).
4. ASSUMES MINIMAL REHAB TO EXISTING STRUCTURE. IF COMPLETE REPLACEMENT IS REQUIRED, DEMO HALF OF THE EXISTING BRIDGE PRIOR TO ERECTING NEW STRUCTURE WHILE SHIFTING TRAFFIC TO REMAINING HALF OF EXISTING BRIDGE.

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CONVENTIONAL ERECTION WIDEN LONG SPAN BRIDGE OVER ROADWAY

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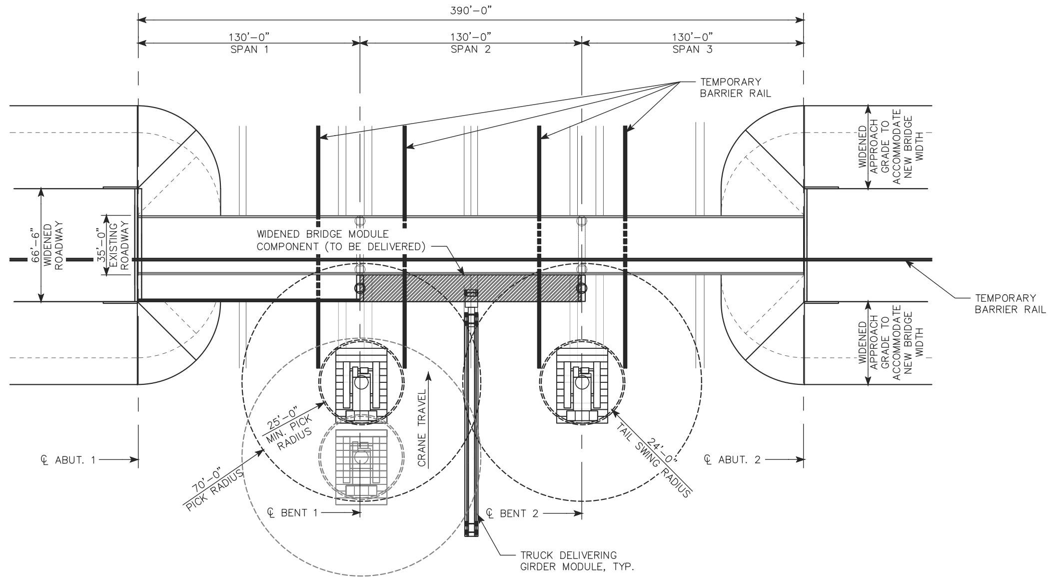
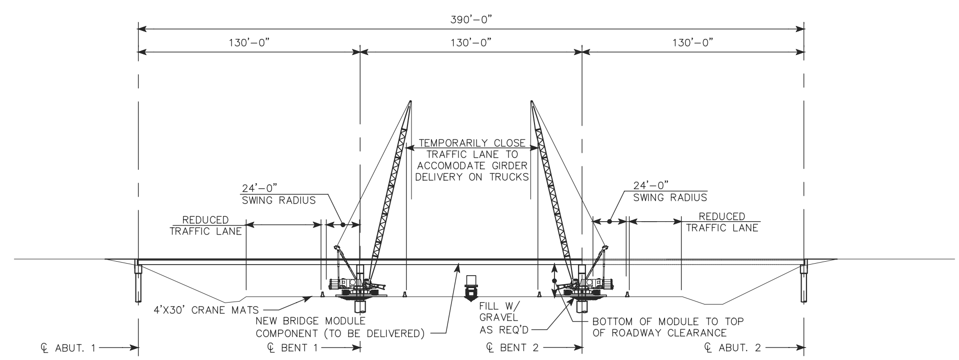


EXISTING LONG SPAN BRIDGE WIDENING OVER ROADWAY USING CONVENTIONAL CRANES (SPAN 2) – PLAN VIEW



EXISTING LONG SPAN BRIDGE WIDENING OVER ROADWAY USING CONVENTIONAL CRANES (SPAN 2) – ELEVATION VIEW

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ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

CONVENTIONAL ERECTION WIDEN LONG SPAN BRIDGE OVER ROADWAY

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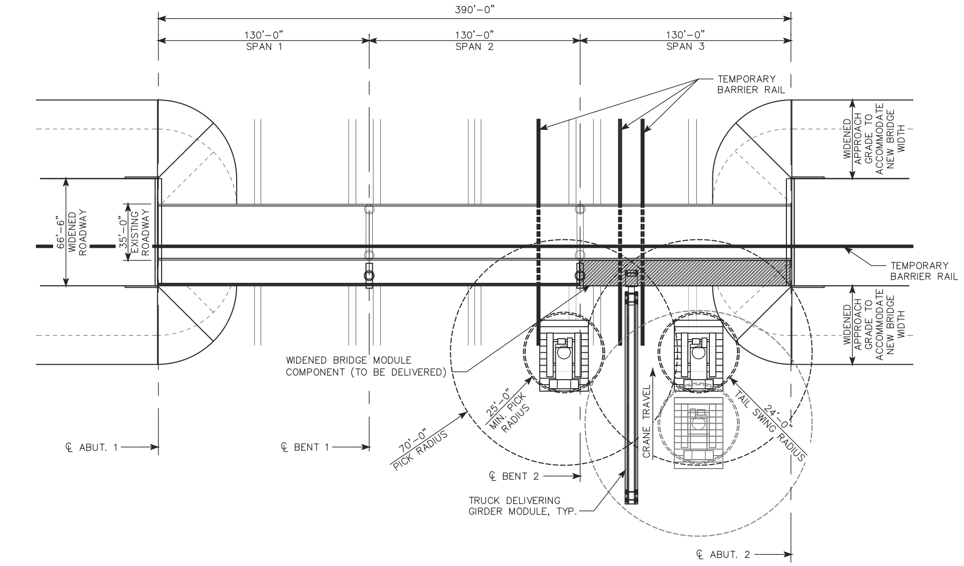
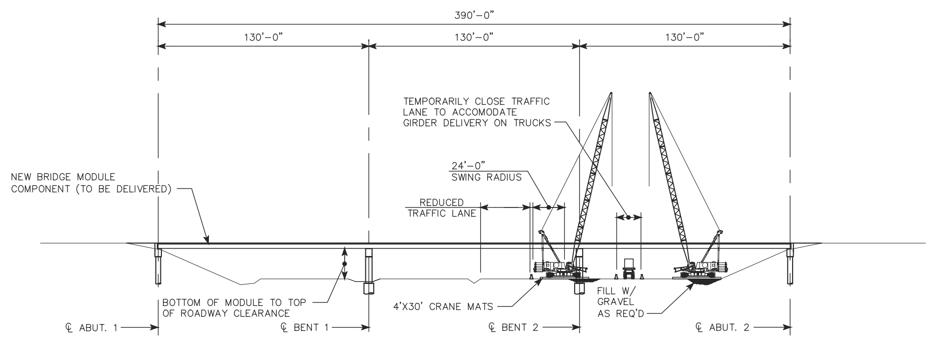


EXISTING LONG SPAN BRIDGE WIDENING OVER ROADWAY USING CONVENTIONAL CRANES (SPAN 3) – PLAN VIEW



EXISTING LONG SPAN BRIDGE WIDENING OVER ROADWAY USING CONVENTIONAL CRANES (SPAN 3) – ELEVATION VIEW

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ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

CONVENTIONAL ERECTION WIDEN LONG SPAN BRIDGE OVER ROADWAY

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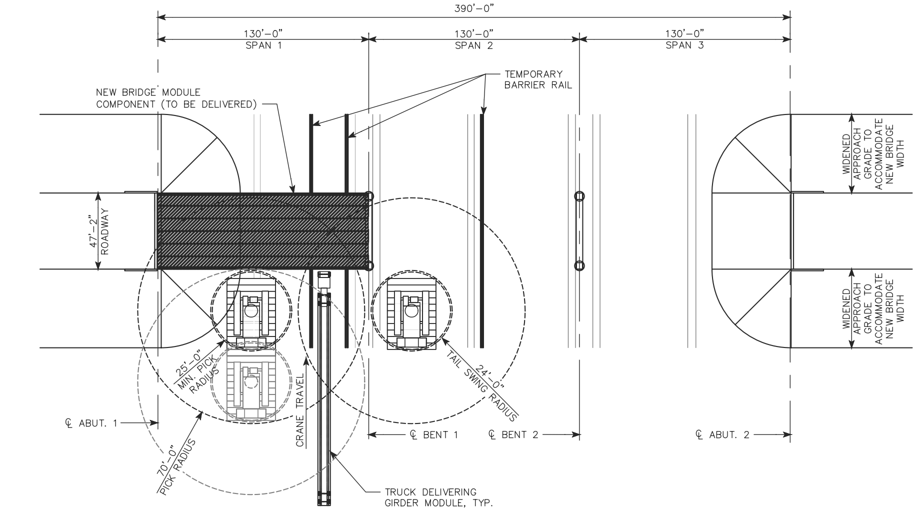
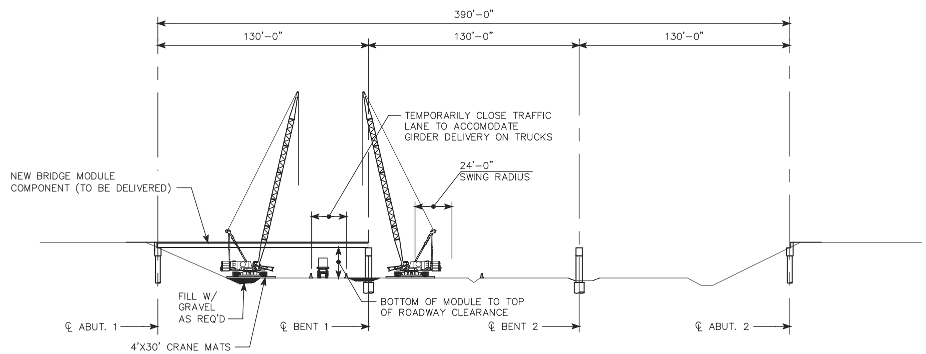


REPLACEMENT LONG SPAN BRIDGE OVER ROADWAY USING CONVENTIONAL CRANES (SPAN 1) – PLAN VIEW



REPLACEMENT LONG SPAN BRIDGE OVER ROADWAY USING CONVENTIONAL CRANES (SPAN 1) – ELEVATION VIEW

TRAFFIC DISRUPTION

DURING SUPERSTRUCTURE ERECTION OPERATION, MAINLINE ROADWAY (BELOW) WILL REQUIRE TEMPORARY CLOSURES AND REDUCED TRAFFIC LANES TO ACCOMMODATE CRANE SWINGS & GIRDER DELIVERY AND NO TRAFFIC ON PROPOSED (NEW CONSTRUCTION).

NOTES:

1. CRANES SELECTED FOR 125,000LB PICKS EACH (PREFABRICATED STEEL OR CONCRETE GIRDER SUPERSTRUCTURE MODULES)
2. DEMO EXISTING PRIOR TO ERECTING REPLACEMENT STRUCTURE.

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CONVENTIONAL ERECTION REPLACEMENT LONG SPAN BRIDGE OVER ROADWAY

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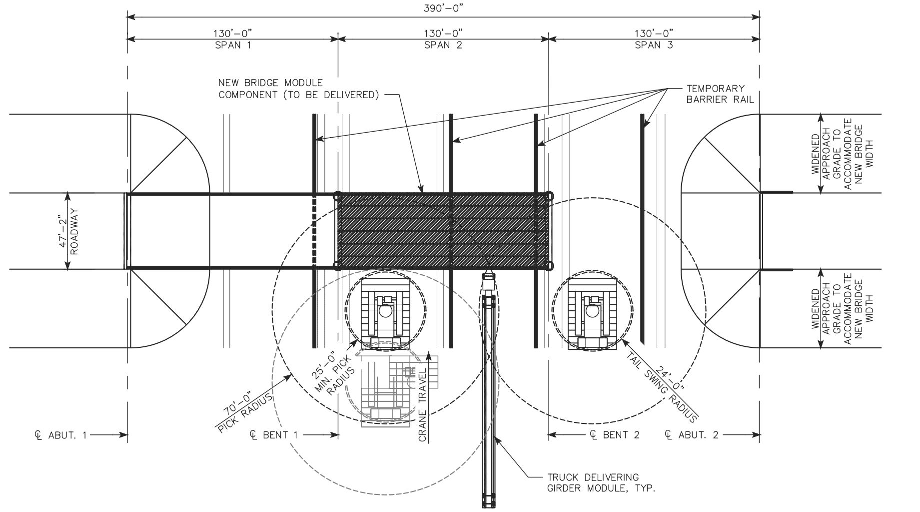
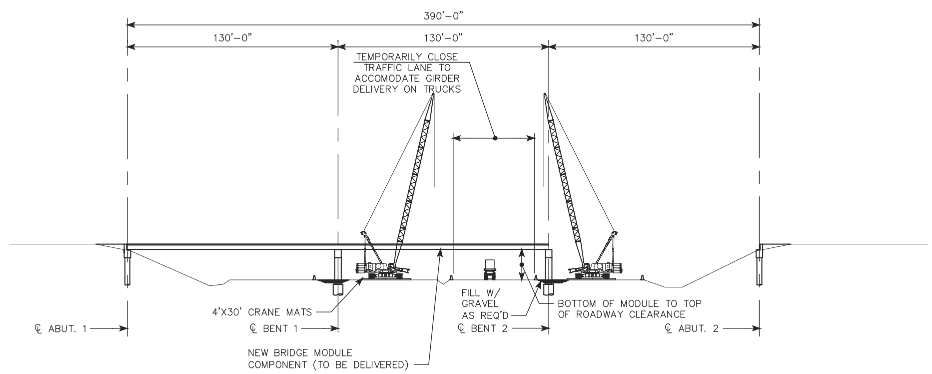


REPLACEMENT LONG SPAN BRIDGE OVER ROADWAY USING CONVENTIONAL CRANES (SPAN 2) – PLAN VIEW



REPLACEMENT LONG SPAN BRIDGE OVER ROADWAY USING CONVENTIONAL CRANES (SPAN 2) – ELEVATION VIEW

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ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

CONVENTIONAL ERECTION REPLACEMENT LONG SPAN BRIDGE OVER ROADWAY

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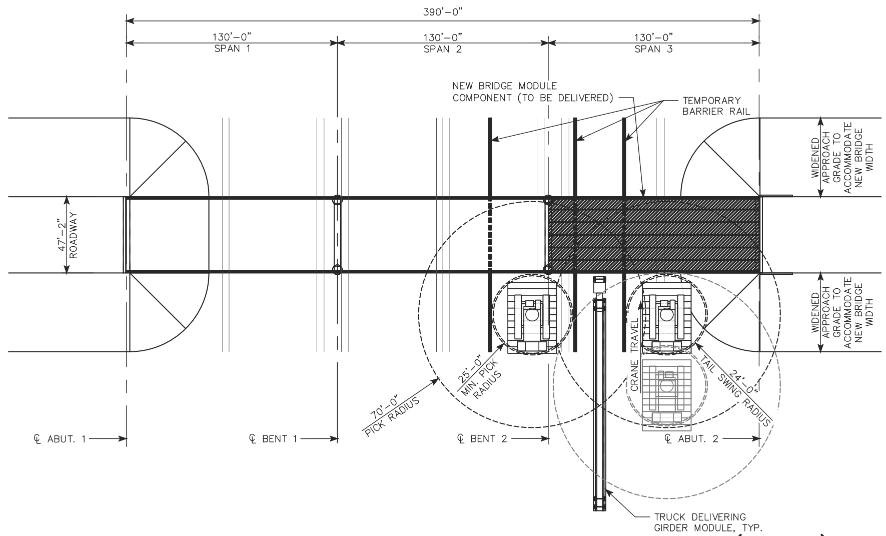
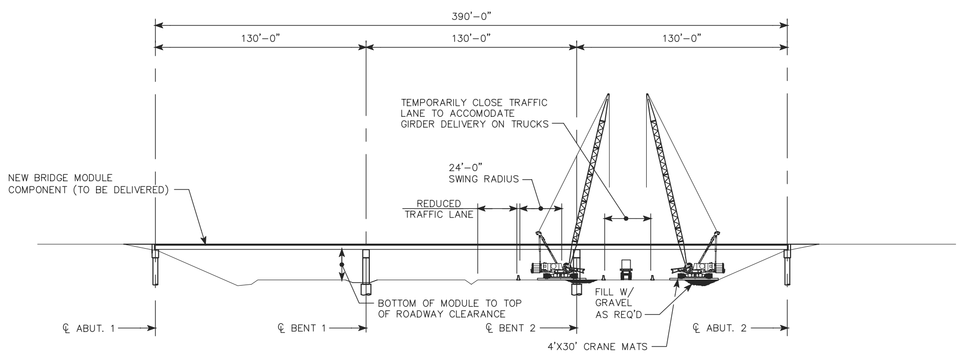


REPLACEMENT LONG SPAN BRIDGE OVER ROADWAY USING CONVENTIONAL CRANES (SPAN 3) – PLAN VIEW



REPLACEMENT LONG SPAN BRIDGE OVER ROADWAY USING CONVENTIONAL CRANES (SPAN 3) – ELEVATION VIEW

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ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

CONVENTIONAL ERECTION REPLACEMENT LONG SPAN BRIDGE OVER ROADWAY

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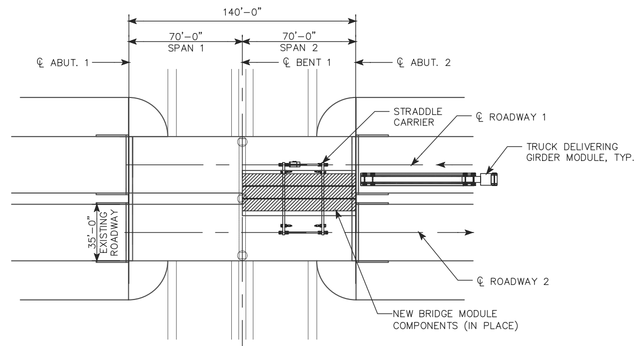
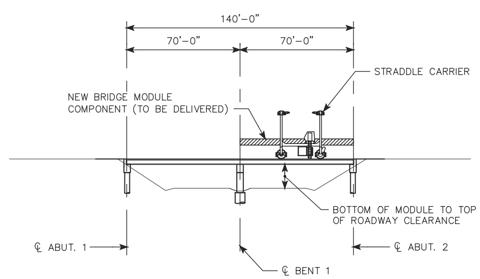


REPLACEMENT BRIDGE USING STRADDLE CARRIER ON PERMANENT BRIDGE
(SPAN 2) – PLAN VIEW



REPLACEMENT BRIDGE USING STRADDLE CARRIER ON PERMANENT BRIDGE
(SPAN 2) – ELEVATION VIEW

TRAFFIC DISRUPTION

DURING SUPERSTRUCTURE ERECTION OPERATION, MAINLINE ROADWAY (BELOW) HAS SHORT–DURATION CLOSURES DURING STRADDLE CARRIER ACTIVITIES. EXISTING ROADWAY (FOR REPLACEMENT) CLOSED DURING CONSTRUCTION.

NOTES:

1. DEMO EXISTING PRIOR TO ERECTING REPLACEMENT STRUCTURE.
2. CENTER STRADDLE CARRIER WHEELS ON EXISTING AND/OR REPLACEMENT GIRDERS.
3. PRIOR TO CONSTRUCTION, VERIFY ADEQUACY OF EXISTING AND REPLACEMENT BRIDGE TO SUPPORT LOADED STRADDLE CARRIER.
4. PARALLEL DUAL BRIDGE SCENE SHOWN HERE. CONCEPT IS APPLICABLE FOR SIMILAR SCENARIO WITH SUITABLE GIRDER ARRANGEMENT. STRADDLE CARRIER OPERATION REQUIRES PARALLEL GIRDER ARRANGEMENT.

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ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

STRADDLE CARRIER ON PERMANENT BRIDGE SHORT SPAN BRIDGE

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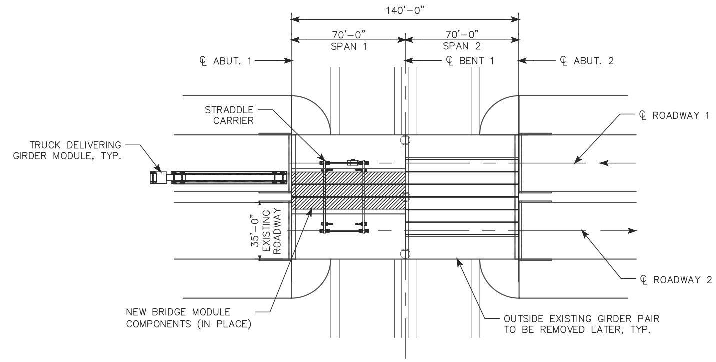
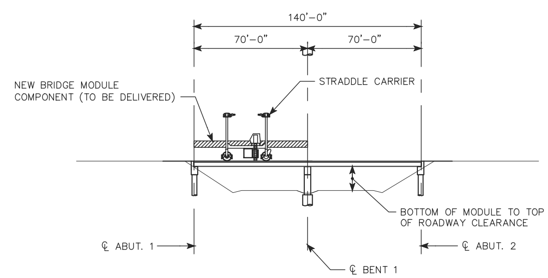


REPLACEMENT BRIDGE USING STRADDLE CARRIER ON PERMANENT BRIDGE (SPAN 1) – PLAN VIEW



REPLACEMENT BRIDGE USING STRADDLE CARRIER ON PERMANENT BRIDGE (SPAN 1) – ELEVATION VIEW

TRAFFIC DISRUPTION

DURING SUPERSTRUCTURE ERECTION OPERATION, MAINLINE ROADWAY (BELOW) HAS SHORT-DURATION CLOSURES DURING STRADDLE CARRIER ACTIVITIES. EXISTING ROADWAY (FOR REPLACEMENT) CLOSED DURING CONSTRUCTION.

NOTES:

1. DEMO EXISTING PRIOR TO ERECTING REPLACEMENT STRUCTURE.
2. CENTER STRADDLE CARRIER WHEELS ON EXISTING AND/OR REPLACEMENT GIRDERS.
3. PRIOR TO CONSTRUCTION, VERIFY ADEQUACY OF EXISTING AND REPLACEMENT BRIDGE TO SUPPORT LOADED STRADDLE CARRIER.
4. PARALLEL DUAL BRIDGE SCENE SHOWN HERE. CONCEPT IS APPLICABLE FOR SIMILAR SCENARIO WITH SUITABLE GIRDER ARRANGEMENT. STRADDLE CARRIER OPERATION REQUIRES PARALLEL GIRDER ARRANGEMENT.

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ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

STRADDLE CARRIER ON PERMANENT BRIDGE SHORT SPAN BRIDGE

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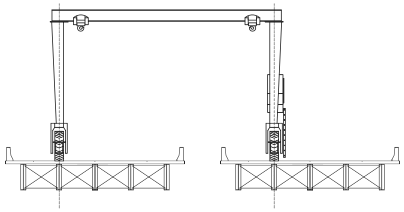
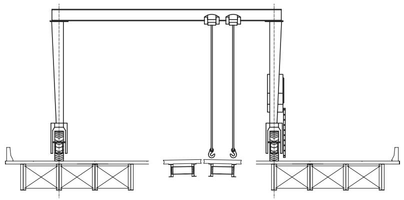


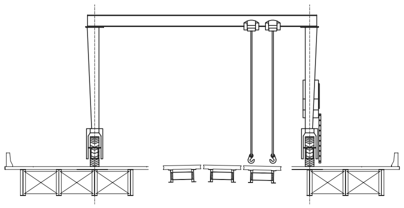
STAGE 1:

1. POSITION STRADDLE CARRIER AT ℄ EXISTING GIRDERS.



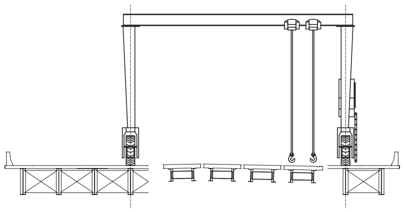
STAGE 2:

1. REMOVE EXISTING INSIDE GIRDERS AND SLAB.
2. ERECT NEW PREFABRICATED SUPERSTRUCTURE MODULES.



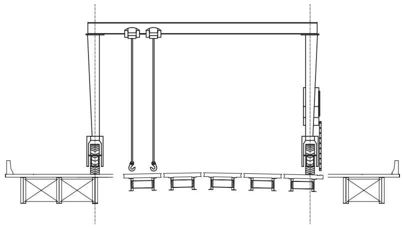
STAGE 3:

1. SHIFT STRADDLE CARRIER.
2. REMOVE EXISTING GIRDER AND REPLACE WITH NEW PREFABRICATED SUPERSTRUCTURE MODULES.



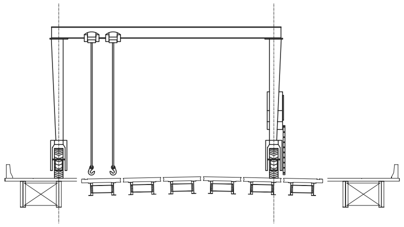
STAGE 4:

1. SHIFT STRADDLE CARRIER.
2. REMOVE EXISTING GIRDER AND REPLACE WITH NEW PREFABRICATED SUPERSTRUCTURE MODULES.



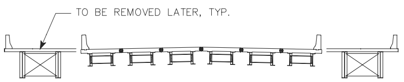
STAGE 5:

1. SHIFT STRADDLE CARRIER PARTIALLY ONTO NEW STRUCTURE.
2. REMOVE EXISTING GIRDER AND REPLACE WITH NEW PREFABRICATED SUPERSTRUCTURE MODULES.



STAGE 6:

1. SHIFT STRADDLE CARRIER.
2. REMOVE EXISTING GIRDER AND REPLACE WITH NEW PREFABRICATED SUPERSTRUCTURE MODULES.



STAGE 7:

1. REMOVE STRADDLE CARRIER.
2. CAST CLOSURE POURS FOR NEW BRIDGE.
3. OUTSIDE EXISTING GIRDER PAIRS TO BE REMOVED LATER.

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ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

STRADDLE CARRIER ON PERMANENT BRIDGE STAGED CONSTRUCTION

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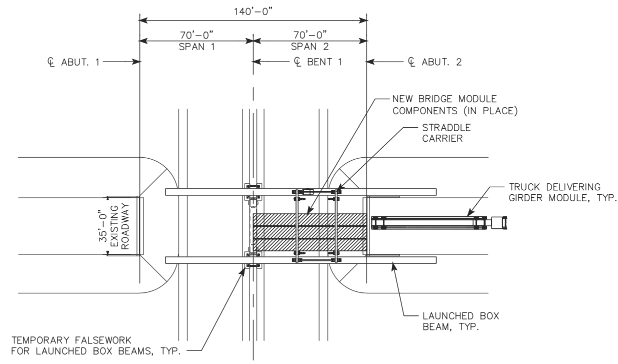
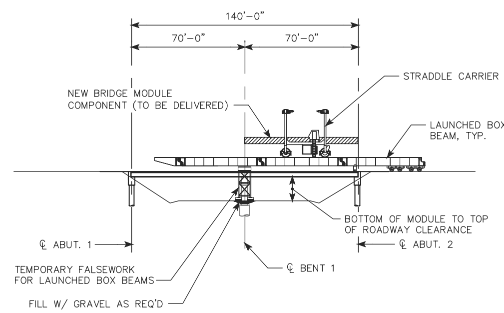


REPLACEMENT BRIDGE USING STRADDLE CARRIER ON LAUNCH BEAMS
(SPAN 2) – PLAN VIEW



REPLACEMENT BRIDGE USING STRADDLE CARRIER ON LAUNCH BEAMS
(SPAN 2) – ELEVATION VIEW

TRAFFIC DISRUPTION

DURING SUPERSTRUCTURE ERECTION OPERATION, MAINLINE ROADWAY (BELOW) HAS SHORT–DURATION CLOSURES DURING LAUNCHING AND STRADDLE CARRIER ACTIVITIES. EXISTING ROADWAY (FOR REPLACEMENT) CLOSED DURING CONSTRUCTION.

NOTES:

1. DEMO EXISTING PRIOR TO ERECTING REPLACEMENT STRUCTURE.

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ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

STRADDLE CARRIER ON LAUNCH BEAMS SHORT SPAN BRIDGE

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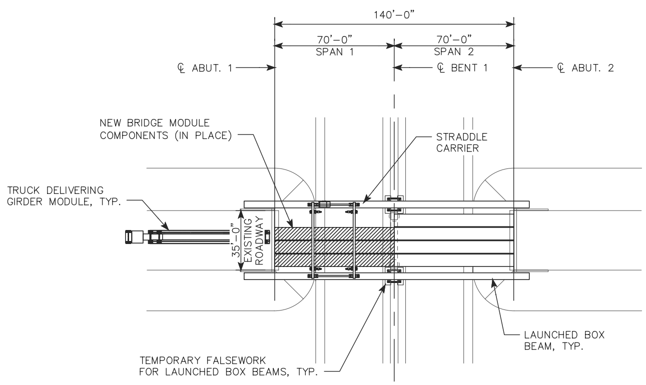
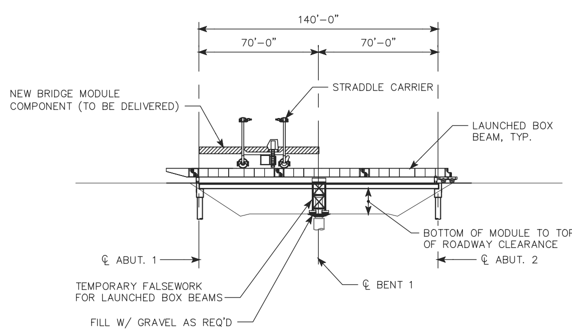


REPLACEMENT BRIDGE USING STRADDLE CARRIER ON LAUNCH BEAMS
(SPAN 1) – PLAN VIEW



REPLACEMENT BRIDGE USING STRADDLE CARRIER ON LAUNCH BEAMS
(SPAN 1) – ELEVATION VIEW

TRAFFIC DISRUPTION

DURING SUPERSTRUCTURE ERECTION OPERATION, MAINLINE ROADWAY (BELOW) HAS SHORT–DURATION CLOSURES DURING LAUNCHING AND STRADDLE CARRIER ACTIVITIES. EXISTING ROADWAY (FOR REPLACEMENT) CLOSED DURING CONSTRUCTION.

NOTES:

1. DEMO EXISTING PRIOR TO ERECTING REPLACEMENT STRUCTURE.

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ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

STRADDLE CARRIER ON LAUNCH BEAMS SHORT SPAN BRIDGE

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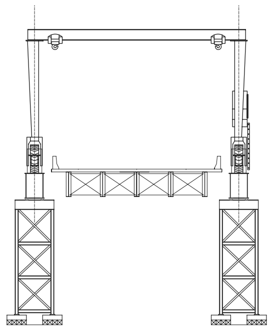
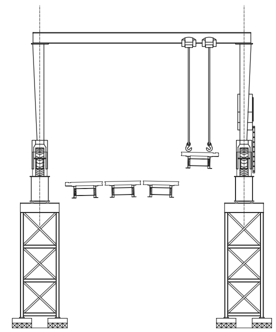


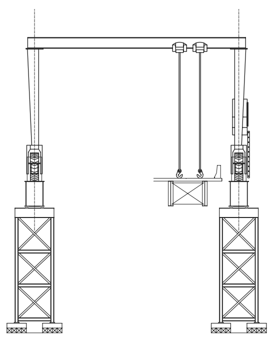
STAGE 1:

1. LAUNCH BOX BEAMS AND POSITION STRADDLE CARRIER.



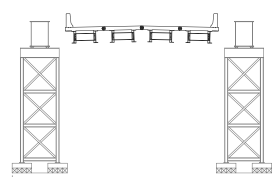
STAGE 3:

1. ERECT REPLACEMENT BRIDGE COMPONENTS USING STRADDLE CARRIER.



STAGE 2:

1. REMOVE EXISTING BRIDGE COMPONENTS USING STRADDLE CARRIER.



STAGE 4:

1. REMOVE STRADDLE CARRIER.
2. CAST DECK CLOSURE POURS FOR NEW BRIDGE.
3. REMOVE BOX BEAMS.

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STRADDLE CARRIER ON LAUNCH BEAM STAGED CONSTRUCTION

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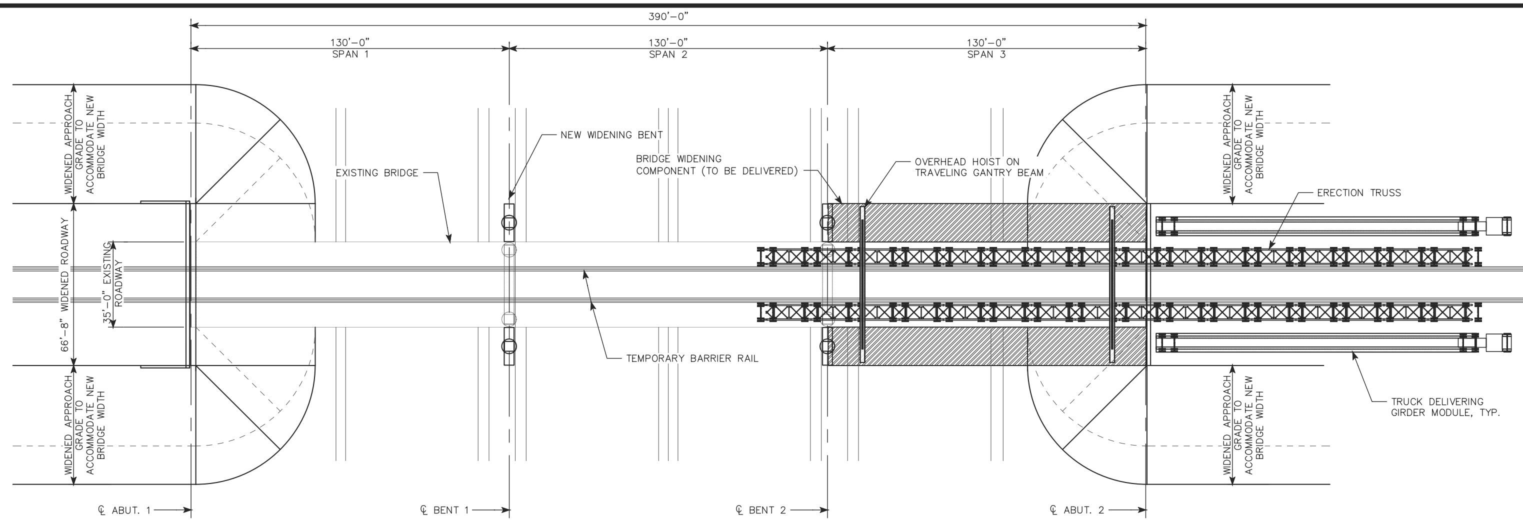
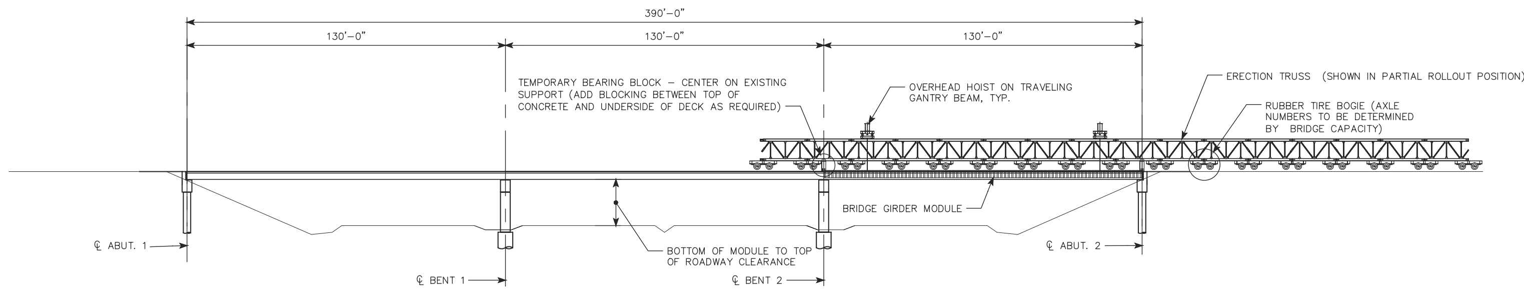


EXISTING BRIDGE WIDENING USING ADDC – PLAN VIEW



EXISTING BRIDGE WIDENING USING ADDC – ELEVATION VIEW

NOTES:

1. ADDC CONCEPT SHOWN FOR LONG SPAN BRIDGE. ADDC CONCEPT SIMILAR FOR SHORT SPAN BRIDGE.
2. FOR TYPICAL TRUSS MODULE DETAILS, SEE TYPICAL ERECTION TRUSS MODULE SHEET.
3. ADDC REFERS TO "ABOVE DECK DRIVEN CARRIER."
4. ADDC SCENARIOS DEVELOPED FOR PREFABRICATED STEEL OR CONCRETE GIRDER SUPERSTRUCTURE MODULES.
5. ERECTION OPERATIONS FOR CENTER AND END SPANS SIMILAR.
6. OVERHEAD BRIDGE SHOWN FOR HIGHWAY INTERSECTION. SIMILAR CONCEPT WOULD BE APPROPRIATE FOR RIVER OR CREEK SETTING.

TRAFFIC DISRUPTION

DURING SUPERSTRUCTURE ERECTION OPERATION, MAINLINE ROADWAY (BELOW) REMAINS OPEN AND EXISTING ROADWAY (FOR WIDENING) REMAINS OPEN WITH REDUCED LANES.

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

ADDC CONCEPT – PLAN & ELEVATION

HNTB
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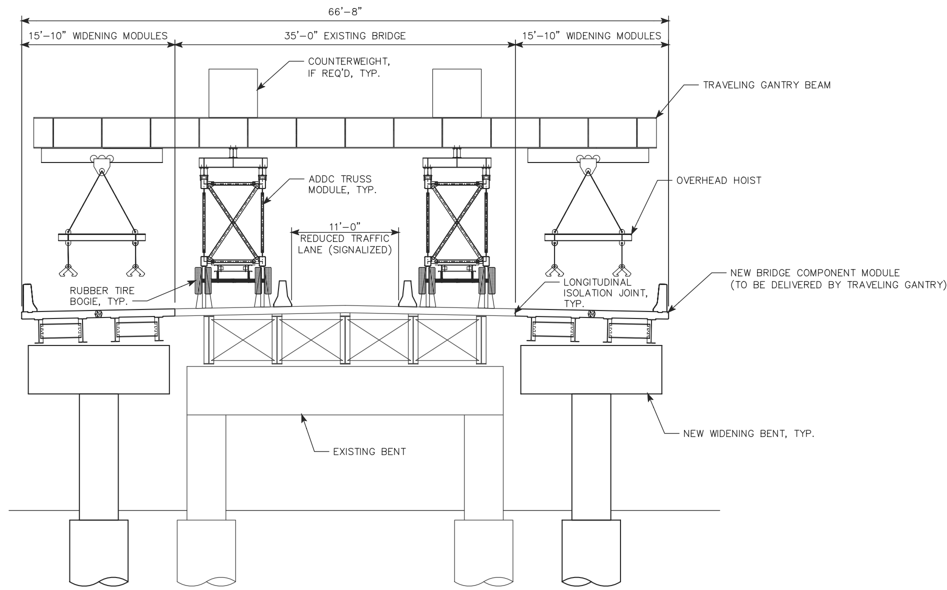
DESIGN TEAM

SHEET NUMBER CC24

DGN$SYTIME0123456 DGN$USERNAME DGN$SPEC



EXISTING BRIDGE WIDENING USING ADDC – TYPICAL CROSS SECTION

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PROJECT R04
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ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

ADDC CONCEPT – TYPICAL CROSS SECTION

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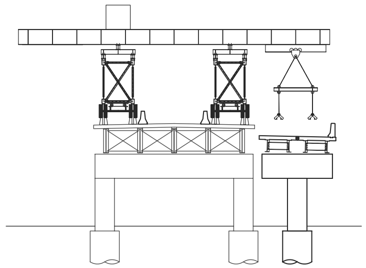
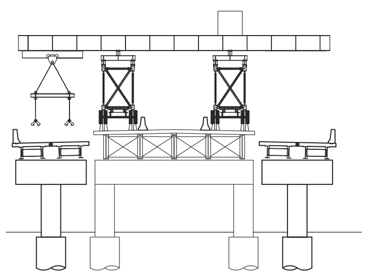
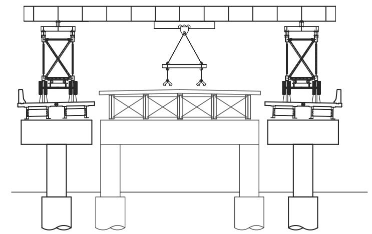


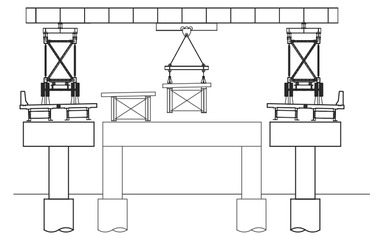
STAGE 1: ERECT NEW WIDENING STRUCTURE SIDE 1



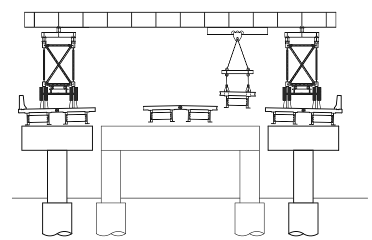
STAGE 2: ERECT NEW WIDENING STRUCTURE SIDE 2



STAGE 3: MOVE ERECTION TRUSSES AND RECONFIGURE GANTRY SYSTEM



STAGE 4: REMOVE EXISTING STRUCTURE



STAGE 5: PLACE NEW PREFABRICATED BRIDGE COMPONENTS

NOTES:

1. CONSTRUCTION STAGING SHOWN IS AN EXAMPLE OF A WIDENING PROJECT.
2. ADDC CONCEPT, INCLUDING ERECTION TRUSSES AND GANTRY SYSTEM, COULD BE MODIFIED AND RECONFIGURED AS REQUIRED.

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

ADDC CONCEPT – STAGED CONSTRUCTION

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DESIGN TEAM

SHEET NUMBER CC26

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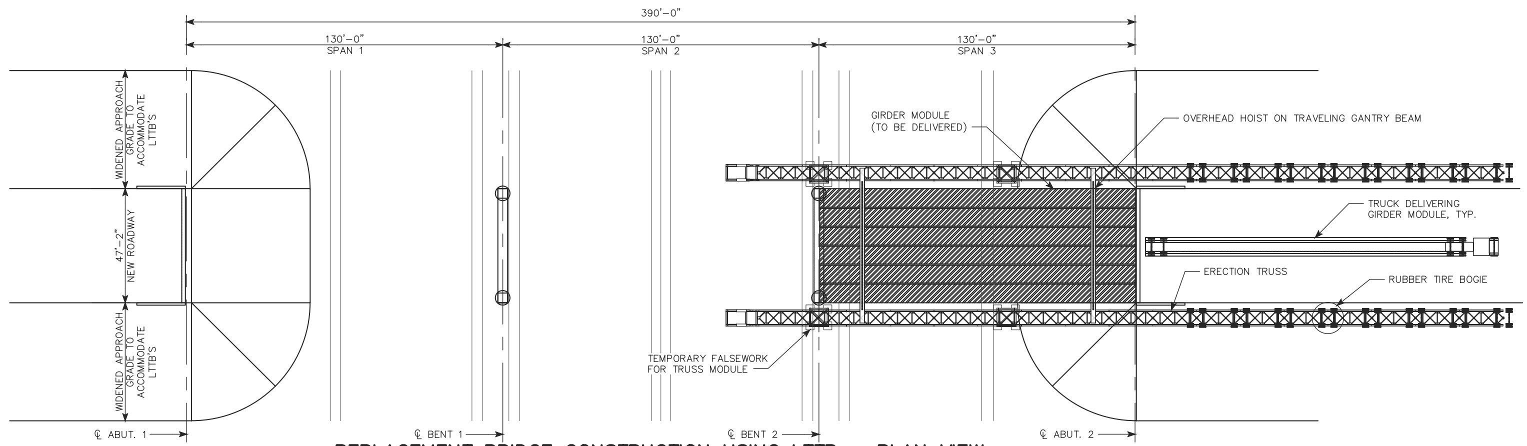
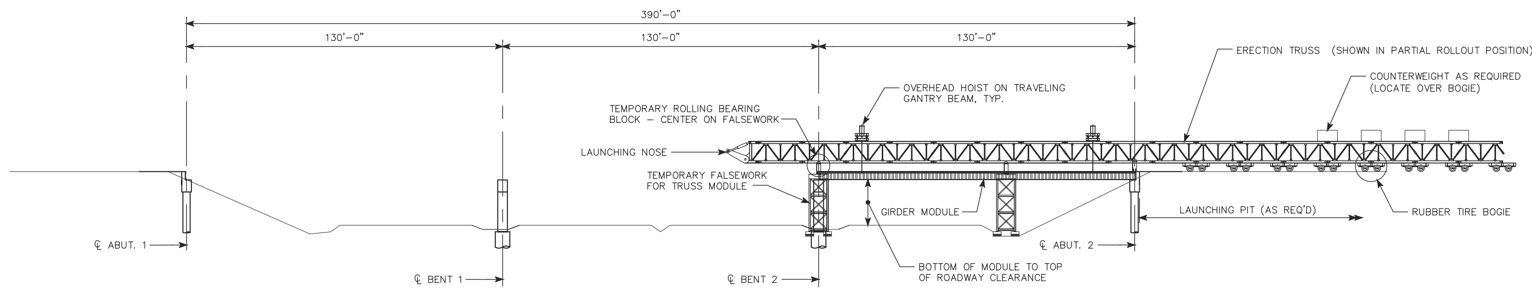


REPLACEMENT BRIDGE CONSTRUCTION USING LTTB – PLAN VIEW



REPLACEMENT BRIDGE CONSTRUCTION USING LTTB – ELEVATION VIEW

TRAFFIC DISRUPTION

DURING SUPERSTRUCTURE ERECTION OPERATION, MAINLINE ROADWAY (BELOW) REMAINS OPEN AND NO TRAFFIC ON PROPOSED (REPLACEMENT BRIDGE).

NOTES:

1. LTTB CONCEPT SHOWN FOR LONG SPAN BRIDGE. LTTB CONCEPT SIMILAR FOR SHORT SPAN BRIDGE.
2. FOR TYPICAL TRUSS MODULE DETAILS, SEE TYPICAL ERECTION TRUSS MODULE SHEET.
3. LTTB REFERS TO "LAUNCHED TEMPORARY TRUSS BRIDGE."
4. LTTB SCENARIOS DEVELOPED FOR PREFABRICATED STEEL OR CONCRETE GIRDER SUPERSTRUCTURE MODULES.
5. ERECTION OPERATIONS FOR CENTER AND END SPANS SIMILAR.
6. OVERHEAD BRIDGE SHOWN FOR HIGHWAY INTERSECTION. SIMILAR CONCEPT WOULD BE APPROPRIATE FOR RIVER OR CREEK SETTING.

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
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INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

LTTB CONCEPT – PLAN & ELEVATION

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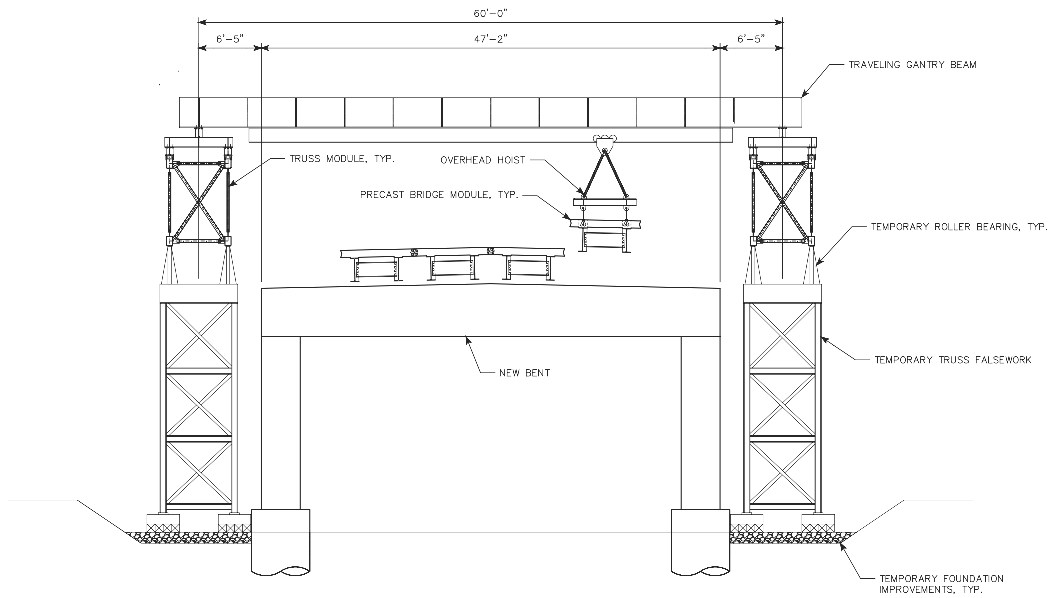

REPLACEMENT BRIDGE CONSTRUCTION USING LTTB – TYPICAL SECTION

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
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INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

LTTB CONCEPT – TYPICAL CROSS SECTION

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SHEET NUMBER CC28

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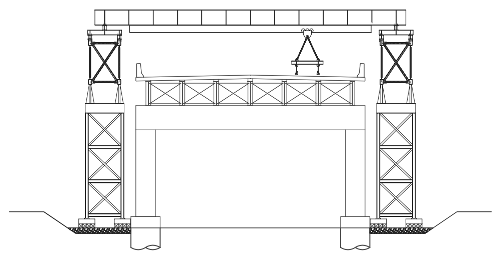
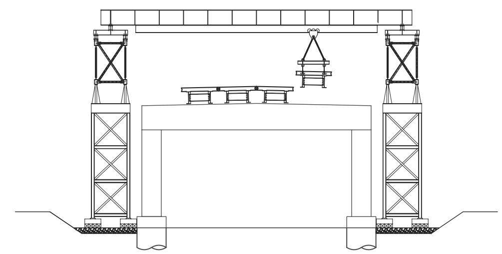
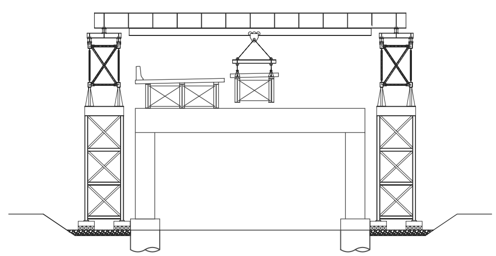


STAGE 1: LAUNCH ERECTION TRUSS & ASSEMBLE ROLLING GANTRY SYSTEM



STAGE 3: REPLACE WITH NEW STRUCTURE



STAGE 2: REMOVE EXISTING STRUCTURE

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

ACCELERATED BRIDGE CONSTRUCTION
CONCEPTS FOR MODULAR SYSTEMS

LTTB CONCEPT – STAGED CONSTRUCTION

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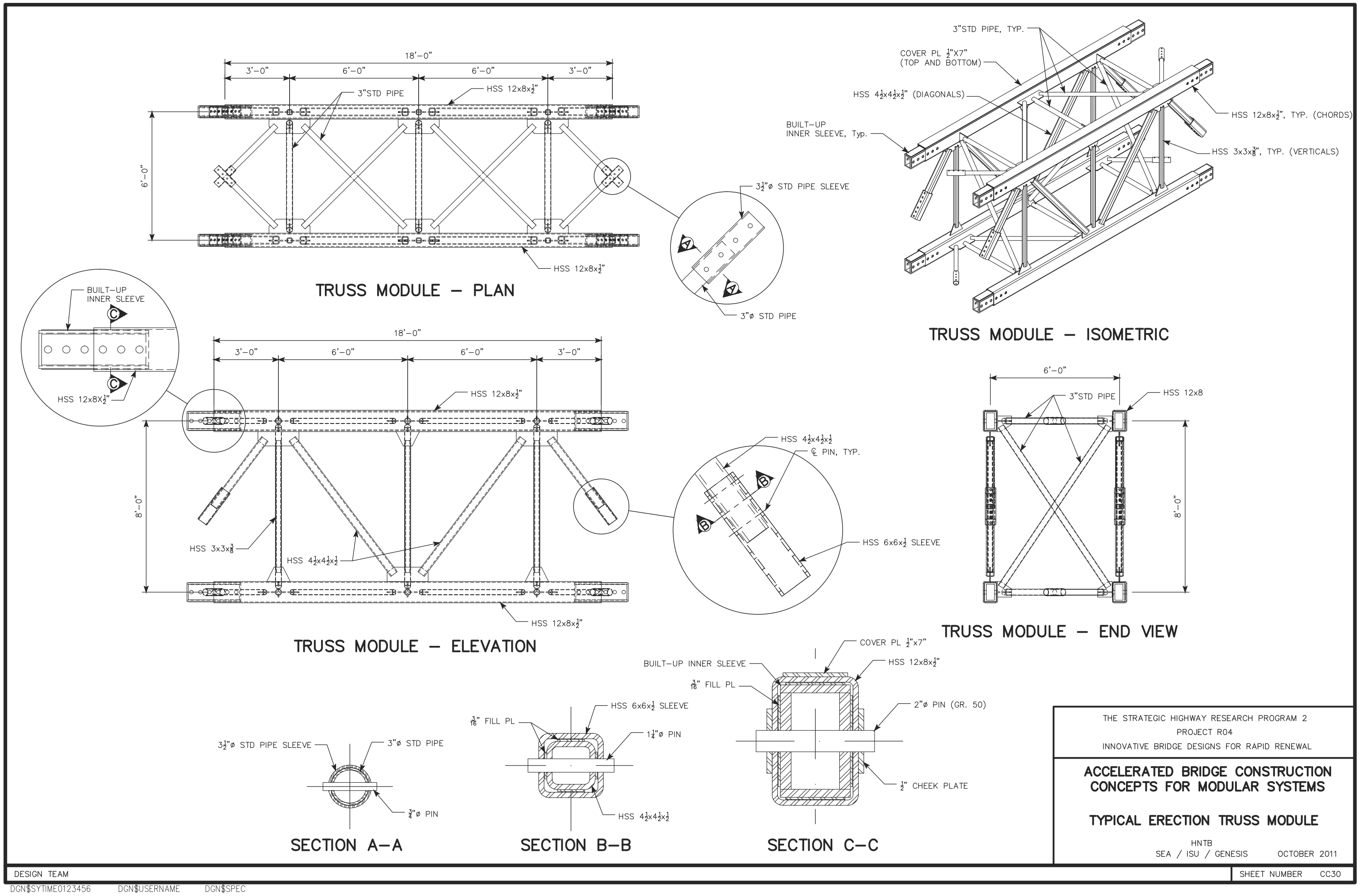
DESIGN TEAM

SHEET NUMBER CC29

DGN$SYTIME0123456 DGN$USERNAME DGN$SPEC


TRUSS MODULE — PLAN
18'–0"
3'–0"
6'–0"
6'–0"
3'–0"
6'–0"
3"STD PIPE
HSS 12x8x½"
HSS 12x8x½"
3½"ø STD PIPE SLEEVE
3"ø STD PIPE
TRUSS MODULE — ISOMETRIC
3"STD PIPE, TYP.
COVER PL ½"X7" (TOP AND BOTTOM)
HSS 4½x4½x½" (DIAGONALS)
BUILT–UP INNER SLEEVE, Typ.
HSS 12x8x½", TYP. (CHORDS)
HSS 3x3x⅜", TYP. (VERTICALS)
BUILT–UP INNER SLEEVE
HSS 12x8X½"
TRUSS MODULE — ELEVATION
18'–0"
3'–0"
6'–0"
6'–0"
3'–0"
8'–0"
HSS 12x8x½"
HSS 3x3x⅜"
HSS 4½x4½x½
HSS 12x8x½"
HSS 4½x4½x½
℄ PIN, TYP.
HSS 6x6x½ SLEEVE
TRUSS MODULE — END VIEW
6'–0"
3"STD PIPE
HSS 12x8
8'–0"
SECTION A–A
3½"ø STD PIPE SLEEVE
3"ø STD PIPE
¾"ø PIN
SECTION B–B
3/16" FILL PL
HSS 6x6x½ SLEEVE
1¼"ø PIN
HSS 4½x4½x½
SECTION C–C
COVER PL ½"x7"
BUILT–UP INNER SLEEVE
HSS 12x8x½"
3/16" FILL PL
2"ø PIN (GR. 50)
½" CHEEK PLATE
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL
ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS
TYPICAL ERECTION TRUSS MODULE
HNTB
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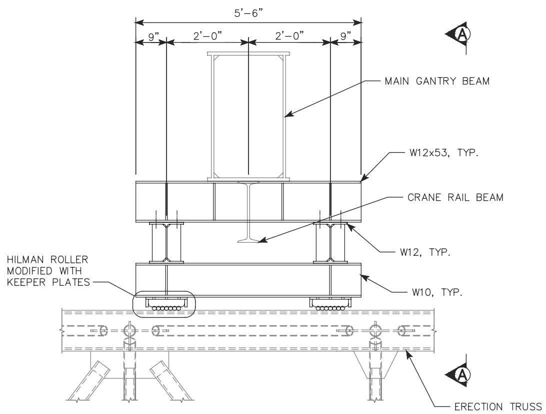
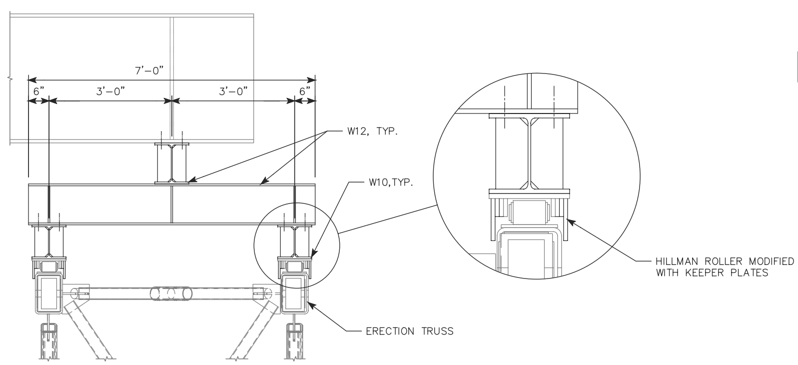
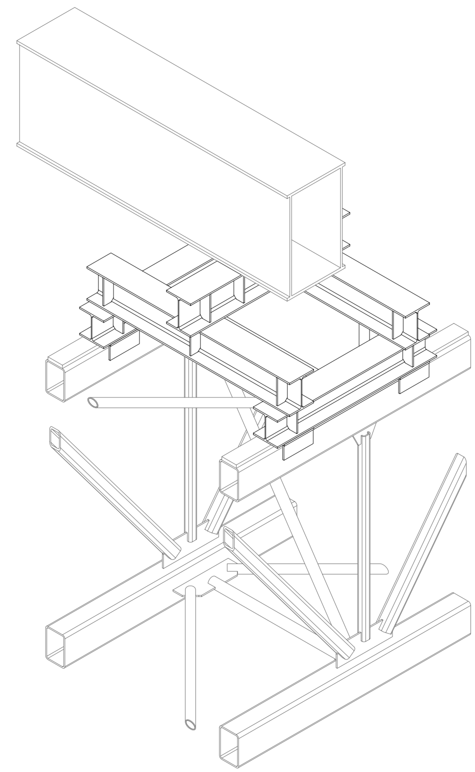
OCTOBER 2011
DESIGN TEAM
SHEET NUMBER CC30
DGN$SYTIME0123456 DGN$USERNAME DGN$SPEC



ROLLING GANTRY SYSTEM — ELEVATION VIEW



VIEW A—A



ISOMETRIC VIEW

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

TYPICAL ROLLING GANTRY CONCEPTS

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DESIGN TEAM

SHEET NUMBER CC31

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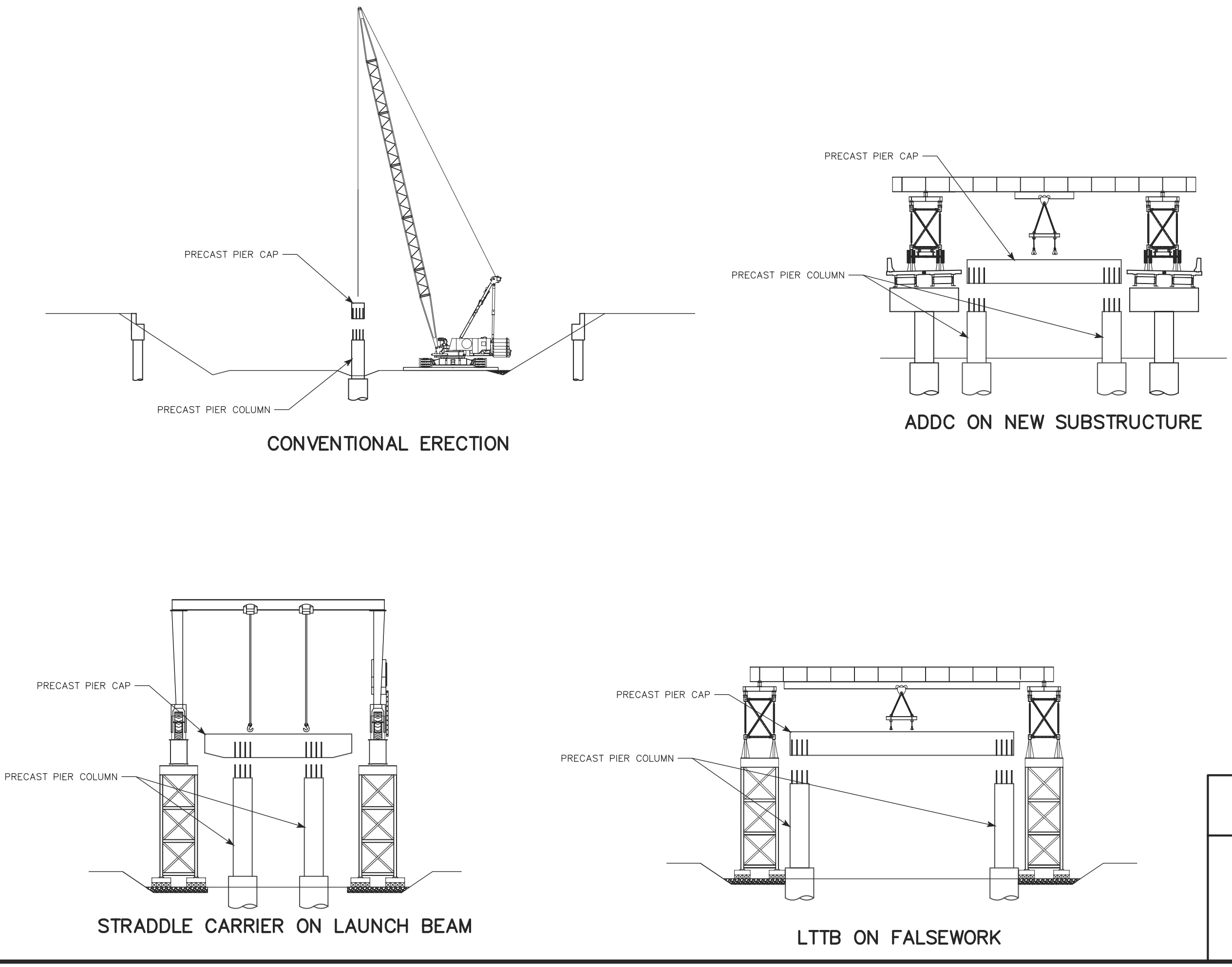

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

ERECTION OF PREFABRICATED CONCRETE SUBSTRUCTURE ELEMENTS

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DESIGN TEAM   SHEET NUMBER CC32

DGN$SYTIME0123456   DGN$USERNAME   DGN$SPEC

APPENDIX F

ABC Sample Design Calculations

Three design examples are presented in this appendix, as follows:

- Sample Calculation 1: Decked Steel Girder Design for ABC
- Sample Calculation 2: Decked Precast Prestressed Concrete Girder Design for ABC
- Sample Calculation 3: Precast Pier Design for ABC

The design examples illustrate the design steps involved in the ABC design process as given in the breakdown below. The ABC design philosophy and design criteria have been described. Annotations have been used for the purpose of providing explanation of the design steps. LRFD code references have also been included to guide the reader.

Sample Calculation 1: Decked Steel Girder Design for ABC

General:
1. Introduction
2. Design Philosophy
3. Design Criteria
4. Material Properties
5. Load Combinations

Girder Design:

6. Beam Section Properties
7. Permanent Loads
8. Precast Lifting Weight
9. Live Load Distribution Factors
10. Load Results
11. Flexural Strength
12. Flexural Strength Checks
13. Flexural Service Checks
14. Shear Strength
15. Fatigue Limit States
16. Bearing Stiffeners
17. Shear Connectors

Deck Design:

18. Slab Properties
19. Permanent Loads
20. Live Loads

21. Load Results
22. Flexural Strength Capacity Check
23. Longitudinal Deck Reinforcing Design
24. Design Checks
25. Deck Overhang Design

Continuity Design:

26. Compression Splice
27. Closure Pour Design

Sample Calculation 2: Decked Precast Prestressed Concrete girder Design for ABC

General:

1. Introduction
2. Design Philosophy
3. Design Criteria

Girder Design:

4. Beam Section
5. Material Properties
6. Permanent Loads
7. Precast Lifting Weight
8. Live Load
9. Prestress Properties
10. Prestress Losses
11. Concrete Stresses
12. Flexural Strength
13. Shear Strength
14. Splitting Resistance
15. Camber and Deflections
16. Negative Moment Flexural Strength

Sample Calculation 3a: Precast Pier Design for ABC (70' Span Straddle Bent)

1. Bent Cap Loading
2. Bent Cap Flexural Design
3. Bent Cap Shear and Torsion Design
4. Column / Drilled Shaft Loading and Design
5. Precast Component Design

Sample Calculation 3b: Precast Pier Design for ABC (70' Span Conventional Pier)

1. Bent Cap Loading
2. Bent Cap Flexural Design
3. Bent Cap Shear and Torsion Design
4. Column / Drilled Shaft Loading and Design
5. Precast Component Design

ABC SAMPLE CALCULATION – 1

Decked Steel Girder Design for ABC

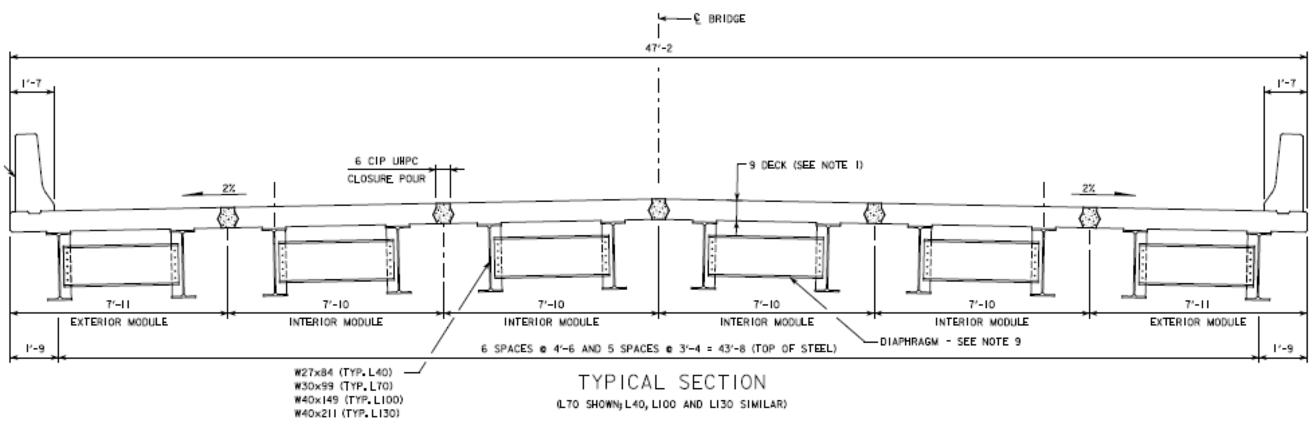
CONCRETE DECKED STEEL GIRDER DESIGN FOR ABC

This document shows the procedure for the design of a steel girder bridge with precast deck elements for use in a rapid bridge replacement design. The procedure in this document employs Accelerated Bridge Construction (ABC) methods. This sample calculation is intended as an informational tool for the practicing bridge engineer. These calculations illustrate the procedure followed to develop a similar design but shall not be considered fully exhaustive.

This sample calculation is based on the *AASHTO LRFD Bridge Design Specifications* (Fifth Edition with 2010 interims). References to the *AASHTO LRFD Bridge Design Specifications* are included throughout the design example. AASHTO references are presented in a dedicated column in the right margin of each page, immediately adjacent to the corresponding design procedure.

An analysis of the superstructure was performed using structural modeling software. The design moments, shears, and reactions used in the design example are taken from the output, but their computation is not shown in the design example.

BRIDGE GEOMETRY



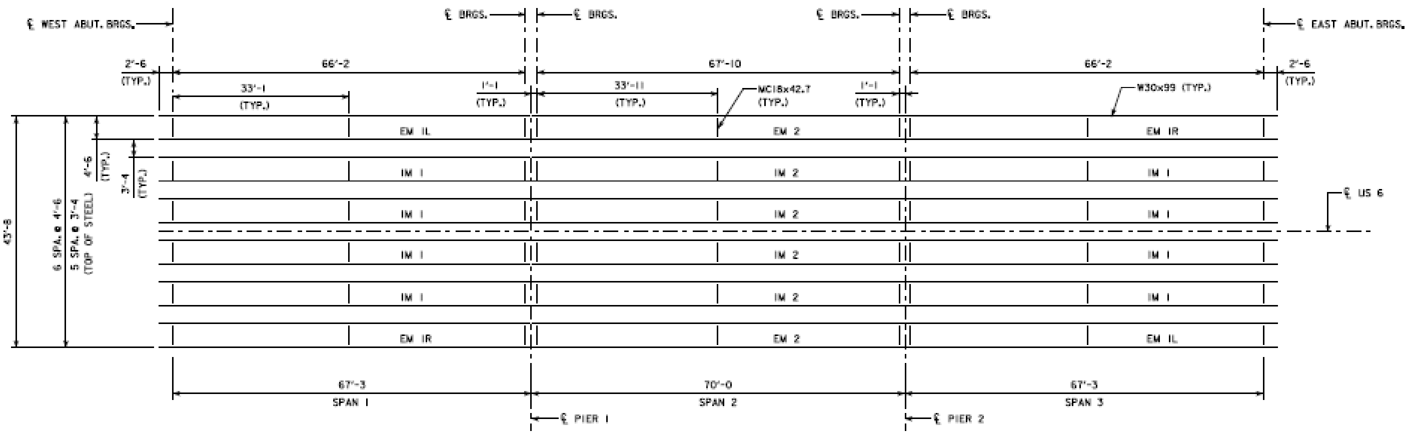
Design member parameters:

Deck Width:	$w_{deck} = 47ft + 2in$	C. to C. Piers:	$Length = 70ft$
Roadway Width:	$w_{roadway} = 44ft$	C. to C. Bearings	$L_{span} = 67ft + 10in$
Skew Angle:	$Skew = 0deg$	Bridge Length:	$L_{total} = 3\ Length = 210\,ft$
Deck Thickness	$t_d = 10.5in$	Stringer	W30x99
Haunch Thickness	$t_h = 2in$	Stringer Weight	$w_{s1} = 99plf$
Haunch Width	$w_h = 10.5in$	Stringer Length	$L_{str} = Length - 6\ in = 69.5\,ft$
Girder Spacing	$spacing_{int} = 3ft + 11in$	Average spacing of adjacent beams. This value is used so that effective deck width is not overestimated.	
	$spacing_{ext} = 4ft$		

TABLE OF CONTENTS:

1. INTRODUCTION

The design of this superstructure system follows AASHTO LRFD and is based on a bridge of three even spans, with no skew. The bridge has two 14-foot lanes and two 8-foot shoulders, for a total roadway width of 44' from curb to curb. The out-to-out width of the bridge is 47'-2". The bridge deck is precast reinforced concrete with overhangs at the outermost girders. The longitudinal girders are placed as simply supported modules, and made continuous with connection plates and cast-in-place deck joints. The design of the continuity at the deck joint is addressed in the final sections of this example.



FRAMING PLAN

The cross-section consists of six modules. The interior modules are identical and consist of two steel girders and a 7'-10" precast composite deck slab. Exterior modules include two steel girders and a 7'-11" precast composite deck slab, with F-shape barriers. Grade 50 steel is used throughout, and the deck concrete has a compressive strength of 5,000 psi. The closure pour joints between the modules use Ultra High Performance Concrete with a strength of 21,000 psi.

The following sections detail the design of the steel girders, including constructability checks, fatigue design for infinite fatigue lift (unless otherwise noted), and bearing stiffener design. The diaphragms are not designed in detail. A brief deck design is also included, with focus on the necessary checks for this type of modular superstructure.

Tips for reading this Design Example:

This calculation was prepared with Mathcad version 14. Mathcad is a computational aide for the practicing engineer. It allows for repetitive calculations in a clear, understandable and presentable fashion. Other computational aides may be used in lieu of Mathcad.

Mathcad is not a design software. Mathcad executes user mathematical and simple logic commands.

Example 1: User inputs are noted with dark shaded boxes. Shading of boxes allows the user to easily find the location of a desired variable. Given that equations are written in mathcad in the same fashion as they are on paper, except that they are interactive, shading input cells allows the user to quickly locate inputs amongst other data on screen. Units are user inputs.

Height of Structure: $H_{structure} = 25\ \text{ft}$

Example 2: Equations are typed directly into the workspace. Mathcad then reads the operators and executes the calculations.

Panels are 2.5 ft $N_{panels} = \dfrac{H_{structure}}{2.5\ \text{ft}}$ $N_{panels} = 10$

Example 3: If Statements are an important operator that allow for the user to dictate a future value with given parameters. They are marked by a solid bar and operate with the use of program specific logic commands.

Operator offers discount per volume of panels

$$\text{Discount} = \begin{vmatrix} 0.75 & \text{if } N_{panels} \geq 6 \\ 0.55 & \text{if } N_{panels} \geq 10 \\ 1 & \text{otherwise} \end{vmatrix} \qquad \text{Discount} = 0.6$$

Example 4: True or False Verification Statements are an important operator that allow for the user to verify a system criteria that has been manually input. They are marked by lighter shading to make a distinction between the user inputs. True or false statements check a single or pairs of variables and return a Zero or One.

Owner to proceed if discounts on retail below 60%

$$\text{Discount} \leq 0.55 = 1$$

2. DESIGN PHILOSOPHY

The geometry of this superstructure uses modules consisting of two rolled steel girders supporting a segment of bridge deck cast along the girder lengths. It is assumed that the initial condition for the girders is simply supported under the weight of the cast deck. Each girder is assumed to carry half the weight of the precast deck.

After the deck and girders are made composite, the barrier is added to the exterior modules. The barrier dead load is assumed to be evenly distributed between the two modules. Under the additional barrier dead load, the girders are again assumed to be simply supported.

During transport, it is assumed that 28-day concrete strength has been reached in the deck and the deck is fully composite with the girders. The self-weight of the module during lifting and placement is assumed as evenly distributed to four pick points (two per girder).

The modules are placed such that there is a bearing on each end and are again simply supported. The continuous span configuration, which includes two bearings per pier on either side of the UHPC joints, is analyzed for positive and negative bending and shear (using simple or refined methods). The negative bending moment above the pier is used to find the force couple for continuity design, between the compression plates at the bottom of the girders and the closure joint in the deck.

The deck design utilizes the equivalent strip method.

3. DESIGN CRITERIA

The first step for any bridge design is to establish the design criteria. The following is a summary of the primary design criteria for this design example:

Governing Specifications: AASTHO LRFD Bridge Design Specifications (5th Edition with 2010 interims)

Design Methodology: Load and Resistance Factor Design (LRFD)

Live Load Requirements: HL-93 S3.6

Section Constraints:

$W_{mod.max} = 200$ kip — Upper limit on the weight of the modules, based on common lifting and transport capabilities without significantly increasing time and/or cost due to unconventional equipment or permits

4. MATERIAL PROPERTIES

Structural Steel Yield Strength:	$F_y = 50ksi$		STable 6.4.1-1
Structural Steel Tensile Strength:	$F_u = 65ksi$		STable 6.4.1-1
Concrete 28-day Compressive Strength:	$f'_c = 5ksi$	$f'_{c\_uhpc} = 21ksi$	S5.4.2.1
Reinforcement Strength:	$F_s = 60ksi$		S5.4.3 & S6.10.3.7
Steel Density:	$w_s = 490pcf$		STable 3.5.1-1
Concrete Density:	$w_c = 150pcf$		STable 3.5.1-1
Modulus of Elasticity - Steel:	$E_s = 29000ksi$		
Modulus of Elasticity - Concrete:	$E_c = 33000\left(\frac{w_c}{1000pcf}\right)^{1.5}\sqrt{f'_c\,ksi} = 4286.8\ ksi$		
Modular Ratio:	$n = ceil\left(\frac{E_s}{E_c}\right) = 7$		
Future Wearing Surface Density:	$W_{fws} = 140pcf$		STable 3.5.1-1
Future Wearing Surface Thickness:	$t_{fws} = 2.5in$	(Assumed)	

5. LOAD COMBINATIONS

The following load combinations will be used in this design example, in accordance with Table 3.4.1-1.

Strength I = 1.25DC + 1.5DW + 1.75(LL+IM), where IM = 33%

Strength III = 1.25DC + 1.5DW + 1.40WS

Strength V = 1.25DC + 1.5DW + 1.35(LL+IM) + 0.40WS + 1.0WL, where IM = 33%

Service I = 1.0DC + 1.0DW + 1.0(LL+IM) + 0.3WS + 1.0WL, where IM = 33%

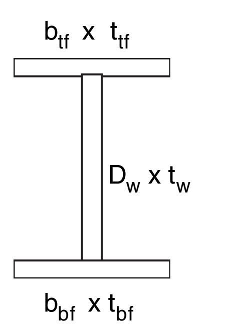
Service II = 1.0DC + 1.0DW + 1.3(LL+IM), where IM = 33%

Fatigue I = 1.5(LL+IM), where IM = 15%

6. BEAM SECTION

Determine Beam Section Properties:

Girder	W30x99	
Top Flange	$b_{tf} = 10.45in$	$t_{tf} = 0.67in$
Bottom Flange	$b_{bf} = 10.45in$	$t_{bf} = 0.67in$
Web	$D_w = 28.31in$	$t_w = 0.52in$
Girder Depth	$d_{gird} = 29.7in$	



Check Flange Proportion Requirements Met: S 6.10.2.2

$$\frac{b_{tf}}{2\ t_{tf}} \le 12.0 = 1 \qquad \frac{b_{bf}}{2\ t_{bf}} \le 12.0 = 1$$

$$b_{tf} \ge \frac{D_w}{6} = 1 \qquad b_{bf} \ge \frac{D_w}{6} = 1$$

$$t_{tf} \ge 1.1\ t_w = 1 \qquad t_{bf} \ge 1.1\ t_w = 1$$

$$0.1 \le \frac{\dfrac{t_{bf}^{\ 3}\ b_{bf}}{12}}{\dfrac{t_{tf}^{\ 3}\ b_{tf}}{12}} \le 10 = 1 \qquad \frac{\dfrac{t_{bf}\ b_{bf}}{12}}{\dfrac{t_{tf}\ b_{tf}}{12}} \ge 0.3 = 1$$

Properties for use when analyzing under beam self weight (steel only):

$$A_{tf} = b_{tf}\ t_{tf} \qquad A_{bf} = b_{bf}\ t_{bf} \qquad A_w = D_w\ t_w$$

$$A_{steel} = A_{bf} + A_{tf} + A_w \qquad A_{steel} = 28.7\ in^2$$

Total steel area.

$$y_{steel} = \frac{A_{tf}\dfrac{t_{tf}}{2} + A_{bf}\left(\dfrac{t_{bf}}{2} + D_w + t_{tf}\right) + A_w\left(\dfrac{D_w}{2} + t_{tf}\right)}{A_{steel}} \qquad y_{steel} = 14.8\ in$$

Steel centroid from top.

Calculate Iz:

Moment of inertia about Z axis.

$$I_{zsteel} = \frac{t_w\ D_w^{\ 3}}{12} + \frac{b_{tf}\ t_{tf}^{\ 3}}{12} + \frac{b_{bf}\ t_{bf}^{\ 3}}{12} + A_w\left(\frac{D_w}{2} + t_{tf} - y_{steel}\right)^2 + A_{tf}\left(y_{steel} - \frac{t_{tf}}{2}\right)^2 + A_{bf}\left(D_w + \frac{t_{bf}}{2} + t_{tf} - y_{steel}\right)^2$$

Calculate Iy:

$$I_{ysteel} = \frac{D_w\ t_w^{\ 3} + t_{tf}\ b_{tf}^{\ 3} + t_{bf}\ b_{bf}^{\ 3}}{12}$$

Moment of inertia about Y axis.

Calculate Ix:

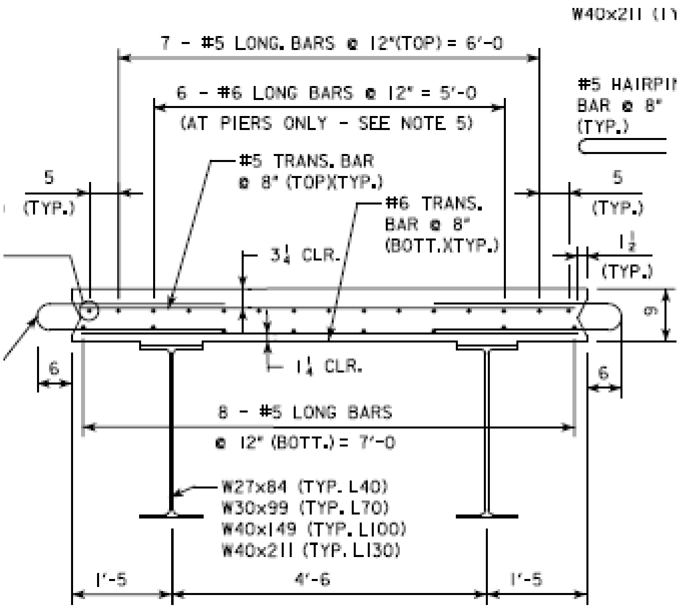
$$I_{xsteel} = \frac{1}{3}\left(b_{tf}\ t_{tf}^{\ 3} + b_{bf}\ t_{bf}^{\ 3} + D_w\ t_w^{\ 3}\right)$$

Moment of inertia about X axis.

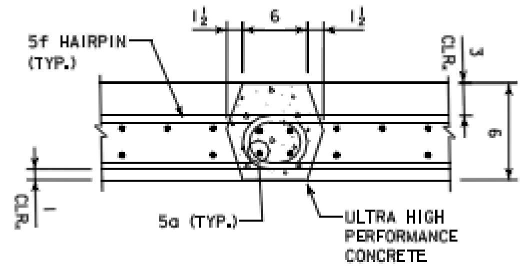
$$I_{zsteel} = 3923.795\ in^4 \qquad I_{ysteel} = 127.762\ in^4 \qquad I_{xsteel} = 3.4\ in^4 \qquad A_{steel} = 28.7\ in^2$$

Composite Section Properties (Uncracked Section - used for barrier dead load and live load positive bending):

Determine composite slab and reinforcing properties



INTERIOR MODULE
REINFORCING DETAIL



LONGITUDINAL CLOSURE POUR DETAIL
(TRANSVERSE REINFORCEMENT NOT SHOWN FOR CLARITY)

Slab thickness assumes some sacrificial thickness; use: $t_{slab} = 8in$

$D_t = \left(t_{slab} + t_{tf} + D_w + t_{bf}\right) = 37.6\ in$ Total section depth

$b_{eff} = spacing_{int} \quad b_{eff} = 47\ in$ Effective width. S 4.6.2.6.1

$b_{tr} = \dfrac{b_{eff}}{n}$ Transformed slab width as steel.

$I_{zslab} = b_{tr}\dfrac{t_{slab}^{\ 3}}{12}$ Transformed slab moment of inertia about z axis as steel.

$A_{slab} = b_{tr}\,t_{slab}$ Transformed slab area as steel.

Slab reinforcement: (Use #5 @ 8" top, and #6 @ 8" bottom; additional bar for continuous segments of #6 @ 12")

Typical Cross Section

$$A_{rt} = 0.465\frac{in^2}{ft}b_{eff} = 1.8\ in^2$$

$$A_r = A_{rt} + A_{rb} = 4.4\ in^2$$

$$c_{rt} = 2.5in + 0.625in + \left(\frac{5}{16}\right)in = 3.4\ in$$

$$c_r = \frac{\left(A_{rt}\,c_{rt} + A_{rb}\,c_{rb}\right)}{A_r} = 4.9\ in$$

Cross Section Over Support

$$A_{rb} = 0.66\frac{in^2}{ft}b_{eff} = 2.6\ in^2 \qquad A_{rtadd} = 0.44\frac{in^2}{ft}b_{eff} = 1.7\ in^2$$

$$A_{rneg} = A_r + A_{rtadd} = 6.1\ in^2$$

$$c_{rb} = t_{slab} - 1.75in - \left(\frac{6}{16}\right)in = 5.9\ in$$ ref from top of slab

$$c_{rneg} = \frac{\left(A_{rt}\,c_{rt} + A_{rb}\,c_{rb} + A_{rtadd}\,c_{rt}\right)}{A_{rneg}} = 4.5\ in$$

Find composite section centroid:

$$A_x = A_{steel} + \frac{A_r (n-1)}{n} + A_{slab} \qquad y_{slab} = \frac{t_{slab}}{2}$$

$$y_{st} = \frac{A_{tf}\left(\frac{t_{tf}}{2} + t_{slab}\right) + A_{bf}\left(\frac{t_{bf}}{2} + D_w + t_{tf} + t_{slab}\right) + A_w\left(\frac{D_w}{2} + t_{tf} + t_{slab}\right)}{A_{steel}}$$

Centroid of steel from top of slab.

$$y_c = \frac{y_{st} A_{steel} + \frac{c_r A_r (n-1)}{n} + A_{slab}\, y_{slab}}{A_x} \qquad y_c = 10.3 \text{ in}$$

Centroid of transformed composite section from top of slab.

Calculate Transformed Iz for composite section:

$$I_z = I_{zsteel} + A_{steel}\left(y_{st} - y_c\right)^2 + I_{zslab} + A_{slab}\left(y_{slab} - y_c\right)^2 + \frac{A_r (n-1)}{n}\left(c_r - y_c\right)^2$$

Transformed moment of inertia about the z axis.

Calculate Transformed Iy for composite section:

$$t_{tr} = \frac{t_{slab}}{n}$$

Transformed slab thickness.

$$I_{yslab} = \frac{t_{tr}\, b_{eff}^{\ 3}}{12}$$

Transformed moment of inertia about y axis of slab.

$$I_y = I_{ysteel} + I_{yslab}$$

Transformed moment of inertia about the y axis (ignoring reinforcement).

Calculate Transformed Ix for composite section:

$$I_x = \frac{1}{3}\left(b_{tf}\, t_{tf}^{\ 3} + b_{bf}\, t_{bf}^{\ 3} + D_w\, t_w^{\ 3} + b_{tr}\, t_{slab}^{\ 3}\right)$$

Transformed moment of inertia about the x axis.

Results: $A_x = 86.2 \text{ in}^2$ $\quad I_y = 10015.7 \text{ in}^4$ $\quad I_z = 10959.8 \text{ in}^4$ $\quad I_x = 1149.3 \text{ in}^4$

Composite Section Properties (Uncracked Section - used for live load negative bending):

Find composite section area and centroid:

$$A_{xneg} = A_{steel} + \frac{A_{rneg}(n-1)}{n} + A_{slab}$$

$$y_{cneg} = \frac{y_{steel} A_{steel} + \frac{c_{rneg} A_{rneg}(n-1)}{n} + A_{slab}\, y_{slab}}{A_{xneg}} \qquad y_{cneg} = 7.6 \text{ in}$$

Centroid of transformed composite section from top of slab.

Calculate Transformed Izneg for composite negative moment section:

$$I_{zneg} = I_{zsteel} + A_{steel}\left(y_{steel} - y_{cneg}\right)^2 + I_{zslab} + A_{slab}\left(y_{slab} - y_{cneg}\right)^2 + \frac{A_{rneg}(n-1)}{n}\left(c_{rneg} - y_{cneg}\right)^2$$

Transformed moment of inertia about the z axis.

$$I_{zneg} = 6457.4 \text{ in}^4$$

Composite Section Properties (Cracked Section - used for live load negative bending):

Find cracked section area and centroid:

$A_{cr} = A_{steel} + A_{rneg} = 34.9\ in^2$

$y_{cr} = \dfrac{\left(A_{steel}\ y_{steel} + A_{rneg}\ c_{rneg}\right)}{A_{cr}} = 13\ in$ $\qquad y_{crb} = t_{slab} + t_{tf} + D_w + t_{bf} - y_{cr} = 24.6\ in$

Find cracked section moments of inertia and section moduli:

$I_{zcr} = I_{zsteel} + A_{steel}\left(y_{steel} - y_{cr}\right)^2 + A_r\left(c_r - y_{cr}\right)^2$ $\qquad I_{zcr} = 4310.8\ in^4$

$I_{ycr} = I_{ysteel}$ $\qquad I_{ycr} = 127.8\ in^4$

$I_{xcr} = \dfrac{1}{3}\left(b_{tf}\ t_{tf}^{\ 3} + b_{bf}\ t_{tf}^{\ 3} + D_w\ t_w^{\ 3}\right)$ $\qquad I_{xcr} = 3.4\ in^4$

$d_{topcr} = y_{cr} - c_{rt}$ $\qquad d_{topcr} = 9.6\ in$

$d_{botcr} = t_{slab} + t_{tf} + D_w + t_{bf} - y_{cr}$ $\qquad d_{botcr} = 24.6\ in$

$S_{topcr} = \dfrac{I_{zcr}}{d_{topcr}}$ $\qquad S_{topcr} = 450.7\ in^3$

$S_{botcr} = \dfrac{I_{zcr}}{d_{botcr}}$ $\qquad S_{botcr} = 174.9\ in^3$

7. PERMANENT LOADS

Phase 1: Steel girders are simply supported, and support their self-weight plus the weight of the slab. Steel girders in each module for this example are separated by three diaphragms - one at each bearing location, and one at midspan. Other module span configurations may require an increase or decrease in the number of diaphragms.

$W_{deck\_int} = w_c\ spacing_{int}\ t_d$ $\qquad W_{deck\_int} = 514.1\ plf$

$W_{deck\_ext} = w_c\ spacing_{ext}\ t_d$ $\qquad W_{deck\_ext} = 525\ plf$

$W_{haunch} = w_c\ w_h\ t_h$ $\qquad W_{haunch} = 21.9\ plf$

$W_{stringer} = w_{s1}$ $\qquad W_{stringer} = 99\ plf$

Diaphragms:	MC18x42.7	Thickness Conn. Plate	$t_{conn} = \frac{5}{8} in$
Diaphragm Weight	$w_{s2} = 42.7 plf$	Width Conn. Plate	$w_{conn} = 5 in$
Diaphragm Length	$L_{diaph} = 4ft + 2.5in$	Height Conn. Plate	$h_{conn} = 28.5 in$

$W_{diaphragm} = w_{s2}\dfrac{L_{diaph}}{2}$ $\qquad W_{diaphragm} = 89.8\ lbf$

$W_{conn} = 2\ w_s\ t_{conn}\ w_{conn}\ h_{conn}$ $\qquad W_{conn} = 50.5\ lbf$

$W_{DCpoint} = \left(W_{diaphragm} + W_{conn}\right)1.05$ $\qquad W_{DCpoint} = 147.4\ lbf$

Equivalent distributed load from DC point loads: $\qquad w_{DCpt\_equiv} = \dfrac{3\ W_{DCpoint}}{L_{str}} = 6.4\ plf$

Interior Uniform Dead Load, Phase 1: $\qquad W_{DCuniform1\_int} = W_{deck\_int} + W_{haunch} + W_{stringer} + w_{DCpt\_equiv} = 641.3\ plf$

Exterior Uniform Dead Load, Phase 1: $\qquad W_{DCuniform1\_ext} = W_{deck\_ext} + W_{haunch} + W_{stringer} + w_{DCpt\_equiv} = 652.2\ plf$

Moments due to Phase 1 DL: $M_{DC1\_int}(x) = \dfrac{W_{DCuniform1\_int}\, x}{2}\left(L_{str} - x\right)$ $M_{DC1\_ext}(x) = \dfrac{W_{DCuniform1\_ext}\, x}{2}\left(L_{str} - x\right)$

Shear due to Phase 1 DL: $V_{DC1\_int}(x) = W_{DCuniform1\_int}\left(\dfrac{L_{str}}{2} - x\right)$ $V_{DC1\_ext}(x) = W_{DCuniform1\_ext}\left(\dfrac{L_{str}}{2} - x\right)$

Phase 2: Steel girders are simply supported and composite with the deck slab, and support their self-weight plus the weight of the slab in addition to barriers on exterior modules. Barriers are assumed to be evenly distributed between the two exterior module girders.

Barrier Area $A_{barrier} = 2.89ft^2$

Barrier Weight $W_{barrier} = \dfrac{\left(w_c\, A_{barrier}\right)}{2}$ $W_{barrier} = 216.8\ \text{plf}$

Interior Dead Load, Phase 2: $W_{DCuniform\_int} = W_{DCuniform1\_int} = 641.3\ \text{plf}$

Exterior Dead Load, Phase 2: $W_{DCuniform\_ext} = W_{DCuniform1\_ext} + W_{barrier} = 869\ \text{plf}$

Moments due to Phase 2 DL: $M_{DC2\_int}(x) = \dfrac{W_{DCuniform\_int}\, x}{2}\left(L_{str} - x\right)$ $M_{DC2\_ext}(x) = \dfrac{W_{DCuniform\_ext}\, x}{2}\left(L_{str} - x\right)$

Shear due to Phase 2 DL: $V_{DC2\_int}(x) = W_{DCuniform\_int}\left(\dfrac{L_{str}}{2} - x\right)$ $V_{DC2\_ext}(x) = W_{DCuniform\_ext}\left(\dfrac{L_{str}}{2} - x\right)$

Phase 3: Girders are composite and have been made continuous. Utilities and future wearing surface are applied.

Unit Weight Overlay $w_{ws} = 30psf$

$W_{ws\_int} = w_{ws}\ spacing_{int}$ $W_{ws\_int} = 117.5\ \text{plf}$

$W_{ws\_ext} = w_{ws}\left(spacing_{ext} - 1\ ft - 7in\right)$ $W_{ws\_ext} = 72.5\ \text{plf}$

Unit Weight Utilities $W_u = 15plf$

$W_{DWuniform\_int} = W_{ws\_int} + W_u$ $W_{DWuniform\_int} = 132.5\ \text{plf}$

$W_{DWuniform\_ext} = W_{ws\_ext} + W_u$ $W_{DWuniform\_ext} = 87.5\ \text{plf}$

Moments due to DW: $M_{DW\_int}(x) = \dfrac{W_{DWuniform\_int}\, x}{2}\left(L_{str} - x\right)$ $M_{DW\_ext}(x) = \dfrac{W_{DWuniform\_ext}\, x}{2}\left(L_{str} - x\right)$

Shears due to DW: $V_{DW\_int}(x) = W_{DWuniform\_int}\left(\dfrac{L_{str}}{2} - x\right)$ $V_{DW\_ext}(x) = W_{DWuniform\_ext}\left(\dfrac{L_{str}}{2} - x\right)$

8. PRECAST LIFTING WEIGHTS AND FORCES

This section addresses the construction loads for lifting the module into place. The module is lifted from four points, at some distance, D_{lift} from each end of each girder.

Distance from end of lifting point: $D_{lift} = 8.75ft$

Assume weight uniformly distributed along girder, with 30% Dynamic Dead Load Allowance:

Dynamic Dead Load Allowance: $DLIM = 30\%$

Interior Module:

Total Interior Module Weight: $W_{int} = \left(L_{str}\ W_{DCuniform\_int} + 3\ W_{DCpoint}\right) 2\ (1 + DLIM) = 117\ kip$

Vertical force at lifting point: $F_{lift\_int} = \dfrac{W_{int}}{4} = 29.3\ kip$

Equivalent distributed load: $w_{int\_IM} = \dfrac{W_{int}}{\left(2\ L_{str}\right)} = 842\ plf$

Min (Neg.) Moment during lifting: $M_{lift\_neg\_max\_int} = -w_{int\_IM} \dfrac{\left(D_{lift}^{\ 2}\right)}{2}$ $\quad M_{lift\_neg\_max\_int} = -32.2\ kip\ ft$

Max (Pos.) Moment during lifting:

$$M_{lift\_pos\_max\_int} = \begin{cases} 0 \ \text{ if } \dfrac{w_{int\_IM}\left(L_{str} - 2\ D_{lift}\right)^2}{8} + M_{lift\_neg\_max\_int} < 0 \\ \dfrac{w_{int\_IM}\left(L_{str} - 2\ D_{lift}\right)^2}{8} + M_{lift\_neg\_max\_int} \end{cases}$$

$M_{lift\_pos\_max\_int} = 252.4\ kip\ ft$

Exterior Module:

Total Exterior Module Weight: $W_{ext} = \left(L_{str}\ W_{DCuniform\_ext} + 3\ W_{DCpoint} + W_{barrier}\ L_{str}\right) 2\ (1 + DLIM) = 197.3\ kip$

Vertical force at lifting point: $F_{lift\_ext} = \dfrac{W_{ext}}{4} = 49.3\ kip$

Equivalent distributed load: $w_{ext\_IM} = \dfrac{W_{ext}}{2\ L_{str}} = 1419.7\ plf$

Min (Neg.) Moment during lifting: $M_{lift\_neg\_max\_ext} = -w_{ext\_IM} \dfrac{D_{lift}^{\ 2}}{2}$ $\quad M_{lift\_neg\_max\_ext} = -54.3\ kip\ ft$

Max (Pos.) Moment during lifting:

$$M_{lift\_pos\_max\_ext} = \begin{cases} 0 \ \text{ if } \dfrac{w_{ext\_IM}\left(L_{str} - 2\ D_{lift}\right)^2}{8} + M_{lift\_neg\_max\_ext} < 0 \\ \dfrac{w_{ext\_IM}\left(L_{str} - 2\ D_{lift}\right)^2}{8} + M_{lift\_neg\_max\_ext} \end{cases}$$

$M_{lift\_pos\_max\_ext} = 425.5\ kip\ ft$

Max Shear during lifting: $V_{lift} = \max\left(w_{ext\_IM}\ D_{lift}, F_{lift\_ext} - w_{ext\_IM}\ D_{lift}\right) = 36.9\ kip$

9. LIVE LOAD DISTRIBUTION FACTORS

These factors represent the distribution of live load from the deck to the girders in accordance with AASHTO Section 4, and assumes the deck is fully continuous across the joints.

Girder Section Modulus: $I_{zsteel} = 3923.8\ in^4$

Girder Area: $A_{steel} = 28.7\ in^2$

Girder Depth: $d_{gird} = 29.7\ in$

Distance between centroid of deck and centroid of beam: $e_g = \frac{t_d}{2} + t_h + \frac{d_{gird}}{2} = 22.1\ in$

Modular Ratio: $n = 7$

Multiple Presence Factors: $MP_1 = 1.2$ $\quad MP_2 = 1.0$ S3.6.1.1.2-1

Interior Stringers for Moment: S4.6.2.2.1-1

One Lane Loaded: $K_g = n\left(I_{zsteel} + A_{steel}\, e_g^{\,2}\right) = 125670.9\ in^4$

$$g_{int\_1m} = \left[0.06 + \left(\frac{spacing_{int}}{14ft}\right)^{0.4}\left(\frac{spacing_{int}}{L_{span}}\right)^{0.3}\left(\frac{K_g}{L_{span}\, t_d^{\,3}}\right)^{0.1}\right] = 0.269$$

Two Lanes Loaded: $$g_{int\_2m} = \left[0.075 + \left(\frac{spacing_{int}}{9.5ft}\right)^{0.6}\left(\frac{spacing_{int}}{L_{span}}\right)^{0.2}\left(\frac{K_g}{L_{span}\, t_d^{\,3}}\right)^{0.1}\right] = 0.347$$

Governing Factor: $g_{int\_m} = \max\left(g_{int\_1m}, g_{int\_2m}\right) = 0.347$

Interior Stringers for Shear:

One Lane Loaded: $$g_{int\_1v} = \left(0.36 + \frac{spacing_{int}}{25ft}\right) = 0.517$$

Two Lanes Loaded: $$g_{int\_2v} = \left[0.2 + \frac{spacing_{int}}{12ft} + -\left(\frac{spacing_{int}}{35ft}\right)^2\right] = 0.514$$

Governing Factor: $g_{int\_v} = \max\left(g_{int\_1v}, g_{int\_2v}\right) = 0.517$

Exterior Stringers for Moment:

One Lane Loaded: Use Lever Rule. Wheel is 2' from barrier; barrier is 2" beyond exterior stringer.

$d_e = 2in$

$L_{spa} = 4.5ft \quad r = L_{spa} + d_e - 2ft = 2.7\ ft$

$$g_{ext\_1m} = MP_1 \frac{0.5r}{L_{spa}} = 0.356$$

Two Lanes Loaded: $$e_{2m} = 0.77 + \frac{d_e}{9.1ft} = 0.7883$$

$g_{ext\_2m} = e_{2m}\, g_{int\_2m} = 0.273$

Governing Factor: $g_{ext\_m} = \max(g_{ext\_1m}, g_{ext\_2m}) = 0.356$

Exterior Stringers for Shear:

One Lane Loaded: Use Lever Rule.

$$g_{ext\_1v} = g_{ext\_1m} = 0.356$$

Two Lanes Loaded: $e_{2v} = 0.6 + \dfrac{d_e}{10ft} = 0.62$

$$g_{ext\_2v} = e_{2v}\, g_{int\_2v} = 0.317$$

Governing Factor: $g_{ext\_v} = \max(g_{ext\_1v}, g_{ext\_2v}) = 0.356$

Factor for Use for Shear: $g_v = \max(g_{int\_v}, g_{ext\_v}) = 0.517$

Factor for Use for Moment: $g_m = \max(g_{int\_m}, g_{ext\_m}) = 0.356$

10. LOAD RESULTS

Case 1: Dead Load on Steel Only (calculated in Section 7). Negative moments are zero and are not considered. Because the girder is simply supported, the maximum moment is at x = Lstr/2 and the maximum shear is at x = 0.

Interior Girder $M_{DC1int} = M_{DC1\_int}\left(\dfrac{L_{str}}{2}\right) = 387.2\ kip\ ft$ $M_{DW1int} = 0\ kip\ ft$ $M_{LL1int} = 0kip\ ft$

$V_{DC1int} = V_{DC1\_int}(0) = 22.3\ kip$ $V_{DW1int} = 0\ kip$ $V_{LL1int} = 0\ kip$

Exterior Girder $M_{DC1ext} = M_{DC1\_ext}\left(\dfrac{L_{str}}{2}\right) = 393.8\ kip\ ft$ $M_{DW1ext} = 0\ kip\ ft$ $M_{LL1ext} = 0\ kip\ ft$

$V_{DC1ext} = V_{DC1\_ext}(0) = 22.7\ kip$ $V_{DW1ext} = 0\ kip$ $V_{LL1ext} = 0\ kip\ ft$

Load Cases:

$$M_{1\_STR\_I} = \max(1.25\ M_{DC1int} + 1.5\ M_{DW1int} + 1.75\ M_{LL1int}, 1.25\ M_{DC1ext} + 1.5\ M_{DW1ext} + 1.75\ M_{LL1ext}) = 492.3\ kip\ ft$$

$$V_{1\_STR\_I} = \max(1.25\ V_{DC1int} + 1.5\ V_{DW1int} + 1.75\ V_{LL1int}, 1.25\ V_{DC1ext} + 1.5\ V_{DW1ext} + 1.75\ V_{LL1ext}) = 28.3\ kip$$

Case 2: Dead Load on Composite Section (calculated in Section 7). Negative moments are zero and are not considered. Again, the maximum moment occur at x = Lstr/2 and the maximum shear is at x = 0.

Interior Girder $M_{DC2int} = M_{DC2\_int}\left(\dfrac{L_{str}}{2}\right) = 387.2\ kip\ ft$ $M_{DW2int} = 0\ kip\ ft$ $M_{LL2int} = 0\ kip\ ft$

$V_{DC2int} = V_{DC2\_int}(0) = 22.3\ kip$ $V_{DW2int} = 0\ kip$ $V_{LL2int} = 0\ kip$

Exterior Girder $M_{DC2ext} = M_{DC2\_ext}\left(\dfrac{L_{str}}{2}\right) = 524.7\ kip\ ft$ $M_{DW2ext} = 0\ kip\ ft$ $M_{LL2ext} = 0\ kip\ ft$

$V_{DC2ext} = V_{DC2\_ext}(0) = 30.2\ kip$ $V_{DW2ext} = 0\ kip$ $V_{LL2ext} = 0\ kip$

Load Cases:

$$M_{2\_STR\_I} = \max(1.25\ M_{DC2int} + 1.5\ M_{DW2int} + 1.75\ M_{LL2int}, 1.25\ M_{DC2ext} + 1.5\ M_{DW2ext} + 1.75\ M_{LL2ext}) = 655.8\ kip\ ft$$

$$V_{2\_STR\_I} = \max(1.25\ V_{DC2int} + 1.5\ V_{DW2int} + 1.75\ V_{LL2int}, 1.25\ V_{DC2ext} + 1.5\ V_{DW2ext} + 1.75\ V_{LL2ext}) = 37.7\ kip$$

Case 3: Composite girders are lifted into place from lifting points located distance D_{lift} from the girder edges. Maximum moments and shears were calculated in Section 8.

Interior Girder

$M_{DC3int} = M_{lift\_pos\_max\_int} = 252.4\ \text{kip ft}$ $\quad M_{DW3int} = 0\ \text{kip ft}$ $\quad M_{LL3int} = 0\ \text{kip ft}$

$M_{DC3int\_neg} = \left|M_{lift\_neg\_max\_int}\right| = 32.2\ \text{kip ft}$ $\quad M_{DW3int\_neg} = 0\ \text{kip ft}$ $\quad M_{LL3int\_neg} = 0\ \text{kip ft}$

$V_{DC3int} = V_{lift} = 36.9\ \text{kip}$ $\quad V_{DW3int} = 0\ \text{kip}$ $\quad V_{LL3int} = 0\ \text{kip}$

Exterior Girder

$M_{DC3ext} = M_{lift\_pos\_max\_ext} = 425.5\ \text{kip ft}$ $\quad M_{DW3ext} = 0\ \text{kip ft}$ $\quad M_{LL3ext} = 0\ \text{kip ft}$

$M_{DC3ext\_neg} = \left|M_{lift\_neg\_max\_ext}\right| = 54.3\ \text{kip ft}$ $\quad M_{DW3ext\_neg} = 0\ \text{kip ft}$ $\quad M_{LL3ext\_neg} = 0\ \text{kip ft}$

$V_{DC3ext} = V_{lift} = 36.9\ \text{kip}$ $\quad V_{DW3ext} = 0\ \text{kip}$ $\quad V_{LL3ext} = 0\ \text{kip}$

Load Cases:

$$M_{3\_STR\_I} = \max\left(1.5\ M_{DC3int} + 1.5\ M_{DW3int}, 1.5\ M_{DC3ext} + 1.5\ M_{DW3ext}\right) = 638.3\ \text{kip ft}$$

$$M_{3\_STR\_I\_neg} = \max\left(1.5\ M_{DC3int\_neg} + 1.5\ M_{DW3int\_neg}, 1.5\ M_{DC3ext\_neg} + 1.5\ M_{DW3ext\_neg}\right) = 81.5\ \text{kip ft}$$

$$V_{3\_STR\_I} = \max\left(1.5\ V_{DC3int} + 1.5\ V_{DW3int}, 1.5\ V_{DC3ext} + 1.5\ V_{DW3ext}\right) = 55.4\ \text{kip}$$

Case 4: Composite girders made continuous. Utilities and future wearing surface are applied, and live load. Maximum moment and shear results are from a finite element analysis not included in this design example. The live load value includes the lane fraction calculated in Section 9, and impact.

Governing Loads:

$M_{DC4} = 440\ \text{kip ft}$ $\quad M_{DW4} = 43.3\ \text{kip ft}$ $\quad M_{LL4} = 590.3\ \text{kip ft}$

$M_{DC4neg} = -328.9\ \text{kip ft}$ $\quad M_{WS4} = 0\text{kip ft}$ $\quad M_{W4} = 0\text{kip ft}$

$V_u = 145.3\text{kip}$ $\quad M_{DW4neg} = -32.3\ \text{kip ft}$ $\quad M_{LL4neg} = -314.4\text{kip ft}$

$M_{WS4neg} = 0\ \text{kip ft}$ $\quad M_{WL4neg} = 0\ \text{kip ft}$

Load Cases:

$$M_{4\_STR\_I} = 1.25\ M_{DC4} + 1.5\ M_{DW4} + 1.75\ M_{LL4} = 1648\ \text{kip ft}$$

$$M_{4\_STR\_I\_neg} = 1.25\ M_{DC4neg} + 1.5\ M_{DW4neg} + 1.75\ M_{LL4neg} = -1009.8\ \text{kip ft}$$

$$M_{4\_STR\_III} = 1.25\ M_{DC4} + 1.5\ M_{DW4} + 1.4\ M_{WS4} = 614.9\ \text{kip ft}$$

$$M_{4\_STR\_III\_neg} = 1.25\ M_{DC4neg} + 1.5\ M_{DW4neg} + 1.4\ M_{WS4} = -459.6\ \text{kip ft}$$

$$M_{4\_STR\_V} = 1.25\ M_{DC4} + 1.5\ M_{DW4} + 1.35\ M_{LL4} + 0.4\ M_{WS4} + 1.0\ M_{W4} = 1411.9\ \text{kip ft}$$

$$M_{4\_STR\_V\_neg} = 1.25\ M_{DC4neg} + 1.5\ M_{DW4neg} + 1.35\ M_{LL4neg} + 0.4\ M_{WS4neg} + 1.0\ M_{WL4neg} = -884\ \text{kip ft}$$

$$M_{4\_SRV\_I} = 1.0\ M_{DC4} + 1.0\ M_{DW4} + 1.0\ M_{LL4} + 0.3\ M_{WS4} + 1.0\ M_{W4} = 1073.6\ \text{kip ft}$$

$$M_{4\_SRV\_I\_neg} = 1.0\ M_{DC4neg} + 1.0\ M_{DW4neg} + 1.0\ M_{LL4neg} + 0.3\ M_{WS4neg} + 1.0\ M_{WL4neg} = -675.6\ \text{kip ft}$$

$$M_{4\_SRV\_II} = 1.0\ M_{DC4} + 1.0\ M_{DW4} + 1.3\ M_{LL4} = 1250.7\ \text{kip ft}$$

$$M_{4\_SRV\_II\_neg} = 1.0\ M_{DC4neg} + 1.0\ M_{DW4neg} + 1.3\ M_{LL4neg} = -769.9\ \text{kip ft}$$

11. FLEXURAL STRENGTH

The flexural resistance shall be determined as specified in LRFD Design Article 6.10.6.2. Determine Stringer Plastic Moment Capacity First.

LFRD Appendix D6 Plastic Moment

Find location of PNA:

Forces:

$P_{rt} = A_{rt}\ F_s = 109.3\ \text{kip}$ $\quad P_s = 0.85\ f'_c\ b_{eff}\ t_{slab} = 1598\ \text{kip}$ $\quad P_w = F_y\ D_w\ t_w = 736.1\ \text{kip}$

$P_{rb} = A_{rb}\ F_s = 155.1\ \text{kip}$ $\quad P_c = F_y\ b_{tf}\ t_{tf} = 350.1\ \text{kip}$ $\quad P_t = F_y\ b_{bf}\ t_{bf} = 350.1\ \text{kip}$

$$PNA_{pos} = \left|\begin{array}{l} \text{"case 1"} \ \text{ if } \left(P_t + P_w\right) \geq \left(P_c + P_s + P_{rt} + P_{rb}\right) \\ \text{otherwise} \\ \quad \left|\begin{array}{l} \text{"case 2"} \ \text{ if } \left[\left(P_t + P_w + P_c\right) \geq \left(P_s + P_{rt} + P_{rb}\right)\right] \\ \text{otherwise} \\ \quad \left|\begin{array}{l} \text{"case 3"} \ \text{ if } \left[\left(P_t + P_w + P_c\right) \geq \left(\dfrac{c_{rb}}{t_{slab}}\ P_s + P_{rt} + P_{rb}\right)\right] \\ \text{otherwise} \\ \quad \left|\begin{array}{l} \text{"case 4"} \ \text{ if } \left[\left(P_t + P_w + P_c + P_{rb}\right) \geq \left(\dfrac{c_{rb}}{t_{slab}}\ P_s + P_{rt}\right)\right] \\ \text{otherwise} \\ \quad \left|\begin{array}{l} \text{"case 5"} \ \text{ if } \left[\left(P_t + P_w + P_c + P_{rb}\right) \geq \left(\dfrac{c_{rt}}{t_{slab}}\ P_s + P_{rt}\right)\right] \\ \text{otherwise} \\ \quad \left|\begin{array}{l} \text{"case 6"} \ \text{ if } \left(P_t + P_w + P_c + P_{rb} + P_{rt}\right) \geq \left(\dfrac{c_{rt}}{t_{slab}}\ P_s\right) \\ \text{"case 7"} \ \text{ if } \left(P_t + P_w + P_c + P_{rb} + P_{rt}\right) \leq \left(\dfrac{c_{rt}}{t_{slab}}\ P_s\right) \ \text{ otherwise} \end{array}\right. \end{array}\right. \end{array}\right. \end{array}\right. \end{array}\right. \end{array}\right.$$

$$PNA_{pos} = \text{"case 4"}$$

$$PNA_{neg} = \left|\begin{array}{l} \text{"case 1"} \ \text{ if } \left(P_c + P_w\right) \geq \left(P_t + P_{rt} + P_{rb}\right) \\ \text{"case 2"} \ \text{ if } \left[\left(P_t + P_w + P_c\right) \geq \left(P_{rt} + P_{rb}\right)\right] \ \text{ otherwise} \end{array}\right. \qquad PNA_{neg} = \text{"case 1"}$$

Calculate Y, Dp, and Mp: $D = D_w \quad t_s = t_{slab} \quad t_h = 0 \quad C_{rt} = c_{rt} \quad C_{rb} = c_{rb}$

Case I : Plastic Neutral Axis in the Steel Web

$$Y_1 = \frac{D}{2}\left(\frac{P_t - P_c - P_s - P_{rt} - P_{rb}}{P_w} + 1\right) \qquad D_{P1} = t_s + t_h + t_{tf} + Y_1$$

$$M_{P1} = \frac{P_w}{2D}\left[Y_1^{\,2} + \left(D - Y_1\right)^2\right] + \begin{bmatrix} P_s\left(Y_1 + \frac{t_s}{2} + t_{tf} + t_h\right) + P_{rt}\left(t_s - C_{rt} + t_{tf} + Y_1 + t_h\right) + P_{rb}\left(t_s - C_{rb} + t_{tf} + Y_1 + t_h\right) \ldots \\ + P_c\left(Y_1 + \frac{t_{tf}}{2}\right) + P_t\left(D - Y_1 + \frac{t_{bf}}{2}\right) \end{bmatrix}$$

$$Y_{1neg} = \left(\frac{D}{2}\right)\left[1 + \frac{\left(P_c - P_t - P_{rt} - P_{rb}\right)}{P_w}\right] \qquad D_{p1neg} = t_s + t_h + t_{tf} + Y_{1neg}$$

$$D_{CP1neg} = \left(\frac{D}{2\ P_w}\right)\left(P_t + P_w + P_{rb} + P_{rt} - P_c\right)$$

$$M_{p1neg} = \begin{bmatrix} \left(\frac{P_w}{2\ D}\right)\left[Y_{1neg}^{\ 2} + \left(D_w - Y_{1neg}\right)^2\right] + P_{rt}\left(t_s - C_{rt} + t_{tf} + Y_{1neg} + t_h\right) + P_{rb}\left(t_s - C_{rb} + t_{tf} + Y_{1neg} + t_h\right) \ldots \\ + P_t\left(D - Y_{1neg} + \frac{t_{bf}}{2}\right) + P_c\left(Y_{1neg} + \frac{t_{tf}}{2}\right) \end{bmatrix}$$

Case II: Plastic Neutral Axis in the Steel Top Flange

$$Y_2 = \frac{t_{tf}}{2}\left(\frac{P_w + P_t - P_s - P_{rt} - P_{rb}}{P_c} + 1\right) \qquad D_{P2} = t_s + t_h + Y_2$$

$$M_{P2} = \frac{P_c}{2t_{tf}}\left[Y_2^{\,2} + \left(t_{tf} - Y_2\right)^2\right] + \begin{bmatrix} P_s\left(Y_2 + \frac{t_s}{2} + t_h\right) + P_{rt}\left(t_s - C_{rt} + t_h + Y_2\right) + P_{rb}\left(t_s - C_{rb} + t_h + Y_2\right) \ldots \\ + P_w\left(\frac{D}{2} + t_{tf} - Y_2\right) + P_t\left(D - Y_2 + \frac{t_{bf}}{2} + t_{tf}\right) \end{bmatrix}$$

$$Y_{2neg} = \left(\frac{t_{tf}}{2}\right)\left[1 + \frac{\left(P_w + P_c - P_{rt} - P_{rb}\right)}{P_t}\right] \qquad D_{P2neg} = t_s + t_h + Y_{2neg} \qquad D_{CP2neg} = D$$

$$M_{p2neg} = \left(\frac{P_t}{2\ t_{tf}}\right)\left[Y_{2neg}^{\ 2} + \left(t_{tf} - Y_{2neg}\right)^2\right] + \begin{bmatrix} P_{rt}\left(t_s - C_{rt} + t_h + Y_{2neg}\right) + P_{rb}\left(t_s - C_{rb} + t_h + Y_{2neg}\right) \ldots \\ + P_w\left(t_{tf} - Y_{2neg} + \frac{D}{2}\right) + P_c\left(\left|t_s + t_h - Y_{2neg} + \frac{t_{tf}}{2}\right|\right) \end{bmatrix}$$

Case III: Plastic Neutral Axis in the Concrete Deck Below the Bottom Reinforcing

$$Y_3 = t_s\left(\frac{P_c + P_w + P_t - P_{rt} - P_{rb}}{P_s}\right) \qquad D_{P3} = Y_3$$

$$M_{P3} = \frac{P_s}{2t_s}\left(Y_3^{\,2}\right) + \begin{bmatrix} P_{rt}\left(Y_3 - C_{rt}\right) + P_{rb}\left(C_{rb} - Y_3\right) + P_c\left(\frac{t_{tf}}{2} + t_s + t_h - Y_3\right) + P_w\left(\frac{D}{2} + t_{tf} + t_h + t_s - Y_3\right) \ldots \\ + P_t\left(D + \frac{t_{bf}}{2} + t_{tf} + t_s + t_h - Y_3\right) \end{bmatrix}$$

<u>Case IV:</u> Plastic Neutral Axis in the Concrete Deck in the bottom reinforcing layer $Y_4 = C_{rb}$

$$D_{P4} = Y_4$$

$$M_{P4} = \frac{P_s}{2t_s}\left(Y_4^{\,2}\right) + \left[\begin{array}{l} P_{rt}\left(Y_4 - C_{rt}\right) + P_c\left(\dfrac{t_{tf}}{2} + t_h + t_s - Y_4\right) + P_w\left(\dfrac{D}{2} + t_{tf} + t_h + t_s - Y_4\right) \ldots \\ + P_t\left(D + \dfrac{t_{bf}}{2} + t_{tf} + t_h + t_s - Y_4\right) \end{array}\right]$$

<u>Case V:</u> Plastic Neutral Axis in the Concrete Deck between top and bottom reinforcing layers

$$Y_5 = t_s\left(\frac{P_{rb} + P_c + P_w + P_t - P_{rt}}{P_s}\right) \qquad D_{P5} = Y_5$$

$$M_{P5} = \frac{P_s}{2t_s}\left(Y_5^{\,2}\right) + \left[\begin{array}{l} P_{rt}\left(Y_5 - C_{rt}\right) + P_{rb}\left[\left(t_s - C_{rb}\right) - Y_5\right] + P_c\left(\dfrac{t_{tf}}{2} + t_s + t_h - Y_5\right) + P_w\left(\dfrac{D}{2} + t_{tf} + t_h + t_s - Y_5\right) \ldots \\ + P_t\left(D + \dfrac{t_{bf}}{2} + t_{tf} + t_s + t_h - Y_5\right) \end{array}\right]$$

$$Y_{pos} = \left| \begin{array}{l} Y_1 \ \text{if } PNA_{pos} = \text{"case 1"} \\ Y_2 \ \text{if } PNA_{pos} = \text{"case 2"} \\ Y_3 \ \text{if } PNA_{pos} = \text{"case 3"} \\ Y_4 \ \text{if } PNA_{pos} = \text{"case 4"} \\ Y_5 \ \text{if } PNA_{pos} = \text{"case 5"} \end{array}\right. \qquad D_{Ppos} = \left| \begin{array}{l} D_{P1} \ \text{if } PNA_{pos} = \text{"case 1"} \\ D_{P2} \ \text{if } PNA_{pos} = \text{"case 2"} \\ D_{P3} \ \text{if } PNA_{pos} = \text{"case 3"} \\ D_{P4} \ \text{if } PNA_{pos} = \text{"case 4"} \\ D_{P5} \ \text{if } PNA_{pos} = \text{"case 5"} \end{array}\right. \qquad M_{Ppos} = \left| \begin{array}{l} M_{P1} \ \text{if } PNA_{pos} = \text{"case 1"} \\ M_{P2} \ \text{if } PNA_{pos} = \text{"case 2"} \\ M_{P3} \ \text{if } PNA_{pos} = \text{"case 3"} \\ M_{P4} \ \text{if } PNA_{pos} = \text{"case 4"} \\ M_{P5} \ \text{if } PNA_{pos} = \text{"case 5"} \end{array}\right.$$

$$Y_{pos} = 5.9 \text{ in} \qquad D_{Ppos} = 5.9 \text{ in} \qquad M_{Ppos} = 2338.1 \text{ kip ft}$$

Dp = distance from the top of slab of composite section to the neutral axis at the plastic moment (neglect positive moment reinforcement in the slab).

$$Y_{neg} = \left| \begin{array}{l} Y_{1neg} \ \text{if } PNA_{neg} = \text{"case 1"} \\ Y_{2neg} \ \text{if } PNA_{neg} = \text{"case 2"} \end{array}\right. \qquad D_{Pneg} = \left| \begin{array}{l} D_{p1neg} \ \text{if } PNA_{neg} = \text{"case 1"} \\ D_{P2neg} \ \text{if } PNA_{neg} = \text{"case 2"} \end{array}\right. \qquad M_{Pneg} = \left| \begin{array}{l} M_{p1neg} \ \text{if } PNA_{neg} = \text{"case 1"} \\ M_{p2neg} \ \text{if } PNA_{neg} = \text{"case 2"} \end{array}\right.$$

$$Y_{neg} = 9.1 \text{ in} \qquad D_{Pneg} = 17.7 \text{ in} \qquad M_{Pneg} = ▮ \text{ kip in}$$

<u>Depth of web in compression at the plastic moment [D6.3.2]:</u>

$$A_t = b_{bf}\, t_{bf} \qquad A_c = b_{tf}\, t_{tf}$$

$$D_{cppos} = \frac{D}{2}\left(\frac{F_y\, A_t - F_y\, A_c - 0.85\, f'_c\, A_{slab} - F_s\, A_r}{F_y\, A_w} + 1\right)$$

$$D_{cppos} = \left| \begin{array}{l} (0\text{in}) \ \text{if } PNA_{pos} \neq \text{"case 1"} \\ (0\text{in}) \ \text{if } \left(D_{cppos} < 0\right) \\ D_{cppos} \ \text{if } PNA_{pos} = \text{"case 1"} \end{array}\right. \qquad D_{cpneg} = \left| \begin{array}{l} D_{CP1neg} \ \text{if } PNA_{neg} = \text{"case 1"} \\ D_{CP2neg} \ \text{if } PNA_{neg} = \text{"case 2"} \end{array}\right.$$

$$D_{cppos} = 0 \text{ in} \qquad D_{cpneg} = 19.2 \text{ in}$$

Positive Flexural Compression Check:

From LRFD Article 6.10.2

Check for compactness:

Web Proportions: Web slenderness Limit:

$$\frac{D_w}{t_w} \le 150 = 1 \qquad 2\,\frac{D_{cppos}}{t_w} \le 3.76\sqrt{\frac{E_s}{F_y}} = 1$$ S 6.10.6.2.2

Therefore Section is considered compact and shall satisfy the requirements of Article 6.10.7.1.

$$M_n = \begin{vmatrix} M_{Ppos} \text{ if } D_{Ppos} \le 0.1\,D_t \\ M_{Ppos}\left(1.07 - 0.7\,\frac{D_{Ppos}}{D_t}\right) \text{ otherwise} \end{vmatrix} \qquad M_n = 2246.4 \text{ kip ft}$$

Negative Moment Capacity Check (Appendix A6):

Web Slenderness: $D_t = 37.6$ in $\quad D_{cneg} = D_t - y_{cr} - t_{bf} = 24$ in

$$\frac{2\,D_{cneg}}{t_w} < 5.7\sqrt{\frac{E_s}{F_y}} = 1$$ S Appendix A6 (for skew less than 20 deg).

Moment ignoring concrete:

$$M_{yt} = F_y\,S_{botcr} = 8745.1 \text{ kip in} \qquad M_{yc} = F_s\,S_{topcr} = 27039.2 \text{ kip in}$$

$$M_y = \min(M_{yc}, M_{yt}) = 8745.1 \text{ kip in}$$

Web Compactness:

Check for Permanent Deformations (6.10.4.2):

$$D_n = \max(t_{slab} + t_{tf} + D_w - y_c, y_c - t_{slab} - t_{tf}) = 26.7 \text{ in}$$

$$Gov = \text{if}(y_c - t_{slab} - t_{tf}, y_c - c_{rt}, D_n) = 6.9 \text{ in}$$

$$f_n = \left|M_{4\_SRV\_II\_neg}\right|\frac{Gov}{I_z} = 5.8 \text{ ksi}$$ Steel stress on side of Dn

$$\rho = \min\left(1.0, \frac{F_y}{f_n}\right) = 1 \qquad \beta = 2\,D_n\,\frac{t_w}{A_{tf}} = 4 \qquad R_h = \frac{\left[12 + \beta\left(3\rho - \rho^3\right)\right]}{(12 + 2\,\beta)} = 1$$

$$\lambda_{rw} = 5.7\sqrt{\frac{E_s}{F_y}}$$

$$\lambda_{PWdcp} = \min\left[\lambda_{rw}\,\frac{D_{cpneg}}{D_{cneg}}, \frac{\sqrt{\frac{E_s}{F_y}}}{\left(0.54\,\frac{M_{Pneg}}{R_h\,M_y} - 0.09\right)^2}\right] = 19.6$$

$$2\,\frac{D_{cpneg}}{t_w} \le \lambda_{PWdcp} = 0$$

Web Plastification: $R_{pc} = \dfrac{M_{Pneg}}{M_{yc}} = 0.7$ $\qquad R_{pt} = \dfrac{M_{Pneg}}{M_{yt}} = 2.2$

Flexure Factor: $\phi_f = 1.0$

Tensile Limit: $M_{r\_neg\_t} = \phi_f\, R_{pt}\, M_{yt} = 1619.2 \text{ kip ft}$

Compressive Limit:

Local Buckling Resistance:

$$\lambda_f = \frac{b_{bf}}{2\,t_{bf}} = 7.8 \qquad \lambda_{rf} = 0.95\sqrt{0.76\,\frac{E_s}{F_y}} = 19.9$$

$$\lambda_{pf} = 0.38\sqrt{\frac{E_s}{F_y}} = 9.2 \qquad F_{yresid} = \max\left(\min\left(0.7\,F_y, R_h\,F_y\,\frac{S_{topcr}}{S_{botcr}}, F_y\right), 0.5\,F_y\right) = 35.0 \text{ ksi}$$

$$M_{ncLB} = \left|\begin{array}{ll} \left(R_{pc}\,M_{yc}\right) & \text{if } \lambda_f \le \lambda_{pf} \\ \left[R_{pc}\,M_{yc}\left[1 - \left(1 - \dfrac{F_{yresid}\,S_{topcr}}{R_{pc}\,M_{yc}}\right)\left(\dfrac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right]\right] & \text{otherwise} \end{array}\right. \qquad M_{ncLB} = 1619.2 \text{ kip ft}$$

Lateral Torsional Buckling Resistance:

$$L_b = \frac{\left(L_{str}\right)}{2\;3} = 11.6 \text{ ft}$$

Inflection point assumed to be at 1/6 span

$$r_t = \frac{b_{bf}}{\sqrt{12\left(1 + \dfrac{1}{3}\,\dfrac{D_{cneg}\,t_w}{b_{bf}\,t_{bf}}\right)}} = 2.4 \text{ in}$$

$$L_p = 1.0\,r_t\sqrt{\frac{E_s}{F_y}} = 57.6 \text{ in} \qquad h = D + t_{bf} = 29 \text{ in} \qquad C_b = 1.0$$

$$J_b = \frac{D\,t_w^{\,3}}{3} + \frac{b_{bf}\,t_{bf}^{\,3}}{3}\left(1 - 0.63\,\frac{t_{bf}}{b_{bf}}\right) + \frac{b_{tf}\,t_{tf}^{\,3}}{3}\left(1 - 0.63\,\frac{t_{tf}}{b_{tf}}\right) = 3.3 \text{ in}^4$$

$$L_r = 1.95\,r_t\,\frac{E_s}{F_{yresid}}\sqrt{\frac{J_b}{S_{botcr}\,h}}\sqrt{1 + \sqrt{1 + 6.76\left(\frac{F_{yresid}}{E_s}\,\frac{S_{botcr}\,h}{J_b}\right)^2}} = 240 \text{ in}$$

$$F_{cr} = \frac{C_b\,\pi^2\,E_s}{\left(\dfrac{L_b}{r_t}\right)^2}\sqrt{1 + 0.078\,\frac{J_b}{S_{botcr}\,h}\left(\frac{L_b}{r_t}\right)^2} = 91.7 \text{ ksi}$$

$$M_{ncLTB} = \left|\begin{array}{ll} \left(R_{pc}\,M_{yc}\right) & \text{if } L_b \le L_p \\ \min\left[C_b\left[1 - \left(1 - \dfrac{F_{yresid}\,S_{botcr}}{R_{pc}\,M_{yc}}\right)\dfrac{\left(L_b - L_p\right)}{\left(L_r - L_p\right)}\right]R_{pc}\,M_{yc}, R_{pc}\,M_{yc}\right] & \text{if } L_p < L_b \le L_r \\ \min\left(F_{cr}\,S_{botcr}, R_{pc}\,M_{yc}\right) & \text{if } L_b > L_r \end{array}\right.$$

$M_{ncLTB} = 1124.2 \text{ kip ft}$

$M_{r\_neg\_c} = \phi_f \min(M_{ncLB}, M_{ncLTB}) = 1124.2 \text{ kip ft}$

Governing negative moment capacity: $M_{r\_neg} = \min(M_{r\_neg\_t}, M_{r\_neg\_c}) = 1124.2 \text{ kip ft}$

12. FLEXURAL STRENGTH CHECKS

Phase 1: First, check the stress due to the dead load on the steel section only. (LRFD 6.10.3 - Constructability Requirements)

Reduction factor for construction $\phi_{const} = 0.9$

Load Combination for construction $1.25\ M_{DC}$

Max Moment applied, Phase 1: (at midspan)

$$M_{int\_P1} = 1.25\, M_{DC1\_int}\left(\frac{L_{str}}{2}\right) = 484 \text{ kip ft} \quad \text{(Interior)}$$

$$M_{ext\_P1} = 1.25\, M_{DC1\_ext}\left(\frac{L_{str}}{2}\right) = 492.3 \text{ kip ft} \quad \text{(Exterior)}$$

Maximum Stress, Phase 1:

$$f_{int\_P1} = \frac{M_{int\_P1}\, y_{steel}}{I_{zsteel}} = 21.9 \text{ ksi} \quad \text{(Interior)}$$

$$f_{ext\_P1} = \frac{M_{ext\_P1}\, y_{steel}}{I_{zsteel}} = 22.3 \text{ ksi} \quad \text{(Exterior)}$$

Stress limits: $f_{P1\_max} = \phi_{const}\, F_y$

$f_{int\_P1} \le f_{P1\_max} = 1 \qquad f_{ext\_P1} \le f_{P1\_max} = 1$

Phase 2: Second, check the stress due to dead load on the composite section (with barriers added)

Reduction factor for construction $\phi_{const} = 0.9$

Load Combination for construction $1.25\ M_{DC}$

Max Moment applied, Phase 2: (at midspan) $M_{2\_STR\_I} = 655.8 \text{ kip ft}$

Capacity for positive flexure: $M_n = 2246.4 \text{ kip ft}$

Check Moment: $M_{2\_STR\_I} \le \phi_{const}\, M_n = 1$

Phase 3: Next, check the flexural stress on the stringer during transport and picking, to ensure no cracking.

Reduction factor for construction $\phi_{const} = 0.9$

Load Combination for construction $1.5\ M_{DC}$ when dynamic construction loads are involved (Section 10).

Loads and stresses on stringer during transport and picking: $M_{3\_STR\_I\_neg} = 81.5 \text{ kip ft}$

Concrete rupture stress $f_r = 0.24\sqrt{f'_c \text{ ksi}} = 0.5 \text{ ksi}$

Concrete stress during construction not to exceed:

$$f_{cmax} = \phi_{const}\, f_r = 0.5 \text{ ksi}$$

$$f_{cconst} = \frac{M_{3\_STR\_I\_neg}\, y_c}{I_z\, n} = 0.1 \text{ ksi}$$

$$f_{cconst} \le f_{cmax} = 1$$

Phase 4: Check flexural capacity under dead load and live load for fully installed continuous composite girders.

Strength I Load Combination $\phi_f = 1.0$

$M_{4\_STR\_I} = 1648 \text{ kip ft}$ $M_{4\_STR\_I\_neg} = -1009.8 \text{ kip ft}$

$M_{4\_STR\_I} \le \phi_f M_n = 1$ $|M_{4\_STR\_I\_neg}| \le M_{r\_neg} = 1$

Strength III Load Combination

$M_{4\_STR\_III} = 614.9 \text{ kip ft}$ $M_{4\_STR\_III\_neg} = -459.6 \text{ kip ft}$

$M_{4\_STR\_III} \le \phi_f M_n = 1$ $|M_{4\_STR\_III\_neg}| \le M_{r\_neg} = 1$

Strength V Load Combination

$M_{4\_STR\_V} = 1411.9 \text{ kip ft}$ $M_{4\_STR\_V\_neg} = -884 \text{ kip ft}$

$M_{4\_STR\_V} \le \phi_f M_n = 1$ $|M_{4\_STR\_V\_neg}| \le M_{r\_neg} = 1$

13. FLEXURAL SERVICE CHECKS

Check service load combinations for the fully continuous beam with live load (Phase 4):

under Service II for stress limits - $M_{4\_SRV\_II} = 1250.7 \text{ kip ft}$

$M_{4\_SRV\_II\_neg} = -769.9 \text{ kip ft}$

under Service I for cracking - $M_{4\_SRV\_I\_neg} = -675.6 \text{ kip ft}$

Ignore positive moment for Service I as there is no tension in the concrete in this case.

Service Load Stress Limits:

Top Flange: $f_{tfmax} = 0.95\, R_h\, F_y = 47.5 \text{ ksi}$

Bottom Flange: $f_{bfmax} = f_{tfmax} = 47.5 \text{ ksi}$

Concrete (Negative bending only): $f_r = 0.5 \text{ ksi}$

Service Load Stresses, Positive Moment:

$$f_{SRVII\_tf} = M_{4\_SRV\_II} \frac{(y_c - t_{slab})}{I_z} = 3.2 \text{ ksi}$$

Top Flange: $f_{SRVII\_tf} \le f_{tfmax} = 1$

Bottom Flange: $$f_{bfs2} = M_{4\_SRV\_II} \frac{(t_{slab} + t_{tf} + D_w + t_{bf} - y_c)}{I_z} = 37.4 \text{ ksi}$$

$f_l = 0$ $\quad f_{bfs2} + \frac{f_l}{2} \le f_{bfmax} = 1$ Using Service I Loading

Service Load Stresses, Negative Moment:

Top (Concrete): $$f_{con.neg} = \frac{M_{4\_SRV\_I\_neg}\, y_{cneg}}{n\, I_{zneg}} = -1.4 \text{ ksi}$$

$|f_{con.neg}| \le |f_r| = 0$

Bottom Flange: $$f_{bfs2.neg} = \frac{M_{4\_SRV\_I\_neg}\left(t_{slab} + t_{tf} + D_w + t_{bf} - y_{cneg}\right)}{I_{zneg}} = -37.8 \text{ ksi}$$

$$f_{bfs2.neg} \le f_{bfmax} = 1$$

Check LL Deflection:

$\Delta_{DT} = 1.104 \text{ in}$ from independent Analysis - includes 100% design truck (w/impact), or 25% design truck (w/impact) + 100% lane load

$DF_\delta = \frac{3}{12} = 0.3$ Deflection distribution factor = (no. lanes)/(no. stringers)

$\frac{L_{str}}{\Delta_{DT}\, DF_\delta} = 3021.7$ Equivalent X, where L/X = Deflection*Distribution Factor

$$\frac{L_{str}}{\Delta_{DT}\, DF_\delta} \ge 800 = 1$$

14. SHEAR STRENGTH

Shear Capacity based on AASHTO LRFD 6.10.9

Nominal resistance of unstiffened web:

$F_y = 50.0 \text{ ksi}$ $D_w = 28.3 \text{ in}$ $t_w = 0.5 \text{ in}$ $\phi_v = 1.0$ $k = 5$

$$V_p = 0.58\, F_y\, D_w\, t_w = 426.9 \text{ kip}$$

$$C_1 = \begin{cases} 1.0 & \text{if } \frac{D_w}{t_w} \le 1.12\sqrt{\frac{E_s\, k}{F_y}} \\ \left[\frac{1.57}{\left(\frac{D_w}{t_w}\right)^2}\left(\frac{E_s\, k}{F_y}\right)\right] & \text{if } \frac{D_w}{t_w} > 1.40\sqrt{\frac{E_s\, k}{F_y}} \\ \left(\frac{1.12}{\frac{D_w}{t_w}}\sqrt{\frac{E_s\, k}{F_y}}\right) & \text{otherwise} \end{cases}$$

$C_1 = 1$

$$V_n = C_1\, V_p = 426.9 \text{ kip}$$

$$V_u \le \phi_v\, V_n = 1$$

15. FATIGUE LIMIT STATES:

Fatigue check shall follow LRFD Article 6.10.5. Moments used for fatigue calculations were found using an outside finite element analysis program.

First check Fatigue I (infinite life); then find maximum single lane ADTT for Fatigue II if needed.

Fatigue Stress Limits:

$\Delta F_{TH\_1} = 16 \text{ ksi}$ Category B: non-coated weathering steel

$\Delta F_{TH\_2} = 12 \text{ ksi}$ Category C': Base metal at toe of transverse stiffener fillet welds

$\Delta F_{TH\_3} = 10 \text{ ksi}$ Category C: Base metal at shear connectors

Fatigue Moment Ranges at Detail Locations (from analysis):

$M_{FAT\_B} = 301 \text{ kip ft}$ $\quad M_{FAT\_CP} = 285.7 \text{ kip ft}$ $\quad M_{FAT\_C} = 207.1 \text{kip ft}$

$\gamma_{FATI} = 1.5$ $\quad \gamma_{FATII} = 0.75$

$$n_{fat} = \begin{vmatrix} 2 \ \text{ if } \ L_{str} \le 40 \text{ ft} \\ 1.0 \ \text{ otherwise} \end{vmatrix}$$

Constants to use for detail checks:

$ADTT_{SL\_INF\_B} = 860$ $\quad A_{FAT\_B} = 120\ 10^8$

$ADTT_{SL\_INF\_CP} = 660$ $\quad A_{FAT\_CP} = 44\ 10^8$

$ADTT_{SL\_INF\_C} = 1290$ $\quad A_{FAT\_C} = 44\ 10^8$

Category B Check: Stress at Bottom Flange, Fatigue I

$$f_{FATI\_B} = \frac{\gamma_{FATI}\, M_{FAT\_B}\left(t_{slab} + t_{tf} + D_w + t_{bf} - y_c\right)}{I_z} = 13.5 \text{ ksi}$$

$$f_{FATI\_B} \le \Delta F_{TH\_1} = 1$$

$$f_{FATII\_B} = \frac{\gamma_{FATII}}{\gamma_{FATI}} f_{FATI\_B} = 6.8 \text{ ksi}$$

$$ADTT_{SL\_B\_MAX} = \begin{vmatrix} \dfrac{ADTT_{SL\_INF\_B}}{n_{fat}} \ \text{ if } \ f_{FATI\_B} \le \Delta F_{TH\_1} \\ \dfrac{A_{FAT\_B}\ \text{ksi}^3}{365\ 75\ n_{fat}\ f_{FATII\_B}^{\ 3}} \ \text{ otherwise} \end{vmatrix} \qquad ADTT_{SL\_B\_MAX} = 860$$

Category C' Check: Stress at base of transverse stiffener (top of bottom flange)

$$f_{FATI\_CP} = \gamma_{FATI}\, M_{FAT\_CP} \frac{\left(t_{slab} + t_{tf} + D_w - y_c\right)}{I_z} = 12.5 \text{ ksi}$$

$$f_{FATI\_CP} \le \Delta F_{TH\_2} = 0$$

$$f_{FATII\_CP} = \frac{\gamma_{FATII}}{\gamma_{FATI}} f_{FATI\_CP} = 6.3 \text{ ksi}$$

$$ADTT_{SL\_CP\_MAX} = \begin{vmatrix} \dfrac{ADTT_{SL\_INF\_CP}}{n_{fat}} \ \text{ if } \ f_{FATI\_CP} \le \Delta F_{TH\_2} \\ \dfrac{A_{FAT\_CP}\ \text{ksi}^3}{365\ 75\ n_{fat}\ f_{FATII\_CP}^{\ 3}} \ \text{ otherwise} \end{vmatrix} \qquad ADTT_{SL\_CP\_MAX} = 656$$

Category C Check: Stress at base of shear connectors (top of top flange)

$$f_{FATI\_C} = \gamma_{FATI}\, M_{FAT\_C} \frac{\left(y_c - t_{slab}\right)}{I_z} = 0.8 \text{ ksi}$$

$$f_{FATI\_C} \le \Delta F_{TH\_3} = 1$$

$$f_{FATII\_C} = \frac{\gamma_{FATII}}{\gamma_{FATI}} f_{FATI\_C} = 0.4 \text{ ksi}$$

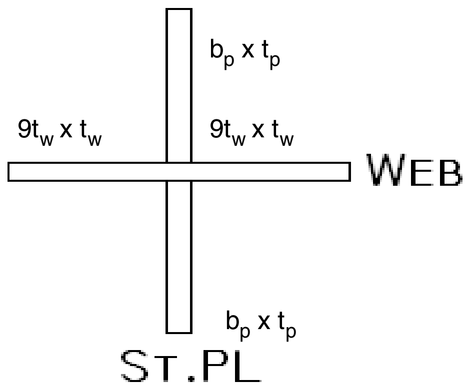
$$ADTT_{SL\_C\_MAX} = \left| \begin{array}{ll} \dfrac{ADTT_{SL\_INF\_C}}{n_{fat}} & \text{if } f_{FATI\_C} \le \Delta F_{TH\_3} \\ \dfrac{A_{FAT\_C}\ ksi^3}{365\ 75\ n_{fat}\ f_{FATII\_C}{}^3} & \text{otherwise} \end{array} \right. \qquad ADTT_{SL\_C\_MAX} = 1290$$

FATIGUE CHECK: $ADTT_{SL\_MAX} = \min\left(ADTT_{SL\_B\_MAX}, ADTT_{SL\_CP\_MAX}, ADTT_{SL\_C\_MAX}\right)$

Ensure that single lane ADTT is less than $ADTT_{SL\_MAX} = 656$

If not, then the beam requires redesign.

16. BEARING STIFFENERS



Using LRFD Article 6.10.11 for stiffeners:

$t_p = \frac{5}{8} in$ $\qquad b_p = 5in$ $\qquad \phi_b = 1.0$ $\qquad t_{p\_weld} = \left(\frac{5}{16}\right) in$

*Check min weld size

Projecting Width Slenderness Check:

$$b_p \le 0.48 t_p \sqrt{\frac{E_s}{F_y}} = 1$$

Stiffener Bearing Resistance:

$A_{pn} = 2\left(b_p - t_{p\_weld}\right) t_p$ $\qquad A_{pn} = 5.9 \text{ in}^2$

$R_{sb\_n} = 1.4 A_{pn} F_y$ $\qquad R_{sb\_n} = 410.2 \text{ kip}$

$R_{sb\_r} = \phi_b R_{sb\_n}$ $\qquad R_{sb\_r} = 410.2 \text{ kip}$

$R_{DC} = 26.721 kip$ $\qquad R_{DW} = 2.62 kip$ $\qquad R_{LL} = 53.943 kip$

$\phi_{DC\_STR\_I} = 1.25$ $\qquad \phi_{DW\_STR\_I} = 1.5$ $\qquad \phi_{LL\_STR\_I} = 1.75$

$R_u = \phi_{DC\_STR\_I} R_{DC} + \phi_{DW\_STR\_I} R_{DW} + \phi_{LL\_STR\_I} R_{LL}$ $\qquad R_u = 131.7 \text{ kip}$

$R_u \le R_{sb\_r} = 1$

Weld Check:

$throat = t_{p\_weld} \frac{\sqrt{2}}{2}$ $\qquad throat = 0.2 \text{ in}$

$L_{weld} = D_w - 2\ 3in$ $\qquad L_{weld} = 22.3 \text{ in}$

$A_{eff\_weld} = throat\ L_{weld}$ $\qquad A_{eff\_weld} = 4.9 \text{ in}^2$

$F_{exx} = 70ksi$ $\qquad \phi_{e2} = 0.8$

$R_{r\_weld} = 0.6\ \phi_{e2}\ F_{exx}$ $\qquad R_{r\_weld} = 33.6 \text{ ksi}$

$R_{u\_weld} = \frac{R_u}{4\ A_{eff\_weld}}$ $\qquad R_{u\_weld} = 6.7 \text{ ksi}$

$R_{u\_weld} \le R_{u\_weld} = 1$

Axial Resistance of Bearing Stiffeners: $\phi_c = 0.9$

$$A_{eff} = \left(2\ 9\ t_w + t_p\right) t_w + 2\ b_p\ t_p \qquad A_{eff} = 11.4\ in^2$$

$$L_{eff} = 0.75\ D_w \qquad L_{eff} = 21.2\ in$$

$$I_{xp} = \frac{2\ 9\ t_w\ t_w^{\ 3}}{12} + \frac{t_p\left(2\ b_p + t_w\right)^3}{12} \qquad I_{xp} = 60.7\ in^4$$

$$I_{yp} = \frac{t_w\left(t_p + 2\ 9\ t_w\right)^3}{12} + \frac{2b_p\ t_p^{\ 3}}{12} \qquad I_{yp} = 43.3\ in^4$$

$$r_p = \sqrt{\frac{\min\left(I_{xp}, I_{yp}\right)}{A_{eff}}} \qquad r_p = 1.9\ in$$

$Q = 1$ for bearing stiffeners $\qquad K_p = 0.75$

$$P_o = Q\ F_y\ A_{eff} = 572.1\ kip$$

$$P_e = \frac{\pi^2 E_s\ A_{eff}}{\left(K_p\ \frac{L_{eff}}{r_p}\right)^2} = 48919.6\ kip$$

$$P_n = \left| \begin{array}{l} \left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] P_o \quad \text{if} \left(\frac{P_e}{P_o}\right) \geq 0.44 \\ 0.877\ P_e \quad \text{otherwise} \end{array} \right. \qquad P_n = 569.3\ kip$$

$P_r = \phi_c\ P_n \qquad P_r = 512.4\ kip \qquad R_u \leq P_r = 1$

17. SHEAR CONNECTORS:

Shear Connector design to follow LRFD 6.10.10.

Stud Properties:

$d_s = \frac{7}{8}\ in$ Diameter $\qquad h_s = 6in$ Height of Stud $\qquad \frac{h_s}{d_s} \geq 4 = 1$

$c_s = t_{slab} - h_s \qquad c_s \geq 2in = 1$

$s_s = 3.5in$ Spacing $\qquad s_s \geq 4d_s = 1$

$n_s = 3$ Studs per row $\qquad \frac{\left[b_{tf} - s_s\left(n_s - 1\right) - d_s\right]}{2} \geq 1.0in = 1$

$$A_{sc} = \pi\left(\frac{d_s}{2}\right)^2 \qquad A_{sc} = 0.6\ in^2$$

$$F_u = 60ksi$$

Fatigue Resistance:

$Z_r = 5.5\ d_s^{\ 2}\ \frac{kip}{in^2} \qquad Z_r = 4.2\ kip \qquad Q_{slab} = A_{slab}\left(y_c - y_{slab}\right) \qquad Q_{slab} = 338.9\ in^3$

$$V_f = 47.0kip$$

$$V_{fat} = \frac{V_f\ Q_{slab}}{I_z} = 1.5\ \frac{kip}{in}$$

$p_s = \frac{n_s\ Z_r}{V_{fat}} = 8.7\ in \qquad 6\ d_s \leq p_s \leq 24in = 1$

Strength Resistance:

$\phi_{sc} = 0.85$

$f'_c = 5\ ksi$

$E_c = 33000\ 0.15^{1.5}\sqrt{f'_c\,ksi} = 4286.8\ ksi$

$Q_n = \min\left(0.5\ A_{sc}\sqrt{f'_c\ E_c}, A_{sc}\ F_u\right)$ $\qquad Q_n = 36.1\ kip$

$Q_r = \phi_{sc}\ Q_n$ $\qquad Q_r = 30.7\ kip$

$P_{simple} = \min\left(0.85\ f'_c\ b_{eff}\ t_s, F_y\ A_{steel}\right)$ $\qquad P_{simple} = 1436.2\ kip$

$P_{cont} = P_{simple} + \min\left(0.45\ f'_c\ b_{eff}\ t_s, F_y\ A_{steel}\right)$ $\qquad P_{cont} = 2282.2\ kip$

$n_{lines} = \dfrac{P_{cont}}{Q_r\ n_s}$ $\qquad n_{lines} = 24.8$

Find required stud spacing along the girder (varies as applied shear varies)

$i = 0..23$

$$x = \begin{pmatrix} 0.00 \\ 1.414 \\ 4.947 \\ 8.480 \\ 12.013 \\ 15.546 \\ 19.079 \\ 22.612 \\ 26.145 \\ 29.678 \\ 33.210 \\ 33.917 \\ 34.624 \\ 36.037 \\ 36.743 \\ 40.276 \\ 43.809 \\ 47.342 \\ 50.875 \\ 54.408 \\ 57.941 \\ 61.474 \\ 65.007 \\ 67.833 \end{pmatrix} ft \quad V_{fi} = \begin{pmatrix} 61.5 \\ 59.2 \\ 56.8 \\ 54.4 \\ 52.0 \\ 49.5 \\ 47.1 \\ 44.7 \\ 42.7 \\ 40.6 \\ 40.6 \\ 40.6 \\ 40.6 \\ 40.6 \\ 40.6 \\ 42.3 \\ 44.2 \\ 46.6 \\ 49.1 \\ 51.5 \\ 53.9 \\ 56.3 \\ 58.7 \\ 61.5 \end{pmatrix} kip$$

$V_{fati} = \dfrac{V_{fi}\ Q_{slab}}{I_z} = $ (table) $\dfrac{kip}{in}$

	0
0	1.9
1	1.8
2	1.8
3	1.7
4	1.6
5	1.5
6	1.5
7	1.4
8	1.3
9	1.3
10	1.3
11	1.3
12	1.3
13	1.3
14	1.3
15	...

$P_{max} = \dfrac{n_s\ Z_r}{V_{fati}} = $ (table) in

	0
0	6.6
1	6.9
2	7.2
3	7.5
4	7.9
5	8.3
6	8.7
7	9.1
8	9.6
9	10.1
10	10.1
11	10.1
12	10.1
13	10.1
14	10.1
15	...

$\min\left(P_{max}\right) = 6.6\ in$

$\max\left(P_{max}\right) = 10.1\ in$

18. SLAB PROPERTIES

This section details the geometric and material properties of the deck. Because the equivalent strip method is used in accordance with AASHTO LRFD Section 4, different loads are used for positive and negative bending.

Unit Weight Concrete $w_c = 150\ \text{pcf}$

Deck Thickness for Design $t_{deck} := 8.0\text{in}$ $\quad t_{deck} \geq 7\text{in} = 1$

Deck Thickness for Loads $t_d = 10.5\ \text{in}$

Rebar yield strength $F_s = 60\ \text{ksi}$ $\quad$ Strength of concrete $f'_c = 5\ \text{ksi}$

Concrete clear cover

Bottom

$c_b := 1.0\text{in}$ $\quad c_b \geq 1.0\text{in} = 1$

Top

$c_t := 2.5\text{in}$ $\quad c_t \geq 2.5\text{in} = 1$

Transverse reinforcement

Bottom Reinforcing $\phi_{tb} := \frac{6}{8}\text{in}$ $\quad$ Top Reinforcing $\phi_{tt} := \frac{5}{8}\text{in}$

Bottom Spacing $s_{tb} := 8\text{in}$ $\quad$ Top Spacing $s_{tt} := 8\text{in}$

$s_{tb} \geq 1.5\phi_{tb} \wedge 1.5\text{in} = 1$ $\quad s_{tt} \geq 1.5\phi_{tt} \wedge 1.5\text{in} = 1$

$s_{tb} \leq 1.5\ t_{deck} \wedge 18\text{in} = 1$ $\quad s_{tt} \leq 1.5\ t_{deck} \wedge 18\text{in} = 1$

$$A_{stb} := \frac{12\text{in}}{s_{tb}}\ \pi \left(\frac{\phi_{tb}}{2}\right)^2 = 0.7\ \text{in}^2 \qquad A_{stt} := \frac{12\text{in}}{s_{tt}}\ \pi \left(\frac{\phi_{tt}}{2}\right)^2 = 0.5\ \text{in}^2$$

Design depth of Bar

$$d_{tb} := t_{deck} - \left(c_b + \frac{\phi_{tb}}{2}\right) = 6.6\ \text{in} \qquad d_{tt} := t_{deck} - \left(c_t + \frac{\phi_{tt}}{2}\right) = 5.2\ \text{in}$$

Girder Spacing $spacing_{int\_max} := 4\text{ft} + 6\text{in}$

$spacing_{ext} = 4\ \text{ft}$

Equivalent Strip, +M

$$w_{posM} := \left(26 + 6.6\ \frac{spacing_{int\_max}}{\text{ft}}\right)\text{in} \qquad w_{posM} = 55.7\ \text{in}$$

Equivalent Strip, –M

$$w_{negM} := \left(48 + 3.0\ \frac{spacing_{int\_max}}{\text{ft}}\right)\text{in} \qquad w_{negM} = 61.5\ \text{in}$$

Once the strip widths are determined, the dead loads can be calculated.

19. PERMANENT LOADS

This section calculates the dead loads on the slab. These are used later for analysis to determine the design moments.

Weight of deck, +M	$w_{deck\_pos} = w_c\ t_d\ w_{posM}$	$w_{deck\_pos} = 609.2\ \text{plf}$
Weight of deck, –M	$w_{deck\_neg} = w_c\ t_d\ w_{negM}$	$w_{deck\_neg} = 672.7\ \text{plf}$
Unit weight of barrier	$w_b = 433.5\text{plf}$	
Barrier point load, +M	$P_{b\_pos} = w_b\ w_{posM}$	$P_{b\_pos} = 2.01\ \text{kip}$
Barrier point load, –M	$P_{b\_neg} = w_b\ w_{negM}$	$P_{b\_neg} = 2.22\ \text{kip}$

20. LIVE LOADS

This section calculates the live loads on the slab. These loads are analyzed in a separate program with the permanent loads to determine the design moments.

Truck wheel load	$P_{wheel} = 16\text{kip}$		
Impact Factor	$IM = 1.33$		
Multiple presence factors	$MP_1 = 1.2$	$MP_2 = 1.0$	$MP_3 = 0.85$
Wheel Loads	$P_1 = IM\ MP_1\ P_{wheel}$	$P_2 = IM\ MP_2\ P_{wheel}$	$P_3 = IM\ MP_3\ P_{wheel}$
	$P_1 = 25.54\ \text{kip}$	$P_2 = 21.3\ \text{kip}$	$P_3 = 18.09\ \text{kip}$

21. LOAD RESULTS

A separate finite element analysis program was used to analyze the deck as an 11-span continuous beam with cantilevered overhangs on either end, with supports stationed at girder locations. The dead and live loads were applied separately. The results are represented here as input values, highlighted.

Design Moments

$$M_{pos} = 38.9\text{kip ft} \qquad M_{pos\_dist} = \frac{M_{pos}}{w_{posM}} \qquad M_{pos\_dist} = 8.38\ \frac{\text{kip ft}}{\text{ft}}$$

$$M_{neg} = -21.0\text{kip ft} \qquad M_{neg\_dist} = \frac{M_{neg}}{w_{negM}} \qquad M_{neg\_dist} = -4.1\ \frac{\text{kip ft}}{\text{ft}}$$

22. FLEXURAL STRENGTH CAPACITY CHECK:

Consider a 1'-0" strip:

$$\phi_b = 0.9 \qquad b = 12\text{in}$$

$$\beta_1 = \begin{cases} 0.85 & \text{if } f'_c \le 4\text{ksi} \\ 0.85 - 0.05\left(\dfrac{f'_c}{\text{ksi}} - 4\right) & \text{otherwise} \end{cases} \qquad \beta_1 = 0.8$$

Bottom:

$$c_{tb} = \frac{A_{stb}\ F_s}{0.85\ f'_c\ \beta_1\ b} = 1\ \text{in}$$

$$a_{tb} = \beta_1\ c_{tb} = 0.8\ \text{in}$$

$$M_{ntb} = \frac{A_{stb}\ F_s}{b}\left(d_{tb} - \frac{a_{tb}}{2}\right) = 20.7\ \frac{\text{kip ft}}{\text{ft}}$$

$$M_{rtb} = \phi_b\ M_{ntb} = 18.6\ \frac{\text{kip ft}}{\text{ft}}$$

$$M_{rtb} \ge \left|M_{pos\_dist}\right| = 1$$

Top:

$$c_{tt} = \frac{A_{stt}\ F_s}{0.85\ f'_c\ \beta_1\ b} = 0.7\ \text{in}$$

$$a_{tt} = \beta_1\ c_{tt} = 0.5\ \text{in}$$

$$M_{ntt} = \frac{A_{stt}\ F_s}{\text{ft}}\left(d_{tt} - \frac{a_{tt}}{2}\right) = 11.3\ \frac{\text{kip ft}}{\text{ft}}$$

$$M_{rtt} = \phi_b\ M_{ntt} = 10.2\ \frac{\text{kip ft}}{\text{ft}}$$

$$M_{rtt} \ge \left|M_{neg\_dist}\right| = 1$$

23. LONGITUDINAL DECK REINFORCEMENT DESIGN:

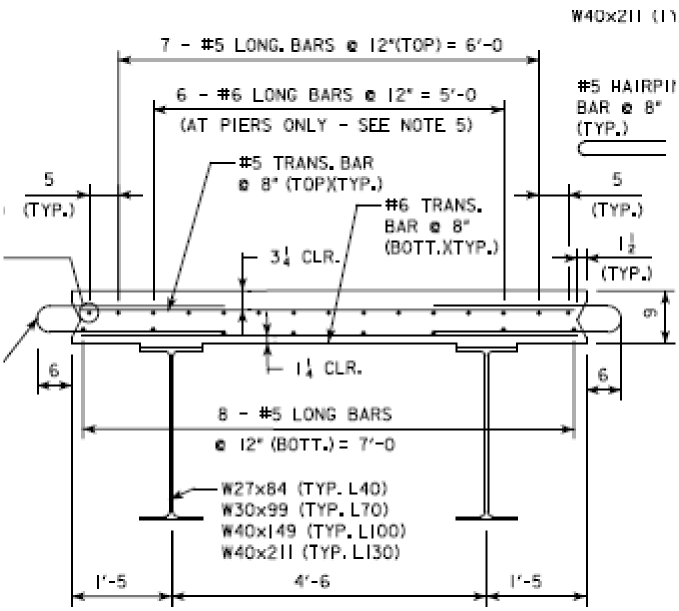
Longitudinal reinforcement $\phi_{lb} = \frac{5}{8}\text{in} \quad s_{lb} = 12\text{in}$ $\qquad \phi_{lt} = \frac{5}{8}\text{in} \quad s_{lt} = 12\text{in}$

$$A_{slb} = \frac{12\text{in}}{s_{lb}}\,\pi\left(\frac{\phi_{lb}}{2}\right)^2 = 0.3\ \text{in}^2 \qquad A_{slt} = \frac{12\text{in}}{s_{lt}}\,\pi\left(\frac{\phi_{lt}}{2}\right)^2 = 0.3\ \text{in}^2$$

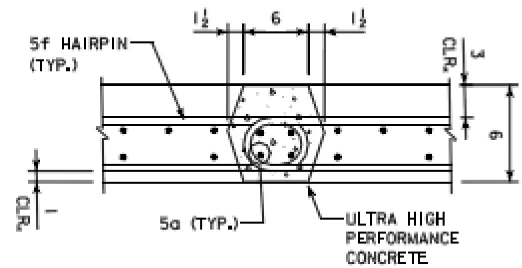
Distribution Reinforcement (AASHTO 9.7.3.2)

$$A_{\%dist} = \frac{\min\left(\dfrac{220}{\sqrt{\dfrac{spacing_{int\_max}}{ft}}}, 67\right)}{100} = 67\ \%$$

$$A_{dist} = A_{\%dist}\left(A_{stb}\right) = 0.4\ \text{in}^2 \qquad A_{slb} + A_{slt} \ge A_{dist} = 1$$



INTERIOR MODULE
REINFORCING DETAIL



LONGITUDINAL CLOSURE POUR DETAIL
(TRANSVERSE REINFORCEMENT NOT SHOWN FOR CLARITY)

24. DESIGN CHECKS

This section will conduct design checks on the reinforcing according to various sections in AASHTO LRFD.

CHECK MINIMUM REINFORCEMENT (AASHTO LRFD 5.7.3.3.2):

Modulus of Rupture $f_r = 0.37\sqrt{f'_c\,\text{ksi}} = 0.8\ \text{ksi}$ $\qquad E_c = 4286.8\ \text{ksi}$

$E_s = 29000\ \text{ksi}$

Section Modulus $S_{nc} = \frac{b\,t_{deck}^{\,2}}{6} = 128\ \text{in}^3$

$$A_{deck} = t_{deck}\,b = 96\ \text{in}^2$$

$$y_{bar\_tb} = \frac{A_{deck}\dfrac{t_{deck}}{2} + (n-1)\,A_{stb}\,d_{tb}}{A_{deck} + (n-1)\,A_{stb}} = 4.1\ \text{in}$$

$$y_{bar\_tt} = \frac{A_{deck} \frac{t_{deck}}{2} + (n-1)\ A_{stt}\ d_{tt}}{A_{deck} + (n-1)\ A_{stt}} = 4\ in$$

$$I_{tb} = \frac{b\ t_{deck}^{\ 3}}{12} + A_{deck}\left(\frac{t_{deck}}{2} - y_{bar\_tb}\right)^2 + (n-1)\ A_{stb}\left(d_{tb} - y_{bar\_tb}\right)^2 = 538.3\ in^4$$

$$I_{tt} = \frac{b\ t_{deck}^{\ 3}}{12} + A_{deck}\left(\frac{t_{deck}}{2} - y_{bar\_tt}\right)^2 + (n-1)\ A_{stt}\left(d_{tt} - y_{bar\_tt}\right)^2 = 515.8\ in^4$$

$$S_{c\_tb} = \frac{I_{tb}}{t_{deck} - y_{bar\_tb}} = 138.2\ in^3 \qquad S_{c\_tt} = \frac{I_{tt}}{t_{deck} - y_{bar\_tt}} = 130\ in^3$$

Unfactored Dead Load

$$M_{dnc\_pos\_t} = 1.25\frac{kip\ ft}{ft} \qquad M_{dnc\_neg\_t} = -0.542\frac{kip\ ft}{ft}$$

Cracking Moment

$$M_{cr\_tb} = \max\left[\frac{S_{c\_tb}\ f_r}{ft} - \left|M_{dnc\_pos\_t}\right|\left(\frac{S_{c\_tb}}{S_{nc}} - 1\right), \frac{S_{c\_tb}\ f_r}{ft}\right] = 9.5\ \frac{kip\ ft}{ft}$$ S 5.7.3.3.2

$$M_{cr\_tt} = \max\left[\frac{S_{c\_tt}\ f_r}{ft} - \left|M_{dnc\_neg\_t}\right|\left(\frac{S_{c\_tt}}{S_{nc}} - 1\right), \frac{S_{c\_tt}\ f_r}{ft}\right] = 9\ \frac{kip\ ft}{ft}$$

Minimum Factored Flexural Resistance

$$M_{r\_min\_tb} = \min\left(1.2\ M_{cr\_tb}, 1.33\ \left|M_{pos\_dist}\right|\right) = 11.1\ \frac{kip\ ft}{ft} \qquad M_{rtb} \ge M_{r\_min\_tb} = 1$$

$$M_{r\_min\_tt} = \min\left(1.2\ M_{cr\_tt}, 1.33\ \left|M_{neg\_dist}\right|\right) = 5.4\ \frac{kip\ ft}{ft} \qquad M_{rtt} \ge M_{r\_min\_tt} = 1$$

CHECK CRACK CONTROL (AASHTO LRFD 5.7.3.4):

$$\gamma_{eb} = 1.0 \qquad \gamma_{et} = 0.75$$

$$M_{SL\_pos} = 29.64 kip\ ft \qquad M_{SL\_neg} = 29.64 kip\ ft$$

$$M_{SL\_pos\_dist} = \frac{M_{SL\_pos}}{w_{posM}} = 6.4\ \frac{kip\ ft}{ft} \qquad M_{SL\_neg\_dist} = \frac{M_{SL\_neg}}{w_{negM}} = 5.8\ \frac{kip\ ft}{ft}$$

$$f_{ssb} = \frac{M_{SL\_pos\_dist}\ b\ n}{\dfrac{I_{tb}}{d_{tb} - y_{bar\_tb}}} = 2.5\ ksi \qquad f_{sst} = \frac{M_{SL\_neg\_dist}\ b\ n}{\dfrac{I_{tt}}{d_{tt} - y_{bar\_tt}}} = 1.1\ ksi$$

$$d_{cb} = c_b + \frac{\phi_{tb}}{2} = 1.4\ in \qquad d_{ct} = c_t + \frac{\phi_{tt}}{2} = 2.8\ in$$

$$\beta_{sb} = 1 + \frac{d_{cb}}{0.7\left(t_{deck} - d_{cb}\right)} = 1.3 \qquad \beta_{st} = 1 + \frac{d_{ct}}{0.7\left(t_{deck} - d_{ct}\right)} = 1.8$$

$$s_b = \frac{700\ \gamma_{eb}\ kip}{\beta_{sb}\ f_{ssb}\ in} - 2\ d_{cb} = 212.2\ in \qquad s_t = \frac{700\ \gamma_{et}\ kip}{\beta_{st}\ f_{sst}\ in} - 2\ d_{ct} = 266.5\ in$$

$$s_{tb} \le s_b = 1 \qquad s_{tt} \le s_t = 1$$

SHRINKAGE AND TEMPERATURE REINFORCING (AASHTO LRFD 5.10.8):

$$A_{st} = \begin{cases} \dfrac{1.30\ b\ t_{deck}}{2\left(b + t_{deck}\right) F_s} \dfrac{kip}{in} & \text{if } 0.11in^2 \le \dfrac{1.30\ b\ t_{deck}}{2\left(b + t_{deck}\right) F_s} \dfrac{kip}{in} \le 0.60in^2 \\ 0.11in^2 & \text{if } \dfrac{1.30\ b\ t_{deck}}{2\left(b + t_{deck}\right) F_s} \dfrac{kip}{in} < 0.11in^2 \\ 0.60in^2 & \text{if } \dfrac{1.30\ b\ t_{deck}}{2\left(b + t_{deck}\right) F_s} \dfrac{kip}{in} > 0.60in^2 \end{cases} = 0.1\ in^2$$

$A_{stb} \ge A_{st} = 1$ $\quad$ $A_{stt} \ge A_{st} = 1$

$A_{slb} \ge A_{st} = 1$ $\quad$ $A_{slt} \ge A_{st} = 1$

SHEAR RESISTANCE (AASHTO LRFD 5.8.3.3):

$\phi = 0.9$ $\quad$ $\beta = 2$ $\quad$ $\theta = 45deg$ $\quad$ $b = 1\ ft$

$$d_{v\_tb} = \max\left(0.72\ t_{deck}, d_{tb} - \frac{a_{tb}}{2}, 0.9\ d_{tb}\right) = 6.2\ in$$

$$d_{v\_tt} = \max\left(0.72\ t_{deck}, d_{tt} - \frac{a_{tt}}{2}, 0.9\ d_{tt}\right) = 5.8\ in$$

$$d_v = \min\left(d_{v\_tb}, d_{v\_tt}\right) = 5.8\ in$$

$$V_c = 0.0316\ \beta \sqrt{f'_c\ ksi}\ b\ d_v = 9.8\ kip$$

$V_s = 0kip$ Shear capacity of reinforcing steel

$V_{ps} = 0kip$ Shear capacity of prestressing steel

$$V_{ns} = \min\left(V_c + V_s + V_{ps}, 0.25\ f'_c\ b\ d_v + V_{ps}\right) = 9.8\ kip$$

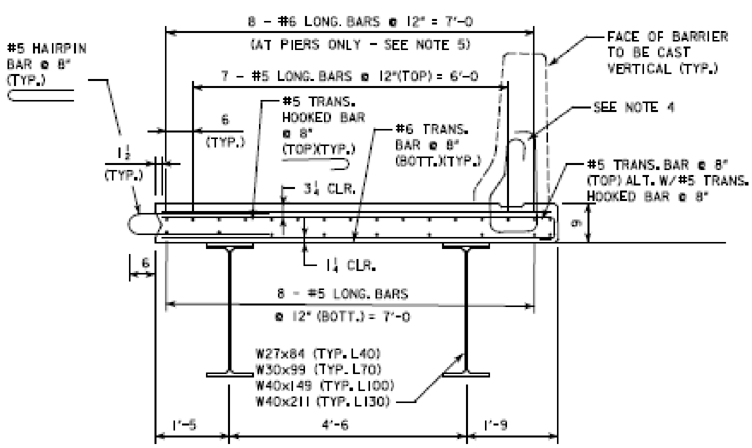
$V_r = \phi\ V_{ns} = 8.8\ kip$ Total factored resistance

$V_{us} = 8.38kip$ Total factored load $\quad$ $V_r \ge V_{us} = 1$

DEVELOPMENT AND SPLICE LENGTHS (AASHTO LRFD 5.11):

Development and splice length design follows standard calculations in AASHTO LRFD 5.11, or as dictated by the State DOT Design Manual.

25. DECK OVERHANG DESIGN (AASHTO LRFD A.13.4):



EXTERIOR MODULE
REINFORCING DETAIL
(SHEAR STUDS OMITTED FOR CLARITY)
(L70 SHOWN; L40, L100 AND L130 SIMILAR)

Deck Properties:

Deck Overhang Length $L_o = 1ft + 9in$

Parapet Properties:

Note: Parapet properties are per unit length. Compression reinforcement is ignored.

Cross Sectional Area	$A_p = 2.84ft^2$	Height of Parapet	$H_{par} = 2ft + 10in$
Parapet Weight	$W_{par} = w_c\, A_p = 426\ plf$		
Width at base	$w_{base} = 1ft + 5in$	Average width of wall	$w_{wall} = \dfrac{13in + 9.5in}{2} = 11.3\ in$
Height of top portion of parapet	$h_1 = 2ft$	Width at top of parapet	$width_1 = 9.5\ in = 9.5\ in$
Height of middle portion of parapet	$h_2 = 7in$	Width at middle transition of parapet	$width_2 = 12\ in = 12\ in$
Height of lower portion of parapet	$h_3 = 3in$	Width at base of parapet	$width_3 = 1ft + 5\ in = 17\ in$

$b_1 = width_1$ $\quad b_2 = width_2 - width_1$ $\quad b_3 = width_3 - width_2$

Parapet Center of Gravity

$$CG_p = \frac{\left(h_1 + h_2 + h_3\right)\dfrac{b_1^{\,2}}{2} + \dfrac{1}{2}h_1\, b_2\left(b_1 + \dfrac{b_2}{3}\right) \ldots + \left(h_2 + h_3\right)\left(b_2 + b_3\right)\left(b_1 + \dfrac{b_2 + b_3}{2}\right) - \dfrac{1}{2}h_2\, b_3\left(b_1 + b_2 + \dfrac{2b_3}{3}\right)}{\left(h_1 + h_2 + h_3\right) b_1 + \dfrac{1}{2}h_1\, b_2 + \left(h_2 + h_3\right)\left(b_2 + b_3\right) - \dfrac{1}{2}h_2\, b_3} = 6.3\ in$$

Parapet Reinforcement	Vertically Aligned Bars in Wall	Horizontal Bars
Rebar spacing:	$s_{pa} = 12in$	$n_{pl} = 5$
Rebar Diameter:	$\phi_{pa} = \dfrac{5}{8}in$	$\phi_{pl} = \dfrac{5}{8}in$
Rebar Area:	$A_{st\_p} = \pi\left(\dfrac{\phi_{pa}}{2}\right)^2 \dfrac{b}{s_{pa}} = 0.3\ in^2$	$A_{sl\_p} = \pi\left(\dfrac{\phi_{pl}}{2}\right)^2 = 0.3\ in^2$
Cover:	$c_{st} = 3in$	$c_{sl} = 2in + \phi_{pa} = 2.6\ in$
Effective Depth:	$d_{st} = w_{base} - c_{st} - \dfrac{\phi_{pa}}{2} = 13.7\ in$	$d_{sl} = w_{wall} - c_{sl} - \dfrac{\phi_{pl}}{2} = 8.3\ in$
Parapet Moment Resistance About Horizontal Axis:	$\phi_{ext} = 1.0$	
Depth of Equivalent Stress Block:	$a_h = \dfrac{A_{st\_p}\, F_s}{0.85\, f'_c\, b} = 0.4\ in$	S 5.7.3.1.2-4 S 5.7.3.2.3

Moment Capacity of Upper Segment of Barrier (about longitudinal axis):

Average width of section $\quad w_1 = \dfrac{width_1 + width_2}{2} = 10.7\ in$

Cover $\quad c_{st1} = 2in$

Depth $\quad d_{h1} = w_1 - c_{st1} - \dfrac{\phi_{pa}}{2} = 8.4\ in$

Factored Moment Resistance

$$\phi M_{nh1} = \frac{\phi_{ext}\, A_{st\_p}\, F_s\left(d_{h1} - \dfrac{a_h}{2}\right)}{b} = 12.7\ \frac{kip\ ft}{ft}$$

Moment Capacity of Middle Segment of Barrier (about longitudinal axis):

Average width of section $w_2 = \dfrac{width_2 + width_3}{2} = 14.5\ in$

Cover $c_{st2} = 3in$

Depth $d_{h2} = w_2 - c_{st2} - \dfrac{\phi_{pa}}{2} = 11.2\ in$

Factored Moment Resistance $\phi M_{nh2} = \dfrac{\phi_{ext}\ A_{st\_p}\ F_s \left(d_{h2} - \dfrac{a_h}{2}\right)}{b} = 16.9\ \dfrac{kip\ ft}{ft}$

Parapet Base Moment Resistance (about longitudinal axis):

Development in tension $c_{st3} = 3in$ $\qquad cover_{base\_vert} = c_{st3} + \dfrac{\phi_{pa}}{2} = 3.3\ in$

$$m_{inc\_ta} = \begin{vmatrix} 1.5 \ \text{ if } c_{st3} < 3\ \phi_{pa} \vee s_{pa} - \phi_{pa} < 6\ \phi_{pa} \\ 1.2 \ \text{ otherwise} \end{vmatrix} = 1.2$$

$$m_{dec\_ta} = \begin{vmatrix} 0.8 \ \text{ if } s_{pa} \geq 6in \\ 1.0 \ \text{ otherwise} \end{vmatrix} = 0.8$$

$$l_{db\_ta} = \begin{vmatrix} \max\left(\dfrac{1.25in\ A_{st\_p}\ \dfrac{F_s}{kip}}{\sqrt{\dfrac{f'_c}{ksi}}}, 0.4\ \phi_{pa}\ \dfrac{F_s}{ksi}\right) \ \text{ if } \phi_{pa} \leq \dfrac{11}{8}in \\ \dfrac{2.70in\ \dfrac{F_s}{ksi}}{\sqrt{\dfrac{f'_c}{ksi}}} \ \text{ if } \phi_{pa} = \dfrac{14}{8}in \\ \dfrac{3.50in\ \dfrac{F_s}{ksi}}{\sqrt{\dfrac{f'_c}{ksi}}} \ \text{ if } \phi_{pa} = \dfrac{18}{8}in \end{vmatrix}$$

$l_{dt\_ta} = l_{db\_ta}\ m_{inc\_ta}\ m_{dec\_ta} = 14.4\ in$

Hooked bar developed in tension $l_{hb\_ta} = \dfrac{38\ \phi_{pa}}{\sqrt{\dfrac{f'_c}{ksi}}} = 10.6\ in$ $\qquad m_{inc} = 1.2$

$l_{dh\_ta} = \max\left(6in, 8\ \phi_{pa}, m_{inc}\ l_{hb\_ta}\right) = 12.7\ in$

Lap splice in tension $l_{lst\_ta} = \max\left(12in, 1.3\ l_{dt\_ta}\right) = 18.7\ in$

$benefit = l_{dt\_ta} - l_{dh\_ta} = 1.7\ in$

$l_{dev\_a} = \left(7 + \dfrac{13}{16}\right)in$

$F_{dev} = \dfrac{benefit + l_{dev\_a}}{l_{dt\_ta}} = 0.7$

$F_d = 0.75$

Distance from NA to Compressive Face $c_{t\_b} = \dfrac{F_d\ A_{st\_p}\ F_s}{0.85\ f'_c\ \beta_1\ b} = 0.3\ in$ $\qquad$ S 5.7.3.1.2-4

Depth of Equivalent Stress Block $a_t = \beta_1 c_{t\_b} = 0.3 \text{ in}$ S 5.7.3.2.3

Nominal Moment Resistance $M_{nt} = F_d A_{st\_p} F_s \left(d_{st} - \frac{a_t}{2}\right) = 15.6 \text{ kip ft}$ S 5.7.3.2.2-1

Factored Moment Resistance $M_{cb} = \phi_{ext} \frac{M_{nt}}{ft} = 15.6 \frac{kip\ ft}{ft}$ S 5.7.3.2

Average Moment Capacity of Barrier (about longitudinal axis):

Factored Moment Resistance about Horizontal Axis $M_c = \frac{\phi M_{nh1} h_1 + \phi M_{nh2} h_2 + M_{cb} h_3}{h_1 + h_2 + h_3} = 13.8 \frac{kip\ ft}{ft}$

Parapet Moment Resistance (about vertical axis):

Height of Transverse Reinforcement in Parapet $y_1 = 5\text{in}$ Width of Parapet at Transverse Reinforcement $x_1 = width_3 - \frac{(y_1 - h_3) b_3}{h_2} = 15.6 \text{ in}$

$y_2 = 11.5\text{in}$ $x_2 = b_1 + b_2 - \frac{(y_2 - h_3 - h_2) b_2}{h_1} = 11.8 \text{ in}$

$y_3 = 18\text{in}$ $x_3 = b_1 + b_2 - \frac{(y_3 - h_3 - h_2) b_2}{h_1} = 11.2 \text{ in}$

$y_4 = 24.5\text{in}$ $x_4 = b_1 + b_2 - \frac{(y_4 - h_3 - h_2) b_2}{h_1} = 10.5 \text{ in}$

$y_5 = 31\text{in}$ $x_5 = b_1 + b_2 - \frac{(y_5 - h_3 - h_2) b_2}{h_1} = 9.8 \text{ in}$

Depth of Equivalent Stress Block $a = \frac{n_{pl} A_{sl\_p} F_s}{0.85 f'_c H_{par}} = 0.6 \text{ in}$

Concrete Cover in Parapet $cover_r = 2\text{in}$ $cover_{rear} = cover_r + \phi_{pa} + \frac{\phi_{pl}}{2} = 2.9 \text{ in}$

$cover_{base} = c_{st3} + \phi_{pa} + \frac{\phi_{pl}}{2} = 3.9 \text{ in}$

$cover_f = 2\text{in}$ $cover_{front} = 2\text{in} + \phi_{pa} + \frac{\phi_{pl}}{2}$

$cover_t = \frac{x_5}{2} = 4.9 \text{ in}$ $cover_{top} = cover_t = 4.9 \text{ in}$

Design depth

$d_{1i} = x_1 - cover_{base} = 11.6 \text{ in}$ $d_{1o} = x_1 - cover_{rear} = 12.6 \text{ in}$

$d_{2i} = x_2 - cover_{front} = 8.9 \text{ in}$ $d_{2o} = x_2 - cover_{rear} = 8.9 \text{ in}$

$d_{3i} = x_3 - cover_{front} = 8.2 \text{ in}$ $d_{3o} = x_3 - cover_{rear} = 8.2 \text{ in}$

$d_{4i} = x_4 - cover_{front} = 7.6 \text{ in}$ $d_{4o} = x_4 - cover_{rear} = 7.6 \text{ in}$

$d_{5i} = x_5 - cover_{top} = 4.9 \text{ in}$ $d_{5o} = x_5 - cover_{top} = 4.9 \text{ in}$

Nominal Moment Resistance - Tension on Inside Face

$\phi Mn_{1i} = \phi_{ext} A_{sl\_p} F_s \left(d_{1i} - \frac{a}{2}\right) = 208.3 \text{ kip in}$

$\phi Mn_{2i} = \phi_{ext} A_{sl\_p} F_s \left(d_{2i} - \frac{a}{2}\right) = 158.1 \text{ kip in}$

$\phi Mn_{3i} = \phi_{ext} A_{sl\_p} F_s \left(d_{3i} - \frac{a}{2}\right) = 145.6 \text{ kip in}$

$\phi Mn_{4i} = \phi_{ext} A_{sl\_p} F_s \left(d_{4i} - \frac{a}{2}\right) = 133.2 \text{ kip in}$

$\phi Mn_{5i} = \phi_{ext} A_{sl\_p} F_s \left(d_{5i} - \frac{a}{2}\right) = 84.5 \text{ kip in}$

$$M_{wi} = \phi Mn_{1i} + \phi Mn_{2i} + \phi Mn_{3i} + \phi Mn_{4i} + \phi Mn_{5i} = 60.8 \text{ kip ft}$$

Nominal Moment Resistance - Tension on Outside Face

$$\phi Mn_{1o} = \phi_{ext} A_{sl\_p} F_s \left(d_{1o} - \frac{a}{2}\right) = 18.9 \text{ kip ft}$$

$$\phi Mn_{2o} = \phi_{ext} A_{sl\_p} F_s \left(d_{2o} - \frac{a}{2}\right) = 13.2 \text{ kip ft}$$

$$\phi Mn_{3o} = \phi_{ext} A_{sl\_p} F_s \left(d_{3o} - \frac{a}{2}\right) = 12.1 \text{ kip ft}$$

$$\phi Mn_{4o} = \phi_{ext} A_{sl\_p} F_s \left(d_{4o} - \frac{a}{2}\right) = 11.1 \text{ kip ft}$$

$$\phi Mn_{5o} = \phi_{ext} A_{sl\_p} F_s \left(d_{5o} - \frac{a}{2}\right) = 7 \text{ kip ft}$$

$$M_{wo} = \phi Mn_{1o} + \phi Mn_{2o} + \phi Mn_{3o} + \phi Mn_{4o} + \phi Mn_{5o} = 62.3 \text{ kip ft}$$

Vertical Nominal Moment Resistance of Parapet

$$M_w = \frac{2\ M_{wi} + M_{wo}}{3} = 61.3 \text{ kip ft}$$

Parapet Design Factors:

Crash Level

$$CL = \text{"TL-4"}$$

Transverse Design Force

$$F_t = \left| \begin{array}{l} 13.5\text{kip} \ \text{ if } CL = \text{"TL-1"} \\ 27.0\text{kip} \ \text{ if } CL = \text{"TL-2"} \\ 54.0\text{kip} \ \text{ if } CL = \text{"TL-3"} \\ 54.0\text{kip} \ \text{ if } CL = \text{"TL-4"} \\ 124.0\text{kip} \ \text{ if } CL = \text{"TL-5"} \\ 175.0\text{kip} \ \text{ otherwise} \end{array} \right. = 54 \text{ kip} \qquad L_t = \left| \begin{array}{l} 4.0\text{ft} \ \text{ if } CL = \text{"TL-1"} \\ 4.0\text{ft} \ \text{ if } CL = \text{"TL-2"} \\ 4.0\text{ft} \ \text{ if } CL = \text{"TL-3"} \\ 3.5\text{ft} \ \text{ if } CL = \text{"TL-4"} \\ 8.0\text{ft} \ \text{ if } CL = \text{"TL-5"} \\ 8.0\text{ft} \ \text{ otherwise} \end{array} \right. = 3.5 \text{ ft}$$

Longitudinal Design Force

$$F_l = \left| \begin{array}{l} 4.5\text{kip} \ \text{ if } CL = \text{"TL-1"} \\ 9.0\text{kip} \ \text{ if } CL = \text{"TL-2"} \\ 18.0\text{kip} \ \text{ if } CL = \text{"TL-3"} \\ 18.0\text{kip} \ \text{ if } CL = \text{"TL-4"} \\ 41.0\text{kip} \ \text{ if } CL = \text{"TL-5"} \\ 58.0\text{kip} \ \text{ otherwise} \end{array} \right. = 18 \text{ kip} \qquad L_l = \left| \begin{array}{l} 4.0\text{ft} \ \text{ if } CL = \text{"TL-1"} \\ 4.0\text{ft} \ \text{ if } CL = \text{"TL-2"} \\ 4.0\text{ft} \ \text{ if } CL = \text{"TL-3"} \\ 3.5\text{ft} \ \text{ if } CL = \text{"TL-4"} \\ 8.0\text{ft} \ \text{ if } CL = \text{"TL-5"} \\ 8.0\text{ft} \ \text{ otherwise} \end{array} \right. = 3.5 \text{ ft}$$

Vertical Design Force (Down)

$$F_v = \left| \begin{array}{l} 4.5\text{kip} \ \text{ if } CL = \text{"TL-1"} \\ 4.5\text{kip} \ \text{ if } CL = \text{"TL-2"} \\ 4.5\text{kip} \ \text{ if } CL = \text{"TL-3"} \\ 18.0\text{kip} \ \text{ if } CL = \text{"TL-4"} \\ 80.0\text{kip} \ \text{ if } CL = \text{"TL-5"} \\ 80.0\text{kip} \ \text{ otherwise} \end{array} \right. = 18 \text{ kip} \qquad L_v = \left| \begin{array}{l} 18.0\text{ft} \ \text{ if } CL = \text{"TL-1"} \\ 18.0\text{ft} \ \text{ if } CL = \text{"TL-2"} \\ 18.0\text{ft} \ \text{ if } CL = \text{"TL-3"} \\ 18.0\text{ft} \ \text{ if } CL = \text{"TL-4"} \\ 40.0\text{ft} \ \text{ if } CL = \text{"TL-5"} \\ 40.0\text{ft} \ \text{ otherwise} \end{array} \right. = 18 \text{ ft}$$

Critical Length of Yield Line Failure Pattern:

$$M_b = 0\text{kip ft}$$

$$L_c = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{8\ H_{par}\left(M_b + M_w\right)}{M_c}} = 11.9\ \text{ft}$$ S A13.3.1-2

$$R_w = \frac{2}{2\ L_c - L_t}\left(8\ M_b + 8\ M_w + \frac{M_c\ L_c^{\ 2}}{H_{par}}\right) = 116.2\ \text{kip}$$ S A13.3.1-1

$$T = \frac{R_w\ b}{L_c + 2\ H_{par}} = 6.6\ \text{kip}$$ S A13.4.2-1

The parapet design must consider three design cases. Design Case 1 is for longitudinal and transverse collision loads under Extreme Event Load Combination II. Design Case 2 represents vertical collision loads under Extreme Event Load Combination II; however, this case does not govern for decks with concrete parapets or barriers. Design Case 3 is for dead and live load under Strength Load Combination I; however, the parapet will not carry wheel loads and therefore this case does not govern. Design Case 1 is the only case that requires a check.

Design Case 1: Longitudinal and Transverse Collision Loads, Extreme Event Load Combination II

DC - 1A: Inside face of parapet

$\phi_{ext} = 1$ $\gamma_{DC} = 1.0$ $\gamma_{DW} = 1.0$ $\gamma_{LL} = 0.5$ S A13.4.1 S Table 3.4.1-1

$l_{lip} = 2\text{in}$ $w_{base} = 17\ \text{in}$

$A_{deck\_1A} = t_{deck}\left(l_{lip} + w_{base}\right) = 152\ \text{in}^2$ $A_p = 2.8\ \text{ft}^2$

$W_{deck\_1A} = w_c\ A_{deck\_1A} = 0.2\ \text{klf}$ $W_{par} = 0.4\ \text{klf}$

$$M_{DCdeck\_1A} = \gamma_{DC}\ W_{deck\_1A}\ \frac{l_{lip} + w_{base}}{2} = 0.1\ \frac{\text{kip ft}}{\text{ft}}$$

$$M_{DCpar\_1A} = \gamma_{DC}\ W_{par}\left(l_{lip} + CG_p\right) = 0.3\ \frac{\text{kip ft}}{\text{ft}}$$

$$M_{total\_1A} = M_{cb} + M_{DCdeck\_1A} + M_{DCpar\_1A} = 16\ \frac{\text{kip ft}}{\text{ft}}$$

$\phi_{tt\_add} = \frac{5}{8}\text{in}$ $s_{tt\_add} = 8\text{in}$

$$A_{stt\_p} = \frac{12\text{in}}{s_{tt}}\ \pi\left(\frac{\phi_{tt}}{2}\right)^2 + \frac{12\text{in}}{s_{tt\_add}}\ \pi\left(\frac{\phi_{tt\_add}}{2}\right)^2 = 0.9\ \text{in}^2$$

$$d_{tt\_add} = t_{deck} - \left(c_t + \frac{\phi_{tt\_add}}{2}\right) = 5.2\ \text{in}$$

$c_{tt\_p} = \frac{A_{stt\_p}\ F_s}{0.85\ f'_c\ \beta_1\ b} = 1.4\ \text{in}$ $a_{tt\_p} = \beta_1\ c_{tt\_p} = 1.1\ \text{in}$

$$M_{ntt\_p} = \frac{A_{stt\_p}\ F_s}{\text{ft}}\left(d_{tt\_add} - \frac{a_{tt\_p}}{2}\right) = 21.4\ \frac{\text{kip ft}}{\text{ft}}$$

$M_{rtt\_p} = \phi_b\ M_{ntt\_p} = 19.2\ \frac{\text{kip ft}}{\text{ft}}$ $M_{rtt\_p} \geq M_{total\_1A} = 1$

$A_{sT} = A_{stt} + A_{stb} = 1.1\ \text{in}^2$

$\phi P_n = \phi_{ext}\ A_{sT}\ F_s = 67.4\ \text{kip}$ $\phi P_n \geq T = 1$

$M_{u\_1A} = M_{rtt\_p}\left(1 - \frac{T}{\phi P_n}\right) = 17.4\ \frac{\text{kip ft}}{\text{ft}}$ $M_{u\_1A} \geq M_{total\_1A} = 1$

DC - 1B: Design Section in Overhang

Notes:

- Distribution length is assumed to increase based on a 30 degree angle from the face of parapet.
- Moment of collision loads is distributed over the length Lc + 30 degree spread from face of parapet to location of overhang design section.
- Axial force of collision loads is distributed over the length Lc + 2Hpar + 30 degree spread from face of parapet to location of overhang design section.
- Future wearing surface is neglected as contribution is negligible.

$$A_{deck\_1B} = t_{deck}\, L_o = 168\ in^2 \qquad A_p = 2.8\ ft^2$$

$$W_{deck\_1B} = w_c\, A_{deck\_1B} = 0.2\ klf \qquad W_{par} = 0.4\ klf$$

$$M_{DCdeck\_1B} = \gamma_{DC}\, W_{deck\_1B}\, \frac{L_o}{2} = 0.2\ \frac{kip\ ft}{ft}$$

$$M_{DCpar\_1B} = \gamma_{DC}\, W_{par}\left(L_o - l_{lip} - CG_p\right) = 0.5\ \frac{kip\ ft}{ft}$$

$$L_{spread\_B} = L_o - l_{lip} - width_3 = 2\ in \qquad spread = 30deg$$

$$w_{spread\_B} = L_{spread\_B}\, \tan(spread) = 1.2\ in$$

$$M_{cb\_1B} = \frac{M_{cb}\, L_c}{L_c + 2\, w_{spread\_B}} = 15.3\ \frac{kip\ ft}{ft}$$

$$M_{total\_1B} = M_{cb\_1B} + M_{DCdeck\_1B} + M_{DCpar\_1B} = 15.9\ \frac{kip\ ft}{ft}$$

$$M_{rtt\_p} = 19.2\ \frac{kip\ ft}{ft} \qquad M_{rtt\_p} \geq M_{total\_1B} = 1$$

$$\phi P_n = 67.4\ kip$$

$$P_u = \frac{T\left(L_c + 2\, H_{par}\right)}{L_c + 2\, H_{par} + 2\, w_{spread\_B}} = 6.5\ kip \qquad \phi P_n \geq P_u = 1$$

$$M_{u\_1B} = M_{rtt\_p}\left(1 - \frac{P_u}{\phi P_n}\right) = 17.4\ \frac{kip\ ft}{ft} \qquad M_{u\_1B} \geq M_{total\_1B} = 1$$

DC - 1C: Design Section in First Span

Assumptions:

- Moment of collision loads is distributed over the length Lc + 30 degree spread from face of parapet to location of overhang design section.
- Axial force of collision loads is distributed over the length Lc + 2Hpar + 30 degree spread from face of parapet to location of overhang design section.
- Future wearing surface is neglected as contribution is negligible.

$$M_{par\_G1} = M_{DCpar\_1B} = 0.5\ \frac{kip\ ft}{ft}$$

$$M_{par\_G2} = -0.137\ \frac{kip\ ft}{ft} \qquad \text{(From model output)}$$

$$M_1 = M_{cb} = 15.6\ \frac{kip\ ft}{ft}$$

$$M_2 = M_1\, \frac{M_{par\_G2}}{M_{par\_G1}} = -4.7\ \frac{kip\ ft}{ft}$$

$$b_f = 10.5in$$

$$M_{c\_M2M1} = M_1 + \frac{\frac{1}{4} b_f \left(-M_1 + M_2\right)}{spacing_{int\_max}} = 14.6 \frac{kip\ ft}{ft}$$

$$L_{spread\_C} = L_o - l_{lip} - w_{base} + \frac{b_f}{4} = 4.6\ in$$

$$w_{spread\_C} = L_{spread\_C} \tan(spread) = 2.7\ in$$

$$M_{cb\_1C} = \frac{M_{c\_M2M1} L_c}{L_c + 2\ w_{spread\_C}} = 14.1 \frac{kip\ ft}{ft}$$

$$M_{total\_1C} = M_{cb\_1C} + M_{DCdeck\_1B} + M_{DCpar\_1B} = 14.7 \frac{kip\ ft}{ft}$$

$$M_{rtt\_p} = 19.2 \frac{kip\ ft}{ft} \qquad M_{rtt\_p} \geq M_{total\_1C} = 1$$

$$\phi P_n = 67.4\ kip$$

$$P_{uC} = \frac{T\left(L_c + 2\ H_{par}\right)}{L_c + 2\ H_{par} + 2\ w_{spread\_C}} = 6.4\ kip \qquad \phi P_n \geq P_{uC} = 1$$

$$M_{u\_1C} = M_{rtt\_p}\left(1 - \frac{P_u}{\phi P_n}\right) = 17.4 \frac{kip\ ft}{ft} \qquad M_{u\_1B} \geq M_{total\_1B} = 1$$

Compute Overhang Reinforcement Cut-off Length Requirement

Maximum crash load moment at theoretical cut-off point:

$$M_{c\_max} = M_{rtt} = 10.2 \frac{kip\ ft}{ft}$$

$$L_{Mc\_max} = \frac{M_2 - M_{rtt}}{M_2 - M_1} spacing_{int\_max} = 3.3\ ft$$

$$L_{spread\_D} = L_o - l_{lip} - w_{base} + L_{Mc\_max} = 41.6\ in$$

$$w_{spread\_D} = L_{spread\_D} \tan(spread) = 24\ in$$

$$M_{cb\_max} = \frac{M_{c\_max} L_c}{L_c + 2\ w_{spread\_D}} = 7.6 \frac{kip\ ft}{ft}$$

$$extension = \max\left(d_{tt\_add}, 12\ \phi_{tt\_add}, 0.0625\ spacing_{int\_max}\right) = 7.5\ in$$

$$cutt\_off = L_{Mc\_max} + extension = 47.1\ in$$

$$A_{tt\_add} = \pi \left(\frac{\phi_{tt\_add}}{2}\right)^2 = 0.3\ in^2$$

$$m_{thick\_tt\_add} = \begin{vmatrix} 1.4 & \text{if } t_{deck} - c_t \geq 12in \\ 1.0 & \text{otherwise} \end{vmatrix} = 1$$

$$m_{epoxy\_tt\_add} = \begin{vmatrix} 1.5 & \text{if } c_t < 3\ \phi_{tt\_add} \vee \frac{s_{tt\_add}}{2} - \phi_{tt\_add} < 6\ \phi_{tt\_add} \\ 1.2 & \text{otherwise} \end{vmatrix} = 1.5$$

$$m_{inc\_tt\_add} = \min(m_{thick\_tt\_add}\ m_{epoxy\_tt\_add}, 1.7) = 1.5$$

$$m_{dec\_tt\_add} = \begin{cases} 0.8 & \text{if } \dfrac{s_{tt\_add}}{2} \geq 6in \\ 1.0 & \text{otherwise} \end{cases} = 1$$

$$l_{db\_tt\_add} = \begin{cases} \max\left(\dfrac{1.25in\ A_{tt\_add} \dfrac{F_s}{kip}}{\sqrt{\dfrac{f'_c}{ksi}}}, 0.4\ \phi_{tt\_add} \dfrac{F_s}{ksi} \right) & \text{if } \phi_{tt\_add} \leq \dfrac{11}{8} in \\ \dfrac{2.70in \dfrac{F_s}{ksi}}{\sqrt{\dfrac{f'_c}{ksi}}} & \text{if } \phi_{tt\_add} = \dfrac{14}{8} in \\ \dfrac{3.50in \dfrac{F_s}{ksi}}{\sqrt{\dfrac{f'_c}{ksi}}} & \text{if } \phi_{tt\_add} = \dfrac{18}{8} in \end{cases}$$

$l_{db\_tt\_add} = 15\ in$

$$l_{dt\_tt\_add} = l_{db\_tt\_add}\ m_{inc\_tt\_add}\ m_{dec\_tt\_add} = 22.5\ in$$

$Cuttoff_{point} = L_{Mc\_max} + l_{dt\_tt\_add} - spacing_{int\_max} = 8.1\ in$ extension past second interior girder

Check for Cracking in Overhang under Service Limit State:

Does not govern - no live load on overhang.

26. COMPRESSION SPLICE:

See sheet S7 from drawing set titled: "*STANDARD CONCEPTS FOR ABC MODULAR SYSTEMS*"

Ensure compression splice and connection can handle the compressive force in the force couple due to the negative moment over the pier.

Live load negative moment over pier: $M_{LLPier} = 541.8\ kip\ ft$

Factored LL moment: $M_{UPier} = 1.75\ M_{LLPier} = 948.1\ kip\ ft$

The compression splice is comprised of a splice plate on the underside of the bottom flange, and built-up angles on either side of the web, connecting to the bottom flange as well.

Calculate Bottom Flange Stress:

Composite moment of inertia: $I_z = 10959.8\ in^4$

Distance to center of bottom flange from composite section centroid: $y_{bf} = \dfrac{t_{bf}}{2} + D_w + t_{tf} + t_{slab} - y_c = 27\ in$

Stress in bottom flange: $f_{bf} = M_{UPier} \dfrac{y_{bf}}{I_z} = 28\ ksi$

Calculate Bottom Flange Force:

Design Stress: $F_{bf} = \max\left(\dfrac{f_{bf} + F_y}{2}, 0.75\ F_y\right) = 39\ ksi$

Effective Flange Area: $A_{ef} = b_{bf}\ t_{bf} = 7\ in^2$

Force in Flange: $C_{nf} = F_{bf} A_{ef} = 273.2 \text{ kip}$

Calculate Bottom Flange Stress, Ignoring Concrete:

Moment of inertia: $I_{zsteel} = 3923.8 \text{ in}^4$

Distance to center of bottom flange: $y_{bfsteel} = \dfrac{t_{bf}}{2} + D_w + t_{tf} - y_{steel} = 14.5 \text{ in}$

Stress in bottom flange: $f_{bfsteel} = M_{UPier} \dfrac{y_{bfsteel}}{I_{zsteel}} = 42 \text{ ksi}$

Bottom Flange Force for design:

Design Stress: $F_{cf} = \max\left(\dfrac{f_{bfsteel} + F_y}{2}, 0.75\, F_y\right) = 46 \text{ ksi}$

Design Force: $C_n = \max(F_{bf}, F_{cf})\, A_{ef} = 322.1 \text{ kip}$

Compression Splice Plate Dimensions:

Bottom Splice Plate: $b_{bsp} = b_{bf} = 10.4 \text{ in}$ $\quad t_{bsp} = 0.75\text{in}$ $\quad A_{bsp} = b_{bsp}\, t_{bsp} = 7.8 \text{ in}^2$

Built-Up Angle Splice Plate Horizontal Leg: $b_{asph} = 4.25\text{in}$ $\quad t_{asph} = 0.75\text{in}$ $\quad A_{asph} = 2\, b_{asph}\, t_{asph} = 6.4 \text{ in}^2$

Built-Up Angle Splice Plate Vertical Leg: $b_{aspv} = 7.75\text{in}$ $\quad t_{aspv} = 0.75\text{in}$ $\quad A_{aspv} = 2\, b_{aspv}\, t_{aspv} = 11.6 \text{ in}^2$

Total Area: $A_{csp} = A_{bsp} + A_{asph} + A_{aspv} = 25.8 \text{ in}^2$

Average Stress: $f_{cs} = \dfrac{C_n}{A_{csp}} = 12.5 \text{ ksi}$

Proportion Load into each plate based on area:

$$C_{bsp} = \frac{C_n\, A_{bsp}}{A_{csp}} = 97.7 \text{ kip} \qquad C_{asph} = \frac{C_n\, A_{asph}}{A_{csp}} = 79.5 \text{ kip} \qquad C_{aspv} = \frac{C_n\, A_{aspv}}{A_{csp}} = 144.9 \text{ kip}$$

Check Plates Compression Capacity:

Bottom Splice Plate: $k_{cps} = 0.75$ for bolted connection

$l_{cps} = 9\text{in}$

$$r_{bsp} = \sqrt{\frac{\min\left(\dfrac{b_{bsp}\, t_{bsp}^{\,3}}{12}, \dfrac{t_{bsp}\, b_{bsp}^{\,3}}{12}\right)}{A_{bsp}}} = 0.2 \text{ in}$$

$$P_{ebsp} = \frac{\pi^2\, E_s\, A_{bsp}}{\left(\dfrac{k_{cps}\, l_{cps}}{r_{bsp}}\right)^2} = 2307.9 \text{ kip}$$

$$Q_{bsp} = \begin{cases} 1.0 \ \text{if} \ \dfrac{b_{bsp}}{t_{bsp}} \le 0.45\sqrt{\dfrac{E_s}{F_y}} \\ \left[1.34 - 0.76\left(\dfrac{b_{bsp}}{t_{bsp}}\right)\sqrt{\dfrac{F_y}{E_s}}\right] \ \text{if} \ 0.45\sqrt{\dfrac{E_s}{F_y}} \le \dfrac{b_{bsp}}{t_{bsp}} \le 0.91\sqrt{\dfrac{E_s}{F_y}} \\ \dfrac{0.53\,E_s}{F_y\left(\dfrac{b_{bsp}}{t_{bsp}}\right)^2} \ \text{otherwise} \end{cases} = 0.9$$

$$P_{obsp} = Q_{bsp}\,F_y\,A_{bsp} = 352.8\ \text{kip}$$

$$P_{nbsp} = \begin{cases} \left[\left[0.658^{\left(\frac{P_{obsp}}{P_{ebsp}}\right)}\right]P_{obsp}\right] \ \text{if} \ \dfrac{P_{ebsp}}{P_{obsp}} \ge 0.44 \\ (0.877\,P_{ebsp}) \ \text{otherwise} \end{cases} = 330.9\ \text{kip}$$

$$P_{nbsp\_allow} = 0.9\,P_{nbsp} = 297.8\ \text{kip}$$

$$\text{Check} = \begin{cases} \text{"NG"} \ \text{if} \ C_{bsp} \ge P_{nbsp\_allow} \\ \text{"OK"} \ \text{if} \ P_{nbsp\_allow} \ge C_{bsp} \end{cases} = \text{"OK"}$$

Horizontal Angle Leg: $k_{cps} = 0.75$ for bolted connection

$$l_{cps} = 9\ \text{in}$$

$$r_{asph} = \sqrt{\frac{\min\left(\dfrac{b_{asph}\,t_{asph}^{\,3}}{12}, \dfrac{t_{asph}\,b_{asph}^{\,3}}{12}\right)}{A_{asph}}} = 0.153\ \text{in}$$

$$P_{easph} = \frac{\pi^2\,E_s\,A_{asph}}{\left(\dfrac{k_{cps}\,l_{cps}}{r_{asph}}\right)^2} = 938.6\ \text{kip}$$

$$Q_{asph} = \begin{cases} 1.0 \ \text{if} \ \dfrac{b_{asph}}{t_{asph}} \le 0.45\sqrt{\dfrac{E_s}{F_y}} \\ \left[1.34 - 0.76\left(\dfrac{b_{asph}}{t_{asph}}\right)\sqrt{\dfrac{F_y}{E_s}}\right] \ \text{if} \ 0.45\sqrt{\dfrac{E_s}{F_y}} \le \dfrac{b_{asph}}{t_{asph}} \le 0.91\sqrt{\dfrac{E_s}{F_y}} \\ \dfrac{0.53\,E_s}{F_y\left(\dfrac{b_{asph}}{t_{asph}}\right)^2} \ \text{otherwise} \end{cases} = 1$$

$$P_{oasph} = Q_{asph}\,F_y\,A_{asph} = 318.7\ \text{kip}$$

$$P_{nasph} = \begin{vmatrix} \left[\left[0.658^{\left(\frac{P_{oasph}}{P_{easph}}\right)}\right] P_{oasph}\right] & \text{if } \frac{P_{easph}}{P_{oasph}} \geq 0.44 \\ \left(0.877\ P_{easph}\right) & \text{otherwise} \end{vmatrix} = 276.5 \text{ kip}$$

$$P_{nasph\_allow} = 0.9\ P_{nasph} = 248.9 \text{ kip} \qquad Check2 = \begin{vmatrix} \text{"NG"} & \text{if } C_{asph} \geq P_{nasph\_allow} \\ \text{"OK"} & \text{if } P_{nasph\_allow} \geq C_{asph} \end{vmatrix} = \text{"OK"}$$

Vertical Angle Leg: $k_{cps} = 0.75$ for bolted connection

$l_{cps} = 9$ in

$$r_{aspv} = \sqrt{\frac{\min\left(\frac{b_{aspv}\ t_{aspv}^{3}}{12}, \frac{t_{aspv}\ b_{aspv}^{3}}{12}\right)}{A_{aspv}}} = 0.153 \text{ in}$$

$$P_{easpv} = \frac{\pi^2\ E_s\ A_{aspv}}{\left(\frac{k_{cps}\ l_{cps}}{r_{aspv}}\right)^2} = 1711.6 \text{ kip}$$

$$Q_{aspv} = \begin{vmatrix} 1.0 \text{ if } \frac{b_{aspv}}{t_{aspv}} \leq 0.45\sqrt{\frac{E_s}{F_y}} \\ \left[1.34 - 0.76\left(\frac{b_{aspv}}{t_{aspv}}\right)\sqrt{\frac{F_y}{E_s}}\right] \text{ if } 0.45\sqrt{\frac{E_s}{F_y}} \leq \frac{b_{aspv}}{t_{aspv}} \leq 0.91\sqrt{\frac{E_s}{F_y}} \\ \frac{0.53\ E_s}{F_y\left(\frac{b_{aspv}}{t_{aspv}}\right)^2} \text{ otherwise} \end{vmatrix} = 1$$

$$P_{oaspv} = Q_{aspv}\ F_y\ A_{aspv} = 581.2 \text{ kip}$$

$$P_{naspv} = \begin{vmatrix} \left[\left[0.658^{\left(\frac{P_{oaspv}}{P_{easpv}}\right)}\right] P_{oaspv}\right] & \text{if } \frac{P_{easpv}}{P_{oaspv}} \geq 0.44 \\ \left(0.877\ P_{easpv}\right) & \text{otherwise} \end{vmatrix} = 504.2 \text{ kip}$$

$$P_{naspv\_allow} = 0.9\ P_{naspv} = 453.8 \text{ kip} \qquad Check3 = \begin{vmatrix} \text{"NG"} & \text{if } C_{aspv} \geq P_{naspv\_allow} \\ \text{"OK"} & \text{if } P_{naspv\_allow} \geq C_{aspv} \end{vmatrix} = \text{"OK'}$$

Additional Checks: Design Bolted Connections of the splice plates to the girders, checking for shear, bearing, and slip critical connections.

27. CLOSURE POUR DESIGN:

See sheet S2 from drawing set titled: "*STANDARD CONCEPTS FOR ABC MODULAR SYSTEMS*" for closure pour drawing.

Check the closure pour according to the negative bending capacity of the section.

Use the minimum reinforcing properties for design, to be conservative.

$A_{steel} = 28.7\ in^2$ $\quad A_{rt} = 1.8\ in^2$ $\quad A_{rb} = 2.6\ in^2$

$cg_{steel} = t_{slab} + y_{steel} = 22.8\ in$ $\quad cg_{rt} = 3in + 1.5\,\frac{5}{8}in = 3.9\ in$ $\quad cg_{rb} = t_{slab} - \left(1in + 1.5\,\frac{5}{8}in\right) = 6.1\ in$

Overall CG: $A_{neg} = A_{steel} + A_{rt} + A_{rb} = 33.1\ in^2$ $\quad cg_{neg} = \frac{A_{steel}\,cg_{steel} + A_{rt}\,cg_{rt} + A_{rb}\,cg_{rb}}{A_{neg}} = 20.5\ in$

Moment of Inertia: $I_{zstl} = 3990in^4$

$$I_{neg} = I_{zstl} + A_{steel}\left(cg_{steel} - cg_{neg}\right)^2 + A_{rt}\left(cg_{rt} - cg_{neg}\right)^2 + A_{rb}\left(cg_{rb} - cg_{neg}\right)^2 = 5183.7\ in^4$$

Section Moduli: $S_{top\_neg} = \frac{I_{neg}}{cg_{neg} - cg_{rt}} = 313.4\ in^3$ $\quad r_{neg} = \sqrt{\frac{I_{neg}}{A_{neg}}} = 12.5\ in$

$$S_{bot\_neg} = \frac{I_{neg}}{\left(t_{slab} + t_{tf} + D_w + t_{bf} - cg_{neg}\right)} = 301.9\ in^3$$

Concrete Properties: $f'_c = 5\ ksi$ $\quad E_c = 4286.8\ ksi$

Steel Properties: $F_y = 50\ ksi$ $\quad L_{bneg} = 13.42ft$ $\quad E_s = 29000\ ksi$ $\quad F_{yr} = 0.7\,F_y = 35\ ksi$

Negative Flexural Capacity:

Slenderness ratio for compressive flange: $\lambda_{fneg} = \frac{b_{bf}}{2\,t_{bf}} = 7.8$

Limiting ratio for compactness: $\lambda_{pfneg} = 0.38\sqrt{\frac{E_s}{F_y}} = 9.2$

Limiting ratio for noncompact: $\lambda_{rfneg} = 0.56\sqrt{\frac{E_s}{F_{yr}}} = 16.1$

Hybrid Factor: $R_h = 1$

$D_{cneg2} = \frac{D_w}{2} = 14.2\ in$ $\quad a_{wc} = \frac{2\,D_{cneg2}\,t_w}{b_{bf}\,t_{bf}} = 2.1$

$$R_b = \begin{cases} 1.0 & \text{if } 2\,\frac{D_{cneg2}}{t_w} \le 5.7\sqrt{\frac{E_s}{F_y}} \\ \min\left[1.0, 1 - \frac{a_{wc}}{1200 + 300\,a_{wc}}\left(2\,\frac{D_{cneg2}}{t_w} - 5.7\sqrt{\frac{E_s}{F_y}}\right)\right] & \text{otherwise} \end{cases}$$

$R_b = 1$

Flange compression resistance:

$$F_{nc1} = \begin{cases} R_b\,R_h\,F_y & \text{if } \lambda_{fneg} \le \lambda_{pfneg} \\ \left[\left[1 - \left(1 - \frac{F_{yr}}{R_h\,F_y}\right)\frac{\left(\lambda_{fneg} - \lambda_{pfneg}\right)}{\left(\lambda_{rfneg} - \lambda_{pfneg}\right)}\right]R_b\,R_h\,F_y\right] & \text{otherwise} \end{cases}$$

$F_{nc1} = 50\ ksi$

Lateral Torsional Buckling Resistance:

$$r_{tneg} = \frac{b_{bf}}{\sqrt{12\left(1 + \frac{D_{cneg2}\ t_w}{3\ b_{bf}\ t_{bf}}\right)}} = 2.6\ \text{in}$$

$$L_{pneg} = 1.0\ r_{tneg}\sqrt{\frac{E_s}{F_y}} = 62.5\ \text{in}$$

$$L_{rneg} = \pi\ r_{tneg}\sqrt{\frac{E_s}{F_{yr}}} = 234.7\ \text{in}$$

$$C_b = 1$$

$$F_{nc2} = \left| \begin{array}{l} R_b\ R_h\ F_y \quad \text{if}\ \ L_{bneg} \le L_{pneg} \\ \min\left[C_b\left[1 - \left(1 - \frac{F_{yr}}{R_h\ F_y}\right)\frac{\left(L_{bneg} - L_{pneg}\right)}{\left(L_{rneg} - L_{pneg}\right)}\right] R_b\ R_h\ F_y, R_b\ R_h\ F_y\right] \end{array} \right.$$

$$F_{nc2} = 41.4\ \text{ksi}$$

Compressive Resistance: $F_{nc} = \min\left(F_{nc1}, F_{nc2}\right) = 41.4\ \text{ksi}$

Tensile Flexural Resistance: $F_{nt} = R_h\ F_y = 50\ \text{ksi}$ For Strength

$F_{nt\_Serv} = 0.95\ R_h\ F_y = 47.5\ \text{ksi}$ For Service

Ultimate Moment Resistance: $M_{n\_neg} = \min\left(F_{nt}\ S_{top\_neg}, F_{nc}\ S_{bot\_neg}\right) = 1042\ \text{kip ft}$

$M_{UPier} = 948.1\ \text{kip ft}$ from external FE analysis

$$\text{Check4} = M_{n\_neg} \ge M_{UPier} = 1$$

For additional design, one may calculate the force couple at the section over the pier to find the force in the UHPC closure joint. This force can be used to design any additional reinforcement used in the joint.

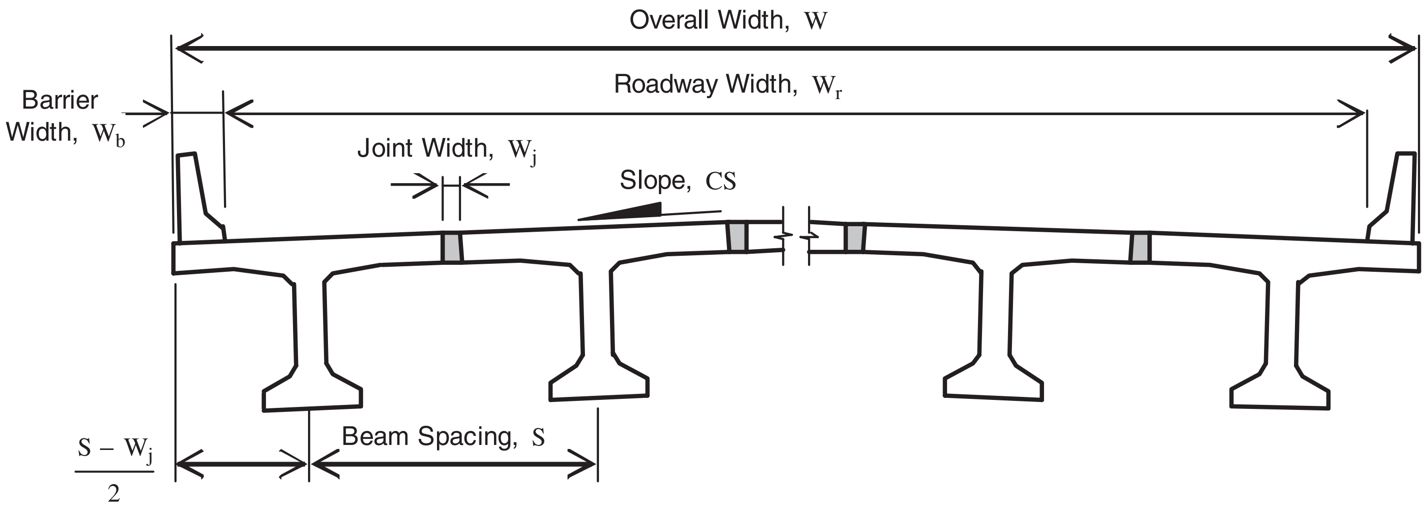
ABC SAMPLE CALCULATION – 2

Decked Precast Prestressed Concrete girder Design for ABC

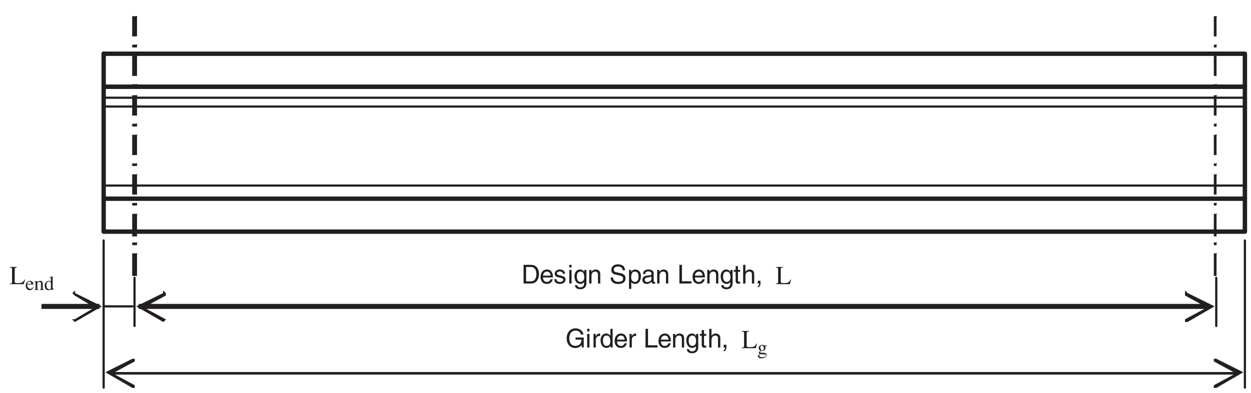
DECKED PRECAST PRESTRESSED CONCRETE GIRDER DESIGN FOR ABC

Unit Definition: $kcf \equiv kip\ ft^{-3}$

This example is for the design of a superstructure system that can be used for rapid bridge replacement in an Accelerated Bridge Construction (ABC) application. The following calculations are intended to provide the designer guidance in developing a similar design with regard to design considerations characteristic of this type of construction, and they shall not be considered fully exhaustive.



TYPICAL SECTION THROUGH SPAN



GIRDER ELEVATION

BRIDGE GEOMETRY

$L = 70\ ft$ $\quad L_{end} = 2\ ft$ $\quad skew = 0\ deg$

$W = 47.167\ ft$ $\quad W_b = 1.5\ ft$

$S_{max} = 8\ ft$ $\quad W_j = 0.5\ ft$

$N_g = ceil\left(\frac{W + W_j}{S_{max}}\right) = 6$ Minimum number of girders in cross-section

$S = \frac{W + W_j}{N_g} = 7.945\ ft$ Girder spacing

ORDER OF CALCULATIONS

1. Introduction
2. Design Philosophy
3. Design Criteria
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5. Material Properties
6. Permanent Loads
7. Precast Lifting Weight
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9. Prestress Properties
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16. Negative Moment Flexural Strength

1. INTRODUCTION

The superstructure system considered here consists of precast prestressed concrete girders with a top flange width nominally equal to the beam spacing, such that the top flange will serve as the riding surface once closure joints between the girders are poured. The intended use of these girders is to facilitate rapid bridge construction by providing a precast deck on the girder, thereby eliminating the need for a cast-in-place deck in the field.

Concepts used in this example are taken from previous and on-going research, the focus of which is overcoming issues detracting from the benefits of decked precast beams and promoting widespread acceptance by transportation agencies and the construction industry. The cross-section is adapted from the optimized girder sections recommended by NCHRP Project No. 12-69, Design and Construction Guidelines for Long-Span Decked Precast, Prestressed Concrete Girder Bridges. The section considered here has an additional 3" added to the top flange to accommodate the joint continuity detail utilized in this project. The girder design does not include the option to re-deck because the final re-decked system, without additional prestressing, is generally expected to have a shorter span length capability, effectively under-utilizing the initial precast section. Sacrificial wearing thickness, use of stainless steel rebars, and the application of a future membrane and wearing surface can mitigate the need to replace the deck, so these characteristics are included in lieu of "re-deckability."

The bridge used in this example represents a typical design problem. The calculations are equally as applicable to a single-span or multiple-span bridge because beam design moments are not reduced for continuity in multiple-span bridges at intermediate support. Design of the continuity details is not addressed in this example. The cross-section consists of a two-lane roadway with normal crown, bordered by standard barrier wall along each fascia. The structural system is made up of uniformly spaced decked precast prestressed concrete girders set normal to the cross-slope to allow for a uniform top flange and to simplify bearing details. The girder flanges are 9" at the tips, emulating an 8" slab with an allowance (1/2") for wear and an additional allowance (1/2") for grinding for smoothness and profile adjustment.

The intent of this example is to illustrate aspects of design unique to decked precast prestressed girders used in an ABC application. Prestress forces and concrete stresses at the service limit states due to the uncommon cross-section, unusually high self-weight, and unconventional sequence of load application are of particular concern, and appropriate detailed calculations are included. Flexure and shear at the strength limit state are not anticipated to differ significantly from a conventional prestressed beam design. With the exception of computing flexural resistance at midspan, flexure and shear are omitted from this example for brevity. Omission of these checks does not indicate they are not necessary, nor does it relieve the designer of the responsibility to satisfy any and all design requirements, as specified by AASHTO and the Owner.

2. DESIGN PHILOSOPHY

Geometry of the section is selected based on availability of standard formwork across many geographic regions, as evidenced by sections commonly used by many state transportation agencies. Depth variations are limited to constant-thickness region of the web, maintaining the shapes of the top flange and bottom bulb.

Concrete strengths can vary widely, and strengths ranging from below 6 ksi to over 10 ksi are common. For the purposes of these calculations, concrete with a 28-day minimum compressive strength of 8 ksi is used. Because this beam is unable to take advantage of the benefits of composite behavior due to its casting sequence, and because allowable tension in the bottom of the beam at the service limit state is limited (discussed in Section 4), end region stresses are expected to be critical. Therefore, minimum concrete strength at release is required to be 80 percent of the 28-day compressive strength of the concrete, increasing the allowable stresses at the top and bottom of the section. The prestressing steel can also be optimized to minimize the stresses in the end region, as discussed below.

Prestressing steel is arranged in a draped, or harped, pattern in order to maximize its effectiveness at midspan while minimizing its eccentricity at the ends of the beam where the concrete is easily overstressed because there is little positive dead load moment to offset the negative prestress moment. Effectiveness of the strand group is optimized at midspan by bundling the harped strands between hold-down points, maximizing the eccentricity of the strand group. The number and deflection angle of the harped strands is constrained by an upper limit on the hold-down force required for a single strand and for a single hold-down device, i.e., the entire group of strands. For longer spans, concrete stresses in the end regions at release will be excessive, and debonding without harped strands is not likely to reduce stresses to within allowable limits. Therefore, since harped strands will be required, this method of stress relief will be used exclusively without debonding. Temporary strands are not considered.

3. DESIGN CRITERIA

In addition to the provisions of AASHTO, several criteria have been selected to govern the design of these beams, based on past and current practice, as well as research related to decked precast sections and accelerated bridge construction. The following is a summary of limiting design values for which the beams are proportioned, and they are categorized as section constraints, prestress limits, and concrete limits:

Section Constraints:

$W_{pc.max} = 200\ \text{kip}$	Upper limit on the weight of the entire precast element, based on common lifting and transport capabilities without significantly increasing time and/or cost due to unconventional equipment or permits
$S_{max} = 8\ \text{ft}$	Upper limit on girder spacing and, therefore, girder flange width (defined on first page)

Prestress Limits:

$F_{hd.single} = 4\ \text{kip}$	Maximum hold-down force for a single strand
$F_{hd.group} = 48\ \text{kip}$	Maximum hold-down force for the group of harped strands

Stress limits in the prestressing steel immediately prior to prestress and at the service limit state after all losses are as prescribed by AASHTO LRFD.

Concrete Limits:

Allowable concrete stresses are generally in line with AASHTO LRFD requirements, with one exception. Allowable tension in the bottom of the section at final, Service III, is limited to 0 ksi, based on the research of NCHRP Project No. 12-69. Imposing this limitation precludes the need to evaluate the flexural effects on the girder section arising from forces applied to correct differential camber between adjacent beams. The reliability of this approach is enhanced without the need for additional calculations by specifying a differential camber tolerance equally as, or more stringent than, the tolerance assumed in the subject project. For the purposes of this example, the differential camber tolerance is assumed be at least as stringent.

$f_{t.all.ser} = 0\ \text{ksi}$ — Allowable bottom fiber tension at the Service III Limit State, when camber leveling forces are to be neglected, regardless of exposure

As previously mentioned, release concrete strength is specified as 80 percent of the minimum 28-day compressive strength to maximize allowable stresses in the end region of beam at release.

$f_{c.rel}(f) = 0.80\ f$ — Minimum strength of concrete at release

At the intermediate erection stage, stresses in the beam due to various lifting and transportation support conditions need to be considered. Using AASHTO LRFD Table 5.9.4.2.1-1, allowable compression during handling can be limited to 60% of the concrete strength. This provision is not explicitly applicable to this case, however, it does apply to handling stresses in prestressed piling and is more appropriate than the more restrictive sustained permanent load limit of 45% due to anticipated dynamic dead load effects. For allowable tension, a "no cracking" approach is considered due to reduced lateral stability after cracking. Therefore, allowable tension is limited to the modulus of rupture, further modified by an appropriate factor of safety. Both allowable values are based on the concrete strength at the time of lifting and transportation. At this stage, assuming the beams will be lifted sometime after release and before the final strength is attained, allowable stresses are based on the average of the release strength and the specified 28-day strength, i.e., 90% of the specified strength.

$DIM = 30\%$ — Dynamic dead load allowance

$f_{c.erec}(f) = 0.90\ f$ — Assumed attained concrete strength during lifting and transportation

$FS_c = 1.5$ — Factor of safety against cracking during lifting transportation

$f_{t.erec}(f) = \dfrac{-0.24\sqrt{f\ \text{ksi}}}{FS_c}$ — Allowable tension in concrete during lifting and transportation to avoid cracking

4. BEAM SECTION

Use trapezoidal areas to define the cross-section. The flange width is defined as the beam spacing less the width of the longitudinal closure joint to reflect pre-erection conditions. Live load can be conservatively applied to this section, as well.

$h = 42\ \text{in}$ — Beam section depth

$t_{flange} = 9\ \text{in}$ — Flange thickness at tip

$t_{sac} = 1\ \text{in}$ — Total sacrificial depth for grinding and wear

$b_1 = 26\ \text{in}$ $\quad b_2 = 26\ \text{in}$ $\quad d_1 = 7\ \text{in}$

$b_2 = 26\ \text{in}$ $\quad b_3 = 6\ \text{in}$ $\quad d_2 = 3\ \text{in}$

$b_3 = 6\ \text{in}$ $\quad b_4 = 6\ \text{in}$

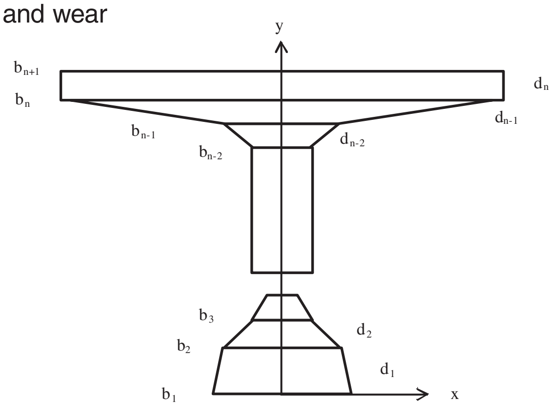
$b_4 = 6\ \text{in}$ $\quad b_5 = 10\ \text{in}$ $\quad d_4 = 2\ \text{in}$

$b_5 = 10\ \text{in}$ $\quad b_6 = 49\ \text{in}$ $\quad d_5 = 3\ \text{in}$

$b_6 = 49\ \text{in}$ $\quad b_7 = S - W_j$ $\quad d_6 = 0\ \text{in}$

$b_7 = 89.334\ \text{in}$ $\quad b_8 = S - W_j$ $\quad d_7 = t_{flange} - t_{sac}$

$d_3 = h - t_{sac} - \sum d$ $\quad d_3 = 18\ \text{in}$



TYPICAL GIRDER SECTION COMPRISED OF n TRAPEZOIDAL REGIONS

$b_f = 89.334\ \text{in}$ — Precast girder flange width

$A_g = 1157.172\ \text{in}^2$ — Cross-sectional area (does not include sacrifical thickness)

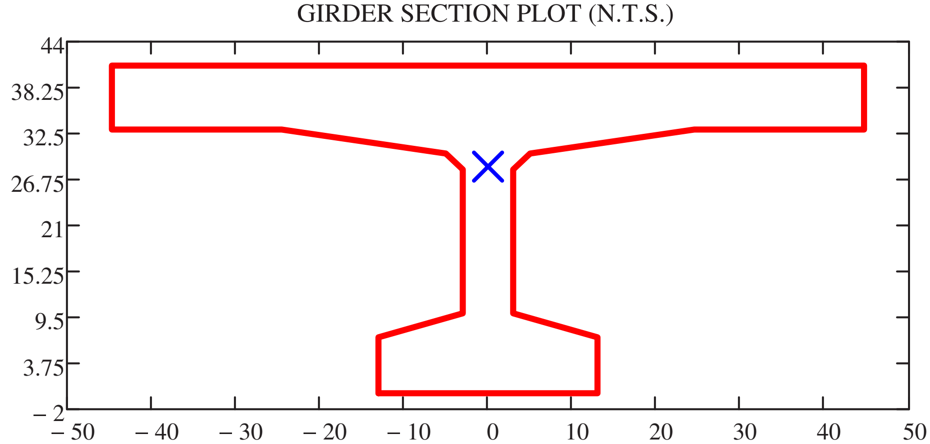
$I_{xg} = 203462\ \text{in}^4$ — Moment of inertia (does not include sacrificial thickness)

$y_{tg} = 12.649\ \text{in}$ $\quad y_{bg} = -28.351\ \text{in}$ — Top and bottom fiber distances from neutal axis (positive up)

$S_{tg} = 16085.5\ \text{in}^3$ $\quad S_{bg} = -7176.5\ \text{in}^3$ — Top and bottom section moduli

$I_{yg} = 493395\ \text{in}^4$ — Weak-axis moment of inertia

GIRDER SECTION PLOT (N.T.S.)



5. MATERIAL PROPERTIES

Concrete:

$f'_c = 8\ \text{ksi}$ — Minimum 28-day compressive strength of concrete

$f_{ci} = f_{c.rel}(f'_c) = 6.4\ \text{ksi}$ — Minimum strength of concrete at release

$\gamma_c = .150\ \text{kcf}$ — Unit weight of concrete

$K_1 = 1.0$ — Correction factor for standard aggregate (5.4.2.4)

$E_{ci} = 33000\ K_1 \left(\frac{\gamma_c}{\text{kcf}}\right)^{1.5} \sqrt{f_{ci}\ \text{ksi}} = 4850\ \text{ksi}$ — Modulus of elasticity at release (5.4.2.4-1)

$E_c = 33000\ K_1 \left(\frac{\gamma_c}{\text{kcf}}\right)^{1.5} \sqrt{f'_c\ \text{ksi}} = 5422\ \text{ksi}$ — Modulus of elasticity (5.4.2.4-1)

$f_{r.cm} = 0.37\sqrt{f'_c\ \text{ksi}} = 1.047\ \text{ksi}$ — Modulus of rupture for cracking moment (5.4.2.6)

$f_{r.cd} = 0.24\sqrt{f'_c\ \text{ksi}} = 0.679\ \text{ksi}$ — Modulus of rupture for camber and deflection (5.4.2.6)

$H = 70$ — Relative humidity (5.4.2.3)

Prestressing Steel:

$f_{pu} = 270\ \text{ksi}$ — Ultimate tensile strength

$f_{py} = 0.9\ f_{pu} = 243\ \text{ksi}$ — Yield strength, low-relaxation strand (Table 5.4.4.1-1)

$f_{pbt.max} = 0.75\ f_{pu} = 202.5\ \text{ksi}$ — Maximum stress in steel immediately prior to transfer

$f_{pe.max} = 0.80\ f_{py} = 194.4\ \text{ksi}$ — Maximum stress in steel after all losses

$E_p = 28500\ \text{ksi}$ — Modulus of elasticity (5.4.4.2)

$d_{ps} = 0.5\ \text{in}$ — Strand diameter

$A_p = 0.153\ \text{in}^2$ — Strand area

$N_{ps.max} = 40$ — Maximum number of strands in section

$n_{pi} = \frac{E_p}{E_{ci}} = 5.9$ — Modular ratio at release

$n_p = \frac{E_p}{E_c} = 5.3$ — Modular ratio

Mild Steel:

$f_y = 60\ \text{ksi}$ — Specified minimum yield strength

$E_s = 29000\ \text{ksi}$ — Modulus of elasticity (5.4.3.2)

6. PERMANENT LOADS

Permanent loads to be considered in the design of this girder are self-weight, diaphragms, barrier, and future wearing surface. The barrier can be cast with the beam, superimposed on the exterior girder only in the field, or superimposed on the bridge after the closure joints have attained sufficient strength. Distribution of the barrier weight to the girders should accurately reflect the stage at which it was installed. In this example, the barrier is assumed to be cast on the exterior girder in the casting yard, after release of prestress, but prior to shipping. This concept increases the dead load to be supported by the exterior girder while eliminating a time-consuming task to be completed in the field.

$\mathrm{BeamLoc} = 1$ — Location of beam within the cross-section (0 - Interior, 1 - Exterior)

Load at Release:

$\gamma_{c.DL} = .155 \text{ kcf}$ — Concrete density used for weight calculations

$A_{g.DL} = A_g + t_{sac}\left(S - W_j\right) = 1246.506 \text{ in}^2$ — Area used for weight calculations, including sacrificial thickness

$w_g = A_{g.DL}\,\gamma_{c.DL} = 1.342 \text{ klf}$ — Uniform load due to self-weight, including sacrificial thickness

$L_g = L + 2\,L_{end} = 74 \text{ ft}$ — Span length at release

$M_{gr}(x) = \dfrac{w_g\,x}{2}\left(L_g - x\right)$ — Moment due to beam self-weight (supported at ends)

$V_{gr}(x) = w_g\left(\dfrac{L_g}{2} - x\right)$ — Shear due to beam self-weight (supported at ends)

Load at Erection:

$M_g(x) = \dfrac{w_g\,x}{2}(L - x)$ — Moment due to beam self-weight

$V_g(x) = w_g\left(\dfrac{L}{2} - x\right)$ — Shear due to beam self-weight

$w_{bar} = 0.430 \text{ klf}$ — Uniform load due to barrier weight, exterior beams only

$w_{bar} = \mathrm{if}\left(\mathrm{BeamLoc} = 1, w_{bar}, 0\right) = 0.43 \text{ klf}$ — Redefine to 0 if interior beam (BeamLoc = 0)

$M_{bar}(x) = \dfrac{w_{bar}\,x}{2}(L - x)$ — Moment due to beam self-weight

$V_{bar}(x) = w_{bar}\left(\dfrac{L}{2} - x\right)$ — Shear due to beam self-weight

Load at Service:

$p_{fws} = 25 \text{ psf}$ — Assumed weight of future wearing surface

$w_{fws} = p_{fws} S = 0.199 \text{ klf}$ — Uniform load due to future wearing surface

$$M_{fws}(x) = \frac{w_{fws}\, x}{2} (L - x)$$ Moment due to future wearing surface

$$V_{fws}(x) = w_{fws} \left(\frac{L}{2} - x\right)$$ Shear due to future wearing surface

$w_j = W_j\, d_7\, \gamma_{c.DL} = 0.052 \text{ klf}$ — Uniform load due to weight of longitudinal closure joint

$$M_j(x) = \frac{w_j\, x}{2} (L - x)$$ Moment due to longitudinal closure joint

$$V_j(x) = w_j \left(\frac{L}{2} - x\right)$$ Shear due to longitudinal closure joint

7. PRECAST LIFTING WEIGHT

Precast Superstructure

$W_g = \left(w_g + w_{bar}\right) L_g = 131.1 \text{ kip}$ — Precast girder, including barrier if necessary

Substructure Precast with Superstructure

$L_{corb} = 1 \text{ ft}$ — Length of approach slab corbel

$B_{corb} = b_f$ $\quad b_f = 89.334 \text{ in}$ — Width of corbel cast with girder

$D_{corb} = 1.5 \text{ ft}$ — Average depth of corbel

$V_{corb} = L_{corb}\, B_{corb}\, D_{corb} = 11.17 \text{ ft}^3$ — Volume of corbel

$L_{ia} = 2.167 \text{ ft}$ — Length of integral abutment

$L_{gia} = 1.167 \text{ ft}$ — Length of girder embedded in integral abutment

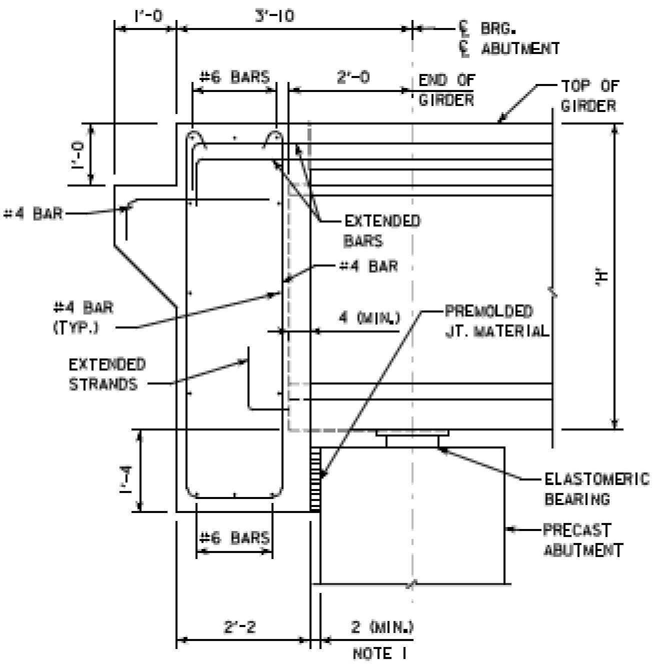
$B_{ia} = S - W_j = 7.444 \text{ ft}$ — Width of integral abutment cast with girder

$D_{ia} = h + 4 \text{ in} = 46 \text{ in}$ — Depth of integral abutment

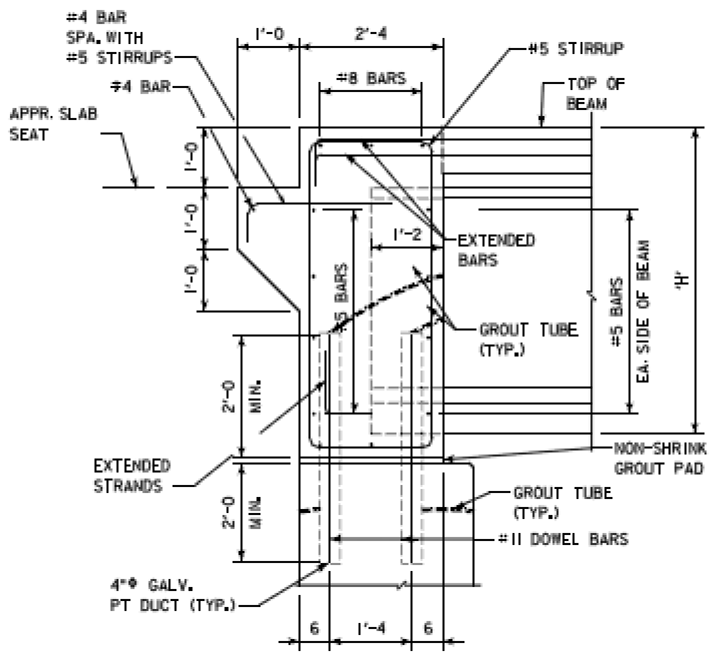
$V_{ia} = V_{corb} + \left[L_{ia}\, B_{ia}\, D_{ia} - \left(A_g - t_{flange}\, b_f\right) L_{gia}\right] = 70.14 \text{ ft}^3$ — Volume of integral abutment cast with girder

$W_{ia} = V_{ia}\, \gamma_c = 11 \text{ kip}$ — Weight of integral abutment cast with girder

$L_{sa} = 2.167 \text{ ft}$	Length of semi-integral abutment
$L_{gsa} = 4 \text{ in}$	Length of girder embedded in semi-integral abutment
$B_{sa} = S - W_j = 7.444 \text{ ft}$	Width of semi-integral abutment cast with girder
$D_{sa} = h + 16 \text{ in} = 58 \text{ in}$	Depth of semi-integral abutment
$V_{sa} = V_{corb} + \left[L_{sa}\, B_{sa}\, D_{sa} - \left(A_g - t_{flange}\, b_f\right) L_{gsa}\right] = 88.32 \text{ ft}^3$	Volume of semi-integral abutment cast with girder
$W_{sa} = V_{sa}\, \gamma_c = 13 \text{ kip}$	Weight of semi-integral abutment cast with girder



Semi-Integral Abutment Backwall



Integral Abutment Backwall

8. LIVE LOAD

Vehicular loading conforms to the HL-93 design load prescribed by AASHTO. If project-specific erection schemes require the bridge to support construction loads at any stage of erection, these loads should be considered as a separate load case and applied to the beam section at an appropriate attained age of the concrete.

Longitudinal joint is designed and detailed for a full moment connection. Therefore, the beams are considered "sufficiently connected to act as a unit" and distribution factors are computed for cross-section type "j", as defined in AASHTO 4.6.2.2. For purposes of computing the longitudinal stiffness parameter, the constant-depth region of the top flange is treated as the slab and the remaining area of the beam section is considered the non-composite beam.

Distribution Factors for Moment:

From Table 4.6.2.2.2b-1 for moment in interior girders,

$$I_{bb} = 59851\ in^4$$ Moment of inertia of section below the top flange

$$A_{bb} = 442.5\ in^2$$ Area of beam section below the top flange

$$e_g = h - \left(t_{sac} + \frac{t_s}{2}\right) + y_{bb} = 22.617\ in$$ Distance between c.g.'s of beam and flange

$$K_g = 1.0\left(I_{bb} + A_{bb}\, e_g^{\,2}\right) = 286209\ in^4$$ Longitudinal stiffness parameter (Eqn. 4.6.2.2.1-1)

Verify this girder design is within the range of applicability for Table 4.6.2.2.2b-1.

$$CheckMint = if\left[(S \le 16\ ft)\ (S \ge 3.5\ ft)\ (t_s \ge 4.5\ in)\ (t_s \le 12.0\ in)\ (L \ge 20\ ft)\ (L \le 240\ ft), \text{"OK"}, \text{"No Good"}\right]$$

$$CheckMint = if\left[(CheckMint = \text{"OK"})\ (N_g \ge 4)\ \left(K_g \ge 10000\ in^4\right)\left(K_g \le 7000000\ in^4\right), \text{"OK"}, \text{"No Good"}\right]$$

$$CheckMint = \text{"OK"}$$

$$g_{mint1} = 0.06 + \left(\frac{S}{14\ ft}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{L\ t_s^{\,3}}\right)^{0.1} = 0.458$$ Single loaded lane

$$g_{mint2} = 0.075 + \left(\frac{S}{9.5\ ft}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{L\ t_s^{\,3}}\right)^{0.1} = 0.633$$ Two or more loaded lanes

$$g_{mint} = \max\left(g_{mint1}, g_{mint2}\right) = 0.633$$ Distribution factor for moment at interior beams

From Table 4.6.2.2.2d-1 for moment in exterior girders,

$$d_e = \frac{S}{2} - W_b = 29.667\ in$$

$$CheckMext = if\left[(d_e \ge -1\ ft)\ (d_e \le 5.5\ ft)\ (N_g \ge 4), \text{"OK"}, \text{"No Good"}\right] = \text{"OK"}$$

For a single loaded lane, use the Lever Rule.

$$g_{mext1} = \frac{\left(S + 0.5\ b_f - W_b - 5\ ft\right)}{S} = 0.65$$ Single loaded lane

$$e_m = 0.77 + \frac{d_e}{9.1\ ft} = 1.042$$

$$g_{mext2} = e_m\ g_{mint} = 0.659$$ Two or more loaded lanes

$$g_{mext} = \max\left(g_{mext1}, g_{mext2}\right) = 0.659$$ Distribution factor for moment at exterior beams

Distribution Factors for Shear:

From Table 4.6.2.2.3a-1 for shear in interior girders,

Verify this girder design is within the range of applicability for Table 4.6.2.2.3a-1.

$$CheckVint = if\left[(S \le 16\ ft)\ (S \ge 3.5\ ft)\ \left(t_s \ge 4.5\ in\right)\ \left(t_s \le 12.0\ in\right)\ (L \ge 20\ ft)\ (L \le 240\ ft), "OK", "No\ Good"\right]$$

$$CheckVint = if\left[(CheckMint = "OK")\ \left(N_g \ge 4\right), "OK", "No\ Good"\right]$$

$$CheckVint = "OK"$$

$$g_{vint1} = 0.36 + \left(\frac{S}{25\ ft}\right) = 0.678$$ Single loaded lane

$$g_{vint2} = 0.2 + \left(\frac{S}{12\ ft}\right) - \left(\frac{S}{35\ ft}\right)^{2.0} = 0.811$$ Two or more loaded lanes

$$g_{vint} = \max\left(g_{vint1}, g_{vint2}\right) = 0.811$$ Distribution factor for shear at interior beams

From Table 4.6.2.2.3b-1 for shear in exterior girders,

For a single loaded lane, use the Lever Rule.

$$CheckVext = if\left[\left(d_e \ge -1\ ft\right)\ \left(d_e \le 5.5\ ft\right)\ \left(N_g \ge 4\right), "OK", "No\ Good"\right] = "OK"$$

$$g_1 = \frac{\left(S + 0.5\ b_f - W_b - 5\ ft\right)}{S} = 0.65$$ Single loaded lane (same as for moment)

$$e_v = 0.6 + \frac{d_e}{10\ ft} = 0.847$$

$$g_2 = e_v\ g_{vint} = 0.687$$ Two or more loaded lanes

$$g_{vext} = \max\left(g_1, g_2\right) = 0.687$$ Distribution factor for shear at exterior beams

From Table 4.6.2.2.3c-1 for skewed bridges,

$$\theta = \text{skew} = 0 \text{ deg}$$

$$\text{CheckSkew} = \text{if}\left[(\theta \le 60 \text{ deg})\ (3.5 \text{ ft} \le S \le 16 \text{ ft})\ (20 \text{ ft} \le L \le 240 \text{ ft})\ \left(N_g \ge 4\right), \text{"OK"}, \text{"No Good"}\right] = \text{"OK"}$$

$$c_{skew} = 1.0 + 0.20 \left(\frac{L\ t_s^{\ 3}}{K_g}\right)^{0.3} \tan(\theta) = 1.00$$ Correction factor for skew

Design Live Load Moment at Midspan:

$$w_{lane} = 0.64 \text{ klf}$$ Design lane load

$$P_{truck} = 32 \text{ kip}$$ Design truck axle load

$$IM = 33\%$$ Dynamic load allowance (truck only)

$$M_{lane}(x) = \frac{w_{lane}\ x}{2}(L - x)$$ Design lane load moment

$$\delta(x) = \frac{x\ L - x^2}{L}$$ Influence coefficient for truck moment calculation

$$M_{truck}(x) = P_{truck}\ \delta(x)\ \max\left[\frac{9\ x\ (L - x) - 14 \text{ ft}\ (3\ x + L)}{4\ x\ (L - x)}, \frac{9\ (L - x) - 84 \text{ ft}}{4\ (L - x)}\right]$$ Design truck moment

$$M_{HL93}(x) = M_{lane}(x) + (1 + IM)\ M_{truck}(x)$$ HL93 design live load moment per lane

$$M_{ll.i}(x) = M_{HL93}(x)\ g_{mint}$$ Design live load moment at interior beam

$$M_{ll.e}(x) = M_{HL93}(x)\ g_{mext}$$ Design live load moment at exterior beam

$$M_{ll}(x) = \text{if}\left(\text{BeamLoc} = 1, M_{ll.e}(x), M_{ll.i}(x)\right)$$ Design live load moment

Design Live Load Shear:

$$V_{lane}(x) = w_{lane}\left(\frac{L}{2} - x\right)$$ Design lane load shear

$$V_{truck}(x) = P_{truck}\left(\frac{9\ L - 9\ x - 84 \text{ ft}}{4\ L}\right)$$ Design truck shear

$$V_{HL93}(x) = V_{lane}(x) + (1 + IM)\ V_{truck}(x)$$ HL93 design live load shear

$$V_{ll.i}(x) = V_{HL93}(x)\ g_{vint}$$ Design live load shear at interior beam

$$V_{ll.e}(x) = V_{HL93}(x)\ g_{vext}$$ Design live load shear at exterior beam

$$V_{ll}(x) = \text{if}\left(\text{BeamLoc} = 1, V_{ll.e}(x), V_{ll.i}(x)\right)$$ Design live load shear

9. PRESTRESS PROPERTIES

Because allowable tension at the service limit state is reduced to account for camber leveling forces, the prestress force required at midspan is expected to be excessive in the ends at release without measures to reduce the prestress moment. Estimate losses and prestress eccentricity at midspan to select a trial prestress force that results in a bottom fiber tension stress less than allowable. Compute instantaneous losses in the prestressing steel and check release stresses at the end of the beam. Once end stresses are satisfied, estimate total loss of prestress. As long as computed losses do not differ significantly from the assumed values, the prestress layout should be adequate. Concrete stresses at all limit states are evaluated in Section 9.

$y_{p.est} = 5\ \text{in}$ Assumed distance from bottom of beam to centroid of prestress at midspan

$y_{cgp.est} = y_{bg} + y_{p.est} = -23.35\ \text{in}$ Eccentricity of prestress from neutral axis, based on assumed location

$\Delta f_{p.est} = 25\%$ Estimate of total prestress losses at the service limit state

Compute bottom fiber service stresses at midspan using gross section properties.

$X = \dfrac{L}{2}$ Distance from support

$M_{dl.ser} = M_g(X) + M_{fws}(X) + M_j(X) + M_{bar}(X) = 1238\ \text{kip ft}$ Total dead load moment

$f_{b.serIII} = \dfrac{M_{dl.ser} + 0.8\,M_{ll}(X)}{S_{bg}} = -3.567\ \text{ksi}$ Total bottom fiber service stress

$f_{pj} = f_{pbt.max} = 202.5\ \text{ksi}$ Prestress jacking force

$f_{pe.est} = f_{pj}\left(1 - \Delta f_{p.est}\right) = 151.9\ \text{ksi}$ Estimate of effective prestress force

$A_{ps.est} = A_g \dfrac{\left(\dfrac{-f_{b.serIII} + f_{t.all.ser}}{f_{pe.est}}\right)}{1 + \dfrac{A_g\, y_{cgp.est}}{S_{bg}}} = 5.703\ \text{in}^2$ Estimated minimum area of prestressing steel

$N_{ps.est} = \text{ceil}\left(\dfrac{A_{ps.est}}{A_p}\right) = 38$ Estimated number of strands required

$N_{ps} = 38$ Number of strands used ($N_{ps.max} = 40$)

This number is used to lay out the strand pattern and compute an actual location and eccentricity of the strand group, after which the required area is computed again. If the location estimate was accurate, the recomputed number of strands should not differ from the number defined here. If the estimate was low, consider increasing the number of strands. It should be noted that the number of strands determined in this section is based on assumed prestressed losses and gross section properties and may not accurately reflect the final number of strands required to satisfy design requirements. Concrete stresses are evaluated in Section 10.

Strand pattern geometry calculations assume a vertical spacing of 2" between straight strands, as well as harped strands at the ends of the beam. Harped strands are bundled at midpsan, where the centroid of these strands is 5" from the bottom.

$$N_h = \begin{cases} 2 & \text{if } N_{ps} \le 12 \\ 4 & \text{if } 12 < N_{ps} \le 24 \\ 6 & \text{if } 24 < N_{ps} \le 30 \\ 6 + (N_{ps} - 30) & \text{if } N_{ps} > 30 \end{cases}$$ $N_h = 14$ Assumes all flange rows are filled prior to filling rows in web above the flange, which maximized efficiency. Use override below to shift strands from flange to web if needed to satisfy end stresses.

$N_{h.add} = 16$ Additional harped strands in web (strands to be moved from flange to web)

$N_h = \min\left(N_h + N_{h.add}, 16, 2\ \text{floor}\left(\frac{N_{ps}}{4}\right)\right)$ $N_h = 16$ 16 strands or half of total strands maximum harped in web

$y_h = 1\ \text{in} + (2\ \text{in})\left(1 + \frac{0.5\ N_h - 1}{2}\right)$ $y_h = 10\ \text{in}$ Centroid of harped strands from bottom, equally spaced

$y_{hb} = 5\ \text{in}$ Centroid of harped strands from bottom, bundled

$N_s = N_{ps} - N_h$ $N_s = 22$ Number of straight strands in flange

$$y_s = 1\ \text{in} + \begin{cases} 2\ \text{in} & \text{if } N_s \le 10 \\ \dfrac{(4\ \text{in})\ N_s - 20\ \text{in}}{N_s} & \text{if } 10 < N_s \le 20 \\ \dfrac{(6\ \text{in})\ N_s - 60\ \text{in}}{N_s} & \text{if } 20 < N_s \le 24 \\ 3.5\ \text{in} & \text{otherwise} \end{cases}$$ $y_s = 4.273\ \text{in}$ Centroid of straight strands from bottom

$y_p = \frac{N_s\ y_s + N_h\ y_{hb}}{N_s + N_h} = 4.579\ \text{in}$ Centroid of prestress from bottom at midspan

$y_{cgp} = y_{bg} + y_p = -23.77\ \text{in}$ Eccentricity of prestress from neutral axis

$$A_{ps.req} = A_g \frac{\left(\dfrac{-f_{b.serIII} + f_{t.all.ser}}{f_{pe.est}}\right)}{1 + \dfrac{A_g\ y_{cgp}}{S_{bg}}} = 5.623\ \text{in}^2$$ Estimated minimum area of prestressing steel

$N_{ps.req} = \text{ceil}\left(\frac{A_{ps.req}}{A_p}\right) = 37$ Estimated number of strands required

$\text{CheckNps} = \text{if}\left[(N_{ps} \le N_{ps.max})\ (N_{ps.req} \le N_{ps}), \text{"OK"}, \text{"No Good"}\right] = \text{"OK"}$

$A_{ps.h} = N_h\ A_p = 2.448\ \text{in}^2$ Area of prestress in web (harped)

$A_{ps.s} = N_s\ A_p = 3.366\ \text{in}^2$ Area of prestress in flange (straight)

$A_{ps} = A_{ps.h} + A_{ps.s} = 5.814\ \text{in}^2$ Total area of prestress

Compute transformed section properties based on prestress layout.

Initial Transformed Section (release):

$A_{ti} = 1185.5\ in^2$

$I_{xti} = 219101\ in^4$

$y_{tti} = 13.217\ in$ $\quad S_{tti} = 16577\ in^3$

$y_{cgpi} = -23.204\ in$ $\quad S_{cgpi} = -9442\ in^3$

$y_{bti} = -27.783\ in$ $\quad S_{bti} = -7886\ in^3$

Final Transformed Section (service):

$A_{tf} = 1181.9\ in^2$

$I_{xtf} = 217153\ in^4$

$y_{ttf} = 13.146\ in$ $\quad S_{ttf} = 16518\ in^3$

$y_{cgpf} = -23.275\ in$ $\quad S_{cgpf} = -9330\ in^3$

$y_{btf} = -27.854\ in$ $\quad S_{btf} = -7796\ in^3$

Determine initial prestress force after instantaneous loss due to elastic shortening. Use transformed properties to compute stress in the concrete at the level of prestress.

$P_j = f_{pj}\ A_{ps} = 1177.3\ kip$ — Jacking force in prestress, prior to losses

$$f_{cgpi} = P_j\left(\frac{1}{A_{ti}} + \frac{y_{cgpi}}{S_{cgpi}}\right) + \frac{M_{gr}\left(\frac{L_g}{2}\right)}{S_{cgpi}} = 2.719\ ksi$$

Stress in concrete at the level of prestress after instantaneous losses

$\Delta f_{pES} = n_{pi}\ f_{cgpi} = 15.978\ ksi$ — Prestress loss due to elastic shortening (5.9.5.2.3a-1)

$f_{pi} = f_{pj} - \Delta f_{pES} = 186.522\ ksi$ — Initial prestress after instantaneous losses

$P_i = f_{pi}\ A_{ps} = 1084.4\ kip$ — Initial prestress force

Determine deflection of harped strands required to satisfy allowable stresses at the end of the beam at release.

$f_{c.all.rel} = 0.6\ f_{ci} = 3.84\ ksi$ — Allowable compression before losses (5.9.4.1.1)

$f_{t.all.rel} = \max\left(-0.0948\sqrt{f_{ci}\ ksi}, -0.2\ ksi\right) = -0.200\ ksi$ — Allowable tension before losses (Table 5.9.4.1.2-1)

$L_t = 60\ d_{ps} = 2.5\ ft$ — Transfer length (AASHTO 5.11.4.1)

$$y_{cgp.t} = \left(\frac{f_{t.all.rel} - \frac{M_{gr}(L_t)}{S_{tti}}}{P_i} - \frac{1}{A_{ti}}\right) S_{tti} = -18.367\ in$$

Prestress eccentricity required for tension

$$y_{cgp.b} = \left(\frac{f_{c.all.rel} - \frac{M_{gr}(L_t)}{S_{bti}}}{P_i} - \frac{1}{A_{ti}}\right) S_{bti} = -22.6\ in$$

Prestress eccentricity required for compression

$y_{cgp.req} = \max\left(y_{cgp.t}, y_{cgp.b}\right) = -18.367\ in$ — Required prestress eccentricity at end of beam

$$y_{h.brg.req} = \frac{(y_{cgp.req} - y_{bti})(N_s + N_h) - y_s N_s}{N_h} = 16.488 \text{ in}$$

Minimum distance to harped prestress centroid from bottom of beam at centerline of bearing

$$y_{top.min} = 18 \text{ in}$$

Minimum distance between uppermost strand and top of beam

$$\alpha_{hd} = 0.4$$

Hold-down point, fraction of the design span length

$$slope_{max} = \text{if}\left(d_{ps} = 0.6 \text{ in}, \frac{1}{12}, \frac{1}{8}\right) = 0.125$$

Maximum slope of an individual strand to limit hold-down force to 4 kips/strand

$$y_{h.brg} = h - y_{top.min} - \left(\frac{0.5\,N_h - 1}{2}\right)(2 \text{ in}) = 17 \text{ in}$$

Set centroid of harped strands as high as possible to minimize release and handling stresses

$$y_{h.brg} = \min\left(y_{h.brg}, y_{hb} + slope_{max}\,\alpha_{hd}\,L\right) = 17 \text{ in}$$

Verify that slope requirement is satisfied at uppermost strand

$$CheckEndPrestress = \text{if}\left(y_{h.brg} \geq y_{h.brg.req}, \text{"OK"}, \text{"Verify release stresses."}\right) = \text{"OK"}$$

$$y_{p.brg} = \frac{N_s\,y_s + N_h\,y_{h.brg}}{N_s + N_h} = 9.632 \text{ in}$$

Centroid of prestress from bottom at bearing

$$slope_{cgp} = \frac{y_{p.brg} - y_p}{\alpha_{hd}\,L} = 0.015$$

Slope of prestress centroid within the harping length

$$y_{px}(x) = \begin{cases} y_p + slope_{cgp}\left(L_{end} + \alpha_{hd}\,L - x\right) & \text{if } x \leq L_{end} + \alpha_{hd}\,L \\ y_p & \text{otherwise} \end{cases}$$

Distance to center of prestress from the bottom of the beam at any position

10. PRESTRESS LOSSES

As with any prestressed concrete design, total prestress loss can be considered as the sum of instantaneous (short-term) and time-dependent (long-term) losses. For pretensioned girders, the instantaneous loss consists of elastic shortening of the beam upon release of the prestress force. The time-dependent losses consist of creep and shrinkage of beam concrete, creep and shrinkage of deck concrete, and relaxation of the prestressing steel. These long-term effects in the girder are further subdivided into two stages to represent a significant event in the construction of the bridge: time between transfer of the prestress force and placement of the deck, and the period of time between placement of the deck and final service. For the specific case of a decked beam, computation of long-term losses is somewhat simplified because the cross-section does not change between these two stages and the term related to shrinkage of the deck concrete is eliminated since the deck is cast monolithically with the beam. There will be no gains or losses in the steel associated with deck placement after transfer.

AASHTO provides two procedures for estimating time-dependent losses:

1. Approximate Estimate (5.9.5.3)
2. Refined Estimate (5.9.5.4)

The approximate method is intended for systems with composite decks and is based upon assumptions related to timing of load application, the cross-section to which load is applied (non-composite or composite), and ratios of dead load and live load to total load. The conditions under which these beams are to be fabricated, erected, and loaded differ from the conditions assumed in development of the approximate method. Therefore, the refined method is used to estimate time-dependent losses in the prestressing steel.

Time-dependent loss equations of 5.9.5.4 include age-adjusted transformed section factors to permit loss computations using gross section properties.

Assumed time sequence in the life of the girder for loss calculations:

$t_i = 1$ — Time (days) between casting and release of prestress

$t_b = 20$ — Time (days) to barrier casting (exterior girder only)

$t_d = 30$ — Time (days) to erection of precast section, closure joint pour

$t_f = 20000$ — Time (days) to end of service life

Terms and equations used in the loss calculations:

$K_L = 45$ — Prestressing steel factor for low-relaxation strands (C5.9.5.4.2c)

$VS = \dfrac{A_g}{Peri} = 4.023 \text{ in}$ — Volume-to-surface ratio of the precast section

$k_s = \max\left(1.45 - 0.13\,\dfrac{VS}{in}, 1.0\right) = 1.00$ — Factor for volume-to-surface ratio (5.4.2.3.2-2)

$k_{hc} = 1.56 - 0.008\ H = 1.00$ — Humidity factor for creep (5.4.2.3.2-3)

$k_{hs} = 2.00 - 0.014\ H = 1.02$ — Humidity factor for shrinkage (5.4.2.3.3-2)

$k_f = \dfrac{5}{1 + \dfrac{f_{ci}}{ksi}} = 0.676$ — Factor for effect of concrete strength (5.4.2.3.2-4)

$k_{td}(t) = \dfrac{t}{61 - 4\,\dfrac{f_{ci}}{ksi} + t}$ — Time development factor (5.4.2.3.2-5)

$\psi\left(t, t_{init}\right) = 1.9\ k_s\ k_{hc}\ k_f\ k_{td}(t)\ \left(t_{init}\right)^{-0.118}$ — Creep coefficient (5.4.2.3.2-1)

$\varepsilon_{sh}(t) = k_s\ k_{hs}\ k_f\ k_{td}(t)\ \left(0.48\ 10^{-3}\right)$ — Concrete shrinkage strain (5.4.2.3.3-1)

Time from Transfer to Erection:

$e_{pg} = -\left(y_p + y_{bg}\right) = 23.772 \text{ in}$ — Eccentricity of prestress force with respect to the neutral axis of the gross non-composite beam, positive below the beam neutral axis

$f_{cgp} = P_i\left(\dfrac{1}{A_g} + \dfrac{e_{pg}^{\,2}}{I_{xg}}\right) + \dfrac{M_g\left(\dfrac{L}{2}\right)}{I_{xg}}\left(y_p + y_{bg}\right) = 2.797 \text{ ksi}$ — Stress in the concrete at the center prestress immediately after transfer

$f_{pt} = \max\left(f_{pi}, 0.55\ f_{py}\right) = 186.522 \text{ ksi}$ — Stress in strands immediately after transfer (5.9.5.4.2c-1)

$\psi_{bid} = \psi\left(t_d, t_i\right) = 0.589$ — Creep coefficient at erection due to loading at transfer

$\psi_{bif} = \psi\left(t_f, t_i\right) = 1.282$ — Creep coefficient at final due to loading at transfer

$\varepsilon_{bid} = \varepsilon_{sh}\left(t_d - t_i\right) = 1.490 \times 10^{-4}$ — Concrete shrinkage between transfer and erection

$$K_{id} = \frac{1}{1 + n_{pi}\frac{A_{ps}}{A_g}\left(1 + \frac{A_g\, e_{pg}^{\,2}}{I_{xg}}\right)\left(1 + 0.7\,\psi_{bif}\right)} = 0.809$$

Age-adjusted transformed section coefficient (5.9.5.4.2a-2)

$$\Delta f_{pSR} = \varepsilon_{bid}\, E_p\, K_{id} = 3.435 \text{ ksi}$$

Loss due to beam shrinkage (5.9.5.4.2a-1)

$$\Delta f_{pCR} = n_{pi}\, f_{cgp}\, \psi_{bid}\, K_{id} = 7.831 \text{ ksi}$$

Loss due to creep (5.9.5.4.2b-1)

$$\Delta f_{pR1} = \left[\frac{f_{pt}}{K_L}\,\frac{\log(24\,t_d)}{\log(24\,t_i)}\left(\frac{f_{pt}}{f_{py}} - 0.55\right)\right]\left[1 - \frac{3\left(\Delta f_{pSR} + \Delta f_{pCR}\right)}{f_{pt}}\right] K_{id} = 1.237 \text{ ksi}$$

Loss due to relaxation (C5.9.5.4.2c-1

$$\Delta f_{pid} = \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1} = 12.502 \text{ ksi}$$

Time from Erection to Final:

$$e_{pc} = e_{pg} = 23.772 \text{ in}$$

Eccentricity of prestress force does not change

$$A_c = A_g \qquad I_c = I_{xg}$$

Section properties remain unchanged

$$\Delta f_{cd} = \frac{M_{fws}\left(\frac{L}{2}\right) + M_j\left(\frac{L}{2}\right)}{S_{cgpf}} + \frac{\Delta f_{pid}}{n_p} = 2.182 \text{ ksi}$$

Change in concrete stress at center of prestress due to initial time-dependent losses and superimposed dead load. Deck weight is not included for this design.

$$\psi_{bdf} = \psi(t_f, t_d) = 0.858$$

Creep coefficient at final due to loading at erection

$$\varepsilon_{bif} = \varepsilon_{sh}(t_f - t_i) = 3.302 \times 10^{-4}$$

Concrete shrinkage between transfer and final

$$\varepsilon_{bdf} = \varepsilon_{bif} - \varepsilon_{bid} = 1.813 \times 10^{-4}$$

Concrete shrinkage between erection and final

$$K_{df} = \frac{1}{1 + n_{pi}\frac{A_{ps}}{A_c}\left(1 + \frac{A_c\, e_{pc}^{\,2}}{I_c}\right)\left(1 + 0.7\,\psi_{bif}\right)} = 0.809$$

Age-adjusted transformed section coefficient remains unchanged

$$\Delta f_{pSD} = \varepsilon_{bdf}\, E_p\, K_{df} = 4.179 \text{ ksi}$$

Loss due to beam shrinkage

$$\Delta f_{pCD} = n_{pi}\, f_{cgp}\left(\psi_{bif} - \psi_{bid}\right) K_{df} + n_p\, \Delta f_{cd}\, \psi_{bdf}\, K_{df} = 17.168 \text{ ksi}$$

Loss due to creep

$$\Delta f_{pR2} = \Delta f_{pR1} = 1.237 \text{ ksi}$$

Loss due to relaxation

$$\Delta f_{pSS} = 0$$

Loss due to deck shrinkage

$$\Delta f_{pdf} = \Delta f_{pSD} + \Delta f_{pCD} + \Delta f_{pR2} + \Delta f_{pSS} = 22.584 \text{ ksi}$$

Prestress Loss Summary

$$\Delta f_{pES} = 15.978\ \text{ksi} \qquad \frac{\Delta f_{pES}}{f_{pj}} = 7.9\ \%$$

$$\Delta f_{pLT} = \Delta f_{pid} + \Delta f_{pdf} = 35.087\ \text{ksi} \qquad \frac{\Delta f_{pLT}}{f_{pj}} = 17.3\ \%$$

$$\Delta f_{pTotal} = \Delta f_{pES} + \Delta f_{pLT} = 51.065\ \text{ksi} \qquad \frac{\Delta f_{pTotal}}{f_{pj}} = 25.2\ \% \qquad \Delta f_{p.est} = 25\ \%$$

$$f_{pe} = f_{pj} - \Delta f_{pTotal} = 151.4\ \text{ksi}$$

Final effective prestress

$$\text{CheckFinalPrestress} = \text{if}\left(f_{pe} \le f_{pe.max}, \text{"OK"}, \text{"No Good"}\right) = \text{"OK"}$$

11. CONCRETE STRESSES

Stresses in the concrete section at release, during handling, and at final service are computed and checked against allowable values appropriate for the stage being considered.

Concrete Stresses at Release

Stresses at release are computed using the overall beam length as the span because the beam will be supported at its ends in the casting bed after the prestress force is transfered.

Define locations for which stresses are to be calculated:

$$x_r = L_g \left(0 \quad \min\left(\frac{L_t}{L_g}, \frac{L_{end}}{L_g}\right) \quad \max\left(\frac{L_t}{L_g}, \frac{L_{end}}{L_g}\right) \quad 0.1 \quad 0.2 \quad 0.3 \quad \alpha_{hd} \quad 0.5 \right)^T \qquad ir = 1\,..\,\text{last}(x_r)$$

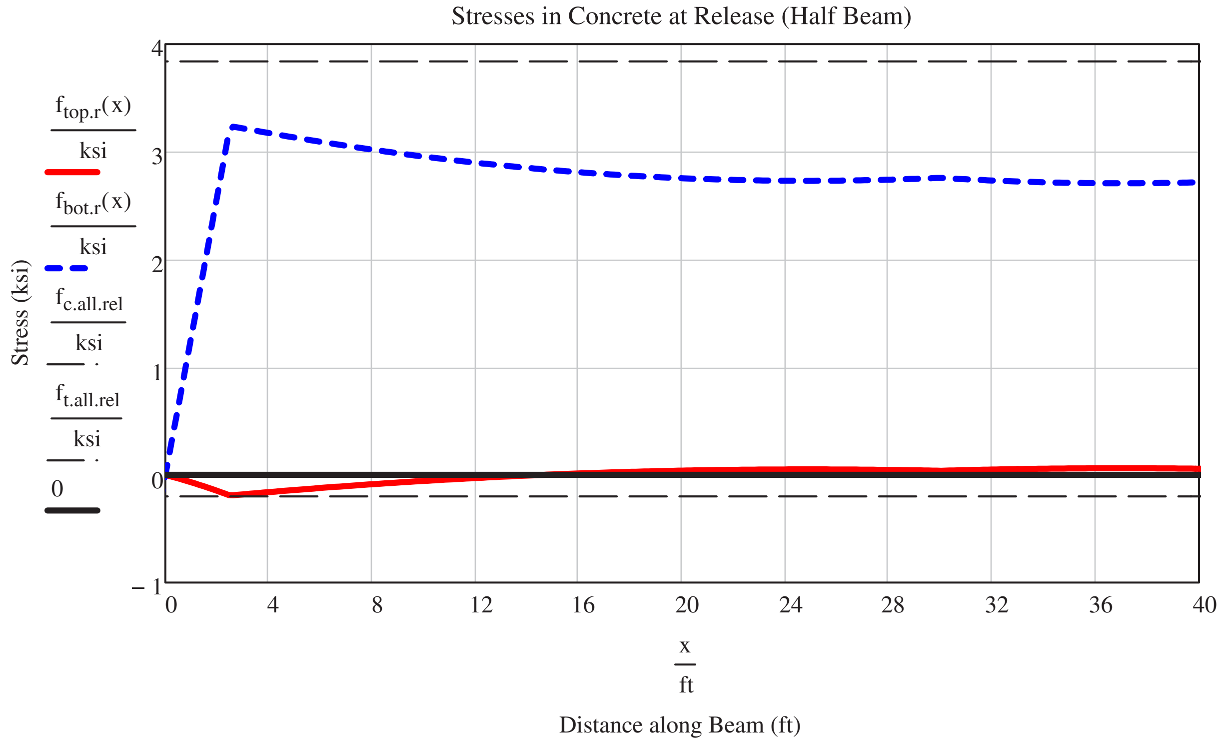
Functions for computing beam stresses:

$$f_{top.r}(x) = \min\left(\frac{x}{L_t}, 1\right) P_i \left(\frac{1}{A_{ti}} + \frac{y_{bti} + y_{px}(x)}{S_{tti}}\right) + \frac{M_{gr}(x)}{S_{tti}}$$

Top fiber stress at release

$$f_{bot.r}(x) = \min\left(\frac{x}{L_t}, 1\right) P_i \left(\frac{1}{A_{ti}} + \frac{y_{bti} + y_{px}(x)}{S_{bti}}\right) + \frac{M_{gr}(x)}{S_{bti}}$$

Bottom fiber stress at release



Compare beam stresses to allowable stresses.

$f_{t.all.rel} = -0.2\ \text{ksi}$ — Allowable tension at release

$f_{c.all.rel} = 3.84\ \text{ksi}$ — Allowable compression at release

$$TopRel_{ir} := f_{top.r}\left(x_{r_{ir}}\right) \qquad TopRel^{T} = (0.000\ \ -0.148\ \ -0.192\ \ -0.097\ \ 0.002\ \ 0.047\ \ 0.040\ \ 0.062\ )\ \text{ksi}$$

$$CheckTopRel := \text{if}\left[\left(\max(TopRel) \le f_{c.all.rel}\right)\left(\min(TopRel) \ge f_{t.all.rel}\right), \text{"OK"}, \text{"No Good"}\right] = \text{"OK"}$$

$$BotRel_{ir} := f_{bot.r}\left(x_{r_{ir}}\right) \qquad BotRel^{T} = (0.000\ \ 2.582\ \ 3.241\ \ 3.042\ \ 2.834\ \ 2.738\ \ 2.754\ \ 2.708\ )\ \text{ksi}$$

$$CheckBotRel := \text{if}\left[\left(\max(BotRel) \le f_{c.all.rel}\right)\left(\min(BotRel) \ge f_{t.all.rel}\right), \text{"OK"}, \text{"No Good"}\right] = \text{"OK"}$$

Concrete Stresses During Lifting and Transportation

Stresses in the beam during lifting and transportation may govern over final service limit state stresses due to different support locations, dynamic effects of dead load during shipment and placement, and lateral bending stresses due to rolling during lifting or superelevation of the roadway during shipping. Assume end diaphragms on both ends of the beam. For prestressing effects, compute the effective prestress force using only the losses occurring between transfer and erection (i.e., the Δf_{pid}).

$a := h = 3.5\ \text{ft}$ — Maximum distance to lift point from bearing line

$a' := a + L_{end} = 5.5\ \text{ft}$ — Distance to lift point from end of beam

$$P_{dia} = \max(W_{ia}, W_{sa}) = 13.2 \text{ kip}$$ Approximate abutment weight

$$P_m = P_j \left[1 - \frac{(\Delta f_{pES} + \Delta f_{pid})}{f_{pj}}\right] = 1011.7 \text{ kip}$$ Effective prestress during lifting and shipping

Define locations for which stresses are to be calculated:

$$x_e = L_g \left(0 \quad \min\left(\frac{L_t}{L_g}, \frac{L_{end}}{L_g}\right) \quad \max\left(\frac{L_t}{L_g}, \frac{L_{end}}{L_g}\right) \quad \frac{a'}{L_g} \quad \alpha_{hd} \quad 0.5\right)^T \qquad ie = 1 .. \text{last}(x_e)$$

Compute moment in the girder during lifting with supports at the lift points.

$$M_{lift}(x) = \begin{vmatrix} -\left[\frac{(w_g + w_{bar})\, x^2}{2} + P_{dia}\, x\right] & \text{if } x \le a' \\ M_{gr}(x) - \left[M_{gr}(a') + \frac{(w_g + w_{bar})\,(a')^2}{2} + P_{dia}\, a'\right] & \text{otherwise} \end{vmatrix}$$

Functions for computing beam stresses:

$$f_{top.lift}(x) = \min\left(\frac{x}{L_t}, 1\right) P_m \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{ttf}}\right) + \frac{M_{lift}(x)}{S_{ttf}}$$ Top fiber stress during lifting

$$f_{top.DIM.inc}(x) = \min\left(\frac{x}{L_t}, 1\right) P_m \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{ttf}}\right) + \frac{M_{lift}(x)}{S_{ttf}} (1 + DIM)$$ Top fiber stress during lifting, impact increasing dead load

$$f_{top.DIM.dec}(x) = \min\left(\frac{x}{L_t}, 1\right) P_m \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{ttf}}\right) + \frac{M_{lift}(x)}{S_{ttf}} (1 - DIM)$$ Top fiber stress during lifting, impact decreasing dead load

$$TopLift1_{ie} = f_{top.lift}(x_{e_{ie}}) \qquad TopLift1^T = (0.000 \quad -0.230 \quad -0.294 \quad -0.371 \quad -0.181 \quad -0.158)\text{ ksi}$$

$$TopLift2_{ie} = f_{top.DIM.inc}(x_{e_{ie}}) \qquad TopLift2^T = (0.000 \quad -0.236 \quad -0.302 \quad -0.393 \quad -0.065 \quad -0.035)\text{ ksi}$$

$$TopLift3_{ie} = f_{top.DIM.dec}(x_{e_{ie}}) \qquad TopLift3^T = (0.000 \quad -0.223 \quad -0.285 \quad -0.349 \quad -0.296 \quad -0.282)\text{ ksi}$$

$$f_{bot.lift}(x) = \min\left(\frac{x}{L_t}, 1\right) P_m \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{btf}}\right) + \frac{M_{lift}(x)}{S_{btf}}$$ Bottom fiber stress during lifting

$$f_{bot.DIM.inc}(x) = \min\left(\frac{x}{L_t}, 1\right) P_m \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{btf}}\right) + \frac{M_{lift}(x)}{S_{btf}} (1 + DIM)$$ Bottom fiber stress during lifting, impact increasing dead load

$$f_{bot.DIM.dec}(x) = \min\left(\frac{x}{L_t}, 1\right) P_m \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{btf}}\right) + \frac{M_{lift}(x)}{S_{btf}} (1 - DIM)$$ Bottom fiber stress during lifting, impact decreasing dead load

$$BotLift1_{ie} = f_{bot.lift}(x_{e_{ie}}) \qquad BotLift1^T = (0.000 \quad 2.623 \quad 3.292 \quad 3.456 \quad 3.052 \quad 3.005)\text{ ksi}$$

$$BotLift2_{ie} = f_{bot.DIM.inc}(x_{e_{ie}}) \qquad BotLift2^T = (0.000 \quad 2.637 \quad 3.310 \quad 3.502 \quad 2.808 \quad 2.744)\text{ ksi}$$

$BotLift3_{ie} = f_{bot.DIM.dec}\left(x_{e_{ie}}\right)$ $\quad$ $BotLift3^{T} = (0.000 \;\; 2.609 \;\; 3.274 \;\; 3.410 \;\; 3.297 \;\; 3.267)\ \text{ksi}$

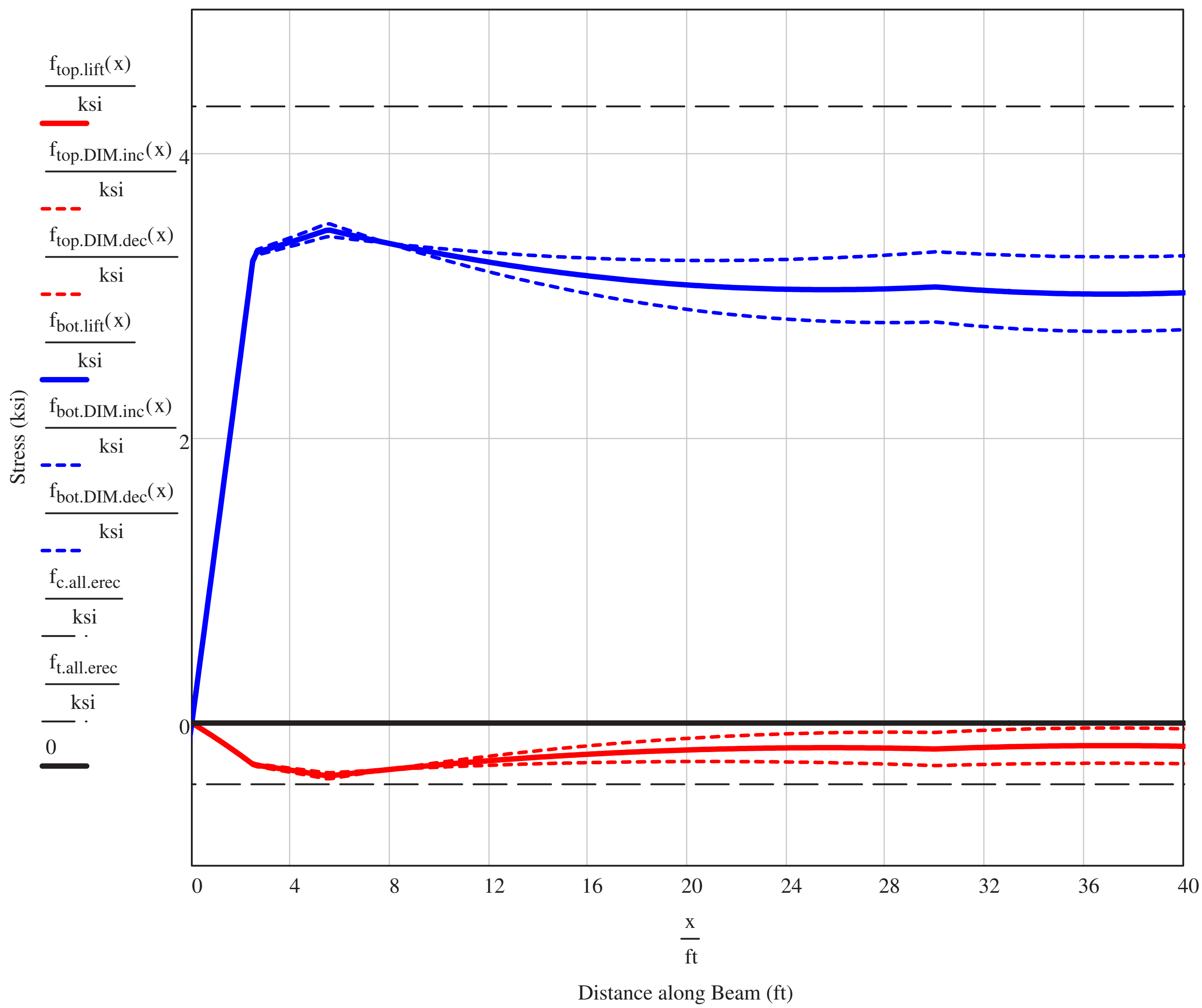
Allowable stresses during handling:

$f_{cm} = f_{c.erec}(f'_c) = 7.2\ \text{ksi}$ — Assumed concrete strength when handling operations begin

$f_{c.all.erec} = 0.6\, f_{cm} = 4.32\ \text{ksi}$ — Allowable compression during lifting and shipping

$f_{t.all.erec} = f_{t.erec}(f_{cm}) = -0.429\ \text{ksi}$ — Allowable tension during lifting and shipping

Stresses in Concrete During Lifting (Half Beam)



Compare beam stresses to allowable stresses.

$TopLiftMax_{ie} = \max\left(TopLift1_{ie}, TopLift2_{ie}, TopLift3_{ie}\right)$ $\quad$ $TopLiftMax^{T} = (0 \;\; -0.223 \;\; -0.285 \;\; -0.349 \;\; -0.065 \;\; -0.035)\ \text{ksi}$

$TopLiftMin_{ie} = \min\left(TopLift1_{ie}, TopLift2_{ie}, TopLift3_{ie}\right)$ $\quad$ $TopLiftMin^{T} = (0 \;\; -0.236 \;\; -0.302 \;\; -0.393 \;\; -0.296 \;\; -0.282)\ \text{ksi}$

$CheckTopLift = \text{if}\left[\left(\max(TopLiftMax) \le f_{c.all.erec}\right)\left(\min(TopLiftMin) \ge f_{t.all.erec}\right), \text{"OK"}, \text{"No Good"}\right] = \text{"OK"}$

$$\mathrm{BotLiftMax}_{ie} = \max\left(\mathrm{BotLift1}_{ie}, \mathrm{BotLift2}_{ie}, \mathrm{BotLift3}_{ie}\right) \qquad \mathrm{BotLiftMax}^{T} = (0 \quad 2.637 \quad 3.31 \quad 3.502 \quad 3.297 \quad 3.267)\ \mathrm{ksi}$$

$$\mathrm{BotLiftMin}_{ie} = \min\left(\mathrm{BotLift1}_{ie}, \mathrm{BotLift2}_{ie}, \mathrm{BotLift3}_{ie}\right) \qquad \mathrm{BotLiftMin}^{T} = (0 \quad 2.609 \quad 3.274 \quad 3.41 \quad 2.808 \quad 2.744)\ \mathrm{ksi}$$

$$\mathrm{CheckBotLift} = \mathrm{if}\left[\left(\max(\mathrm{BotLiftMax}) \le f_{c.all.erec}\right)\left(\min(\mathrm{BotLiftMin}) \ge f_{t.all.erec}\right), \text{"OK"}, \text{"No Good"}\right] = \text{"OK"}$$

Concrete Stresses at Final

Stresses at final are also computed using the design span length. Top flange compression and bottom flange tension are evaluated at the Service I and Service III limit states, respectively.

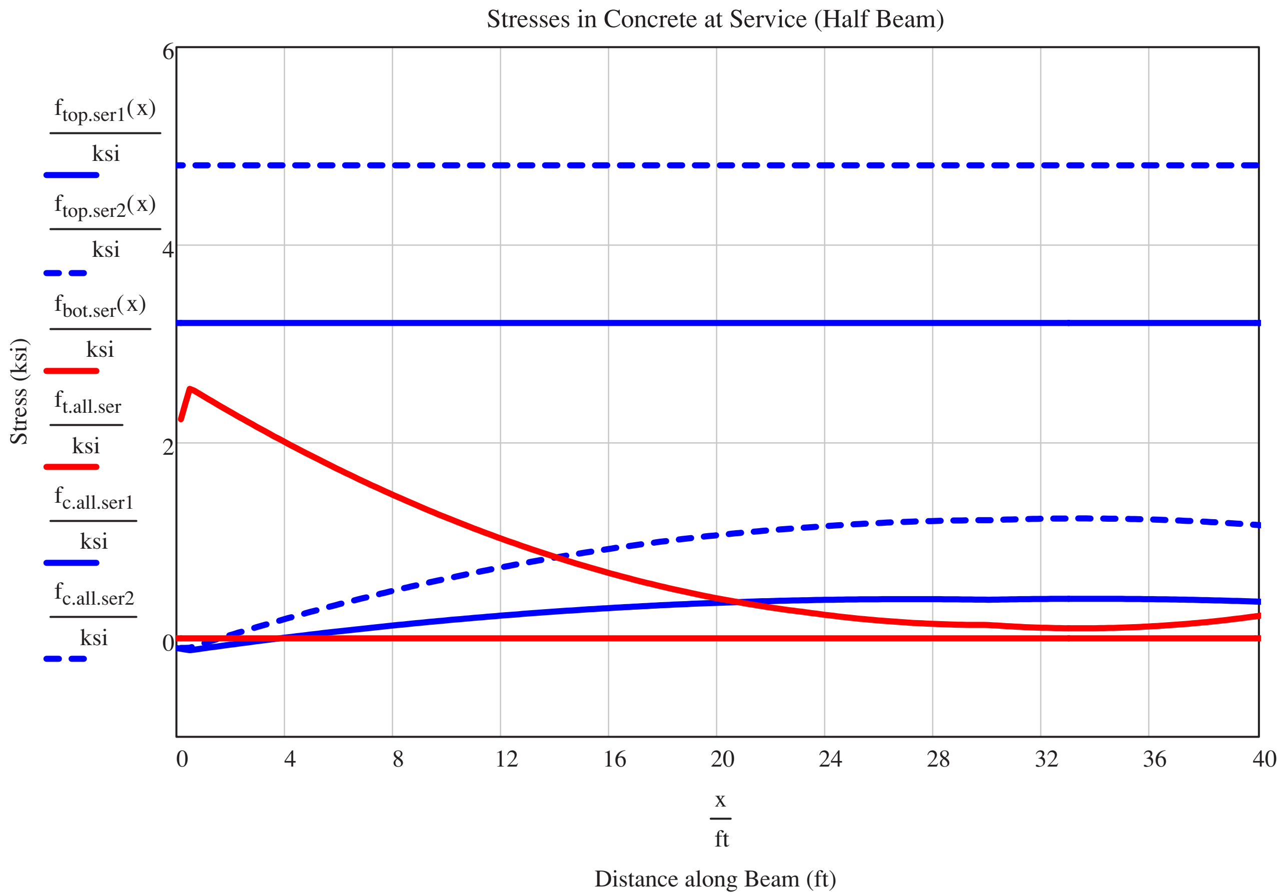
$f_{c.all.ser1} = 0.4\ f'_c = 3.2\ \mathrm{ksi}$	Allowable compression due to effective prestress and dead load (Table 5.9.4.2.1-1)
$f_{c.all.ser2} = 0.6\ f'_c = 4.8\ \mathrm{ksi}$	Allowable compression due to effective prestress, permanent load, and transient loads, as well as stresses during shipping and handling (Table 5.9.4.2.1-1)
$f_{t.all.ser} = 0\ \mathrm{ksi}$	Allowable tension (computed previously)
$P_e = f_{pe}\ A_{ps} = 880.4\ \mathrm{kip}$	Effective prestress after all losses

Compute stresses at midspan and compare to allowable values.

$$f_{top.ser1}(x) = \min\left(\frac{L_{end} + x}{L_t}, 1\right) P_e \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{ttf}}\right) + \frac{M_g\left(x + L_{end}\right)}{S_{tti}} + \frac{M_{bar}(x) + M_{fws}(x) + M_j(x)}{S_{ttf}}$$

$$f_{top.ser2}(x) = \min\left(\frac{L_{end} + x}{L_t}, 1\right) P_e \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{ttf}}\right) + \frac{M_g\left(x + L_{end}\right)}{S_{tti}} + \frac{M_{bar}(x) + M_{fws}(x) + M_j(x) + M_{ll}(x)}{S_{ttf}}$$

$$f_{bot.ser}(x) = \min\left(\frac{L_{end} + x}{L_t}, 1\right) P_e \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{btf}}\right) + \frac{M_g\left(x + L_{end}\right)}{S_{bti}} + \frac{M_{bar}(x) + M_{fws}(x) + M_j(x) + 0.8\ M_{ll}(x)}{S_{btf}}$$



Compare beam stresses to allowable stresses.

$$x_s = L \left(\frac{L_t}{L} \quad 0.1 \quad 0.15 \quad 0.2 \quad 0.25 \quad 0.3 \quad 0.35 \quad \alpha_{hd} \quad 0.45 \quad 0.5 \right)^T \qquad is = 1 .. \mathrm{last}(x_s)$$

$$TopSer1_{is} = f_{top.ser1}\left(x_{s_{is}}\right) \qquad TopSer1^T = (-0.046 \quad 0.101 \quad 0.195 \quad 0.272 \quad 0.330 \quad 0.370 \quad 0.393 \quad 0.397 \quad 0.398 \quad 0.400)\ ksi$$

$$TopSer2_{is} = f_{top.ser2}\left(x_{s_{is}}\right) \qquad TopSer2^T = (0.075 \quad 0.415 \quad 0.636 \quad 0.820 \quad 0.966 \quad 1.074 \quad 1.148 \quad 1.191 \quad 1.211 \quad 1.212)\ ksi$$

$$CheckCompSerI = \mathrm{if}\left[\left(\max(TopSer1) \le f_{c.all.ser1}\right)\left(\max(TopSer2) \le f_{c.all.ser2}\right), \text{"OK"}, \text{"No Good"}\right] = \text{"OK"}$$

$$BotSer_{is} = f_{bot.ser}\left(x_{s_{is}}\right) \qquad BotSer^T = (2.218 \quad 1.581 \quad 1.168 \quad 0.825 \quad 0.554 \quad 0.355 \quad 0.221 \quad 0.146 \quad 0.112 \quad 0.109)\ ksi$$

$$CheckTenSerIII = \mathrm{if}\left(\min(BotSer) \ge f_{t.all.ser}, \text{"OK"}, \text{"No Good"}\right) = \text{"OK"}$$

12. FLEXURAL STRENGTH

Verify flexural resistance at the Strength Limit State. Compute the factored moment at midspan due to the Strength I load combination, then compare it to the factored resistance calculated in accordance with AASHTO LRFD 5.7.3.

$M_{DC}(x) = M_g(x) + M_{bar}(x) + M_j(x)$ Self weight of components

$M_{DW}(x) = M_{fws}(x)$ Weight of future wearing surface

$M_{LL}(x) = M_{ll}(x)$ Live load

$M_{StrI}(x) = 1.25\ M_{DC}(x) + 1.5\ M_{DW}(x) + 1.75\ M_{LL}(x)$ Factored design moment

For minimum reinforcement check, per 5.7.3.3.2

$f_{cpe} = P_e\left(\frac{1}{A_g} + \frac{y_{cgp}}{S_{bg}}\right) = 3.677\ \text{ksi}$ Concrete compression at extreme fiber due to effective prestress

$M_{cr} = -\left(f_{r.cm} + f_{cpe}\right) S_{bg} = 2825\ \text{kip ft}$ Cracking moment (5.7.3.3.2-1)

$M_u(x) = \max\left(M_{StrI}(x), \min\left(1.33\ M_{StrI}(x), 1.2\ M_{cr}\right)\right)$ Design moment

Compute factored flexural resistance.

$\beta_1 = \max\left[0.65, 0.85 - 0.05\left(\frac{f'_c}{ksi} - 4\right)\right] = 0.65$ Stress block factor (5.7.2.2)

$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right) = 0.28$ Tendon type factor (5.7.3.1.1-2)

$d_p(x) = h - y_{px}\left(x + L_{end}\right)$ $d_p(X) = 37.421\ \text{in}$ Distance from compression fiber to prestress centroid

$h_f = d_7 = 8\ \text{in}$ Structural flange thickness

$b_{taper} = \frac{b_6 - b_5}{2} = 19.5\ \text{in}$ Average width of taper at bottom of flange

$h_{taper} = d_5 = 3\ \text{in}$ Depth of taper at bottom of flange

$a(x) = \frac{A_{ps}\ f_{pu}}{0.85\ f'_c\ b_f + \frac{k}{\beta_1} A_{ps}\left(\frac{f_{pu}}{d_p(x)}\right)}$ $a(X) = 2.509\ \text{in}$ Depth of equivalent stress block for rectangular section

$c(x) = \frac{a(x)}{\beta_1}$ $c(X) = 3.861\ \text{in}$ Neutral axis location

$CheckTC = \text{if}\left[\frac{c(X)}{d_p(X)} \le \left(\frac{.003}{.003 + .005}\right), \text{"OK"}, \text{"NG"}\right] = \text{"OK"}$ Tension-controlled section check (midspan)

$\varphi_f = \min\left[1.0, \max\left[0.75, 0.583 + 0.25\left(\frac{d_p(X)}{c(X)} - 1\right)\right]\right] = 1.00$ Resistance factor for prestressed concrete (5.5.4.2)

$$f_{ps} = f_{pu}\left(1 - k\,\frac{c(X)}{d_p(X)}\right) = 262.2\ \text{ksi}$$

Average stress in the prestressing steel (5.7.3.1.1-1)

$$L_d = \frac{1.6}{\text{ksi}}\left(f_{ps} - \frac{2}{3}\,f_{pe}\right) d_{ps} = 10.75\ \text{ft}$$

Bonded strand devlepment length (5.11.4.2-1)

$$f_{px}(x) = \left|\begin{array}{l} \dfrac{f_{pe}\left(x + L_{end}\right)}{L_t} \quad \text{if } x \le L_t - L_{end} \\ f_{pe} + \dfrac{\left(x + L_{end}\right) - L_t}{L_d - L_t}\left(f_{ps} - f_{pe}\right) \quad \text{if } L_t - L_{end} < x \le L_d - L_{end} \\ f_{ps} \quad \text{otherwise} \end{array}\right.$$

Stress in prestressing steel along the length for bonded strand (5.11.4.2)

$$M_r(x) = \varphi_f\left[A_{ps}\,f_{px}(x)\left(d_p(x) - \frac{a(x)}{2}\right)\right]$$

Flexure resistance along the length

$$x_{mom} = L\left(0.01 \quad \frac{L_t - L_{end}}{L} \quad \frac{L_d - L_{end}}{L} \quad \alpha_{hd} \quad 0.5\right)^T$$

$$imom = 1\,..\,\text{last}\left(x_{mom}\right)$$

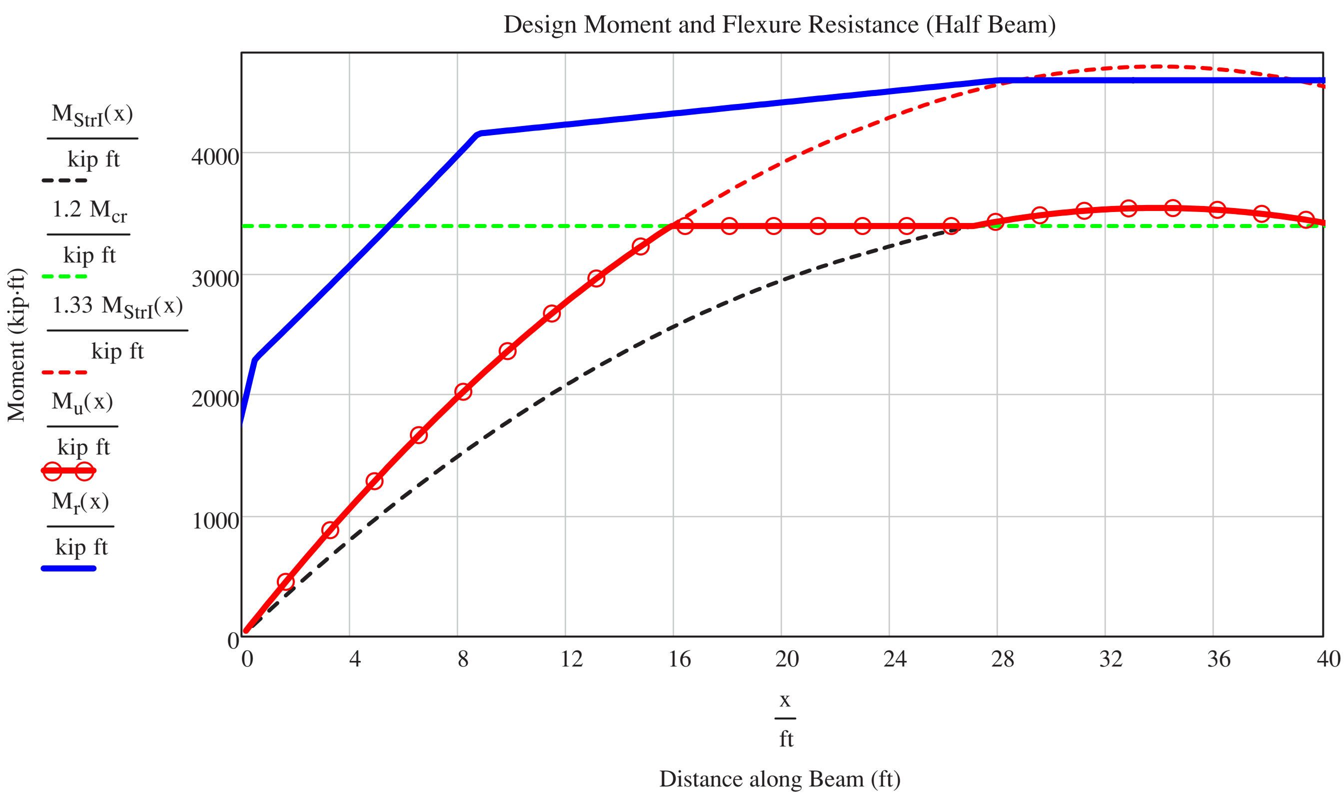
$$M_{rx_{imom}} = M_r\left(x_{mom_{imom}}\right) \qquad M_{ux_{imom}} = M_u\left(x_{mom_{imom}}\right)$$

$$DC_{mom} = \frac{M_{ux}}{M_{rx}} \qquad \max\left(DC_{mom}\right) = 0.769$$

Demand-Capacity ratio for moment

$$\text{CheckMom} = \text{if}\left(\max\left(DC_{mom}\right) \le 1.0, \text{"OK"}, \text{"No Good"}\right) = \text{"OK"}$$

Flexure resistance check



13. SHEAR STRENGTH

Shear Resistance

Compute the factored shear at the critical shear section and at tenth points along the span due to the Strength I load combination, then compare it to the factored resistance calculated in accordance with AASHTO LRFD 5.8.

$V_{DC}(x) = V_g(x) + V_{bar}(x) + V_j(x)$ Self weight of components

$V_{DW}(x) = V_{fws}(x)$ Weight of future wearing surface

$V_{LL}(x) = V_{ll}(x)$ Live load

$V_u(x) = 1.25\ V_{DC}(x) + 1.5\ V_{DW}(x) + 1.75\ V_{LL}(x)$ Factored design shear

$\varphi_v = 0.90$ Resistance factor for shear in normal weight concrete (AASHTO LRFD 5.5.4.2)

$d_{end} = h - y_{px}(L_{end}) = 32.368\ \text{in}$ Depth to steel centroid at bearing

$d_v = \min(0.9\ d_{end}, 0.72\ h) = 29.132\ \text{in}$ Effective shear depth lower limit at end

$$V_p(x) = \left| \begin{array}{l} P_e\ slope_{cgp}\ \dfrac{x + L_{end}}{L_t}\ \ \text{if}\ x \le L_t - L_{end} \\ P_e\ slope_{cgp}\ \ \text{if}\ L_t - L_{end} < x \le \alpha_{hd}\ L \\ 0\ \ \text{otherwise} \end{array} \right.$$

Vertical component of effective prestress force

$b_v = b_3 = 6\ \text{in}$ Web thickness

$$v_u(x) = \frac{\left|V_u(x) - \varphi_v\ V_p(x)\right|}{\varphi_v\ b_v\ d_v}$$

Shear stress on concrete (5.8.2.9-1)

$M_{ushr}(x) = \max\left(M_{StrI}(x), \left|V_u(x) - V_p(x)\right|\ d_v\right)$ Factored moment for shear

$f_{po} = 0.7\ f_{pu} = 189\ \text{ksi}$ Stress in prestressing steel due to locked-in strain after casting concrete

$$\varepsilon_s(x) = \max\left(-0.4\ 10^{-3}, \frac{\dfrac{\left|M_u(x)\right|}{d_v} + \left|V_u(x) - V_p(x)\right| - A_{ps}\ f_{po}}{E_p\ A_{ps}}\right)$$

Steel strain at the centroid of the prestressing steel

$$\beta(x) = \frac{4.8}{1 + 750\ \varepsilon_s(x)}$$

Shear resistance parameter

$\theta(x) = \left(29 + 3500\ \varepsilon_s(x)\right)\ \text{deg}$ Principal compressive stress angle

$$V_c(x) = 0.0316\ \text{ksi}\ \beta(x)\ \sqrt{\frac{f'_c}{\text{ksi}}}\ b_v\ d_v$$

Concrete contribution to total shear resistance

$\alpha = 90\ \text{deg}$ Angle of inclination of transverse reinforcement

$A_v = (1.02\ \ 0.62\ \ 0.62\ \ 0.62\ \ 0.31)^T\ in^2 \qquad s_v = (3\ \ 6\ \ 6\ \ 12\ \ 12)^T\ in$ Transverse reinforcement area and spacing provided

$x_v = (0\ \ 0.25\,h\ \ 1.5\,h\ \ 0.3\,L\ \ 0.5\,L\ \ 0.6\,L)^T \qquad x_v^{\,T} = (0\ \ 0.875\ \ 5.25\ \ 21\ \ 35\ \ 42)\ ft$

$$A_{vs}(x) = \left|\begin{array}{l} \text{for } i \in 1..\text{last}(A_v) \\ \quad out \leftarrow \dfrac{A_{v_i}}{s_{v_i}} \ \text{ if } x_{v_i} \le x \le x_{v_{i+1}} \\ out \end{array}\right.$$

$V_s(x) = A_{vs}(x)\, f_y\, d_v\, (\cot(\theta(x)) + \cot(\alpha))\, \sin(\alpha)$ Steel contribution to total shear resistance

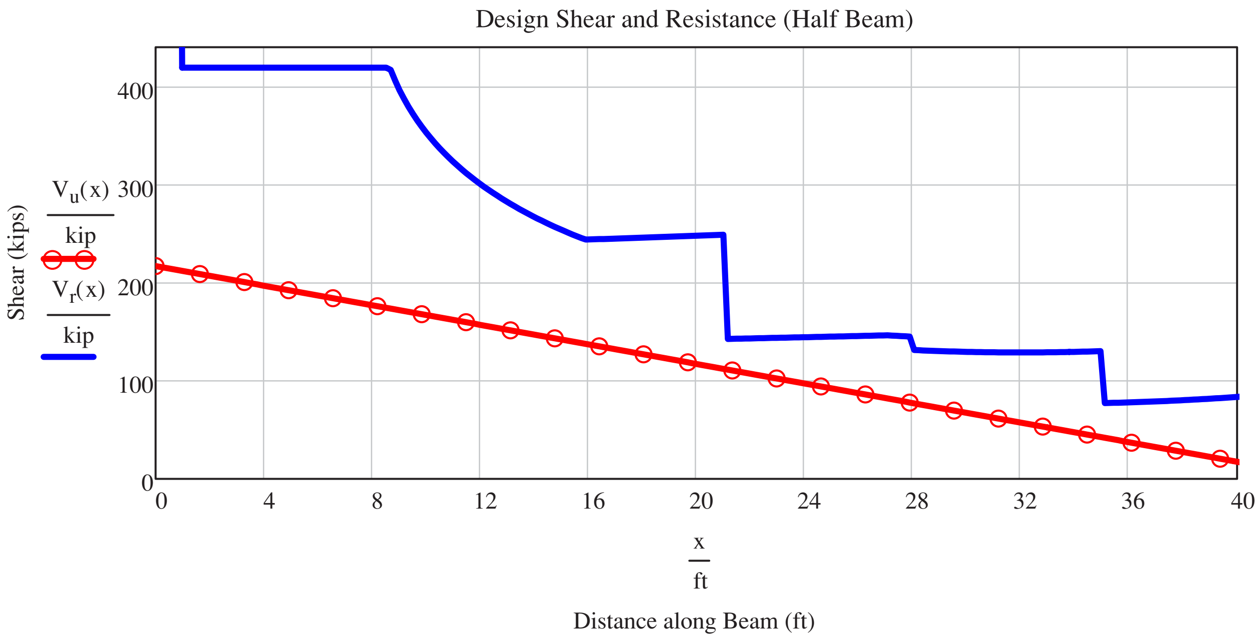
$V_r(x) = \varphi_v\left(V_c(x) + V_s(x) + V_p(x)\right)$ Factored shear resistance

$$x_{shr} = \left|\begin{array}{l} \text{for } i \in 1..100 \\ \quad out_i \leftarrow i\,\dfrac{0.5\,L}{100} \\ out \end{array}\right. \qquad ishr = 1..\text{last}(x_{shr})$$

$V_{ux_{ishr}} = V_u\left(x_{shr_{ishr}}\right) \qquad V_{rx_{ishr}} = V_r\left(x_{shr_{ishr}}\right)$

$DC_{shr} = \dfrac{V_{ux}}{V_{rx}} \qquad \max(DC_{shr}) = 0.787$

$CheckShear = \text{if}\left(\max(DC_{shr}) \le 1.0, \text{"OK"}, \text{"No Good"}\right) = \text{"OK"}$ Shear resistance check



Longitudinal Reinforcement

$$A_{l.req}(x) = \left| \begin{array}{l} a1 \leftarrow \dfrac{M_{StrI}(x)}{\varphi_f\, f_{px}(x)\left(d_p(x) - \dfrac{a(x)}{2}\right)} \\ a2 \leftarrow \dfrac{\left(\dfrac{V_u(x)}{\varphi_v} - 0.5\, V_s(x) - V_p(x)\right)\cot(\theta(x))}{f_{px}(x)} \\ a3 \leftarrow \dfrac{\dfrac{M_{ushr}(x)}{d_v\, \varphi_f} + \left(\left|\dfrac{V_u(x)}{\varphi_v} - V_p(x)\right| - 0.5\, V_s(x)\right)\cot(\theta(x))}{f_{px}(x)} \\ \min(a1, a2) \quad \text{if } x \le d_v + 5\text{ in} \\ \min(a1, a3) \quad \text{otherwise} \end{array} \right.$$

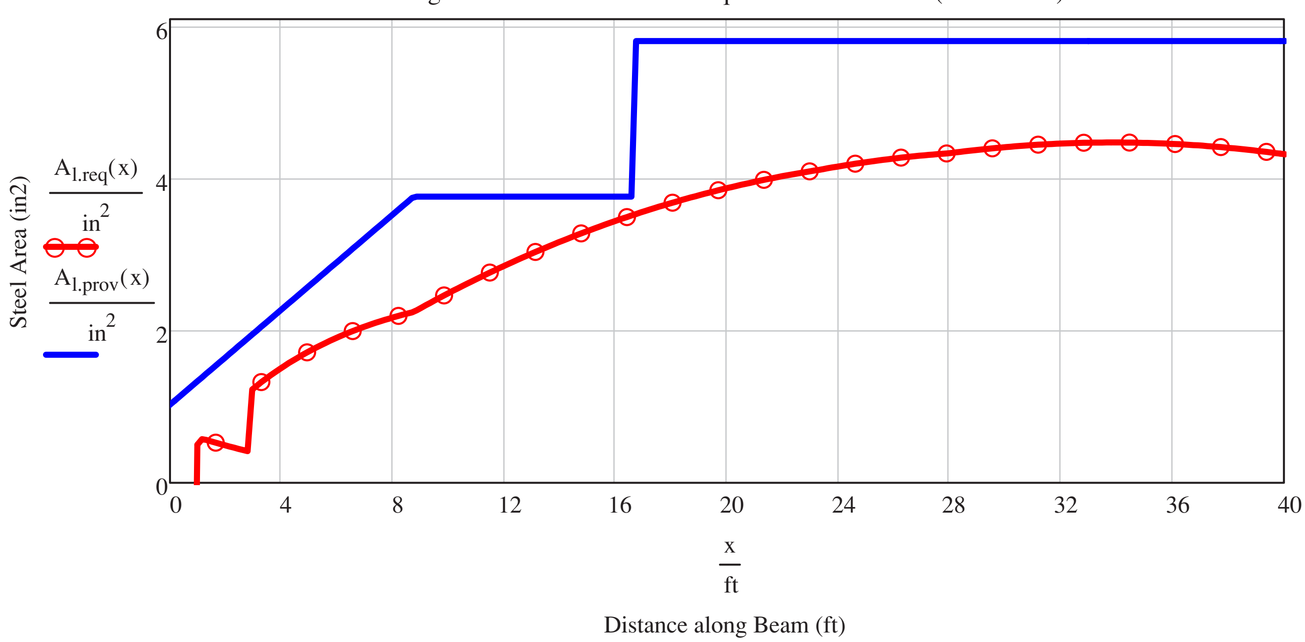
Longitudinal reinforcement required for shear (5.8.3.5)

$A_{s.add} = 0.40\text{ in}^2 \qquad L_{d.add} = 18.67\text{ ft}$

Additional longitudinal steel and developed length from end of beam

$$A_{l.prov}(x) = \text{if}\left(x < L_{d.add} - L_{end}, A_{s.add}, 0\right) + \left| \begin{array}{l} A_p\, N_s\, \dfrac{x + L_{end}}{L_d} \quad \text{if } x \le L_d - L_{end} \\ A_p\, N_s \quad \text{if } L_d - L_{end} < x \le \dfrac{y_{h.brg} - 0.5\, h}{slope_{cgp}} + \left(\dfrac{0.5\, N_h - 1}{2}\right)(2\text{ in})\cot\left(slope_{cgp}\right) \\ A_p\left(N_h + N_s\right) \quad \text{otherwise} \end{array} \right.$$

Longitudinal Reinforcement Required and Provided (Half Beam)



$A_{l.req_{ishr}} = A_{l.req}\left(x_{shr_{ishr}}\right) \qquad A_{l.prov_{ishr}} = A_{l.prov}\left(x_{shr_{ishr}}\right)$

$$DC_{long} = \frac{A_{l.req}}{A_{l.prov}} \qquad \max(DC_{long}) = 0.93$$

$$CheckLong = \text{if}\left(\max(DC_{long}) \le 1.0, \text{"OK"}, \text{"No Good"}\right) = \text{"OK"}$$ Longitudinal reinforcement check

14. SPLITTING RESISTANCE

Splitting Resistance

Checking splitting resistance provided by first zone of transverse reinforcement defined in the previous section for shear design.

$$A_s = \frac{A_{v_1} x_{v_2}}{s_{v_1}} = 3.57 \text{ in}^2$$

$f_s = 20 \text{ ksi}$ Limiting stress in steel for crack control (5.10.10.1)

$P_r = f_s A_s = 71.4 \text{ kip}$ Splitting resistance provided (5.10.10.1-1)

$P_{r.min} = 0.04 P_j = 47.1 \text{ kip}$ Minimum splitting resistance required

$CheckSplit = \text{if}\left(P_r \ge P_{r.min}, \text{"OK"}, \text{"No Good"}\right) = \text{"OK"}$ Splitting resistance check

15. CAMBER AND DEFLECTIONS

$$\Delta_{ps} = \frac{-P_i}{E_{ci} I_{xg}} \left[\frac{y_{cgp} L_g^2}{8} - \frac{(y_{bg} + y_{p.brg}) (\alpha_{hd} L + L_{end})^2}{6} \right] = 2.131 \text{ in}$$ Deflection due to prestress at release

$$\Delta_{gr} = \frac{-5}{384} \frac{w_g L_g^4}{E_{ci} I_{xg}} = -0.917 \text{ in}$$ Deflection due to self-weight at release

$$\Delta_{bar} = \frac{-5}{384} \frac{w_{bar} L_g^4}{E_c I_{xg}} = -0.263 \text{ in}$$ Deflection due to barrier weight

$$\Delta_j = \frac{-5}{384} \frac{w_j L^4}{E_c I_{xg}} \text{ if}(BeamLoc = 0, 1, 0.5) = -0.013 \text{ in}$$ Deflection due to longitudinal joint

$$\Delta_{fws} = \frac{-5}{384} \frac{w_{fws} L^4}{E_c I_{xg}} \text{ if}\left(BeamLoc = 0, 1, \frac{S - W_b}{S}\right) = -0.079 \text{ in}$$ Deflection due to future wearing surface

$t_{bar} = 20$ Age at which barrier is assumed to be cast

$$T = \begin{pmatrix} t_i & 7 & 14 & 21 & 28 & 60 & 120 & 240 & \infty \end{pmatrix}^T$$ Concrete ages at which camber is computed

$$\Delta_{cr1}(t) = \psi(t - t_i, t_i)(\Delta_{gr} + \Delta_{ps})$$

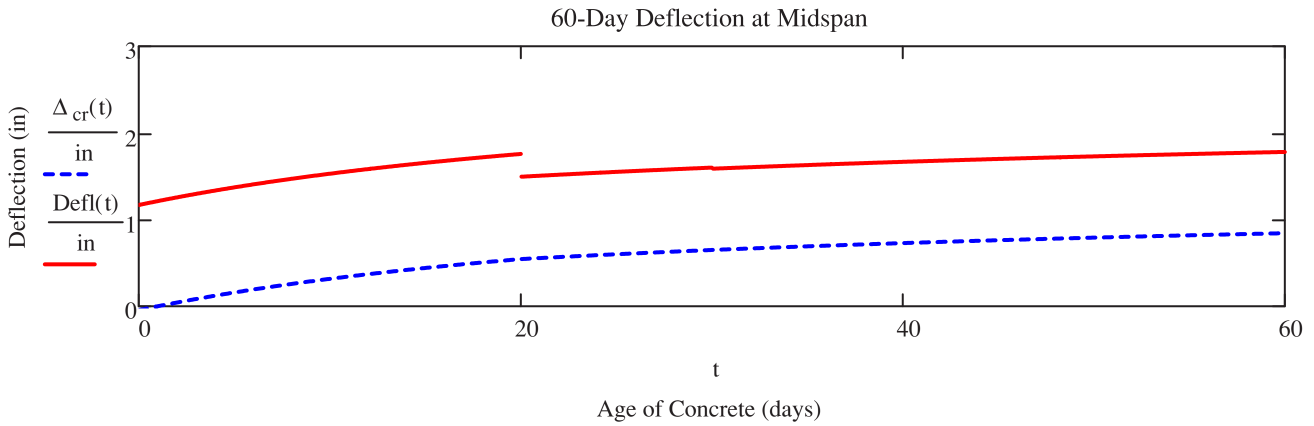
$$\Delta_{cr2}(t) = \left(\psi(t - t_i, t_i) - \psi(t_{bar} - t_i, t_i)\right) (\Delta_{gr} + \Delta_{ps}) + \psi(t - t_{bar}, t_{bar}) \Delta_{bar}$$

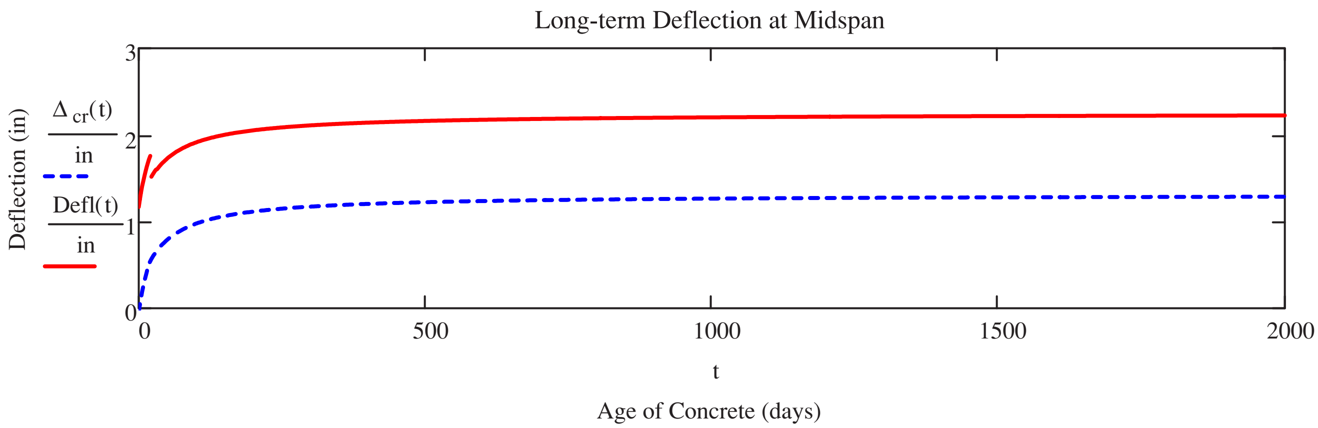
$$\Delta_{cr3}(t) = \left(\psi(t - t_i, t_i) - \psi(t_d - t_i, t_i)\right) (\Delta_{gr} + \Delta_{ps}) + \left(\psi(t - t_{bar}, t_{bar}) - \psi(t_d - t_{bar}, t_{bar})\right) \Delta_{bar} \ldots$$
$$+ \psi(t - t_d, t_d) (\Delta_j)$$

$$\Delta_{cr}(t) = \begin{vmatrix} \Delta_{cr1}(t) \quad \text{if } t \le t_{bar} \\ \Delta_{cr1}(t_{bar}) + \Delta_{cr2}(t) \quad \text{if } t_{bar} < t \le t_d \\ \Delta_{cr1}(t_{bar}) + \Delta_{cr2}(t_d) + \Delta_{cr3}(t) \quad \text{if } t > t_d \end{vmatrix}$$

$$\text{Defl}(t) = \begin{vmatrix} (\Delta_{gr} + \Delta_{ps}) + \Delta_{cr1}(t) \quad \text{if } t \le t_{bar} \\ (\Delta_{gr} + \Delta_{ps}) + \Delta_{cr1}(t_{bar}) + \Delta_{bar} + \Delta_{cr2}(t) \quad \text{if } t_{bar} < t \le t_d \\ (\Delta_{gr} + \Delta_{ps}) + \Delta_{cr1}(t_{bar}) + \Delta_{bar} + \Delta_{cr2}(t_d) + \Delta_j + \Delta_{cr3}(t) \quad \text{if } t > t_d \end{vmatrix}$$

$$C = \begin{vmatrix} \text{for } j \in 1 .. \text{last}(T) \\ \quad \text{out}_j \leftarrow \text{Defl}(T_j) \\ \text{out} \end{vmatrix} \qquad C^T = (1.213 \quad 1.439 \quad 1.632 \quad 1.506 \quad 1.581 \quad 1.78 \quad 1.955 \quad 2.081 \quad 2.247)\ \text{in}$$





16. NEGATIVE MOMENT FLEXURAL STRENGTH

Compute the factored moment to be resisted across the interior pier and determine the required reinforcing steel to be fully developed in the top flange.

Negative Live Load Moment

Compute the negative moment over the interior support due to the design live load load, in accordance with AASHTO LRFD 3.6.1.3.1.

$$\min\left(M_{truck}\right) = -889 \text{ kip ft}$$
Maximum negative moment due to a single truck

$$\min\left(M_{train}\right) = -1650 \text{ kip ft}$$
Maximum negative moment due to two trucks in a single lane

$$M_{neg.lane} = \frac{-w_{lane} L^2}{2} = -1568 \text{ kip ft}$$
Negative moment due to lane load on adjacent spans

$$M_{neg.truck} = M_{neg.lane} + (1 + IM) \min\left(M_{truck}\right) = -2750 \text{ kip ft}$$
Live load negative moment for single truck

$$M_{neg.train} = 0.9\left[M_{neg.lane} + (1 + IM) \min\left(M_{train}\right)\right] = -3387 \text{ kip ft}$$
Live load negative moment for two trucks in a single lane

$$M_{HL93.neg} = \min\left(M_{neg.truck}, M_{neg.train}\right) = -3387 \text{ kip ft}$$
Design negative live load moment, per design lane

$$M_{ll.neg.i} = M_{HL93.neg}\, g_{mint} = -2144 \text{ kip ft}$$
Design negative live load moment at interior beam

$$M_{ll.neg.e} = M_{HL93.neg}\, g_{mext} = -2233 \text{ kip ft}$$
Design negative live load moment at exterior beam

$$M_{LL.neg} = \text{if}\left(BeamLoc = 1, M_{ll.neg.e}, M_{ll.neg.i}\right) = -2233 \text{ kip ft}$$
Design negative live load moment

Factored Negative Design Moment

Dead load applied to the continuity section at interior supports is limited to the future overlay.

$$M_{DW.neg} = \frac{-w_{fws} L^2}{2} = -487 \text{ kip ft}$$
Superimposed dead load resisted by continuity section

$$M_{u.neg.StrI} = 1.5\, M_{DW.neg} + 1.75\, M_{LL.neg} = -4638 \text{ kip ft}$$
Strength Limit State

$$M_{u.neg.StrI} = 1.0\, M_{DW.neg} + 1.0\, M_{LL.neg} = -2720 \text{ kip ft}$$
Service Limit State

Reinforcing Steel Requirement in the Top Flange for Strength

$\varphi_f = 0.90$ — Reduction factor for strength in tension-controlled reinforced concrete (5.5.4.2)

$b_c = b_1 = 26\ \text{in}$ — Width of compression block at bottom flange

$d_{nms} = h - t_{sac} - 0.5\left(t_{flange} - t_{sac}\right) = 37\ \text{in}$ — Distance to centroid of negative moment steel, taken at mid-depth of top flange

$R_u = \dfrac{\left|M_{u.neg.StrI}\right|}{\varphi_f\ b_c\ d_{nms}^{\ 2}} = 1.019\ \text{ksi}$ — Factored load, in terms of stress in concrete at depth of steel, for computing steel requirement

$m = \dfrac{f_y}{0.85\ f'_c} = 8.824$ — Steel-to-concrete strength ratio

$\rho_{req} = \dfrac{1}{m}\left(1 - \sqrt{1 - \dfrac{2\ m\ R_u}{f_y}}\right) = 0.0185$ — Required negative moment steel ratio

$A_{nms.req} = \rho_{req}\ b_c\ d_{nms} = 17.787\ \text{in}^2$ — Required negative moment steel in top flange

$A_{s.long.t} = 2.0\ \text{in}^2 \qquad A_{s.long.b} = 2.0\ \text{in}^2$ — Full-length longitudinal reinforcement to be made continuous across joint

$A_{bar} = 0.44\ \text{in}^2$ — Additional negative moment reinforcing bar area

$A_{nms.t} = \dfrac{2}{3}\ A_{nms.req} - A_{s.long.t} = 9.858\ \text{in}^2$ — Additional reinforcement area required in the top mat (2/3 of total)

$n_{bar.t} = \text{ceil}\left(\dfrac{A_{nms.t}}{A_{bar}}\right) = 23$ — Additional bars required in the top mat

$A_{nms.b} = \dfrac{1}{3}\ A_{nms.req} - A_{s.long.b} = 3.929\ \text{in}^2$ — Additional reinforcement area required in the bottom mat

$n_{bar.b} = \text{ceil}\left(\dfrac{A_{nms.b}}{A_{bar}}\right) = 9$ — Additional bars required in the top mat

$s_{bar.top} = \dfrac{S - W_j - 6\ \text{in}}{n_{bar.t} - 1} = 3.788\ \text{in}$ — Spacing of bars in top mat

$A_{s.nms} = \left(n_{bar.t} + n_{bar.b}\right)\ A_{bar} + A_{s.long.t} + A_{s.long.b} = 18.08\ \text{in}^2$ — Total reinforcing steel provided over pier

$a = \dfrac{A_{s.nms}\ f_y}{0.85\ f'_c\ b_c} = 6.136\ \text{in}$ — Depth of compression block

$M_{r.neg} = \varphi_f\ A_{s.nms}\ f_y\left(d_{nms} - \dfrac{a}{2}\right) = 2761\ \text{kip ft}$ — Factored flexural resistance at interior pier

$$DC_{neg.mom} = \frac{\left|M_{u.neg.StrI}\right|}{M_{r.neg}} = 0.985$$

$\text{CheckNegMom} = \text{if}\left(DC_{neg.mom} \le 1.0, \text{"OK"}, \text{"No Good"}\right) = \text{"OK"}$ — Negative flexure resistance check

ABC SAMPLE CALCULATION – 3a

Precast Pier Design for ABC (70’ Span Straddle Bent)

PRECAST PIER DESIGN FOR ABC (70' SPAN STRADDLE BENT)

FNofBm = Total Number of Beams in Forward Span

FSpan = Forward Span Length

FDeckW = Out to Out Forward Span Deck Width FBmAg

= Forward Span Beam X Sectional Area FBmFlange =

Forward Span Beam Top Flange Width FHaunch =

Forward Span Haunch Thickness

FBmD = Forward Span Beam Depth or Height

FBmIg = Forward Span Beam Moment of Inertia

y_{Ft} = Forward Span Beam Top Distance from cg SlabTh =

Slab Thickness

RailWt = Railing Weight

RailH = Railing Height

RailW = Rail Base Width

LeftOH = Left Overhang Distance

RightOH = Right Overhang Distance

DeckW = Out to Out Deck Width at Bent

RoadW = Roadway Width

BrgTh = Bearing Pad Thickness +Bearing Seat Thickness

NofLane = Number of Lanes

wCap = Cap Width

hCap = Cap Depth

CapL = Cap Length

wFoam = Width of Foam for Blockout

hFoam = height of Foam for Blockout

LFoam = Length of Foam for Blockout

BNofBm = Total Number of Beams in Backward Span

BSpan = Backward Span Length

BDeckW = Out to Out Backward Span Deck Width

BBmAg = Backward Span Beam X Sectional Area

BBmFlange = Backward Span Beam Top Flange Width

BHaunch = Backward Span Haunch Thickness

BBmD = Backward Span Beam Depth or Height

BBmIg = Backward Span Beam Moment of Inertia

y_{Bt} = Backward Span Beam Top Distance from cg

NofCol = Number of Columns per Bents

NofDs = Number of Drilled Shaft per Bents

wCol = Width of Column Section

bCol = Breadth of Column Section

DsDia = Drilled Shaft Diameter

HCol = Height of Column

wEarWall = Width of Ear Wall

hEarWall = Height of Ear Wall

tEarWall = Thickness of Ear Wall

tSWalk = Thickness of Side Walk

bSWalk = Breadth of Side Walk

BmMat = Beam Material either Steel or Concrete

h_{bS} = Bottom Solid Height at Foam

h_{tS} = Top Solid Height at Foam

γ_{st} = Unit Weight of Steel

γ_c, w_c = Unit Weight of Concrete

$SlabDC_{Int}$ = Dead Load for Slab per Interior Beam

$SlabDC_{Ext}$ = Dead Load for Slab per Exterior Beam

BeamDC = Self Weight of Beam

HaunchDC = Dead Load of Haunch Concrete per Beam

RailDC = Weight of Rail per Beam

$FSuperDC_{Int}$ = Half of Forward Span Super Structure Dead Load Component per Interior Beam

$FSuperDC_{Ext}$ = Half of Forward Span Super Structure Dead Load Component per Exterior Beam

FSuperDW = Half of Forward Span Overlay Dead Load Component per Beam

$BSuperDC_{Int}$ = Half of Backward Span Super Structure Dead Load Component per Interior Beam

$BSuperDC_{Ext}$ = Half of Backward Span Super Structure Dead Load Component per Exterior Beam

BSuperDW = Half of Backward Span Overlay Dead Load Component per Beam

$TorsionDC_{Int}$ = DeadLoad Torsion in a Cap due to difference in Forward and Backward span length per Interior Beam

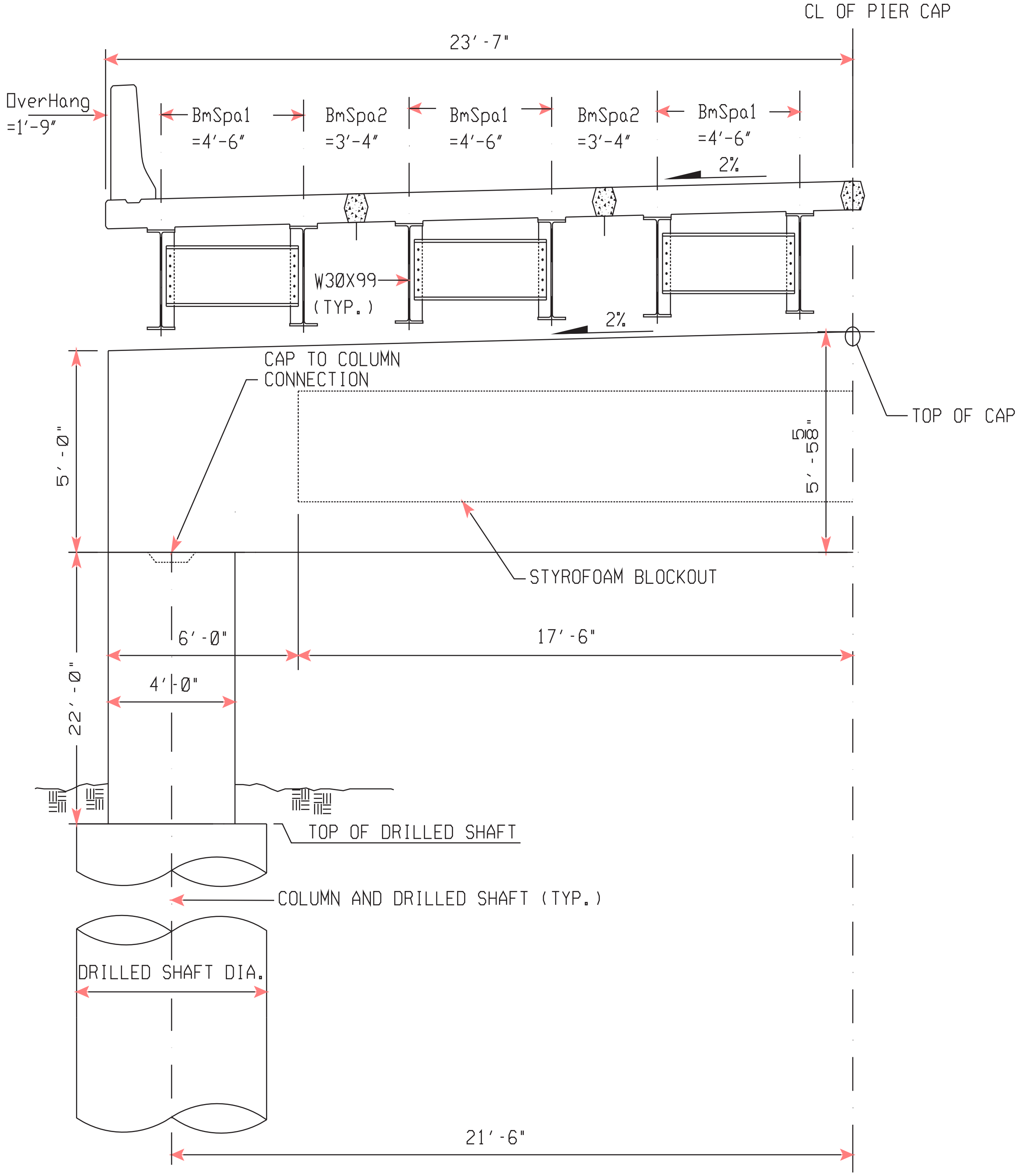
$TorsionDC_{Ext}$ = DeadLoad Torsion in a Cap due to difference in Forward and Backward span length per Exterior Beam

TorsionDW = DW Torsion in a Cap due to difference in Forward and Backward span length per Beam

DiapWt = Weight of Diaphragm

tBrgSeat = Thickness of Bearing Seat

bBrgSeat = Breadth of Bearing Seat



TYPICAL SECTION & PIER ELEVATION

Note: Use of Light-Weight-Concrete (LWC) may be considered to reduce the weight of the pier cap instead of styrofoam blockouts.

FORWARD SPAN PARAMETER INPUT:

$FNofBm = 12$	$FSpan = 70\ ft$	$FDeckW = \frac{283}{6}\ ft$	$FBmAg = 29.1\ in^2$	$FBmFlange = 10.5\ in$
$FHaunch = 0\ in$	$FBmD = 29.7\ in$	$FBmIg = 3990\ in^4$	$y_{Ft} = 14.85\ in$	

BACKWARD SPAN PARAMETER INPUT:

$BNofBm = 12$	$BSpan = 70\ ft$	$BDeckW = \frac{283}{6}\ ft$	$BBmAg = 29.1\ in^2$	$BBmFlange = 10.5\ in$
$BHaunch = 0\ in$	$BBmD = 29.7\ in$	$BBmIg = 3990\ in^4$	$y_{Bt} = 14.85\ in$	

COMMON BRIDGE PARAMETER INPUT: Intermediate Bent between Forward and Backward span Parameters

$SlabTh = 9\ in$	$Overlay = 25\ psf$	$\theta = 0\ deg$	$DeckOH = 1.75\ ft$	$BrgTh = 3.5\ in$
$RailWt = 0.43\ klf$	$RailW = 19\ in$	$RailH = 34.0\ in$	$tBrgSeat = 0\ in$	$bBrgSeat = 0\ ft$
$DeckW = \frac{283}{6}\ ft$	$NofLane = 3$	$m = 0.85$	$w_c = 0.150\ kcf$	$f'_c = 5\ ksi$ (Cap)
$wCap = 4.5\ ft$	$hCap = 5\ ft$	$CapL = 47\ ft$	$NofDs = 2$	$DsDia = 6\ ft$
$wCol = 4\ ft$	$bCol = 4\ ft$	$NofCol = 2$	$HCol = 22.00\ ft$	$f'_{cs} = 4\ ksi$ (Slab)
$\gamma_c = 0.150\ kcf$	$e_{brg} = 13\ in$	$NofBm = 12$	$Sta = 0.25\ \frac{ft}{incr}$	$DiapWt = 0.2\ kip$
$wEarWall = 0\ ft$	$hEarWall = 0\ ft$	$tEarWall = 0\ in$	$IM = 0.33$	$BmMat = Steel$
$LFoam = 35\ ft$	$wFoam = 14\ in$	$hFoam = 31\ in$	$h_{bS} = 15\ in$ (Bottom Solid Depth of Section)	
$E_s = 29000\ ksi$	$\gamma_{st} = 490\ pcf$ (steel)			

Modulus of elasticity of Concrete:

$E(f'_c) = 33000\,(w_c)^{1.5}\sqrt{f'_c\ ksi}$ (AASHTO LRFD EQ 5.4.2.4-1 for $K_1 = 1$)

$E_{slab} = E(f'_{cs})$ $\qquad E_{slab} = 3834.254\ ksi$

$E_{cap} = E(f'_c)$ $\qquad E_{cap} = 4286.826\ ksi$

Modulus of Beam or Girder: Input Beam Material, BmMat = Steel or Concrete

$E_{beam} = \text{if}(BmMat = Steel, E_s, E(f'_c))$ $\qquad E_{beam} = 29000\ ksi$

1. BENT CAP LOADING

DEAD LOAD FROM SUPERSTRUCTURE:

The permanent dead load components (DC) consist of slab, rail, sidewalk, haunch weight, and beam self weight. Slab dead weight components will be distributed to each beam by slab tributary width between beams. Interior Beam tributary width (IntBmTriW) is taken as the average of consecutive beam spacing for a particular interior beam. Exterior Beam tributary width (ExtBmTriW) is taken as half of beam spacing plus the overhang distance. Rail, sidewalk dead load components and future wearing surface weight components (DW) can be distributed evenly among each beam. Half of DC and DW components from forward span and backward span comprise the total superstructure load or dead load reaction per beam on the pier cap or the bent cap.

FORWARD SPAN SUPERSTRUCTURE DEAD LOAD: consists of 12 W30x99 Beams

12 beams were spaced 4.5' and 3'-4" alternately in forward span. For beam spacing see Typical Section Details sheet

$$FBmSpa1 = 4.5\ ft \qquad FBmSpa2 = \frac{10}{3}\ ft$$

$$FIntBmTriW = \frac{FBmSpa1}{2} + \frac{FBmSpa2}{2} \qquad FIntBmTriW = 3.917\ ft$$

$$FExtBmTriW = \frac{FBmSpa1}{2} + DeckOH \qquad FExtBmTriW = 4\ ft$$

$$RoadW = 0.25\ (FDeckW + 3\ DeckW) - 2\ RailW \qquad RoadW = 44\ ft$$

$$SlabDC_{Int} = \gamma_c\ FIntBmTriW\ SlabTh\left(\frac{FSpan}{2}\right) \qquad SlabDC_{Int} = 15.422\ \frac{kip}{beam}$$

$$SlabDC_{Ext} = \gamma_c\ FExtBmTriW\ SlabTh\left(\frac{FSpan}{2}\right) \qquad SlabDC_{Ext} = 15.75\ \frac{kip}{beam}$$

$$BeamDC = \gamma_{st}\ FBmAg\left(\frac{FSpan}{2}\right) \qquad BeamDC = 3.466\ \frac{kip}{beam}$$

$$HaunchDC = \gamma_c\ FHaunch\ FBmFlange\left(\frac{FSpan}{2}\right) \qquad HaunchDC = 0\ \frac{kip}{beam}$$

NOTE: Permanent loads such as the weight of the Rail (Barrier), Future wearing surface may be distributed uniformly among all beams if following conditions are met. Apply for live load distribution factors too. AASHTO LRFD 4.6.2.2.1

1. Width of deck is constant
2. Number of Beams >= 4 beams
3. Beams are parallel and have approximately same stiffness
4. The Roadway part of the overhang, d_e<= 3ft
5. Curvature in plan is < 4º
6. Bridge cross-section is consistent with one of the x-section shown in AASHTO LRFD TABLE 4.6.2.2.1-1

$$RailDC = \frac{2\ RailWt}{FNofBm}\left(\frac{FSpan}{2}\right) \qquad RailDC = 2.508\ \frac{kip}{beam}$$

$$OverlayDW = \frac{RoadW\ Overlay}{FNofBm}\left(\frac{FSpan}{2}\right) \qquad OverlayDW = 3.208\ \frac{kip}{beam}$$

Forward Span Superstructure DC & DW per Interior and Exterior Beam:

$$FSuperDC_{Int} = RailDC + BeamDC + SlabDC_{Int} + HaunchDC + DiapWt \qquad FSuperDC_{Int} = 21.596\ \frac{kip}{beam}$$

$$FSuperDC_{Ext} = RailDC + BeamDC + SlabDC_{Ext} + HaunchDC + 0.5\ DiapWt \qquad FSuperDC_{Ext} = 21.824\ \frac{kip}{beam}$$

$$FSuperDW = OverlayDW \qquad FSuperDW = 3.208\ \frac{kip}{beam}$$

BACKWARD SPAN SUPERSTRUCTURE DEAD LOAD: consists of 12 W30x99 beams

12 beams were spaced 4.5' and 3'-4" alternately in backward span. For beam spacing see Typical Section Details sheet

$$BBmSpa1 = 4.5\ ft \qquad BBmSpa2 = \frac{10}{3}\ ft$$

$$BIntBmTriW = \frac{BBmSpa1}{2} + \frac{BBmSpa2}{2} \qquad BIntBmTriW = 3.917\ ft$$

$$BExtBmTriW = \frac{BBmSpa1}{2} + DeckOH \qquad BExtBmTriW = 4\ ft$$

$$RoadW = 0.25\ (BDeckW + 3\ DeckW) - 2\ RailW \qquad RoadW = 44\ ft$$

$$SlabDC_{Int} = \gamma_c\ BIntBmTriW\ SlabTh\left(\frac{BSpan}{2}\right) \qquad SlabDC_{Int} = 15.422\ \frac{kip}{beam}$$

$$SlabDC_{Ext} = \gamma_c\ BExtBmTriW\ SlabTh\left(\frac{BSpan}{2}\right) \qquad SlabDC_{Ext} = 15.75\ \frac{kip}{beam}$$

$$BeamDC = \gamma_{st}\ BBmAg\left(\frac{BSpan}{2}\right) \qquad BeamDC = 3.466\ \frac{kip}{beam}$$

$$HaunchDC = \gamma_c\ BHaunch\ BBmFlange\left(\frac{BSpan}{2}\right) \qquad HaunchDC = 0\ \frac{kip}{beam}$$

$$RailDC = \frac{2\ RailWt}{BNofBm}\left(\frac{BSpan}{2}\right) \qquad RailDC = 2.508\ \frac{kip}{beam}$$

$$OverlayDW = \frac{RoadW\ Overlay}{BNofBm}\left(\frac{BSpan}{2}\right) \qquad OverlayDW = 3.208\ \frac{kip}{beam}$$

Total Backward Span Superstructure DC & DW per Interior and Exterior Beam:

$$BSuperDC_{Int} = RailDC + BeamDC + SlabDC_{Int} + HaunchDC + DiapWt \qquad BSuperDC_{Int} = 21.596\ \frac{kip}{beam}$$

$$BSuperDC_{Ext} = RailDC + BeamDC + SlabDC_{Ext} + HaunchDC + 0.5\ DiapWt \qquad BSuperDC_{Ext} = 21.824\ \frac{kip}{beam}$$

$$BSuperDW = OverlayDW \qquad BSuperDW = 3.208\ \frac{kip}{beam}$$

Total Superstructure DC & DW per Beam on Bent Cap:

$SuperDC_{Int} = FSuperDC_{Int} + BSuperDC_{Int}$ $\qquad SuperDC_{Int} = 43.192 \, \frac{kip}{beam}$

$SuperDC_{Ext} = FSuperDC_{Ext} + BSuperDC_{Ext}$ $\qquad SuperDC_{Ext} = 43.648 \, \frac{kip}{beam}$

$SuperDW = FSuperDW + BSuperDW$ $\qquad SuperDW = 6.417 \, \frac{kip}{beam}$

$TorsionDC_{Int} = \left(\max\left(FSuperDC_{Int}, BSuperDC_{Int}\right) - \min\left(FSuperDC_{Int}, BSuperDC_{Int}\right)\right) e_{brg}$ $\qquad TorsionDC_{Int} = 0 \, \frac{kft}{beam}$

$TorsionDC_{Ext} = \left(\max\left(FSuperDC_{Ext}, BSuperDC_{Ext}\right) - \min\left(FSuperDC_{Ext}, BSuperDC_{Ext}\right)\right) e_{b}$ $\qquad TorsionDC_{Ext} = 0 \, \frac{kft}{beam}$

$TorsionDW = (\max(FSuperDW, BSuperDW) - \min(FSuperDW, BSuperDW)) \, e_{brg}$ $\qquad TorsionDW = 0 \, \frac{kft}{beam}$

CAP, EAR WALL & BEARING SEAT WEIGHT:

The Bent cap has two sections along the length. One is a solid rectangular section 6ft from both ends. The middle section is made hollow by placing foam blockouts in two sides of mid section as can be seen in the typical section and pier elevation figure. CapDC1 is the weight of the solid section and CapDC2 is the weight of the hollow section.

$CapDC1 = wCap \; hCap \; \gamma_c$ Applicable for $(0 \text{ ft} \le CapL \le 6 \text{ ft}), (41 \text{ ft} \le CapL \le 47 \text{ ft})$ $\qquad CapDC1 = 3.375 \, \frac{kip}{ft}$

$CapDC2 = (wCap \; hCap - 2 \; wFoam \; hFoam) \, \gamma_c$ Applicable for $(6 \text{ ft} \le CapL \le 41 \text{ ft})$ $\qquad CapDC2 = 2.471 \, \frac{kip}{ft}$

$EarWallDC = (wEarWall \; hEarWall \; tEarWall) \, \gamma_c$ $\qquad EarWallDC = 0 \text{ kip}$

$BrgSeatDC = tBrgSeat \; bBrgSeat \; (wCap) \, \gamma_c$ $\qquad BrgSeatDC = 0 \, \frac{kip}{beam}$

RESULTS OF DISTRIBUTION FACTORS:

Forward Span Distribution Factors:

$DFM_{Fmax} = 0.391$ (Distribution Factor for Moment)

$DFS_{Fmax} = 0.558$ (Distribution Factor for Shear)

Backward Span Distribution Factors:

$DFM_{Bmax} = 0.391$ (Distribution Factor for Moment)

$DFS_{Bmax} = 0.558$ (Distribution Factor for Shear)

LIVE LOAD FOR SIMPLY SUPPORTED BRIDGE:

HL-93 Loading: According to AASHTO LRFD 3.6.1.2.1, HL-93 consists of Design Truck + Design Lane Load or Design Tandem + Design Lane Load. Design Truck rather than Design Tandem + Design Lane Load controls the maximum Live Load Reactions at an interior bent for a span longer than 26'. For maximum reaction, place middle axle (P_2 = 32 kip) of design truck over the support at a bent between the forward and the backward span and place rear axle (P_3 = 32 kip) 14' away from P_2 on the longer span while placing P_1 14' away from P_1 on either spans yielding maximum value.

P_1 **=** Front Axle of Design Truck P_2 **=** Middle Axle of Design Truck P_3 **=** Rear Axle of Design Truck

Design Truck Axle Load: $P_1 = 8\text{ kip}$ $P_2 = 32\text{ kip}$ $P_3 = 32\text{ kip}$ (AASHTO LRFD 3.6.1.2.2) $TruckT = P_1 + P_2 + P_3$

Design Lane Load: $w_{lane} = 0.64\text{ klf}$ (AASHTO LRFD 3.6.1.2.4)

$LongSpan = \max(FSpan, BSpan)$ $ShortSpan = \min(FSpan, BSpan)$

$L_{long} = LongSpan$ $L_{short} = ShortSpan$

Lane Load Reaction

$$Lane = w_{lane}\left(\frac{L_{long} + L_{short}}{2}\right) \qquad Lane = 44.8\ \frac{kip}{lane}$$

Truck Load Reaction

$$Truck = P_2 + P_3\,\frac{(L_{long} - 14ft)}{L_{long}} + P_1\,\max\left[\frac{(L_{long} - 28ft)}{L_{long}}, \frac{(L_{short} - 14ft)}{L_{short}}\right] \qquad Truck = 64\ \frac{kip}{lane}$$

Maximum Live Load Reaction with Impact (LLRxn) over support on Bent:

The Dynamic Load Allowance or Impact Factor, $IM = 0.33$ (AASHTO LRFD Table 3.6.2.1 − 1)

$$LLRxn = Lane + Truck\ (1 + IM) \qquad LLRxn = 129.92\ \frac{kip}{lane}$$

Live Load Model for Cap Loading Program:

AASHTO LRFD Recommended Live Load Model For Cap Loading Program: Live Load reaction on the pier cap using distribution factors are not sufficient to design bent cap for moment and shear. Therefore, the reaction from live load is uniformly distributed to over a 10' width (which becomes W) and the reaction from the truck is applied as two concentrated loads (P and P) 6' apart. The loads act within a 12' wide traffic lane. The reaction W and the truck move across the width of the traffic lane. However, neither of the P loads can be placed closer than 2' from the edge of the traffic lane. One lane, two lanes, three lanes and so forth loaded traffic can be moved across the width of the roadway to create maximum load effects.

Load on one rear wheel out of rear axle of the truck with Impact:

$$P = (0.5\,P_3)\,(1 + IM) \qquad P = 21.28\text{ kip}$$

The Design Lane Load Width Transversely in a Lane

$wlaneTransW = 10\text{ ft}$ AASHTO LRFD Article 3.6.1.2.1

The uniform load portion of the Live Load, kip/station for Cap Loading Program:

$$W = \frac{(LLRxn - 2\,P)\ Sta}{wlaneTransW} \qquad W = 2.184\ \frac{kip}{incr}$$

LOADS generated above will be placed into a CAP LOADING PROGRAM to obtain moment and shear values for Bent Cap.

Torsion on Bent Cap per Beam and per Drilled Shaft:

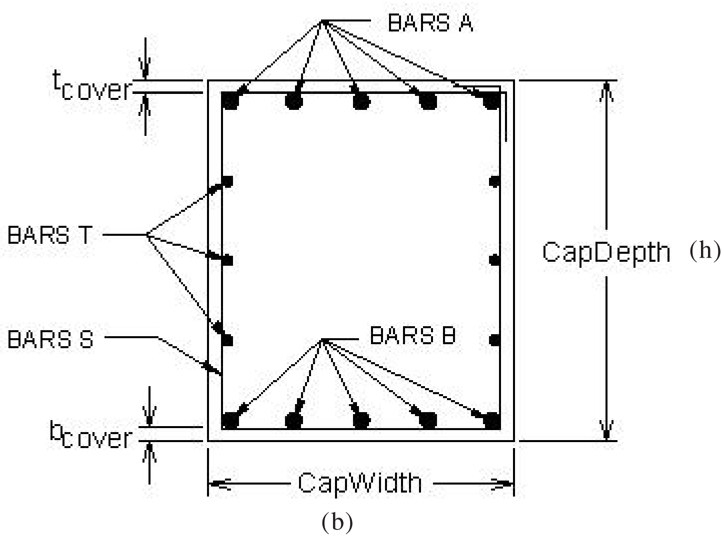
Torsional load about center line of bent cap occurs due to horizontal loads acting on the superstructure perpendicular to the bent length or along the bridge length. Braking force, Centrifugal force, WS on superstructure, and WL cause torsion on bent.

In addition, torque about center line of bent cap for the dead load reaction on beam brg location occurs due to differences in forward and backward span length and eccentricity between center line of bent cap and brg location. Torsion can be neglected if $T_u < 0.25 \phi T_{cr}$ (AASHTO LRFD 5.8.2.1).

The maximum torsional effects on the pier cap will be obtained from RISA frame analysis under loading as stated in AASHTO LRFD SECTION 3 for different load combinations using AASHTO LRFD Table 3.4.1-1.

2. BENT CAP FLEXURAL DESIGN

FLEXURAL DESIGN OF BENT CAP:



$f'_c = 5.0\ \text{ksi}$ $\quad f_y = 60\ \text{ksi}$ $\quad E_s = 29000\ \text{ksi}$ $\quad \phi_m = 0.9$ $\quad \phi_v = 0.9$ $\quad \phi_n = 1$

$\gamma_c = 0.150\ \text{kcf}$ $\quad b_{cover} = 2\ \text{in}$ $\quad t_{cover} = 2\ \text{in}$ $\quad h = 5\ \text{ft}$ $\quad b = 4.5\ \text{ft}$ $\quad E_c = E_{cap}$

$$n = \text{round}\left(\frac{E_s}{E_c}, 0\right)$$ (AASHTO LRFD 5.7.1) $\quad n = 7$

$$EI_{cap1} = E_c \frac{\left(b\ h^3\right)}{12}$$ Applicable for $0 \le CapL \le 6, 41 \le CapL \le 47$ $\quad EI_{cap1} = 2.894 \times 10^7\ \text{kip ft}^2$

$$y_{cg2} = \frac{wCap\ hCap\ \dfrac{hCap}{2} - 2\ (wFoam\ hFoam)\left(\dfrac{hFoam}{2} + h_{bS}\right)}{wCap\ hCap - 2\ (wFoam\ hFoam)}$$ (y_{cg} of from Bottom of Cap Section) $\quad y_{cg2} = 29.817\ \text{in}$

$$I_{cap2} = \frac{wCap\ hCap^3}{12} + wCap\ hCap\left(\frac{hCap}{2} - y_{cg2}\right)^2 \dots$$
$$+ -2\left[\frac{wFoam\ hFoam^3}{12} + wFoam\ hFoam\left(\frac{hFoam}{2} + h_{bS} - y_{cg2}\right)^2\right] \qquad I_{cap2} = 902191.259\ in^4$$

$EI_{cap2} = E_c\ I_{cap2}$ Applicable for $6 \le CapL \le 41$ $EI_{cap2} = 2.686 \times 10^7\ kip\ ft^2$

OUTPUT of BENT CAP LOADING PROGRAM: The maximum load effects from different applicable limit states:

DEAD LOAD	$M_{dlPos} = 3309.6\ kft$	$M_{dlNeg} = 30.1\ kft$
SERVICE I	$M_{sPos} = 5377.1\ kft$	$M_{sNeg} = 45.1\ kft$
STRENGTH I	$M_{uPos} = 7830.6\ kft$	$M_{uNeg} = 64.6\ kft$

FLEXURE DESIGN:

MINIMUM FLEXURAL REINFORCEMENT *AASHTO LRFD 5.7.3.3.2*

Factored Flexural Resistance, M_r, must be greater than or equal to the lesser of $1.2M_{cr}$ or $1.33\ M_u$. Applicable to both positive and negative moment.

Modulus of rupture

$f_r = 0.37\sqrt{f'_c\ ksi}$ (AASHTO LRFD EQ 5.4.2.6) $f_r = 0.827\ ksi$

$S = \frac{I_{cap2}}{y_{cg2}}$ (Bottom Section Modulus for Positive Moment) $S = 30257.581\ in^3$

Cracking moment

$M_{cr} = S\ f_r$ (AASHTO LRFD EQ 5.7.3.3.2-1) $M_{cr} = 2086.122\ kip\ ft$

$M_{cr1} = 1.2\ M_{cr}$ $M_{cr1} = 2503.346\ kip\ ft$

$M_{cr2} = 1.33\ \max(M_{uPos}, M_{uNeg})$ $M_{cr2} = 10414.698\ kip\ ft$

$M_{cr\_min} = \min(M_{cr1}, M_{cr2})$ Therefore M_r must be <u>greater</u> than $M_{cr\_min} = 2503.346\ kip\ ft$

Moment Capacity Design (Positive Moment, Bottom Bars B) *AASHTO LRFD 5.7.3.2*

Bottom Steel arrangement for the Cap:

Input no. of total rebar in a row from bottom of cap up to 12 rows (in unnecessary rows input zero)

$N_p = (9\ \ 9\ \ 9\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0)$

Input area of rebar corresponding to above rows from bottom of cap, not applicable for mixed rebar in a single row

$A_{bp} = (1.56\ \ 1.56\ \ 1.56\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0)\ in^2$

Input center to center vertical distance between each rebar row starting from bottom of cap

$clp = (3.5\ \ 4\ \ 4\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0)\ in$

$ns_{Pos} = 3$ (No. of Bottom or Positive Steel Layers)

Distance from centroid of positive rebar to extreme bottom tension fiber (d_{cPos}):

$$d_{cPos} = \left(Ayp_{0,0}\right) \text{ in} \qquad d_{cPos} = 7.5 \text{ in}$$

Effective depth from centroid of bottom rebar to extreme compression fiber (d_{Pos}):

$$d_{Pos} = h - d_{cPos} \qquad d_{Pos} = 52.5 \text{ in}$$

Compression Block depth under ultimate load *AASHTO LRFD 5.7.2.2*

$$\beta_1 = \min\left[0.85, \max\left[0.65, 0.85 - \frac{0.05}{\text{ksi}}\left(f'_c - 4 \text{ ksi}\right)\right]\right] \qquad \beta_1 = 0.8$$

The Amount of Bottom or Positive Steel A_s Required,

$$A_{sReq} = \left(\frac{0.85\ f'_c\ b\ d_{Pos}}{f_y}\right)\left(1 - \sqrt{1 - \frac{2\ M_{uPos}}{0.85\ \phi_m\ f'_c\ b\ d_{Pos}^{\ 2}}}\right) \qquad A_{sReq} = 36.454 \text{ in}^2$$

The Amount of Positive A_s Provided,

$$NofBars_{Pos} = \sum N_p \qquad NofBars_{Pos} = 27$$

$$A_{sPos} = \left(Ayp_{0,1}\right) \text{ in}^2 \qquad A_{sPos} = 42.12 \text{ in}^2$$

$$h_{tS} = h - hFoam - h_{bS} \quad \text{(Top solid depth)} \qquad h_{tS} = 14 \text{ in}$$

Compression depth under ultimate load

$$c_{Pos} = \frac{A_{sPos}\ f_y}{0.85\ f'_c\ \beta_1\ b} \quad \text{(AASHTO LRFD EQ 5.7.3.1.1-4)} \qquad c_{Pos} = 13.765 \text{ in}$$

$$a_{Pos} = \beta_1\ c_{Pos} \quad \left(a_{Pos} < h_{tS}, \text{OK}\right) \quad \text{(AASHTO LRFD 5.7.3.2.2)} \qquad a_{Pos} = 11.012 \text{ in}$$

Nominal flexural resistance:

$$M_{nPos} = A_{sPos}\ f_y\left(d_{Pos} - \frac{a_{Pos}}{2}\right) \quad \text{(AASHTO LRFD EQ 5.7.3.2.2-1)} \qquad M_{nPos} = 9896.961 \text{ kip ft}$$

Tension controlled resistance factor for flexure

$$\phi_{mPos} = \min\left[0.65 + 0.15\left(\frac{d_{Pos}}{c_{Pos}} - 1\right), 0.9\right] \quad \text{(AASHTO LRFD EQ 5.5.4.2.1-2)} \qquad \phi_{mPos} = 0.9$$

or simply use, $\phi_m = 0.9$ (AASHTO LRFD 5.5.4.2)

$M_{rPos} = \phi_{mPos} M_{nPos}$ (AASHTO LRFD EQ 5.7.3.2.1-1) $M_{rPos} = 8907.265 \text{ kip ft}$

$M_{uPos} = 7830.6 \text{ kip ft}$

$MinReinChkPos = if\left[\left(M_{rPos} \geq M_{cr\_min}\right), \text{"OK"}, \text{"NG"}\right]$ $MinReinChkPos = \text{"OK"}$

$UltimateMomChkPos = if\left[\left(M_{rPos} \geq M_{uPos}\right), \text{"OK"}, \text{"NG"}\right]$ $UltimateMomChkPos = \text{"OK"}$

Moment Capacity Design (Negative Moment, Top Bars A) *AASHTO LRFD 5.7.3.2*

Top Steel arrangement for the Cap:

Input no. of total rebar in a row from top of cap up to 12 rows (in unnecessary rows input zero)

$N_n = (6\ \ 6\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0)$

Input area of rebar corresponding to above rows from top of cap, not applicable for mixed rebar in a single row

$A_{bn} = (0.6\ \ 1.27\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0)\ in^2$

Input center to center vertical distance between each rebar row starting from top of cap

$cln = (3.5\ \ 4\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0)\ in$

$ns_{Neg} = 2$ (No. of Negative or Top Steel Layers)

Distance from centroid of negative rebar to top extreme tension fiber (d_{cNeg}):

$d_{cNeg} = \left(Ayn_{0,0}\right) in$ $d_{cNeg} = 6.217 \text{ in}$

Effective depth from centroid of top rebar to extreme compression fiber (d_{Neg}):

$d_{Neg} = h - d_{cNeg}$ $d_{Neg} = 53.783 \text{ in}$

The Amount of Negative A_s Required,

$$A_{sReq} = \left(\frac{0.85\ f'_c\ b\ d_{Neg}}{f_y}\right)\left(1 - \sqrt{1 - \frac{2\ M_{uNeg}}{0.85\ \phi_m\ f'_c\ b\ d_{Neg}^{\ 2}}}\right) \qquad A_{sReq} = 0.267\ in^2$$

The Amount of Negative A_s Provided,

$NofBars_{Neg} = \sum N_n$ $NofBars_{Neg} = 12$

$A_{sNeg} = \left(Ayn_{0,1}\right) in^2$ $A_{sNeg} = 11.22\ in^2$

Compression depth under ultimate load

$$c_{Neg} = \frac{A_{sNeg}\ f_y}{0.85\ f'_c\ \beta_1\ b}$$ $c_{Neg} = 3.667$ in

$$a_{Neg} = \beta_1\ c_{Neg}$$ $a_{Neg} = 2.933$ in

Thus, nominal flexural resistance:

$$M_{nNeg} = A_{sNeg}\ f_y \left(d_{Neg} - \frac{a_{Neg}}{2}\right)$$ $M_{nNeg} = 2934.97$ kip ft

Factored flexural resistance

$$M_{rNeg} = \phi_m\ M_{nNeg}$$ $M_{rNeg} = 2641.473$ kip ft

$M_{uNeg} = 64.6$ kip ft

$\text{MinReinChkNeg} = \text{if}\left[\left(M_{rNeg} \geq M_{cr\_min}\right), \text{"OK"}, \text{"NG"}\right]$ MinReinChkNeg = "OK"

$\text{UltimateMomChkNeg} = \text{if}\left[\left(M_{rNeg} \geq M_{uNeg}\right), \text{"OK"}, \text{"NG"}\right]$ UltimateMomChkNeg = "OK"

Control of Cracking at Service Limit State *AASHTO LRFD 5.7.3.4*

exposure_cond = 1 (for exposure condition, input Class 1 = 1 and Class 2 = 2)

$\gamma_e = \text{if}(\text{exposure\_cond} = 1, 1, 0.75)$ (Exposure condition factor) $\gamma_e = 1$

$\left(side_{cTop}\ \ side_{cBot}\right) = (5.625\ \ 4.75)$ in (Input side cover for Top and Bottom Rebars)

<u>Positive Moment (Bottom Bars B)</u> To find S_{max}: S is spacing of first layer of rebar closest to tension face

$$n = \text{round}\left(\frac{E_s}{E_c}, 0\right)$$ (modular ratio) $n = 7$

$$\rho_{Pos} = \frac{A_{sPos}}{b\ d_{Pos}}$$ $\rho_{Pos} = 0.0149$

$$k_{Pos} = \sqrt{\left(\rho_{Pos}\ n + 1\right)^2 - 1} - \rho_{Pos}\ n$$ (Applicable for Solid Rectangular Section) $k_{Pos} = 0.364$

$kd_P = k_{Pos}\ d_{Pos}$ Location of NA from Top of Cap for Pos Moment $kd_P = 19.098$ in

$\text{StressBlock}_{Pos} = \text{if}\left(kd_P \geq h_{tS}, \text{"T-Section"}, \text{"Rec-Section"}\right)$ StressBlock_{Pos} = "T-Section"

Compression Force = Tension Force OR Moment of Compression Area = Moment of Tension Area about NA

$$b\left(kd_{Pos}\right)^2 - 2\ wFoam\left(kd_{Pos} - hStop\right)^2 = 2\ n\ A_{sPos}\left(d_{Pos} - kd_{Pos}\right)$$

$$(b - 2\ wFoam)\left(kd_{Pos}\right)^2 + \left(2\ n\ A_{sPos} + 4\ wFoam\ hStop\right)\left(kd_{Pos}\right) - 2\left(wFoam\ hStop^2 + n\ A_{sPos}\ d_{Pos}\right) = 0$$

$$kd_{Pos} = \frac{-\left(2\ n\ A_{sPos} + 4\ wFoam\ h_{tS}\right) + \sqrt{\left(2\ n\ A_{sPos} + 4\ wFoam\ h_{tS}\right)^2 \ldots + 4\ (b - 2\ wFoam)\ 2\left(wFoam\ h_{tS}^2 + n\ A_{sPos}\ d_{Pos}\right)}}{2\ (b - 2\ wFoam)}$$

$kd_{Pos} = 19.405\ in$ Location of NA from Top of Cap

Location of Resultant Compression force from NA for Positive Moment:

$$x_{Pos} = \frac{b\frac{\left(kd_{Pos}\right)^2}{3} - \frac{2}{3}\ wFoam\left(kd_{Pos} - h_{tS}\right)^2\left(1 - \frac{h_{tS}}{kd_{Pos}}\right)}{\frac{1}{2}\ b\ kd_{Pos} - wFoam\left(kd_{Pos} - h_{tS}\right)\left(1 - \frac{h_{tS}}{kd_{Pos}}\right)} \qquad x_{Pos} = 13.328\ in$$

$$jd_{Pos} = d_{Pos} - kd_{Pos} + x_{Pos} \qquad jd_{Pos} = 46.423\ in$$

Tensile Stress at Service Limit State

$$f_{ssPos} = \frac{M_{sPos}}{A_{sPos}\ jd_{Pos}} \qquad f_{ssPos} = 33\ ksi$$

$d_{c1Pos} = clp_{0,0}$ (Distance of bottom first row rebar closest to tension face) $d_{c1Pos} = 3.5\ in$

$$\beta_{sPos} = 1 + \frac{d_{c1Pos}}{0.7\left(h - d_{c1Pos}\right)} \qquad \beta_{sPos} = 1.088$$

$$s_{maxPos} = \frac{700\frac{kip}{in}\ \gamma_e}{\beta_{sPos}\ f_{ssPos}} - 2\ d_{c1Pos}$$ AASHTO LRFD EQ (5.7.3.4-1) $s_{maxPos} = 12.488\ in$

$$s_{ActualPos} = \frac{b - 2\ side_{cBot}}{N_{p_{0,0}} - 1}$$ (Equal horizontal spacing of bottom first rebar row closest to tension face) $s_{ActualPos} = 5.563\ in$

Actual Max Spacing Provided in Bottom first row closest to Tension Face, $s_{aPosProvided} = 7\ in$

$$s_{ActualPos} = \max\left(s_{aPosProvided}, s_{ActualPos}\right) \qquad s_{ActualPos} = 7\ in$$

$$SpacingCheckPos = if\left[\left(s_{maxPos} \geq s_{ActualPos}\right), \text{"OK"}, \text{"NG"}\right] \qquad SpacingCheckPos = \text{"OK"}$$

Negative Moment (Top Bars A)

$$\rho_{Neg} = \frac{A_{sNeg}}{b\ d_{Neg}} \qquad \rho_{Neg} = 3.863 \times 10^{-3}$$

$$k_{Neg} = \sqrt{\left(\rho_{Neg}\ n + 1\right)^2 - 1} - \rho_{Neg}\ n$$ (Applicable for Solid Rectangular Section) $k_{Neg} = 0.207$

$$kd_N = k_{Neg}\ d_{Neg}$$ Location of NA from Bottom of Cap for Neg Moment $kd_N = 11.138\ \text{in}$

$$StressBlock_{Neg} = \text{if}\left(kd_N \ge h_{bS}, \text{"T-Section"}, \text{"Rec-Section"}\right) \qquad StressBlock_{Neg} = \text{"Rec-Section"}$$

$$j_{Neg} = 1 - \frac{k_{Neg}}{3} \qquad j_{Neg} = 0.931$$

$$f_{ssNeg} = \frac{M_{sNeg}}{A_{sNeg}\ j_{Neg}\ d_{Neg}} \qquad f_{ssNeg} = 0.963\ \text{ksi}$$

$$d_{c1Neg} = cln_{0,0}$$ (Distance of top first row rebar closest to tension face) $d_{c1Neg} = 3.5\ \text{in}$

$$\beta_{sNeg} = 1 + \frac{d_{c1Neg}}{0.7\left(h - d_{c1Neg}\right)} \qquad \beta_{sNeg} = 1.088$$

$$s_{maxNeg} = \frac{700\ \frac{kip}{in}\ \gamma_e}{\beta_{sNeg}\ f_{ssNeg}} - 2\ d_{c1Neg} \qquad s_{maxNeg} = 660.561\ \text{in}$$

$$s_{ActualNeg} = \frac{b - 2\ side_{cTop}}{N_{n_{0,0}} - 1}$$ (Equal horizontal spacing of top first rebar row closest to tension face) $s_{ActualNeg} = 8.55\ \text{in}$

Actual Max Spacing Provided in Top first row closest to Tension Face, $s_{aNegProvided} = 11.125\ \text{in}$

$$s_{ActualNeg} = \max\left(s_{aNegProvided}, s_{ActualNeg}\right) \qquad s_{ActualNeg} = 11.125\ \text{in}$$

$$SpacingCheckNeg = \text{if}\left[\left(s_{maxNeg} \ge s_{ActualNeg}\right), \text{"OK"}, \text{"NG"}\right] \qquad SpacingCheckNeg = \text{"OK"}$$

SUMMARY OF FLEXURE DESIGN:

Bottom Rebar or B Bars: use 27~#11 bars @ 9 bars in each row of 3 rows

Top Rebar or A Bars: use 6~#7 bars and 6~#10 bars in first and 2nd row from top

SKIN REINFORCEMENT (BARS T) *AASHTO LRFD 5.7.3.4*

$SkBarNo = 8$ (Size of a skin bar) Area of a skin bar, $A_{skBar} = 0.79\ in^2$

$d_{cTop} = \sum cln$ $\qquad d_{cTop} = 7.5\ in$

$d_{cBot} = \sum clp$ $\qquad d_{cBot} = 11.5\ in$

Effective Depth from centroid of ExtremeTension Steel to Extreme compression Fiber (d_l):

$d_l = \max\left(h - clp_{0,0}, h - cln_{0,0}\right)$ $\qquad d_l = 56.5\ in$

Effective Depth from centroid of Tension Steel to Extreme compression Fiber (d_e):

$d_e = \max\left(d_{Pos}, d_{Neg}\right)$ $\qquad d_e = 53.783\ in$

$A_s = \min\left(A_{sNeg}, A_{sPos}\right)$ min. of negative and positive reinforcement $\qquad A_s = 11.22\ in^2$

$d_{skin} = h - \left(d_{cTop} + d_{cBot}\right)$ $\qquad d_{skin} = 41\ in$

Skin Reinforcement Requirement: AASHTO LRFD EQ 5.7.3.4-2

$$A_{skReq} = \text{if}\left[d_l > 3ft, \min\left[0.012\ \frac{in}{ft}\left(d_l - 30\ in\right) d_{skin}, \frac{A_s + A_{ps}}{4}\right], 0in^2\right]$$

$A_{skReq} = 1.087\ in^2$

$$NoA_{skbar1} = R\left(\frac{A_{skReq}}{A_{skBar}}\right)$$

$NoA_{skbar1} = 2$ per Side

Maximum Spacing of Skin Reinforcement:

$$S_{skMax} = \min\left(\frac{d_e}{6}, 12\ in\right)$$ AASHTO LRFD 5.7.3.4

$S_{skMax} = 8.964\ in$

$$NoA_{skbar2} = \text{if}\left(d_l > 3ft, R\left(\frac{d_{skin}}{S_{skMax}} - 1\right), 1\right)$$

$NoA_{skbar2} = 4$ per Side

$NofSideBars_{req} = \max\left(NoA_{skbar1}, NoA_{skbar2}\right)$ $\qquad NofSideBars_{req} = 4$

$$S_{skRequired} = \frac{d_{skin}}{1 + NofSideBars_{req}}$$

$S_{skRequired} = 8.2\ in$

$NofSideBars = 5$ (No. of Side Bars Provided)

$$S_{skProvided} = \frac{d_{skin}}{1 + NofSideBars}$$

$S_{skProvided} = 6.833\ in$

$S_{skChk} = \text{if}\left(S_{skProvided} < S_{skMax}, \text{"OK"}, \text{"N.G."}\right)$ $\qquad S_{skChk} = \text{"OK"}$

Therefore Use: $NofSideBars = 5$ and Size $SkBarNo = 8$

3. BENT CAP SHEAR AND TORSION DESIGN

SHEAR DESIGN OF CAP:

Effective Shear Depth, $d_v = \max\left(\begin{pmatrix} d_e - \frac{a}{2} \\ 0.9\ d_e \\ 0.72\ h \end{pmatrix}\right)$ (*AASHTO LRFD 5.8.2.9*)

d_v = Distance between the resultants of tensile and compressive Force

d_s = Effective depth from cg of the nonprestressed tensile steel to extreme compression fiber

d_p = Effective depth from cg of the prestressed tendon to extreme compression fiber

d_e = Effective depth from centroid of the tensile force to extreme compression fiber at critical shear Location

θ = Angle of inclination diagonal compressive stress

A_o = Area enclosed by shear flow path including area of holes therein

A_c = Area of concrete on flexural tension side of member shown in AASHTO LRFD Figure 5.8.3.4.2 − 1

A_{oh} = Area enclosed by centerline of exterior closed transverse torsion reinforcement including area of holes therein

Total Pos Flexural Steel Area,	$A_s = A_{sPos}$		$A_s = 42.12\ in^2$
Nominal Flexure,	$M_n = M_{nPos}$		$M_n = 9896.961\ kft$
Stress block Depth,	$a = a_{Pos}$		$a = 11.012\ in$
Effective Depth,	$d_e = d_{Pos}$		$d_e = 52.5\ in$
Effective web Width at critical Location,	$b_v = b$		$b_v = 4.5\ ft$
Input initial θ,	$\theta = 35\ deg$		$\cot\theta = \cot(\theta)$
Shear Resistance Factor,	$\phi_v = 0.9$		
Cap Depth & Width,	$h = 60\ in$		$b = 54\ in$
Moment Arm,	$\left(d_e - \frac{a}{2}\right) = 46.994\ in$	$0.9\ d_e = 47.25\ in$	$0.72\ h = 43.2\ in$
Effective Shear Depth at Critical Location,	$d_v = \max\left(\begin{pmatrix} d_e - \frac{a}{2} \\ 0.9\ d_e \\ 0.72\ h \end{pmatrix}\right)$	(*AASHTO LRFD 5.8.2.9*)	$d_v = 47.25\ in$

$h_h = h - t_{cover} - b_{cover}$ (Height of shear reinforcement) $h_h = 56$ in

$b_h = b - 2\,b_{cover}$ (Width of shear reinforcement) $b_h = 50$ in

$p_h = 2(h_h + b_h)$ (Perimeter of shear reinforcement) $p_h = 212$ in

$A_{oh} = (h_h)(b_h)$ (Area enclosed by the shear reinforcement) $A_{oh} = 2800\ in^2$

$A_o = 0.85\,A_{oh}$ (AASHTO LRFD C5.8.2.1) $A_o = 2380\ in^2$

$A_c = 0.5\,b\,h$ (AASHTO LRFD FIGURE 5.8.3.4.2 − 1) $A_c = 1620\ in^2$

Yield strength & Modulus of Elasticity of Steel Reinforcement:

$(f_y\ \ E_s) = (60\ \ 29000)$ ksi (AASHTO LRFD 5.4.3.1, 5.4.3.2)

Input Mu, Tu, Vu, Nu for the critical section to be investigated: (Loads from Bent Cap & RISA Analysis)

$(M_u\ \ T_u) = (1314.8\ \ 964.6)$ kft $(V_u\ \ N_u) = (665.4\ \ 0)$ kip

$M'_u = \max(M_u, |V_u - V_p|\,d_v)$ AASHTO LRFD B5.2 $M'_u = 2620.013$ kip ft

$$V'_u = \sqrt{V_u^2 + \left(\frac{0.9\,p_h\,T_u}{2\,A_o}\right)^2}$$ (Equivalent shear) AASHTO LRFD EQ (5.8.2.1-6) for solid section $V'_u = 811.194$ kip

Assuming at least minimum transverse reinforcement is provided (Always provide min. transverse reinf.)

$$\varepsilon_x = \frac{\left(\frac{M'_u}{d_v}\right) + 0.5\,N_u + 0.5\,(V'_u - V_p)\cot\theta - A_{ps}\,f_{po}}{2\,(E_s\,A_s + E_p\,A_{ps})}$$ (Strain from Appendix B5) AASHTO LRFD EQ (B5.2-1)

$$v_u = \frac{(V_u - \phi_v\,V_p)}{\phi_v\,b_v\,d_v}$$ (Shear Stress) AASHTO LRFD EQ (5.8.2.9-1) $v_u = 0.29$ ksi

$$r = \max\left(0.075, \frac{v_u}{f'_c}\right)$$ (Shear stress ratio) $r = 0.075$

After Interpolating the value of $(\Theta \;\; B)$

$\Theta = 30.773\ \text{deg}$ $B = 2.572$

Nominal Shear Resistance by Concrete,

$V_c = 0.0316\ B\ \sqrt{f'_c\ \text{ksi}}\ b_v\ d_v$ AASHTO LRFD EQ (5.8.3.3-3) $V_c = 463.7\ \text{kip}$

$V_u = 665.4\ \text{kip}$ $0.5\ \phi_v\left(V_c + V_p\right) = 208.673\ \text{kip}$

REGION REQUIRING TRANSVERSE REINFORCEMENT: AASHTO LRFD 5.8.2.4

$V_u > 0.5\ \phi_v\left(V_c + V_p\right)$ AASHTO LRFD EQ (5.8.2.4-1)

$\text{check} = \text{if}\left[V_u > 0.5\ \phi_v\left(V_c + V_p\right), \text{"Provide Shear Reinf"}, \text{"No reinf."}\right]$ $\text{check} = \text{"Provide Shear Reinf"}$

$$V_n = \min\left(\begin{pmatrix} V_c + V_s + V_p \\ 0.25\ f'_c\ b_v\ d_v + V_p \end{pmatrix}\right)$$ (Nominal Shear Resistance) AASHTO LRFD EQ (5.8.3.3 − 1, 2)

$$V_s = \frac{A_v\ f_y\ d_v\ (\cot\theta + \cot\alpha)\ \sin\alpha}{S}$$ (Shear Resistance of Steel) AASHTO LRFD EQ (5.8.3.3 − 4)

$$V_s = \frac{A_v\ f_y\ d_v\ \cot\theta}{S}$$ (Shear Resistance of Steel when, $\alpha = 90$ deg) AASHTO LRFD EQ (C5.8.3.3-1)

$S_v = 6\ \text{in}$ (Input Stirrup Spacing) $V_p = 0\ \text{kip}$ $\left(V_u \;\; V_c\right) = (665.4 \;\; 463.718)\ \text{kip}$

$f_y = 60\ \text{ksi}$ $d_v = 47.25\ \text{in}$ $\Theta = 30.773\ \text{deg}$

$$A_{v\_req} = \left(\frac{V_u}{\phi_v} - V_c - V_p\right)\left(\frac{S_v}{f_y\ d_v\ \cot\Theta}\right)$$ (Derive from AASHTO LRFD EQ 5.8.3.3-1, C5.8.3.3-1 and ϕVn >= Vu) $A_{v\_req} = 0.3474\ \text{in}^2$

Torsional Steel:

$$A_t = \frac{T_u}{2\ \phi_v\ A_o\ f_y\ \cot\Theta}\ S_v$$ (Derive from AASHTO LRFD EQ 5.8.3.6.2-1 and ϕTn >= Tu) $A_t = 0.161\ \text{in}^2$

$A_{vt\_req} = A_{v\_req} + 2\ A_t$ (Shear + Torsion) $A_{vt\_req} = 0.669\ \text{in}^2$

$A_{vt} = 4\left(0.44\ \text{in}^2\right)$ (Use 2 #6 double leg Stirrup at S_v c/c,) *Provided*, $A_{vt} = 1.76\ \text{in}^2$

$A_{vt\_check} = \text{if}\left(A_{vt} > A_{vt\_req}, \text{"OK"}, \text{"NG"}\right)$ $A_{vt\_check} = \text{"OK"}$

Maximum Spacing Check: AASHTO LRFD Article 5.8.2.7

$V_u = 665.4\ \text{kip}$

$0.125\ f'_c\ b_v\ d_v = 1594.69\ \text{kip}$

$S_{vmax} = \text{if}\left(V_u < 0.125\ f'_c\ b_v\ d_v, \min\left(0.8\ d_v, 24\ \text{in}\right), \min\left(0.4\ d_v, 12\ \text{in}\right)\right)$

$S_{vmax} = 24\ \text{in}$

$S_{vmax\_check} = \text{if}\left(S_v < S_{vmax}, \text{"OK"}, \text{"use lower spacing"}\right)$

$S_{vmax\_check} = \text{"OK"}$

$A_v = A_{vt} - A_t$ (Shear Reinf. without Torsion Reinf.)

$A_v = 1.599\ \text{in}^2$

$$V_s = \frac{A_v\ f_y\ d_v\ \cot\Theta}{S_v}$$

$V_s = 1268.855\ \text{kip}$

Minimum Transverse Reinforcement Check: AASHTO LRFD Article 5.8.2.5

$b_v = 54\ \text{in}$

$$A_{vmin} = 0.0316\sqrt{f'_c\ \text{ksi}}\ \frac{b_v\ S_v}{f_y}$$ AASHTO LRFD EQ (5.8.2.5 − 1)

$A_{vmin} = 0.382\ \text{in}^2$

$A_{vmin\_check} = \text{if}\left(A_{vt} > A_{vmin}, \text{"OK"}, \text{"NG"}\right)$

$A_{vmin\_check} = \text{"OK"}$

Maximum Nominal Shear: *To ensure that the concrete in the web of beam will not crush prior to yield of shear reinforcement, LRFD Specification has given an upper limit of*

$0.25\ f'_c\ b_v\ d_v + V_p = 3189.375\ \text{kip}$

$V_c + V_s + V_p = 1732.573\ \text{kip}$

$$V_n = \min\left(\begin{pmatrix} V_c + V_s + V_p \\ 0.25\ f'_c\ b_v\ d_v + V_p \end{pmatrix}\right)$$ AASHTO LRFD EQ (5.8.3.3 − 1, 2)

$V_n = 1732.573\ \text{kip}$

$\phi_v\ V_n = 1559.316\ \text{kip}$

$V_u = 665.4\ \text{kip}$

$\phi V_{n\_check} = \text{if}\left(\phi_v\ V_n > V_u, \text{"OK"}, \text{"NG"}\right)$

$\phi V_{n\_check} = \text{"OK"}$

Torsional Resistance,

$$T_n = \frac{2\ A_o\ \left(0.5\ A_{vt}\right)\ f_y\ \cot\Theta}{S_v}$$ AASHTO LRFD EQ (5.8.3.6.2 − 1)

$\phi_v\ T_n = 5275.8\ \text{kip ft}$

Longitudinal Reinforcement Requirements including Torsion: AASHTO LRFD 5.8.3.6.3

AASHTO LRFD EQ(5.8.3.6.3 − 1) Applicable for solid section with Torsion

$$A_{ps}\ f_{ps} + A_s\ f_y \geq \left(\frac{M'_u}{\phi_m\ d_v}\right) + \frac{0.5\ N_u}{\phi_n} + \cot\Theta\sqrt{\left(\frac{V_u}{\phi_v} - V_p - 0.5\ V'_s\right)^2 + \left(\frac{0.45\ p_h\ T_u}{2\ \phi_v\ A_o}\right)^2}$$

$\left(\phi_m\ \ \phi_v\ \ \phi_n\right) = \left(0.9\ \ 0.9\ \ 1\right)$

$A_s\ f_y + A_{ps}\ f_{ps} = 2527.2\ \text{kip}$

$M'_u = 2620.013\ \text{kip ft}$ $\quad V_u = 665.4\ \text{kip}$ $\quad N_u = 0\ \text{kip}$ $\quad V_s = 1268.855\ \text{kip}$

$T_u = 964.6\ \text{kip ft}$ $\quad p_h = 212\ \text{in}$ $\quad V_p = 0\ \text{kip}$ $\quad A_s = 42.12\ \text{in}^2$

$$V'_s = \min\left(\frac{V_u}{\phi_v}, V_s\right)$$ AASHTO LRFD 5.8.3.5 $\quad V'_s = 739.333\ \text{kip}$

$$F = \left(\frac{M'_u}{\phi_m\ d_v}\right) + \frac{0.5\ N_u}{\phi_n} + \cot\Theta\sqrt{\left(\frac{V_u}{\phi_v} - V_p - 0.5\ V'_s\right)^2 + \left(\frac{0.45\ T_u\ p_h}{2\ \phi_v\ A_o}\right)^2}$$ $\quad F = 1496.141\ \text{kip}$

$F_{check} = \text{if}\left(A_{ps}\ f_{ps} + A_s\ f_y \geq F, \text{"OK"}, \text{"NG"}\right)$ AASHTO LRFD EQ(5.8.3.6.3 − 1) $\quad F_{check} = \text{"OK"}$

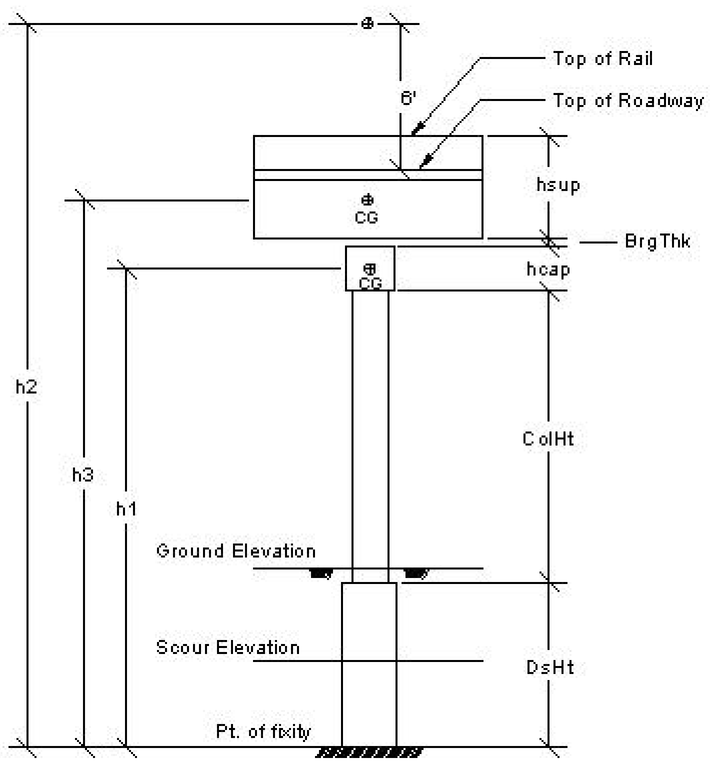
4. COLUMN/DRILLED SHAFT LOADING AND DESIGN

Superstructure to substructure force: AASHTO LRFD SECTION 3 LOADS and LOAD COMBINATIONS

Subscript*: *X* = *Parallel to the Bent cap Length and* *Z* = *Perpendicular to the bent Cap Length

$t_h = 2\ \text{in}$ (Haunch Thickness) $\quad$ Beam Depth, $BmH = FBmD$

$ColH = HCol + 0\ \text{ft}$ (Column height + 0 ft Column Capital) $\quad TribuLength = \dfrac{FSpan + BSpan}{2}$



Scour Depth:

$h_{scour} = 0\ \text{ft}$

Scour to Fixity Depth:

$h_{scf} = \min(3\ DsDia, 10\ \text{ft})$

Total Drilled Shaft height:

$DsH = h_{scour} + h_{scf}$

$DsH = 10\ \text{ft}$

$h_o = BrgTh + BmH + t_h + SlabTh$ (Top of cap to top of slab height) $h_o = 3.683$ ft

$h_6 = h_o + 6ft$ (Top of cap to top of slab height + 6 ft) $h_6 = 9.683$ ft

$hsup = BmH + t_h + SlabTh + RailH$ (Height of Superstructure) $hsup = 6.225$ ft

$h1 = DsH + ColH + \frac{hCap}{2}$ (Height of Cap cg from Fixity of Dshaft) $h1 = 34.5$ ft

$h2 = DsH + ColH + hCap + h_6$ $h2 = 46.683$ ft

$h3 = DsH + ColH + hCap + BrgTh + \frac{hsup}{2}$ $h3 = 40.404$ ft

Tributary area for Superstructure,

$A_{super} = (hsup)\ (TribuLength)$ $A_{super} = 435.75\ ft^2$

LIVE LOAD REACTIONS: LL

Live load Reaction LL on cap can be taken only the vertical Rxn occurs when HL93 is on both the forward and backward span or when HL93 Loading is on one span only which causes torsion too. To maximize the torsion, LL only acts on the longer span between forward and backward span. For maximum reaction, place rear axle (P_3 = 32 kip) over the support at bent while the design truck traveling along the span.

Maximum Forward Span Design Truck (FTruck) & Lane Load Reaction (FLane):

$$FTruck = P_3 + P_2\left[\frac{(FSpan - 14\ ft)}{FSpan}\right] + P_1\ \frac{(FSpan - 28ft)}{FSpan}$$ $FTruck = 62.4$ kip

$$FLane = w_{lane}\left(\frac{FSpan}{2}\right)$$ $FLane = 22.4\ \frac{kip}{lane}$

Forward Span Live Load Reactions with Impact (FLLRxn):

$$FLLRxn = FLane + FTruck\ (1 + IM)$$ $FLLRxn = 105.392\ \frac{kip}{lane}$

Maximum Backward Span Design Truck (BTruck) & Lane Load Reaction (BLane):

$$BTruck = P_3 + P_2\left[\frac{(BSpan - 14\ ft)}{BSpan}\right] + P_1\ \frac{(BSpan - 28ft)}{BSpan}$$ $BTruck = 62.4$ kip

$$BLane = w_{lane}\left(\frac{BSpan}{2}\right)$$ $BLane = 22.4\ \frac{kip}{lane}$

Backward Span Live Load Reactions with Impact (BLLRxn):

$$BLLRxn = BLane + BTruck\ (1 + IM)$$ $BLLRxn = 105.392\ \frac{kip}{lane}$

Live Load Reactions per Beam with Impact (BmLLRxn) using Distribution Factors:

$$BmLLRxn = (LLRxn)\ \max\left(DFS_{Fmax}, DFS_{Bmax},\ (\text{Max reaction when mid axle on support})\right)$$ $BmLLRxn = 72.556\ \frac{kip}{beam}$

$FBmLLRxn = (FLLRxn)\ DFS_{Fmax}$ (Only Forward Span is Loaded) $FBmLLRxn = 58.858\ \frac{kip}{beam}$

$BBmLLRxn = (BLLRxn)\ DFS_{Bmax}$ (Only Backward Span is Loaded) $BBmLLRxn = 58.858\ \frac{kip}{beam}$

Torsion due to the eccentricity from CL of Bearing to CL of Bent when only Longer Span is loaded with HL-93 Loading

$TorsionLL = \max(FBmLLRxn, BBmLLRxn)\ e_{brg}$ $TorsionLL = 63.763\ \frac{kip\ ft}{beam}$

***Live Load Reactions per Beam without Impact ($BmLLRxn_n$) using Distribution Factors*:**

$BmLLRxn_n = (Lane + Truck)\ \max\left(DFS_{Fmax}, DFS_{Bmax}\right)$ $BmLLRxn_n = 60.761\ \frac{kip}{beam}$

$FBmLLRxn_n = (FLane + FTruck)\left(DFS_{Fmax}\right)$ $FBmLLRxn_n = 47.358\ \frac{kip}{beam}$

$BBmLLRxn_n = (BLane + BTruck)\left(DFS_{Bmax}\right)$ $BBmLLRxn_n = 47.358\ \frac{kip}{beam}$

Torsion due to the eccentricity of CL of Bearing and CL of Bent without Impact

$TorsionLL_n = \max\left(FBmLLRxn_n, BBmLLRxn_n\right)\ e_{brg}$ $TorsionLL_n = 51.305\ \frac{kft}{beam}$

CENTRIFUGAL FORCE: CF (*AASHTO LRFD 3.6.3*) Skew Angle of Bridge, $\theta = 0\ deg$

Design Speed $v = 45\ mph$

Degree of Curve, $\phi_c = 0.00001\ deg$ (Input 4° curve or 0.00001° for 0° curve) $(f\quad g) = \left(\frac{4}{3}\quad 32.2\ \frac{ft}{sec^2}\right)$

Radius of Curvature, $R_c = \frac{(360\ deg)\ 100\ ft}{2\ \pi\ \phi_c}$ $R_c = 572957795.131\ ft\ \left(R_c = \infty\ ft\right)$

Centri. Force Factor, $C = f\ \frac{v^2}{R_c\ g}$ (AASHTO LRFD EQ 3.6.3 − 1) $C = 0$

$P_{cf} = C\ TruckT\ (NofLane)\ (m)$ $P_{cf} = 0\ kip$

Centrifugal force **parallel** to bent (X-direction)

$CF_X = \left(\frac{P_{cf}\ \cos(\theta)}{NofBm}\right)$ $CF_X = 0\ \frac{kip}{beam}$

Centrifugal force **normal** to bent (Z-direction)

$CF_Z = \left(\frac{P_{cf}\ \sin(\theta)}{NofBm}\right)$ $CF_Z = 0\ \frac{kip}{beam}$

Moments at cg of the Bent Cap due to Centrifugal Force

$M_{CF\_X} = CF_Z\left(h_6 + \frac{hCap}{2}\right)$ $M_{CF\_X} = 0\ \frac{kft}{beam}$

$M_{CF\_Z} = CF_X\left(h_6 + \frac{hCap}{2}\right)$ $M_{CF\_Z} = 0\ \frac{kft}{beam}$

BRAKING FORCE: BR (*AASHTO LRFD 3.6.4*)

The braking force shall be taken as maximum of 5% of the Resultant Truck plus lane load OR 5% of the Design Tandem plus Lane Load or 25% of the design truck.

$P_{br1} = 5\%\ (\text{Lane} + \text{TruckT})\ (\text{NofLane})\ (m)$ (Truck + Lane) $P_{br1} = 14.892\ \text{kip}$

$P_{br2} = 5\%\ (\text{Lane} + 50\ \text{kip})\ (\text{NofLane})\ (m)$ (Tandem + Lane) $P_{br2} = 12.087\ \text{kip}$

$P_{br3} = 25\%\ (\text{TruckT})\ (\text{NofLane})\ (m)$ (DesignTruck) $P_{br3} = 45.9\ \text{kip}$

$P_{br} = \max\left(P_{br1}, P_{br2}, P_{br3}\right)$ $P_{br} = 45.9\ \text{kip}$

Braking force **parallel** to bent (X-direction)

$$BR_X = \frac{P_{br}\sin(\theta)}{\text{NofBm}} \qquad BR_X = 0\ \frac{\text{kip}}{\text{beam}}$$

Braking force **normal** to bent (Z-direction)

$$BR_Z = \frac{P_{br}\cos(\theta)}{\text{NofBm}} \qquad BR_Z = 3.825\ \frac{\text{kip}}{\text{beam}}$$

Moments at cg of the Bent Cap due to Braking Force

$$M_{BR\_X} = BR_Z\left(h_6 + \frac{hCap}{2}\right) \qquad M_{BR\_X} = 46.601\ \frac{\text{kft}}{\text{beam}}$$

$$M_{BR\_Z} = BR_X\left(h_6 + \frac{hCap}{2}\right) \qquad M_{BR\_Z} = 0\ \frac{\text{kft}}{\text{beam}}$$

WATER LOADS: WA (*AASHTO LRFD 3.7*)

Note : To be applied <u>only</u> on bridge components below design high water surface.

<u>Substructure:</u>

$V = 0\ \frac{\text{ft}}{\text{sec}}$ (Design Stream Velocity) Specific Weight, $\gamma_{water} = 62.4\ \text{pcf}$

<u>Longitudinal Stream Pressure:</u> *AASHTO LRFD 3.7.3.1*

AASHTO LRFD Table 3.7.3.1-1 for Drag Coefficient, C_D

semicircular-nosed pier	0.7
square-ended pier	1.4
debris lodged against the pier	1.4
wedged-nosed pier with nose angle 90 deg or less	0.8

Columns and Drilled Shafts: Longitudinal Drag Force Coefficient for Column, $C_{D\_col} = 1.4$

Longitudinal Drag Force Coefficient for Drilled Shaft, $C_{D\_ds} = 0.7$

$$p_T = C_D \frac{V^2}{2\ g} \gamma_{water}$$ (Longitudinal stream pressure) AASHTO LRFD EQ (C3.7.3.1-1)

$$p_{T\_col} = C_{D\_col} \frac{V^2}{2\ g} \gamma_{water}$$ $p_{T\_col} = 0\ ksf$

$$p_{T\_ds} = C_{D\_ds} \frac{V^2}{2\ g} \gamma_{water}$$ $p_{T\_ds} = 0\ ksf$

Lateral Stream Pressure: AASHTO LRFD 3.7.3.2

AASHTO LRFD Table 3.7.3.2-1 for Lateral Drag Coefficient, C_L

Angle,θ, between direction of flow and longitudinal axis of the pile	CL
0deg	0
5deg	0.5
10deg	0.7
20deg	0.9
>30deg	1

Lateral Drag Force Coefficient, $C_L = 0.0$

Lateral stream pressure, $p_L = C_L \frac{V^2}{2\ g} \gamma_{water}$ $p_L = 0\ ksf$

Bent Cap: Longitudinal stream pressure $C_L = 1.4$

$$p_{Tcap} = C_L \frac{V^2}{2\ g} \gamma_{water}$$ $p_{Tcap} = 0\ ksf$

WA on Columns

Water force on column **parallel** to bent (X-direction)

$$WA_{col\_X} = wCol\ p_{T\_col}$$ $WA_{col\_X} = 0\ \frac{kip}{ft}$

If angle between direction of flow and longitudinal axis of pile = 0 then apply load at one exterior column only; otherwise apply it on all columns. WA at all columns will be distributed uniformly rather than triangular distribution on Column Height.

Water force on column **normal** to bent (Z-direction)

$$WA_{col\_Z} = bCol\ p_L$$ $WA_{col\_Z} = 0\ \frac{kip}{ft}$

WA on Drilled Shafts

Water force on drilled shaft **parallel** to bent (X-direction)

$$WA_{dshaft\_X} = DsDia\ p_{T\_ds}$$ $WA_{dshaft\_X} = 0\ \frac{kip}{ft}$

Water force on drilled shaft **normal** to bent (Z-direction)

$$WA_{dshaft\_Z} = DsDia\ p_L$$ $WA_{dshaft\_Z} = 0\ \frac{kip}{ft}$

WA on Bent Cap (input as a punctual load)

Water force on bent cap **parallel** to bent (X-direction)

$WA_{cap\_X} = wCap\ hCap\ (p_{Tcap})$ (If design HW is below cap then input zero) $WA_{cap\_X} = 0\ kip$

Water force on bent cap **normal** to bent (Z-direction)

$WA_{cap\_Z} = hCap\ p_L$ (If design HW is below cap then input zero) $WA_{cap\_Z} = 0\ \frac{kip}{ft}$

WIND ON SUPERSTRUCTURE: WS (*AASHTO LRFD 3.8.1.2.2*)

Note : Wind Loads to be applied only on bridge exposed components above water surface

AASHTO LRFD Table 3.8.1.2.2-1 specifies the wind load components for various angles of attack. In order to simplify the analysis, this calculation considers as default values those for girders which generate the maximum effect on structure. The results can be considered as conservative. For a superstructure other than a girder type and/or for a more detailed analysis, use the proper values as specified in the above mentioned table.

AASHTO LRFD table 3.8.1.2.2-1 (modified)

Skew Angle	Girders	
	Lateral	Longitudinal
Degrees	(Ksf)	(Ksf)
0	0.05	0
15	0.044	0.006
30	0.041	0.012
45	0.033	0.016
60	0.017	0.019

If the bridge is approximately 30' high and local wind velocities are known to be less than 100 mph, wind load for this bridge should be from AASHTO LRFD TABLE 3.8.2.2-1. Otherwise use AASHTO LRFD EQ 3.8.1.2.1-1 as mentioned above.

$p_{tsup} = 0.05ksf$ Normal to superstructure (conservative suggested value 0.050 ksf)

$p_{lsup} = 0.012ksf$ Along Superstructure (conservative suggested value 0.019 ksf)

$WS_{chk} = if(p_{tsup}\ hsup \geq 0.3\ klf, "OK", "N.G.")$ $WS_{chk} = "OK"$

$$Wsup_{Long} = \frac{p_{lsup}\ hsup\ TribuLength}{NofBm} \qquad Wsup_{Long} = 0.436\ \frac{kip}{beam}$$

$$Wsup_{Trans} = \frac{p_{tsup}\ hsup\ TribuLength}{NofBm} \qquad Wsup_{Trans} = 1.816\ \frac{kip}{beam}$$

Wind force on superstructure **parallel** to bent (X-direction)

$$WS_{super\_X} = Wsup_{Long}\ \sin(\theta) + Wsup_{Trans}\ \cos(\theta) \qquad WS_{super\_X} = 1.816\ \frac{kip}{beam}$$

Wind force on superstructure **normal** to bent (Z-direction)

$$WS_{super\_Z} = Wsup_{Long}\ \cos(\theta) + Wsup_{Trans}\ \sin(\theta) \qquad WS_{super\_Z} = 0.436\ \frac{kip}{beam}$$

Moments at cg of the Bent Cap due to Wind load on superstructure

$$M_{super\_X} = WS_{super\_Z}\left(\frac{hCap}{2} + BrgTh + \frac{hsup}{2}\right) \qquad M_{super\_X} = 2.573\ \frac{kft}{beam}$$

$$M_{super\_Z} = WS_{super\_X}\left(\frac{hCap}{2} + BrgTh + \frac{hsup}{2}\right) \qquad M_{super\_Z} = 10.72\ \frac{kft}{beam}$$

WIND ON SUBSTRUCTURE: WS (*AASHTO LRFD 3.8.1.2.3*)

Base Wind pressure, $p_{sub} = 0.04\ ksf$ will be applied on exposed substructure both transverse & longitudinal direction

Wind on Columns

Wind force on columns **parallel** to bent (X-direction)

$$WS_{col\_X} = \left[p_{sub}\ (bCol\ \cos(\theta) + wCol\ \sin(\theta))\right] \qquad WS_{col\_X} = 0.16\ \frac{kip}{ft}$$

Apply WS loads at all columns even with zero degree attack angle.

Wind force on columns **normal** to bent (Z-direction)

$$WS_{col\_Z} = \left[p_{sub}\ (bCol\ \sin(\theta) + wCol\ \cos(\theta))\right] \qquad WS_{col\_Z} = 0.16\ \frac{kip}{ft}$$

Wind on Bent Cap & Ear Wall

$$WS_{ew\_X} = p_{sub}\ hEarWall\ (wEarWall\ \sin(\theta) + wCap\ \cos(\theta)) \qquad WS_{ew\_X} = 0\ kip$$

$$WS_{ew\_Z} = p_{sub}\ hEarWall\ (wEarWall\ \cos(\theta) + wCap\ \sin(\theta)) \qquad WS_{ew\_Z} = 0\ kip$$

Wind force on bent cap **parallel** to bent (X-direction)

$$WS_{cap\_X} = \left[p_{sub}\ hCap\ (CapL\ \sin(\theta) + wCap\ \cos(\theta))\right] + WS_{ew\_X} \quad \text{(punctual load)} \qquad WS_{cap\_X} = 0.9\ kip$$

Wind force on bent cap **normal** to bent (Z-direction)

$$WS_{cap\_Z} = \frac{\left[p_{sub}\ hCap\ (CapL\ \cos(\theta) + wCap\ \sin(\theta))\right] + WS_{ew\_Z}}{CapL} \qquad WS_{cap\_Z} = 0.2\ \frac{kip}{ft}$$

WIND ON VEHICLES: WL (*AASHTO LRFD 3.8.1.3*)

AASHTO LRFD Table 3.8.1.3-1 specifies the wind on live load components for various angles of attack. In order to simplify the analysis, this calculation considers as default values the maximum wind components as defined in the above mentioned table. The results can be considered conservative. For a more detailed analysis, use the proper skew angle according to the table.

AASHTO LRFD table 3.8.1.3-1

Skew Angle	Normal Componen	Parallel Component
Degrees	(Klf)	(Klf)
0	0.1	0
15	0.088	0.012
30	0.082	0.024
45	0.066	0.032
60	0.034	0.038

(suggested value 0.1 kip/ft) $p_{WLt} = 0.1\ \frac{kip}{ft}$

(suggested value 0.038 kip/ft) $p_{WLl} = 0.04\ \frac{kip}{ft}$

$$WL_{Par} = \frac{p_{WLl}\ TribuLength}{NofBm} \qquad WL_{Par} = 0.233\ \frac{kip}{beam}$$

$$WL_{Nor} = \frac{p_{WLt}\ TribuLength}{NofBm} \qquad WL_{Nor} = 0.583\ \frac{kip}{beam}$$

Wind force on live load **parallel** to bent (X-direction)

$$WL_X = WL_{Nor}\cos(\theta) + WL_{Par}\sin(\theta) \qquad WL_X = 0.583\ \frac{kip}{beam}$$

Wind force on live load **normal** to bent (Z-direction)

$$WL_Z = WL_{Nor}\sin(\theta) + WL_{Par}\cos(\theta) \qquad WL_Z = 0.233\ \frac{kip}{beam}$$

Moments at cg of the Bent Cap due to Wind load on Live Load

$$M_{WL\_X} = WL_Z\left(h_6 + \frac{hCap}{2}\right) \qquad M_{WL\_X} = 2.843\ \frac{kft}{beam}$$

$$M_{WL\_Z} = WL_X\left(h_6 + \frac{hCap}{2}\right) \qquad M_{WL\_Z} = 7.107\ \frac{kft}{beam}$$

Vertical Wind Pressure: *(AASHTO LRFD 3.8.2)*

$DeckWidth = FDeckW$ Bridge deck width including parapet and sidewalk

$P_{uplift} = -(0.02ksf)\ DeckWidth\ TribuLength$ (Acts upword Y-direction) $P_{uplift} = -66.033\ kip$

Applied at the windward quarter-point of the deck width.

Note: Applied only for Strength III and for Service IV for minimum permanent loads only. (*AASHTO LRFD table 3.4,1-2*, factors for permanent loads)

Load Combinations: using AASHTO LRFD Table 3.4.1-1

STRENGTH_I = 1.25 DC + 1.5 DW + 1.75 (LL + BR + CF) + 1.0 WA

STRENGTH_IA = 0.9 DC + 0.65 DW + 1.75 (LL + BR + CF) + 1.0 WA

STRENGTH_III = 1.25 DC + 1.5 DW + 1.4 WS + 1.0 WA + 1.4 P_{uplift}

STRENGTH_IIIA = 0.9 DC + 0.65 DW + 1.4 WS + 1.0 WA + 1.4 P_{uplift}

STRENGTH_V = 1.25 DC + 1.5 DW + 1.35 (LL + BR + CF) + 0.4 WS + 1.0 WA + 1.0 WL

STRENGTH_VA = 0.9 DC + 0.65 DW + 1.35 (LL + BR + CF) + 0.4 WS + 1.0 WA + 1.0 WL

SERVICE_I = 1.0 DC + 1.0 DW + 1.0 $\left(LL_{no\_Impact} + BR + CF\right)$ + 0.3 WS + 1.0 WA + 1.0 WL

All these loadings as computed above such as DC, DW, LL, WL, WA, WS, etc., are placed on the bent frame composed of bent cap and columns and drilled shafts. The frame is analyzed in RISA using load combinations as stated above. Output Loadings for various load combinations for column and drilled shaft are used to run PCA Column program to design the columns. It is found that **4'X4' Column with 20~#11 bars** is sufficient for the loadings. Drilled shaft or other foundation shall be designed for appropriate loads.

Total Vertical Foundation Load at Service I Limit State:

Forward Span Superstructure DC (F_{FDC}) & DW (F_{FDW}):

$F_{FDC} = (FNofBm - 2)\ FSuperDC_{Int} + 2\ FSuperDC_{Ext}$ $\qquad F_{FDC} = 259.607\ kip$

$F_{FDW} = (FNofBm)\ FSuperDW$ $\qquad F_{FDW} = 38.5\ kip$

Backward Span Superstructure DC (F_{BDC}) & DW (F_{BDW}):

$F_{BDC} = (BNofBm - 2)\ BSuperDC_{Int} + 2\ BSuperDC_{Ext}$ $\qquad F_{BDC} = 259.607\ kip$

$F_{BDW} = (BNofBm)\ BSuperDW$ $\qquad F_{BDW} = 38.5\ kip$

Total Cap Dead Load Weight (TCapDC):

$CapDC = CapDC1\ (CapL - LFoam) + CapDC2\ LFoam$ $\qquad CapDC = 126.979\ kip$

$TCapDC = CapDC + (NofBm)\ (BrgSeatDC) + EarWallDC$ $\qquad TCapDC = 126.979\ kip$

Total DL on columns including Cap weight (F_{DC}):

$F_{DL} = \left(F_{FDC} + F_{FDW}\right) + \left(F_{BDC} + F_{BDW}\right) + TCapDC$ $\qquad F_{DL} = 723.194\ kip$

Column & Drilled Shaft Self Weight:

DShaft Length, $DsHt = 0\ ft$ $\qquad$ if Rounded Col, $ColDia = 0\ ft$

$ColDC = if\left[ColDia > 0ft, \frac{\pi}{4}\ (ColDia)^2\ (HCol)\ \gamma_c, wCol\ bCol\ HCol\ \gamma_c\right]$ $\qquad$ Column Wt, $ColDC = 52.8\ kip$

$DsDC = \frac{\pi}{4}\ (DsDia)^2\ (DsHt)\ \gamma_c$ $\qquad$ Dr Shaft Wt, $DsDC = 0\ kip$

Total Dead Load on Drilled Shaft (DL_on_DShaft):

$DL\_on\_DShaft = F_{DL} + (NofCol)\ (ColDC) + (NofDs)\ (DsDC)$ $\qquad DL\_on\_DShaft = 828.794\ kip$

Live Load on Drilled Shaft:

$m = 0.85$ (Multiple Presence Factors for 3 Lanes) $\qquad$ (AASHTO LRFD Table 3.6.1.1.2 − 1

$R_{LL} = (Lane + Truck)\ (NofLane)\ (m)$ (Total LIVE LOAD without Impact) $\qquad R_{LL} = 277.44\ kip$

Total Load, DL+LL per Drilled Shaft of Intermediate Bent:

$$Load\_on\_DShaft = \frac{DL\_on\_DShaft + R_{LL}}{NofDs}$$ $Load\_on\_DShaft = 276.6\ ton$

5. PRECAST COMPONENT DESIGN

Precast Cap Construction and Handling:

$w_1 = b\ h\ \gamma_c$ applicable for $0\ ft \le L_{cap} \le 6\ ft$ $w_1 = 3.375\ klf$ (Cap selfweight)

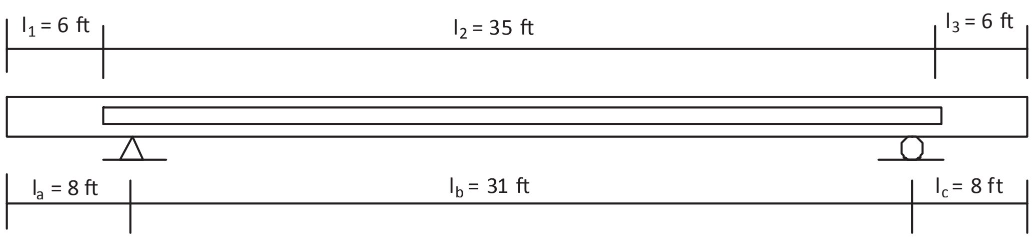
$w_2 = (b\ h - 2\ wFoam\ hFoam)\ \gamma_c$ applicable for $6\ ft \le L_{cap} \le 41\ ft$ $w_2 = 2.471\ klf$ (Cap selfweight)

$w_3 = b\ h\ \gamma_c$ applicable for $41\ ft \le L_{cap} \le 47\ ft$ $w_3 = 3.375\ klf$ (Cap selfweight)

$l_1 = 6\ ft$ $l_2 = 35\ ft$ $l_3 = 6\ ft$

$L_{cap} = l_1 + l_2 + l_3$ (Total Cap Length) $L_{cap} = 47\ ft$

Due to the location of girder bolts, pickup points at 8' from both ends. Indeed, we can model cap lifting points as simply supported beam under self weight supported at 8' and 39' respectively from very end.



$l_a = 8\ ft$ $l_b = 31\ ft$ $l_c = 8ft$

Construction factor:

$\lambda_{cons} = 1.25$ $\lambda_{cons} = 1.25$

Maximum Positive Moment (M_{maxP}) & Negative Moment (M_{maxN}):

$R_{xn} = 0.5\left(w_1\ l_1 + w_2\ l_2 + w_3\ l_3\right)$ $R_{xn} = 63.49\ kip$

$$M_{maxP} = R_{xn}\frac{l_b}{2} - w_1\ l_1\left(\frac{l_1}{2} + l_a - l_1 + \frac{l_b}{2}\right) - \frac{w_2}{2}\left(l_a - l_1 + \frac{l_b}{2}\right)^2$$ $M_{maxP} = 190.617\ kft$

$$M_{maxN} = w_1\ l_1\left(\frac{l_1}{2} + l_a - l_1\right) + \frac{w_2}{2}\left(l_a - l_1\right)^2$$ $M_{maxN} = 106.192\ kft$

Factored Maximum Positive Moment (M_{uP}) & Negative Moment (M_{uN}):

$M_{uP} = \lambda_{cons}\ M_{maxP}$ (Positive Moment at the middle of the cap) $M_{uP} = 238.271\ kft$

$M_{uN} = \lambda_{cons}\ M_{maxN}$ (Negative Moment at the support point) $M_{uN} = 132.74\ kft$

Maximum Positive Stress (f_{tP}) & Negative Stress (f_{tN}):

$$f_{tP} = \frac{M_{uP}\left(h - y_{cg2}\right)}{I_{cap2}} \qquad f_{tP} = 95.657 \text{ psi}$$

$$f_{tN} = \frac{M_{uN}\, y_{cg2}}{I_{cap2}} \qquad f_{tN} = 52.644 \text{ psi}$$

Modulus of Rupture: According PCI hand book 6th edition, modulus of rupture, fr = 7.5√f'c is divided by a safety factor 1.5 in order to design a member without cracking

$f'_c = 5$ ksi (Compressive Strength of Concrete) Unit weight factor, $\lambda = 1$

$f_r = 5\,\lambda\,\sqrt{f'_c \text{ psi}}$ (PCI EQ 5.3.3.2) $f_r = 353.553$ psi

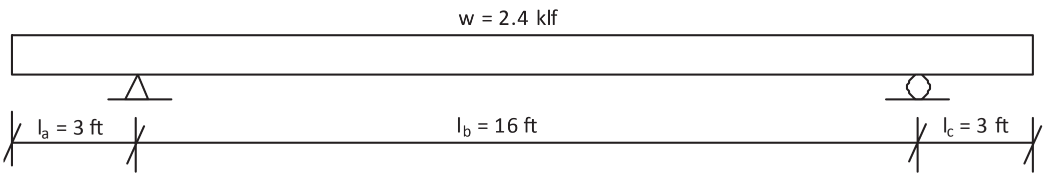
$f_{r\_check} = \text{if}\left[\left(f_r > f_{tP}\right)\left(f_r > f_{tN}\right), \text{"OK"}, \text{"N.G."}\right]$ $f_{r\_check} = \text{"OK"}$

Precast Column Construction and Handling:

$wCol = 4$ ft (Column width) Column breadth, $bCol = 4$ ft

$w_{col} = wCol\ bCol\ \gamma_c$ (Column self weight) $w_{col} = 2.4$ klf

Due to the location of girder bolts on column, pick up points at 3' from both ends. Indeed, we can model column lifting points as simply supported beam under self weight supported at 3' and 19' respectively from very end.



$l_a = 3$ ft $l_b = 16$ ft $l_c = 3$ ft

Maximum Positive Moment (M_{maxP}) & Negative Moment (M_{maxN}):

$$M_{maxP} = \frac{w_{col}\ HCol}{2}\left(\frac{HCol}{4} - l_a\right) \qquad M_{maxP} = 66 \text{ kft}$$

$$M_{maxN} = \frac{w_{col}\ l_a^2}{2} \qquad M_{maxN} = 10.8 \text{ kft}$$

Factored Maximum Positive Moment (M_{uP}) & Negative Moment (M_{uN}):

$$M_{uP} = \lambda_{cons}\ M_{maxP} \qquad M_{uP} = 82.5 \text{ kft}$$

$$M_{uN} = \lambda_{cons}\ M_{maxN} \qquad M_{uN} = 13.5 \text{ kft}$$

$$S_{col} = \frac{wCol\ bCol^2}{6} \quad \text{(Column Section Modulus)} \qquad S_{col} = 18432\ in^3$$

Maximum Positive Stress (f_{tP}) & Negative Stress (f_{tN}):

$$f_{tP} = \frac{M_{uP}}{S_{col}} \qquad f_{tP} = 53.711\ psi$$

$$f_{tN} = \frac{M_{uN}}{S_{col}} \qquad f_{tN} = 8.789\ psi$$

Modulus of Rupture: According PCI hand book 6th edition modulus of rupture, fr = 7.5√f'c is divided by a safety factor 1.5 in order to design a member without cracking

$f'_c = 5\ ksi$ (Compressive Strength of Concrete) Unit weight factor, $\lambda = 1$

$$f_r = 5\ \lambda \sqrt{f'_c\ psi} \quad \text{(PCI EQ 5.3.3.2)} \qquad f_r = 353.553\ psi$$

$$f_{r\_check} = \text{if}\left[\left(f_r > f_{tP}\right)\left(f_r > f_{tN}\right), \text{"OK"}, \text{"N.G."}\right] \qquad f_{r\_check} = \text{"OK"}$$

DEVELOPMENT LENGTH: AASHTO LRFD 5.11

$A_b = 1.56\ in^2$ (Area of Bar) $d_b = 1.41\ in$ (Diameter of Bar) $f'_c = 5\ ksi$

Modification Factor: According to AASHTO LRFD 5.11.2.1.2, the basic development length, l_{db} is required to multiply by the modification factor to obtain the development length l_d for tension or compression.

$\lambda_{mod} = 1.0$

Basic Tension Development: AASHTO LRFD 5.11.2.1 for bars up to #11

$$l_{db} = \max\left[1.25\left(\frac{A_b}{in}\right)\frac{f_y}{\sqrt{f'_c\ ksi}}, 0.4\ d_b\ \frac{f_y}{ksi}, 12\ in\right] \quad \text{(AASHTO LRFD 5.11.2.1.1)} \qquad l_{db} = 52.324\ in$$

$$l_d = \left(\lambda_{mod}\right) l_{db} \qquad l_d = 4.36\ ft$$

Basic Compression Development: AASHTO LRFD 5.11.2.2

$$l_{db} = \max\left(\frac{0.63\ d_b\ f_y}{\sqrt{f'_c\ ksi}}, 0.3\ d_b\ \frac{f_y}{ksi}, 8\ in\right) \quad \text{AASHTO LRFD EQ (5.11.2.2.1 - 1, 2)} \qquad l_{db} = 25.38\ in$$

$$l_d = \left(\lambda_{mod}\right) l_{db} \qquad l_d = 2.115\ ft$$

ABC SAMPLE CALCULATION – 3b

Precast Pier Design for ABC (70’ Conventional Pier)

PRECAST PIER DESIGN FOR ABC (70' SPAN CONVENTIONAL PIER)

FNofBm = Total Number of Beams in Forward Span

FSpan = Forward Span Length

FDeckW = Out to Out Forward Span Deck Width

FBmAg = Forward Span Beam X Sectional Area

FBmFlange = Forward Span Beam Top Flange Width

FHaunch = Forward Span Haunch Thickness

FBmD = Forward Span Beam Depth or Height

FBmIg = Forward Span Beam Moment of Inertia

y_{Ft} = Forward Span Beam Top Distance from cg

SlabTh = Slab Thickness

RailWt = Railing Weight

RailH = Railing Height

RailW = Rail Base Width

DeckOH = Deck Overhang Distance

DeckW = Out to Out Deck Width at Bent

RoadW = Roadway Width

BrgTh = Bearing Pad Thickness + Bearing Seat Thickness

NofLane = Number of Lanes

wCap = Cap Width

hCap = Cap Depth

CapL = Cap Length

γ_c = Unit Weight of Concrete

w_c = Unit Weight of Concrete

$SlabDC_{Int}$ = Dead Load for Slab per Interior Beam

BNofBm = Total Number of Beams in Backward Span

BSpan = Backward Span Length

BDeckW = Out to Out Backward Span Deck Width

BBmAg = Backward Span Beam X Sectional Area

BBmFlange = Backward Span Beam Top Flange Width

BHaunch = Backward Span Haunch Thickness

BBmD = Backward Span Beam Depth or Height

BBmIg = Backward Span Beam Moment of Inertia

y_{Bt} = Backward Span Beam Top Distance from cg

NofCol = Number of Columns per Bent

NofDs = Number of Drilled Shaft per Bent

wCol = Width of Column Section

bCol = Breadth of Column Section

DsDia = Drilled Shaft Diameter

HCol = Height of Column

wEarWall = Width of Ear Wall

hEarWall = Height of Ear Wall

tEarWall = Thickness of Ear Wall

tSWalk = Thickness of Side Walk

bSWalk = Breadth of Side Walk

BmMat = Beam Material either Steel or Concrete

DiapWt = Weight of Diaphragm

γ_{st} = Unit Weight of Steel

$SlabDC_{Ext}$ = Dead Load for Slab per Exterior Beam

BeamDC = Self Weight of Beam

HaunchDC = Dead Load of Haunch Concrete per Beam

RailDC = Weight of Rail per Beam

$FSuperDC_{Int}$ = Half of Forward Span Super Structure Dead Load Component per Interior Beam

$FSuperDC_{Ext}$ = Half of Forward Span Super Structure Dead Load Component per Exterior Beam

FSuperDW = Half of Forward Span Overlay Dead Load Component per Beam

$BSuperDC_{Int}$ = Half of Backward Span Super Structure Dead Load Component per Interior Beam

$BSuperDC_{Ext}$ = Half of Backward Span Super Structure Dead Load Component per Exterior Beam

BSuperDW = Half of Backward Span Overlay Dead Load Component per Beam

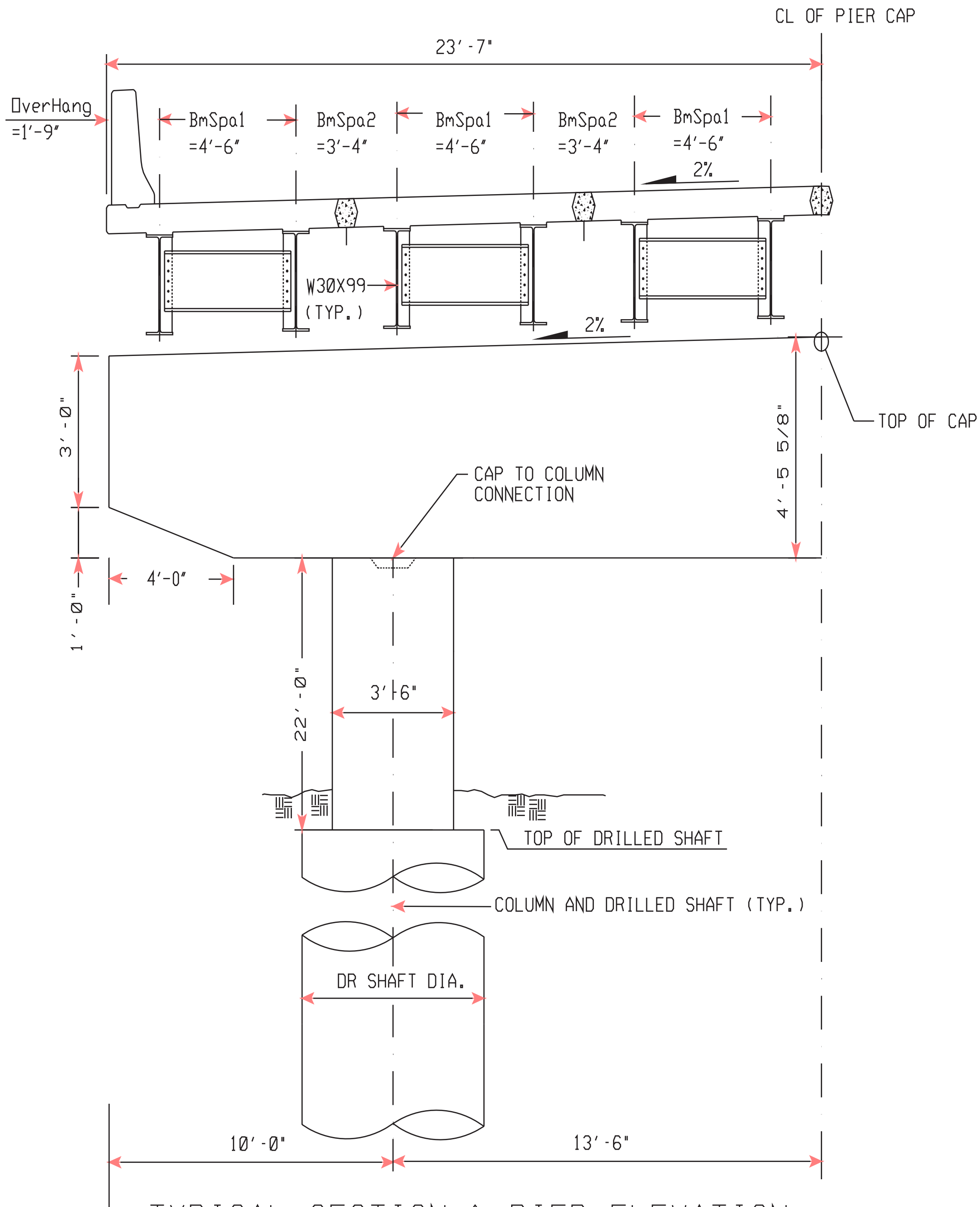
$TorsionDC_{Int}$ = DeadLoad Torsion in a Cap due to difference in Forward and Backward span length per Interior Beam

$TorsionDC_{Ext}$ = DeadLoad Torsion in a Cap due to difference in Forward and Backward span length per Exterior Beam

TorsionDW = DW Torsion in a Cap due to difference in Forward and Backward span length per Beam

tBrgSeat = Thickness of Bearing Seat

bBrgSeat = Breadth of Bearing Seat



TYPICAL SECTION & PIER ELEVATION

Note: Use of Light Weight Concrete (LWC) may be considered to reduce the weight of the pier cap instead of using styrofoam blockouts.

FORWARD SPAN PARAMETER INPUT:

$FNofBm = 12$ $FSpan = 70\ ft$ $FDeckW = \frac{283}{6}\ ft$ $FBmAg = 29.1\ in^2$ $FBmFlange = 10.5\ in$

$FHaunch = 0\ in$ $FBmD = 29.7\ in$ $FBmIg = 3990\ in^4$ $y_{Ft} = 14.85\ in$

BACKWARD SPAN PARAMETER INPUT:

$BNofBm = 12$ $BSpan = 70\ ft$ $BDeckW = \frac{283}{6}\ ft$ $BBmAg = 29.1\ in^2$ $BBmFlange = 10.5\ in$

$BHaunch = 0\ in$ $BBmD = 29.7\ in$ $BBmIg = 3990\ in^4$ $y_{Bt} = 14.85\ in$

COMMON BRIDGE PARAMETER INPUT: Bent in Question Parameters

$SlabTh = 9\ in$ $Overlay = 25\ psf$ $\theta = 0\ deg$ $DeckOH = 1.75\ ft$ $BrgTh = 3.5\ in$

$RailWt = 0.43\ klf$ $RailW = 19\ in$ $RailH = 34.0\ in$ $tBrgSeat = 0\ in$ $bBrgSeat = 0\ ft$

$DeckW = \frac{283}{6}\ ft$ $NofLane = 3$ $m = 0.85$ $w_c = 0.150\ kcf$ $f'_c = 5\ ksi$ (Cap)

$wCap = 4.0\ ft$ $hCap = 4.0\ ft$ $CapL = 47\ ft$ $NofDs = 2$ $DsDia = 5\ ft$

$wCol = 3.5\ ft$ $bCol = 3.5\ ft$ $NofCol = 2$ $HCol = 22.00\ ft$ $f'_{cs} = 4\ ksi$ (Slab)

$\gamma_c = 0.150\ kcf$ $e_{brg} = 13\ in$ $NofBm = 12$ $Sta = 0.25\ \frac{ft}{incr}$ $DiapWt = 0.2\ kip$

$wEarWall = 0\ ft$ $hEarWall = 0\ ft$ $tEarWall = 0\ in$ $IM = 0.33$ $BmMat = Steel$

$E_s = 29000\ ksi$ $\gamma_{st} = 490\ pcf$ (steel)

Modulus of elasticity of Concrete:

$E(f'_c) = 33000\ (w_c)^{1.5}\sqrt{f'_c\ ksi}$ (AASHTO LRFD EQ 5.4.2.4-1 for $K_1 = 1$)

$E_{slab} = E(f'_{cs})$ $E_{slab} = 3834.254\ ksi$

$E_{cap} = E(f'_c)$ $E_{cap} = 4286.826\ ksi$

Modulus of Beam or Girder: Input Beam Material, BmMat = Steel or Concrete

$E_{beam} = if(BmMat = Steel, E_s, E(f'_c))$ $E_{beam} = 29000\ ksi$

1. BENT CAP LOADING

DEAD LOAD FROM SUPERSTRUCTURE:

The permanent dead load components (DC) consist of slab, rail, sidewalk, haunch weight, and beam self weight. Slab Dead weight components will be distributed to each beam by slab tributary width between beams. Interior Beam tributary width (IntBmTriW) is taken as the average of consecutive beam spacing for a particular interior beam. Exterior Beam tributary width (ExtBmTriW) is taken as half of beam spacing plus the overhang distance. Rail, sidewalk dead load components and future wearing surface weight components (DW) can be distributed evenly among each beam. Half of DC and DW components from forward span and backward span comprise the total superstructure load or dead load reaction per beam on the pier cap or the bent cap.

FORWARD SPAN SUPERSTRUCTURE DEAD LOAD: consists of 12 W30x99 Beams

12 beams were spaced 4.5' and 3'-4" alternately in forward span. For beam spacing see Typical Section Details sheet

$\text{FBmSpa1} = 4.5\ \text{ft}$ $\qquad$ $\text{FBmSpa2} = \dfrac{10}{3}\ \text{ft}$

$$\text{FIntBmTriW} = \frac{\text{FBmSpa1}}{2} + \frac{\text{FBmSpa2}}{2} \qquad \text{FIntBmTriW} = 3.917\ \text{ft}$$

$$\text{FExtBmTriW} = \frac{\text{FBmSpa1}}{2} + \text{DeckOH} \qquad \text{FExtBmTriW} = 4\ \text{ft}$$

$$\text{RoadW} = 0.25\,(\text{FDeckW} + 3\ \text{DeckW}) - 2\ \text{RailW} \qquad \text{RoadW} = 44\ \text{ft}$$

$$\text{SlabDC}_{\text{Int}} = \gamma_c\ \text{FIntBmTriW}\ \text{SlabTh}\left(\frac{\text{FSpan}}{2}\right) \qquad \text{SlabDC}_{\text{Int}} = 15.422\ \frac{\text{kip}}{\text{beam}}$$

$$\text{SlabDC}_{\text{Ext}} = \gamma_c\ \text{FExtBmTriW}\ \text{SlabTh}\left(\frac{\text{FSpan}}{2}\right) \qquad \text{SlabDC}_{\text{Ext}} = 15.75\ \frac{\text{kip}}{\text{beam}}$$

$$\text{BeamDC} = \gamma_{st}\ \text{FBmAg}\left(\frac{\text{FSpan}}{2}\right) \qquad \text{BeamDC} = 3.466\ \frac{\text{kip}}{\text{beam}}$$

$$\text{HaunchDC} = \gamma_c\ \text{FHaunch}\ \text{FBmFlange}\left(\frac{\text{FSpan}}{2}\right) \qquad \text{HaunchDC} = 0\ \frac{\text{kip}}{\text{beam}}$$

NOTE: Permanent loads such as the weight of the Rail (Barrier), Future wearing surface may be distributed uniformly among all beams if following conditions are met. Apply for live load distribution factors too. AASHTO LRFD 4.6.2.2.1

1. Width of deck is constant
2. Number of Beams >= 4 beams
3. Beams are parallel and have approximately same stiffness
4. The Roadway part of the overhang, d_e<= 3ft
5. Curvature in plan is < 4°
6. Bridge cross-section is consistent with one of the x-section shown in AASHTO LRFD TABLE 4.6.2.2.1-1

$$\text{RailDC} = \frac{2\ \text{RailWt}}{\text{FNofBm}}\left(\frac{\text{FSpan}}{2}\right) \qquad \text{RailDC} = 2.508\ \frac{\text{kip}}{\text{beam}}$$

$$\text{OverlayDW} = \frac{\text{RoadW}\ \text{Overlay}}{\text{FNofBm}}\left(\frac{\text{FSpan}}{2}\right) \qquad \text{OverlayDW} = 3.208\ \frac{\text{kip}}{\text{beam}}$$

Forward Span Superstructure DC & DW per Interior and Exterior Beam:

$$FSuperDC_{Int} = RailDC + BeamDC + SlabDC_{Int} + HaunchDC + DiapWt \qquad FSuperDC_{Int} = 21.596\ \frac{kip}{beam}$$

$$FSuperDC_{Ext} = RailDC + BeamDC + SlabDC_{Ext} + HaunchDC + 0.5\ DiapWt \qquad FSuperDC_{Ext} = 21.824\ \frac{kip}{beam}$$

$$FSuperDW = OverlayDW \qquad FSuperDW = 3.208\ \frac{kip}{beam}$$

BACKWARD SPAN SUPERSTRUCTURE DEAD LOAD: Consists of 12 W30x99 beams

12 beams were spaced 4.5' and 3'-4" alternately in Backward span. For beam spacing see Typical Section Details sheet

$$BBmSpa1 = 4.5\ ft \qquad BBmSpa2 = \frac{10}{3}\ ft$$

$$BIntBmTriW = \frac{BBmSpa1}{2} + \frac{BBmSpa2}{2} \qquad BIntBmTriW = 3.917\ ft$$

$$BExtBmTriW = \frac{BBmSpa1}{2} + DeckOH \qquad BExtBmTriW = 4\ ft$$

$$RoadW = 0.25\ (BDeckW + 3\ DeckW) - 2\ RailW \qquad RoadW = 44\ ft$$

$$SlabDC_{Int} = \gamma_c\ BIntBmTriW\ SlabTh\left(\frac{BSpan}{2}\right) \qquad SlabDC_{Int} = 15.422\ \frac{kip}{beam}$$

$$SlabDC_{Ext} = \gamma_c\ BExtBmTriW\ SlabTh\left(\frac{BSpan}{2}\right) \qquad SlabDC_{Ext} = 15.75\ \frac{kip}{beam}$$

$$BeamDC = \gamma_{st}\ BBmAg\left(\frac{BSpan}{2}\right) \qquad BeamDC = 3.466\ \frac{kip}{beam}$$

$$HaunchDC = \gamma_c\ BHaunch\ BBmFlange\left(\frac{BSpan}{2}\right) \qquad HaunchDC = 0\ \frac{kip}{beam}$$

$$RailDC = \frac{2\ RailWt}{BNofBm}\left(\frac{BSpan}{2}\right) \qquad RailDC = 2.508\ \frac{kip}{beam}$$

$$OverlayDW = \frac{RoadW\ Overlay}{BNofBm}\left(\frac{BSpan}{2}\right) \qquad OverlayDW = 3.208\ \frac{kip}{beam}$$

Total Backward Span Superstructure DC & DW per Interior and Exterior Beam:

$$BSuperDC_{Int} = RailDC + BeamDC + SlabDC_{Int} + HaunchDC + DiapWt \qquad BSuperDC_{Int} = 21.596\ \frac{kip}{beam}$$

$$BSuperDC_{Ext} = RailDC + BeamDC + SlabDC_{Ext} + HaunchDC + 0.5\ DiapWt \qquad BSuperDC_{Ext} = 21.824\ \frac{kip}{beam}$$

$$BSuperDW = OverlayDW \qquad BSuperDW = 3.208\ \frac{kip}{beam}$$

Total Superstructure DC & DW Reactions per Beam on Bent Cap:

$SuperDC_{Int} = FSuperDC_{Int} + BSuperDC_{Int}$ $\qquad SuperDC_{Int} = 43.192 \frac{kip}{beam}$

$SuperDC_{Ext} = FSuperDC_{Ext} + BSuperDC_{Ext}$ $\qquad SuperDC_{Ext} = 43.648 \frac{kip}{beam}$

$SuperDW = FSuperDW + BSuperDW$ $\qquad SuperDW = 6.417 \frac{kip}{beam}$

$TorsionDC_{Int} = \left(max\left(FSuperDC_{Int}, BSuperDC_{Int}\right) - min\left(FSuperDC_{Int}, BSuperDC_{Int}\right)\right) e_{brg}$ $\qquad TorsionDC_{Int} = 0 \frac{kft}{beam}$

$TorsionDC_{Ext} = \left(max\left(FSuperDC_{Ext}, BSuperDC_{Ext}\right) - min\left(FSuperDC_{Ext}, BSuperDC_{Ext}\right)\right) e_{b}$ $\qquad TorsionDC_{Ext} = 0 \frac{kft}{beam}$

$TorsionDW = (max(FSuperDW, BSuperDW) - min(FSuperDW, BSuperDW))\ e_{brg}$ $\qquad TorsionDW = 0 \frac{kft}{beam}$

CAP, EAR WALL & BEARING SEAT WEIGHT:

The bent cap has only one solid section along the length. The solid rectangular section of 4'X4' can be seen in typical section and pier elevation figure. CapDC is the weight of the section of the bent or pier cap.

$CapDC = wCap\ hCap\ \gamma_c$ $\qquad CapDC = 2.4\ klf$

$CapDC_{sta} = \left(wCap\ hCap\ \gamma_c\right)(Sta)$ $\qquad CapDC_{sta} = 0.6 \frac{kip}{incr}$

$EarWallDC = (wEarWall\ hEarWall\ tEarWall)\ \gamma_c$ $\qquad EarWallDC = 0\ kip$

$BrgSeatDC = tBrgSeat\ bBrgSeat\ (wCap)\ \gamma_c$ $\qquad BrgSeatDC = 0 \frac{kip}{beam}$

$EI_{cap} = E_{cap}\left(\frac{wCap\ hCap^3}{12}\right)$ $\qquad EI_{cap} = 1.317 \times 10^7\ kip\ ft^2$

RESULTS OF DISTRIBUTION FACTORS:

Forward Span Distribution Factors:

$DFM_{Fmax} = 0.391$ (Distribution Factor for Moment)

$DFS_{Fmax} = 0.558$ (Distribution Factor for Shear)

Backward Span Distribution Factors:

$DFM_{Bmax} = 0.391$ (Distribution Factor for Moment)

$DFS_{Bmax} = 0.558$ (Distribution Factor for Shear)

LIVE LOAD FOR SIMPLY SUPPORTED BRIDGE:

HL-93 Loading: According to AASHTO LRFD 3.6.1.2.1, HL-93 consists of Design Truck + Design Lane Load or Design Tandem + Design Lane Load. Design Truck rather than Design Tandem + Design Lane Load controls the maximum Live Load Reactions at an interior bent for a span longer than 26'. For maximum reaction, place middle axle (P_2 = 32 kip) of design truck over the support at a bent between the forward and the backward span and place rear axle (P_3 = 32 kip) 14' away from P_2 on the longer span while placing P_1 14' away from P_1 on either spans yielding maximum value.

P_1 **=** Front Axle of Design Truck P_2 **=** Middle Axle of Design Truck P_3 **=** Rear Axle of Design Truck

Design Truck Axle Load: $P_1 = 8\text{ kip}\ P_2 = 32\text{ kip}\ P_3 = 32\text{ kip}$ (AASHTO LRFD 3.6.1.2.2) $\text{TruckT} = P_1 + P_2 + P_3$

Design Lane Load: $w_{lane} = 0.64\text{ klf}$ (AASHTO LRFD 3.6.1.2.4)

Longer Span Length, $L_{long} = \max(\text{FSpan}, \text{BSpan})$ Shorter Span Length, $L_{short} = \min(\text{FSpan}, \text{BSpan})$

Lane Load Reaction:

$$\text{Lane} = w_{lane}\left(\frac{L_{long} + L_{short}}{2}\right) \qquad \text{Lane} = 44.8\,\frac{\text{kip}}{\text{lane}}$$

Truck Load Reaction:

$$\text{Truck} = P_2 + P_3\,\frac{(L_{long} - 14\text{ft})}{L_{long}} + P_1\,\max\left[\frac{(L_{long} - 28\text{ft})}{L_{long}}, \frac{(L_{short} - 14\text{ft})}{L_{short}}\right] \qquad \text{Truck} = 64\,\frac{\text{kip}}{\text{lane}}$$

Maximum Live Load Reaction with Impact (LLRxn) over support on Bent:

The Dynamic Load Allowance or Impact Factor, $IM = 0.33$ (AASHTO LRFD Table 3.6.2.1 − 1)

$$\text{LLRxn} = \text{Lane} + \text{Truck}\,(1 + IM) \qquad \text{LLRxn} = 129.92\,\frac{\text{kip}}{\text{lane}}$$

Live Load Model for Cap Loading Program:

AASHTO LRFD Recommended Live Load Model For Cap Loading Program: Live Load reaction on the pier cap using distribution factors are not sufficient to design bent cap for moment and shear. Therefore, the reaction from live load is uniformly distributed to over a 10' width (which becomes W) and the reaction from the truck is applied as two concentrated loads (P and P) 6' apart. The loads act within a 12' wide traffic lane. The reaction W and the truck move across the width of the traffic lane. However, neither of the P loads can be placed closer than 2' from the edge of the traffic lane. One lane, two lane, three lane and so forth loaded traffic can be moved across the width of the roadway to create maximum load effects.

Load on one rear wheel out of rear axle of the truck with Impact:

$$P = (0.5\,P_3)\,(1 + IM) \qquad P = 21.28\text{ kip}$$

The Design Lane Load Width Transversely in a Lane

$\text{wlaneTransW} = 10\text{ ft}$ AASHTO LRFD Article 3.6.1.2.1

The uniform load portion of the Live Load, kip/station for Cap Loading Program:

$$W = \frac{(\text{LLRxn} - 2\,P)\,\text{Sta}}{\text{wlaneTransW}} \qquad W = 2.184\,\frac{\text{kip}}{\text{incr}}$$

LOADS generated above will be placed into a CAP LOADING PROGRAM to obtain moment and shear values for Bent Cap design.

Torsion on Bent Cap per Beam and per Drilled Shaft:

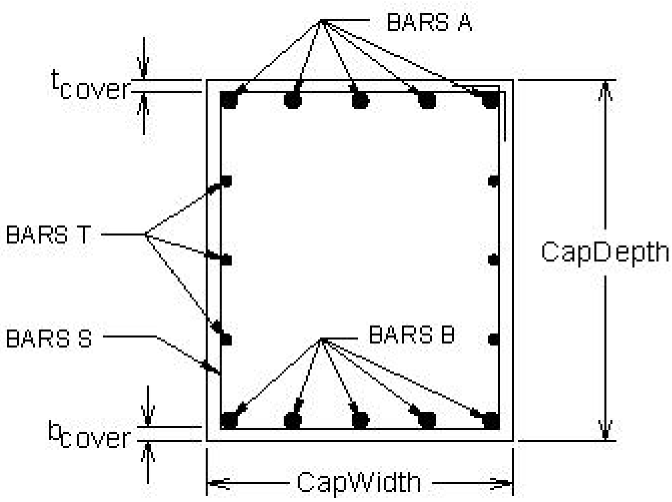
Torsional load about center line of bent cap occurs due to horizontal loads acting on the superstructure perpendicular to the bent length or along the bridge length. Braking force, Centrifugal force, WS on superstructure, and WL cause torsion on bent.

In addition, torque about center line of bent cap for the dead load reaction on beam brg location occurs due to differences in forward and backward span length and eccentricity between center line of bent cap and brg location. Torsion can be neglected if $Tu<0.25\phi Tcr$ (AASHTO LRFD 5.8.2.1)

The maximum torsional effects on the pier cap will be obtained from RISA frame analysis under loading as stated in AASHTO LRFD SECTION 3 for different load combinations using AASHTO LRFD Table 3.4.1-1

2. BENT CAP FLEXURAL DESIGN

FLEXURAL DESIGN OF BENT CAP:



(h)

(b)

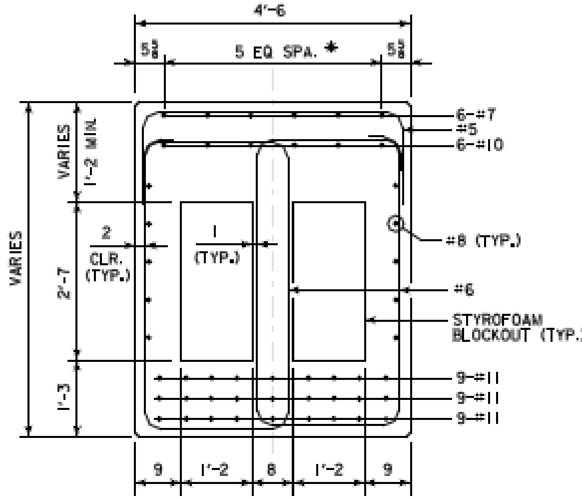
$f'_c = 5.0$ ksi $f_y = 60$ ksi $E_s = 29000$ ksi $\phi_m = 0.9$ $\phi_v = 0.9$ $\phi_n = 1$

$\gamma_c = 0.150$ kcf $b_{cover} = 2.5$ in $t_{cover} = 2.5$ in $h = 4.0$ ft $b = 4.0$ ft $E_c = E_{cap}$

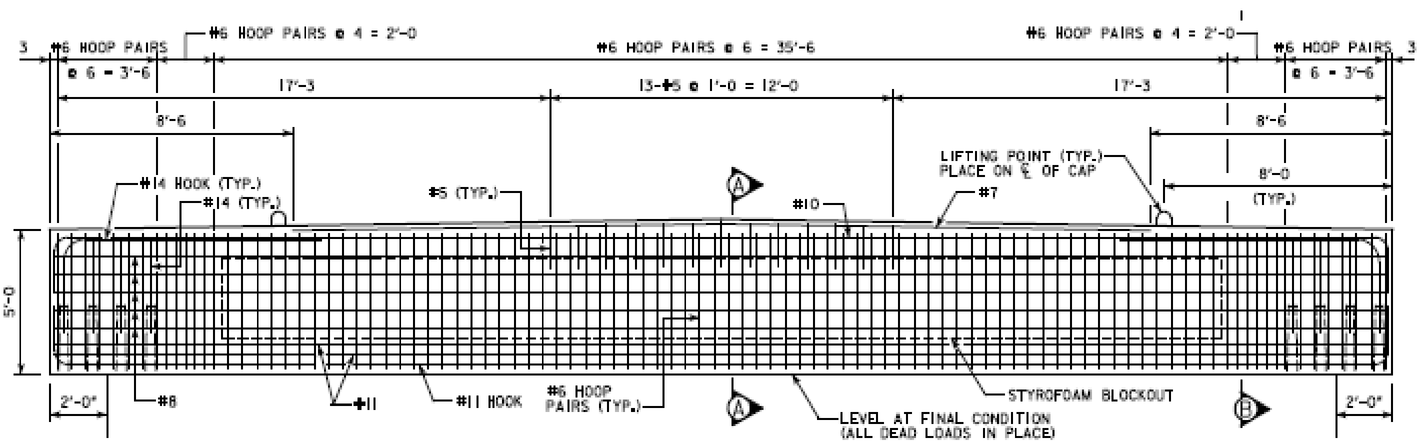
OUTPUT of BENT CAP LOADING PROGRAM: The maximum load effects from different applicable limit states:

DEAD LOAD	$M_{dlPos} = 627.2$ kft	$M_{dlNeg} = 783.4$ kft
SERVICE I	$M_{sPos} = 1462.5$ kft	$M_{sNeg} = 1297.7$ kft
STRENGTH I	$M_{uPos} = 1900.5$ kft	$M_{uNeg} = 2262.8$ kft

FLEXURE DESIGN:



SECTION A-A
PRECAST PIER CAP
(70' SPAN)



PIER CAP ELEVATION

Minimum Flexural Reinforcement *AASHTO LRFD 5.7.3.3.2*

Factored Flexural Resistance, M_r, must be greater than or equal to the lesser of $1.2M_{cr}$ or $1.33\ M_u$. Applicable to both positive and negative moment.

Modulus of rupture

$f_r = 0.37\sqrt{f'_c\ ksi}$ (AASHTO LRFD EQ 5.4.2.6) $f_r = 0.827$ ksi

$S = \dfrac{b\,h^2}{6}$ (Section Modulus) $S = 18432\ in^3$

Cracking moment

$M_{cr} = S\,f_r$ (AASHTO LRFD EQ 5.7.3.3.2-1) $M_{cr} = 1270.802$ kip ft

$M_{cr1} = 1.2\ M_{cr}$ $M_{cr1} = 1524.963$ kip ft

$M_{cr2} = 1.33\ \max(M_{uPos}, M_{uNeg})$ $\qquad M_{cr2} = 3009.524\ kip\ ft$

$M_{cr\_min} = \min(M_{cr1}, M_{cr2})$ Therefore M_r must be <u>greater</u> than $\qquad M_{cr\_min} = 1524.963\ kip\ ft$

Moment Capacity Design (Positive Moment, Bottom Bars B) *AASHTO LRFD 5.7.3.2*

Bottom Steel arrangement for the Cap:

Input no. of total rebar in a row from bottom of cap up to 12 rows (in unnecessary rows input zero)

$N_p = (5\ \ 5\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0)$

Input area of rebar corresponding to above rows from bottom of cap, not applicable for mixed rebar in a single row

$A_{bp} = (1.56\ \ 1.56\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0)\ in^2$

Input center to center vertical distance between each rebar row starting from bottom of cap

$clp = (3.5\ \ 4\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0)\ in$

$ns_{Pos} = 2$ (No. of Bottom or Positive Steel Layers)

Distance from centroid of positive rebar to extreme bottom tension fiber (d_{cPos}):

$d_{cPos} = (Ayp_{0,0})\ in$ $\qquad d_{cPos} = 5.5\ in$

Effective depth from centroid of bottom rebar to extreme compression fiber (d_{Pos}):

$d_{Pos} = h - d_{cPos}$ $\qquad d_{Pos} = 42.5\ in$

Compression Block depth under ultimate load *AASHTO LRFD 5.7.2.2*

$$\beta_1 = \min\left[0.85, \max\left[0.65, 0.85 - \frac{0.05}{ksi}(f'_c - 4\ ksi)\right]\right] \qquad \beta_1 = 0.8$$

The Amount of Bottom or Positive Steel A_s Required, $\qquad b = 48\ in$

$$A_{sReq} = \left(\frac{0.85\ f'_c\ b\ d_{Pos}}{f_y}\right)\left(1 - \sqrt{1 - \frac{2\ M_{uPos}}{0.85\ \phi_m\ f'_c\ b\ {d_{Pos}}^2}}\right) \qquad A_{sReq} = 10.305\ in^2$$

The Amount of Positive A_s Provided,

$NofBars_{Pos} = \sum N_p$ $\qquad NofBars_{Pos} = 10$

$A_{sPos} = (Ayp_{0,1})\ in^2$ $\qquad A_{sPos} = 15.6\ in^2$

Compression depth under ultimate load

$$c_{Pos} = \frac{A_{sPos} f_y}{0.85 f'_c \beta_1 b}$$ (AASHTO LRFD EQ 5.7.3.1.1-4) $c_{Pos} = 5.735$ in

$$a_{Pos} = \beta_1 c_{Pos}$$ (AASHTO LRFD 5.7.3.2.2) $a_{Pos} = 4.588$ in

Nominal flexural resistance:

$$M_{nPos} = A_{sPos} f_y \left(d_{Pos} - \frac{a_{Pos}}{2}\right)$$ (AASHTO LRFD EQ 5.7.3.2.2-1) $M_{nPos} = 3136.059$ kip ft

Tension controlled resistance factor for flexure

$$\phi_{mPos} = \min\left[0.65 + 0.15\left(\frac{d_{Pos}}{c_{Pos}} - 1\right), 0.9\right]$$ (AASHTO LRFD EQ 5.5.4.2.1-2) $\phi_{mPos} = 0.9$

or simply use, $\phi_m = 0.9$ (AASHTO LRFD 5.5.4.2)

$$M_{rPos} = \phi_{mPos} M_{nPos}$$ (AASHTO LRFD EQ 5.7.3.2.1-1) $M_{rPos} = 2822.453$ kip ft

$$MinReinChkPos = if\left[\left(M_{rPos} \ge M_{cr\_min}\right), \text{"OK"}, \text{"NG"}\right]$$ MinReinChkPos = "OK"

$$UltimateMomChkPos = if\left[\left(M_{rPos} \ge M_{uPos}\right), \text{"OK"}, \text{"NG"}\right]$$ UltimateMomChkPos = "OK"

Moment Capacity Design (Negative Moment, Top Bars A) *AASHTO LRFD 5.7.3.2*

Top Steel arrangement for the Cap:

Input no. of total rebar in a row from top of cap up to 12 rows (in unnecessary rows input zero)

$$N_n = (8\ 0\ 0\ 0\ 0\ 0\ 0\ 0\ 0\ 0\ 0\ 0)$$

Input area of rebar corresponding to above rows from top of cap, not applicable for mixed rebar in a single row

$$A_{bn} = (1.56\ 0\ 0\ 0\ 0\ 0\ 0\ 0\ 0\ 0\ 0\ 0)\ in^2$$

Input center to center vertical distance between each rebar row starting from top of cap

$$cln = (3.5\ 0\ 0\ 0\ 0\ 0\ 0\ 0\ 0\ 0\ 0\ 0)\ in$$

$ns_{Neg} = 1$ (No. of Negative or Top Steel Layers)

Distance from centroid of negative rebar to top extreme tension fiber (d_{cNeg}):

$$d_{cNeg} = \left(Ayn_{0,0}\right)\ in$$ $d_{cNeg} = 3.5$ in

Effective depth from centroid of top rebar to extreme compression fiber (d_{Neg}):

$$d_{Neg} = h - d_{cNeg}$$ $d_{Neg} = 44.5$ in

The Amount of Negative A_s Required,

$$A_{sReq} = \left(\frac{0.85\ f'_c\ b\ d_{Neg}}{f_y}\right)\left(1 - \sqrt{1 - \frac{2\ M_{uNeg}}{0.85\ \phi_m\ f'_c\ b\ d_{Neg}^{\ 2}}}\right) \qquad A_{sReq} = 11.757\ in^2$$

The Amount of Negative A_s Provided,

$$NofBars_{Neg} = \sum N_n \qquad NofBars_{Neg} = 8$$

$$A_{sNeg} = \left(Ayn_{0,1}\right)\ in^2 \qquad A_{sNeg} = 12.48\ in^2$$

Compression depth under ultimate load

$$c_{Neg} = \frac{A_{sNeg}\ f_y}{0.85\ f'_c\ \beta_1\ b} \qquad c_{Neg} = 4.588\ in$$

$$a_{Neg} = \beta_1\ c_{Neg} \qquad a_{Neg} = 3.671\ in$$

Thus, nominal flexural resistance:

$$M_{nNeg} = A_{sNeg}\ f_y\left(d_{Neg} - \frac{a_{Neg}}{2}\right) \qquad M_{nNeg} = 2662.278\ kip\ ft$$

$$M_{rNeg} = \phi_m\ M_{nNeg} \quad \text{(Factored flexural resistance)} \qquad M_{rNeg} = 2396.05\ kip\ ft$$

$$MinReinChkNeg = if\left[\left(M_{rNeg} \geq M_{cr\_min}\right), \text{"OK"}, \text{"NG"}\right] \qquad MinReinChkNeg = \text{"OK"}$$

$$UltimateMomChkNeg = if\left[\left(M_{rNeg} \geq M_{uNeg}\right), \text{"OK"}, \text{"NG"}\right] \qquad UltimateMomChkNeg = \text{"OK"}$$

Control of Cracking at Service Limit State *AASHTO LRFD 5.7.3.4*

$exposure\_cond = 1$ (for exposure condition, input Class 1 = 1 and Class 2 = 2)

$\gamma_e = if(exposure\_cond = 1, 1, 0.75)$ (Exposure condition factor) $\gamma_e = 1$

$\left(side_{cTop}\ \ side_{cBot}\right) = (4.75\ \ 4.75)\ in$ (Input side cover for Top and Bottom Rebars)

<u>Positive Moment (Bottom Bars B)</u> To find S_{max}: S is spacing of first layer of rebar closest to tension face

$$n = round\left(\frac{E_s}{E_c}, 0\right) \quad \text{(modular ratio)} \quad \text{(AASHTO LRFD 5.7.1)} \qquad n = 7$$

$$\rho_{Pos} = \frac{A_{sPos}}{b\ d_{Pos}} \qquad \rho_{Pos} = 0.0076$$

$k_{Pos} = \sqrt{\left(\rho_{Pos}\, n + 1\right)^2 - 1} - \rho_{Pos}\, n$ (Applicable for Solid Rectangular Section) $k_{Pos} = 0.278$

$j_{Pos} = 1 - \frac{k_{Pos}}{3}$ $j_{Pos} = 0.907$

$f_{ssPos} = \frac{M_{sPos}}{A_{sPos}\, j_{Pos}\, d_{Pos}}$ (Tensile Stress at Service Limit State) $f_{ssPos} = 29.174\ \text{ksi}$

$d_{c1Pos} = clp_{0,0}$ (Distance of bottom first row rebar closest to tension face) $d_{c1Pos} = 3.5\ \text{in}$

$\beta_{sPos} = 1 + \frac{d_{c1Pos}}{0.7\left(h - d_{c1Pos}\right)}$ $\beta_{sPos} = 1.112$

$s_{maxPos} = \frac{700\,\frac{kip}{in}\,\gamma_e}{\beta_{sPos}\, f_{ssPos}} - 2\, d_{c1Pos}$ AASHTO LRFD EQ (5.7.3.4-1) $s_{maxPos} = 14.57\ \text{in}$

$s_{ActualPos} = \frac{b - 2\, side_{cBot}}{N_{p_{0,0}} - 1}$ (Equal horizontal spacing of Bottom first Rebar row closest to Tension Face) $s_{ActualPos} = 9.625\ \text{in}$

Actual Max Spacing in Bottom first Layer, $s_{aPosProvided} = 7\ \text{in}$

$s_{ActualPos} = \max\left(s_{aPosProvided}, s_{ActualPos}\right)$ $s_{ActualPos} = 9.625\ \text{in}$

$SpacingCheckPos = if\left[\left(s_{maxPos} \geq s_{ActualPos}\right), \text{"OK"}, \text{"NG"}\right]$ $SpacingCheckPos = \text{"OK"}$

Negative Moment (Top Bars A)

$\rho_{Neg} = \frac{A_{sNeg}}{b\, d_{Neg}}$ $\rho_{Neg} = 0.006$

$k_{Neg} = \sqrt{\left(\rho_{Neg}\, n + 1\right)^2 - 1} - \rho_{Neg}\, n$ (Applicable for Solid Rectangular Section) $k_{Neg} = 0.248$

$j_{Neg} = 1 - \frac{k_{Neg}}{3}$ $j_{Neg} = 0.917$

$f_{ssNeg} = \frac{M_{sNeg}}{A_{sNeg}\, j_{Neg}\, d_{Neg}}$ $f_{ssNeg} = 30.567\ \text{ksi}$

$d_{c1Neg} = cln_{0,0}$ (Distance of Top first layer rebar closest to tension face) $d_{c1Neg} = 3.5\ \text{in}$

$\beta_{sNeg} = 1 + \frac{d_{c1Neg}}{0.7\left(h - d_{c1Neg}\right)}$ $\beta_{sNeg} = 1.112$

$$s_{maxNeg} = \frac{700 \frac{kip}{in} \gamma_e}{\beta_{sNeg} f_{ssNeg}} - 2 d_{c1Neg}$$ $s_{maxNeg} = 13.587$ in

$$s_{ActualNeg} = \frac{b - 2 side_{cTop}}{N_{n_{0,0}} - 1}$$ (Equal horizontal spacing of top first Rebar row closest to Tension Face) $s_{ActualNeg} = 5.5$ in

Actual Max Spacing Provided in Top first row closest to Tension Face, $s_{aNegProvided} = 11.125$ in

$$s_{ActualNeg} = \max\left(s_{aNegProvided}, s_{ActualNeg}\right)$$ $s_{ActualNeg} = 11.125$ in

$$SpacingCheckNeg = if\left[\left(s_{maxNeg} \geq s_{ActualNeg}\right), \text{"OK"}, \text{"NG"}\right]$$ SpacingCheckNeg = "OK"

SUMMARY OF FLEXURE DESIGN:

Bottom Rebar or B Bars: use 10~#11 bars @ 5 bars in each row of 2 rows

Top Rebar or A Bars: use 8~#11 bars @ 8 bars in top row

SKIN REINFORCEMENT (BARS T) *AASHTO LRFD 5.7.3.4*

SkBarNo = 5 (Size of a skin bar) Area of a skin bar, $A_{skBar} = 0.31\ in^2$

$$d_{cTop} = \sum cln$$ $d_{cTop} = 3.5$ in

$$d_{cBot} = \sum clp$$ $d_{cBot} = 7.5$ in

Effective Depth from centroid of Extreme Tension Steel to Extreme compression Fiber (d_l):

$$d_l = \max\left(h - clp_{0,0}, h - cln_{0,0}\right)$$ $d_l = 44.5$ in

Effective Depth from centroid of Tension Steel to Extreme compression Fiber (d_e):

$$d_e = \max\left(d_{Pos}, d_{Neg}\right)$$ $d_e = 44.5$ in

$$A_s = \min\left(A_{sNeg}, A_{sPos}\right)$$ min. of negative and positive reinforcement $A_s = 12.48\ in^2$

$$d_{skin} = h - \left(d_{cTop} + d_{cBot}\right)$$ $d_{skin} = 37$ in

Skin Reinforcement Requirement: AASHTO LRFD EQ 5.7.3.4-2

$$A_{skReq} = if\left[d_l > 3ft, \min\left[0.012 \frac{in}{ft}\left(d_l - 30\ in\right) d_{skin}, \frac{A_s + A_{ps}}{4}\right], 0in^2\right]$$ $A_{skReq} = 0.537\ in^2$

$$NoA_{skbar1} = R\left(\frac{A_{skReq}}{A_{skBar}}\right)$$ $NoA_{skbar1} = 2$ per Side

Maximum Spacing of Skin Reinforcement:

$$S_{skMax} = \min\left(\frac{d_e}{6}, 12\ in\right)$$ AASHTO LRFD 5.7.3.4 $S_{skMax} = 7.417\ in$

$$NoA_{skbar2} = if\left(d_l > 3ft, R\left(\frac{d_{skin}}{S_{skMax}} - 1\right), 1\right)$$ $NoA_{skbar2} = 4$ per Side

$$NofSideBars_{req} = \max\left(NoA_{skbar1}, NoA_{skbar2}\right)$$ $NofSideBars_{req} = 4$

$$S_{skRequired} = \frac{d_{skin}}{1 + NofSideBars_{req}}$$ $S_{skRequired} = 7.4\ in$

$NofSideBars = 4$ (No. of Side Bars Provided)

$$S_{skProvided} = \frac{d_{skin}}{1 + NofSideBars}$$ $S_{skProvided} = 7.4\ in$

$$S_{skChk} = if\left(S_{skProvided} < S_{skMax}, "OK", "N.G."\right)$$ $S_{skChk} = "OK"$

Therefore Use: $NofSideBars = 4$ and Size $SkBarNo = 5$

3. BENT CAP SHEAR AND TORSION DESIGN

SHEAR DESIGN OF CAP:

Effective Shear Depth, $$d_v = \max\left(\begin{pmatrix} d_e - \frac{a}{2} \\ 0.9\ d_e \\ 0.72\ h \end{pmatrix}\right)$$ (*AASHTO LRFD 5.8.2.9*)

d_v = Distance between the resultants of tensile and compressive Force

d_s = Effective depth from cg of the nonprestressed tensile steel to extreme compression fiber

d_p = Effective depth from cg of the prestressed tendon to extreme compression fiber

d_e = Effective depth from centroid of the tensile force to extreme compression fiber at critical shear Location

θ = Angle of inclination diagonal compressive stress

A_o = Area enclosed by shear flow path including area of holes therein

A_c = Area of concrete on flexural tension side of member shown in AASHTO LRFD Figure 5.8.3.4.2 − 1

A_{oh} **=** Area enclosed by centerline of exterior closed transverse torsion reinforcement including area of holes therein

Total Flexural Steel Area,	$A_s = A_{sNeg}$		$A_s = 12.48\ in^2$
Nominal Flexure,	$M_n = M_{nNeg}$		$M_n = 2662.278\ kft$
Stress block Depth,	$a = a_{Neg}$		$a = 3.671\ in$
Effective Depth,	$d_e = d_{Neg}$		$d_e = 44.5\ in$
Effective web Width at critical Location,	$b_v = b$		$b_v = 4\ ft$
Input initial θ,	$\theta = 35\ deg$		$\cot\theta = \cot(\theta)$
Shear Resistance Factor,	$\phi_v = 0.9$		
Cap Depth & Width,	$h = 48\ in$		$b = 48\ in$
Moment Arm,	$\left(d_e - \frac{a}{2}\right) = 42.665\ in$	$0.9\ d_e = 40.05\ in$	$0.72\ h = 34.56\ in$
Effective Shear Depth at Critical Location,	$d_v = \max\left(\begin{pmatrix} d_e - \frac{a}{2} \\ 0.9\ d_e \\ 0.72\ h \end{pmatrix}\right)$	(*AASHTO LRFD 5.8.2.9*)	$d_v = 42.665\ in$

$h_h = h - t_{cover} - b_{cover}$	(Height of shear reinforcement)	$h_h = 43\ in$
$b_h = b - 2\ b_{cover}$	(Width of shear reinforcement)	$b_h = 43\ in$
$p_h = 2(h_h + b_h)$	(Perimeter of shear reinforcement)	$p_h = 172\ in$
$A_{oh} = (h_h)\ (b_h)$	(Area enclosed by the shear reinforcement)	$A_{oh} = 1849\ in^2$
$A_o = 0.85\ A_{oh}$	(AASHTO LRFD C5.8.2.1)	$A_o = 1571.65\ in^2$
$A_c = 0.5\ b\ h$	(AASHTO LRFD FIGURE 5.8.3.4.2 − 1)	$A_c = 1152\ in^2$

Yield strength & Modulus of Elasticity of Steel Reinforcement:

$\left(f_y\ \ E_s\right) = (60\ \ 29000)\ ksi$ (AASHTO LRFD 5.4.3.1, 5.4.3.2)

Input M_u, T_u, V_u, N_u for the critical section to be investigated: (Loads from Bent Cap & RISA Analysis)

$\left(M_u\ \ T_u\right) = (1398.6\ \ 570.2)\ kft$

$\left(V_u\ \ N_u\right) = (463.4\ \ 0)\ kip$

$M'_u = \max\left(M_u, \left|V_u - V_p\right| d_v\right)$ AASHTO LRFD B5.2 $M'_u = 1647.569\ \text{kip ft}$

$V'_u = \sqrt{V_u^{\,2} + \left(\frac{0.9\, p_h\, T_u}{2\, A_o}\right)^2}$ (Equivalent shear) AASHTO LRFD EQ (5.8.2.1-6) for solid section $V'_u = 572.966\ \text{kip}$

Assuming at least minimum transverse reinforcement is provided (Always provide min. transverse reinf.)

$$\varepsilon_x = \frac{\left(\frac{M'_u}{d_v}\right) + 0.5\, N_u + 0.5\left(V'_u - V_p\right)\cot\theta - A_{ps}\, f_{po}}{2\left(E_s\, A_s + E_p\, A_{ps}\right)}$$ (Strain from Appendix B5) AASHTO LRFD EQ (B5.2-1)

$v_u = \frac{\left(V_u - \phi_v\, V_p\right)}{\phi_v\, b_v\, d_v}$ (Shear Stress) AASHTO LRFD EQ (5.8.2.9-1) $v_u = 0.251\ \text{ksi}$

$r = \max\left(0.075, \frac{v_u}{f'_c}\right)$ (Shear stress ratio) $r = 0.075$

After Interpolating the value of $(\Theta \quad B)$

$\Theta = \blacksquare\ \text{deg}$ $B = 2.23$

Nominal Shear Resistance by Concrete,

$V_c = 0.0316\, B\, \sqrt{f'_c\, \text{ksi}}\, b_v\, d_v$ AASHTO LRFD EQ (5.8.3.3-3) $V_c = 322.7\ \text{kip}$

$V_u = 463.4\ \text{kip}$ $0.5\, \phi_v\left(V_c + V_p\right) = 145.211\ \text{kip}$

REGION REQUIRING TRANSVERSE REINFORCEMENT: AASHTO LRFD 5.8.2.4

$V_u > 0.5\, \phi_v\left(V_c + V_p\right)$ AASHTO LRFD EQ (5.8.2.4-1)

check = if$\left[V_u > 0.5\, \phi_v\left(V_c + V_p\right)\right.$, "Provide Shear Reinf", "No reinf."$\left.\right]$ check = "Provide Shear Reinf"

$$V_n = \min\left(\begin{pmatrix} V_c + V_s + V_p \\ 0.25\, f'_c\, b_v\, d_v + V_p \end{pmatrix}\right)$$ (Nominal Shear Resistance) AASHTO LRFD EQ (5.8.3.3 − 1, 2)

$$V_s = \frac{A_v\, f_y\, d_v\, (\cot\theta + \cot\alpha)\, \sin\alpha}{S}$$ (Shear Resistance of Steel) AASHTO LRFD EQ (5.8.3.3 − 4)

$$V_s = \frac{A_v\, f_y\, d_v\, \cot\theta}{S}$$ (Shear Resistance of Steel when, $\alpha = 90$ deg) AASHTO LRFD EQ (C5.8.3.3-1)

$S_v = 9\ in$ (Input Stirrup Spacing) $V_p = 0\ kip$ $\left(V_u \ \ V_c\right) = (463.4 \ \ 322.691\ )\ kip$

$f_y = 60\ ksi$ $d_v = 42.665\ in$ $\Theta = 36.4\ deg$

$$A_{v\_req} = \left(\frac{V_u}{\phi_v} - V_c - V_p\right)\left(\frac{S_v}{f_y\ d_v\ \cot\Theta}\right)$$ (Derive from AASHTO LRFD EQ 5.8.3.3-1, C5.8.3.3-1 and ϕVn >= Vu) $A_{v\_req} = 0.4982\ in^2$

Torsional Steel:

$$A_t = \frac{T_u}{2\ \phi_v\ A_o\ f_y\ \cot\Theta}\ S_v$$ (Derive from AASHTO LRFD EQ 5.8.3.6.2-1 and ϕTn >= Tu) $A_t = 0.267\ in^2$

$A_{vt\_req} = A_{v\_req} + 2\ A_t$ (Shear + Torsion) $A_{vt\_req} = 1.033\ in^2$

$A_{vt} = 4\left(0.44\ in^2\right)$ (Use 2 #6 double leg Stirrup at S_v c/c,) *Provided,* $A_{vt} = 1.76\ in^2$

$A_{vt\_check} = if\left(A_{vt} > A_{vt\_req}, "OK", "NG"\right)$ $A_{vt\_check} = "OK"$

Maximum Spacing Check: AASHTO LRFD Article 5.8.2.7

$V_u = 463.4\ kip$ $0.125\ f'_c\ b_v\ d_v = 1279.94\ kip$

$S_{vmax} = if\left(V_u < 0.125\ f'_c\ b_v\ d_v, \min\left(0.8\ d_v, 24\ in\right), \min\left(0.4\ d_v, 12\ in\right)\right)$ $S_{vmax} = 24\ in$

$S_{vmax\_check} = if\left(S_v < S_{vmax}, "OK", "use lower spacing"\right)$ $S_{vmax\_check} = "OK"$

$A_v = A_{vt} - A_t$ (Shear Reinf. without Torsion Reinf.) $A_v = 1.493\ in^2$

$$V_s = \frac{A_v\ f_y\ d_v\ \cot\Theta}{S_v}$$ $V_s = 575.804\ kip$

Minimum Transverse Reinforce Check: AASHTO LRFD Article 5.8.2.5 $b_v = 48\ in$

$$A_{vmin} = 0.0316\sqrt{f'_c\ ksi}\ \frac{b_v\ S_v}{f_y}$$ AASHTO LRFD EQ (5.8.2.5 − 1) $A_{vmin} = 0.509\ in^2$

$A_{vmin\_check} = if\left(A_{vt} > A_{vmin}, "OK", "NG"\right)$ $A_{vmin\_check} = "OK"$

Maximum Nominal Shear: *To ensure that the concrete in the web of beam will not crush prior to yield of shear reinforcement, LRFD Specification has given an upper limit of*

$0.25\ f'_c\ b_v\ d_v + V_p = 2559.882\ kip$ $V_c + V_s + V_p = 898.495\ kip$

$$V_n = \min\left(\begin{pmatrix} V_c + V_s + V_p \\ 0.25\ f'_c\ b_v\ d_v + V_p \end{pmatrix}\right)$$ AASHTO LRFD EQ (5.8.3.3 − 1, 2) $V_n = 898.495\ kip$

$\phi_v \ V_n = 808.645 \text{ kip}$ $\qquad$ $V_u = 463.4 \text{ kip}$

$\phi V_{n\_check} = \text{if}\left(\phi_v \ V_n > V_u, \text{"OK"}, \text{"NG"}\right)$ $\qquad$ $\phi V_{n\_check} = \text{"OK"}$

Torsional Resistance,

$$T_n = \frac{2 \ A_o \left(0.5 \ A_{vt}\right) f_y \cot\Theta}{S_v}$$ AASHTO LRFD EQ (5.8.3.6.2 − 1) $\qquad$ $\phi_v \ T_n = 1875.9 \text{ kip ft}$

Longitudinal Reinforcement Requirements including Torsion: AASHTO LRFD 5.8.3.6.3

AASHTO LRFD EQ(5.8.3.6.3 − 1) Applicable for solid section with Torsion

$$A_{ps} \ f_{ps} + A_s \ f_y \geq \left(\frac{M'_u}{\phi_m \ d_v}\right) + \frac{0.5 \ N_u}{\phi_n} + \cot\Theta \sqrt{\left(\frac{V_u}{\phi_v} - V_p - 0.5 \ V'_s\right)^2 + \left(\frac{0.45 \ p_h \ T_u}{2 \ \phi_v \ A_o}\right)^2}$$

$\left(\phi_m \ \ \phi_v \ \ \phi_n\right) = (0.9 \ \ 0.9 \ \ 1)$ $\qquad$ $A_s \ f_y + A_{ps} \ f_{ps} = 748.8 \text{ kip}$

$M'_u = 1647.569 \text{ kip ft}$ $\qquad$ $V_u = 463.4 \text{ kip}$ $\qquad$ $N_u = 0 \text{ kip}$ $\qquad$ $V_s = 575.804 \text{ kip}$

$T_u = 570.2 \text{ kip ft}$ $\qquad$ $p_h = 172 \text{ in}$ $\qquad$ $V_p = 0 \text{ kip}$ $\qquad$ $A_s = 12.48 \text{ in}^2$

$$V'_s = \min\left(\frac{V_u}{\phi_v}, V_s\right)$$ AASHTO LRFD 5.8.3.5 $\qquad$ $V'_s = 514.889 \text{ kip}$

$$F = \left(\frac{M'_u}{\phi_m \ d_v}\right) + \frac{0.5 \ N_u}{\phi_n} + \cot\Theta \sqrt{\left(\frac{V_u}{\phi_v} - V_p - 0.5 \ V'_s\right)^2 + \left(\frac{0.45 \ T_u \ p_h}{2 \ \phi_v \ A_o}\right)^2}$$ $\qquad$ $F = 946.64 \text{ kip}$

$F_{check} = \text{if}\left(A_{ps} \ f_{ps} + A_s \ f_y \geq F, \text{"OK"}, \text{"N.G."}\right)$ AASHTO LRFD EQ(5.8.3.6.3 − 1) $\qquad$ $F_{check} = \text{"N.G."}$

N.B.-The longitudinal reinforcement check can be ignored for typical multi-column pier cap. This check must be considered for straddle pier cap with no overhangs. Refer to AASHTO LRFD 5.8.3.5 for further information.

4. COLUMN/DRILLED SHAFT LOADING AND DESIGN

Superstructure to substructure force: AASHTO LRFD SECTION 3 LOADS and LOAD COMBINATIONS

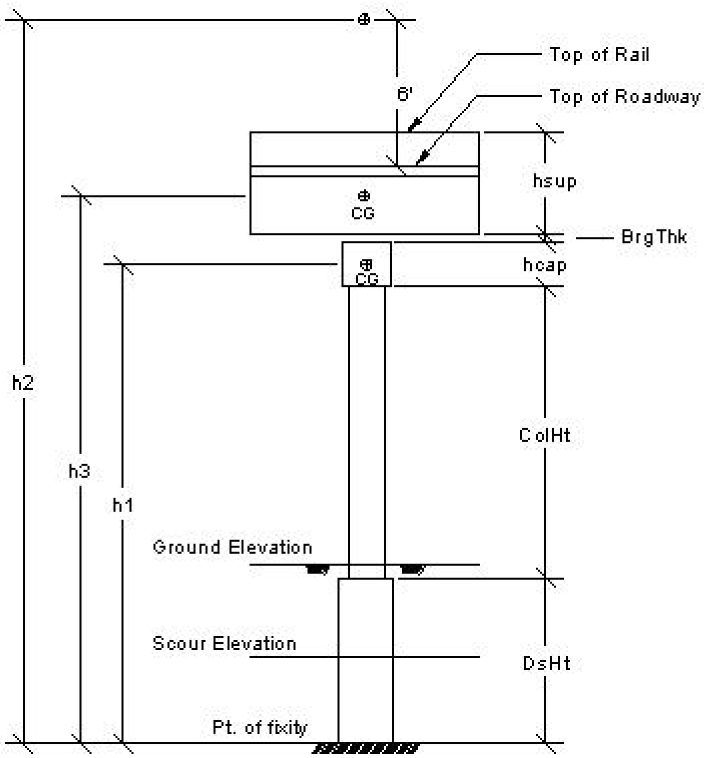
Subscript*: *X* = *Parallel to the Bent cap Length and* *Z* = *Perpendicular to the bent Cap Length

$t_h = 2.5$ in (Haunch Thickness)

Beam Depth, $BmH = FBmD$

$ColH = HCol + 0$ ft (Column height + 0 ft Column Capital)

$$TribuLength = \frac{FSpan + BSpan}{2}$$



Scour Depth:

$h_{scour} = 0$ ft

Scour to Fixity Depth:

$h_{scf} = \min(3\ DsDia, 10\ ft)$

Total Drilled Shaft height:

$DsH = h_{scour} + h_{scf}$

$DsH = 10$ ft

$h_o = BrgTh + BmH + t_h + SlabTh$ (Top of cap to top of slab height) $h_o = 3.725$ ft

$h_6 = h_o + 6ft$ (Top of cap to top of slab height + 6 ft) $h_6 = 9.725$ ft

$hsup = BmH + t_h + SlabTh + RailH$ (Height of Superstructure) $hsup = 6.267$ ft

$h1 = DsH + ColH + \frac{hCap}{2}$ (Height of Cap cg from Fixity of Dshaft) $h1 = 34$ ft

$h2 = DsH + ColH + hCap + h_6$ $h2 = 45.725$ ft

$h3 = DsH + ColH + hCap + BrgTh + \frac{hsup}{2}$ $h3 = 39.425$ ft

Tributary area for Superstructure,

$A_{super} = (hsup)\ (TribuLength)$ $A_{super} = 438.667\ ft^2$

LIVE LOAD REACTIONS: LL

Live load Reaction LL on cap can be taken only the vertical Rxn occurs when HL93 is on both the forward and backward span or when HL93 Loading is on one span only which causes torsion too. To maximize the torsion, LL only acts on the longer span between forward and backward span. For maximum reaction, place rear axle (P_3 = 32 kip) over the support at bent while the design truck traveling along the span.

Maximum Forward Span Design Truck (FTruck) & Lane Load Reaction (FLane):

$$FTruck = P_3 + P_2\left[\frac{(FSpan - 14\ ft)}{FSpan}\right] + P_1\frac{(FSpan - 28ft)}{FSpan} \qquad FTruck = 62.4\ kip$$

$$FLane = w_{lane}\left(\frac{FSpan}{2}\right) \qquad FLane = 22.4\ \frac{kip}{lane}$$

Forward Span Live Load Reactions with Impact (FLLRxn):

$$FLLRxn = FLane + FTruck\ (1 + IM) \qquad FLLRxn = 105.392\ \frac{kip}{lane}$$

Maximum Backward Span Design Truck (BTruck) & Lane Load Reaction (BLane):

$$BTruck = P_3 + P_2\left[\frac{(BSpan - 14\ ft)}{BSpan}\right] + P_1\frac{(BSpan - 28ft)}{BSpan} \qquad BTruck = 62.4\ kip$$

$$BLane = w_{lane}\left(\frac{BSpan}{2}\right) \qquad BLane = 22.4\ \frac{kip}{lane}$$

Backward Span Live Load Reactions with Impact (BLLRxn):

$$BLLRxn = BLane + BTruck\ (1 + IM) \qquad BLLRxn = 105.392\ \frac{kip}{lane}$$

Live Load Reactions per Beam with Impact (BmLLRxn) using Distribution Factors:

$$BmLLRxn = (LLRxn)\ \max\left(DFS_{Fmax}, DFS_{Bmax}, \text{(Max reaction when mid axle on support)}\right) \qquad BmLLRxn = 72.556\ \frac{kip}{beam}$$

$$FBmLLRxn = (FLLRxn)\ DFS_{Fmax} \qquad \text{(Only Forward Span is Loaded)} \qquad FBmLLRxn = 58.858\ \frac{kip}{beam}$$

$$BBmLLRxn = (BLLRxn)\ DFS_{Bmax} \qquad \text{(Only Backward Span is Loaded)} \qquad BBmLLRxn = 58.858\ \frac{kip}{beam}$$

Torsion due to the eccentricity from CL of Bearing to CL of Bent when only Longer Span is loaded with HL-93 Loading

$$TorsionLL = \max(FBmLLRxn, BBmLLRxn)\ e_{brg} \qquad TorsionLL = 63.763\ \frac{kip\ ft}{beam}$$

Live Load Reactions per Beam without Impact ($BmLLRxn_n$) using Distribution Factors:

$$BmLLRxn_n = (Lane + Truck)\ \max\left(DFS_{Fmax}, DFS_{Bmax}\right) \qquad BmLLRxn_n = 60.761\ \frac{kip}{beam}$$

$$FBmLLRxn_n = (FLane + FTruck)\ \left(DFS_{Fmax}\right) \qquad FBmLLRxn_n = 47.358\ \frac{kip}{beam}$$

$$BBmLLRxn_n = (BLane + BTruck)\ \left(DFS_{Bmax}\right) \qquad BBmLLRxn_n = 47.358\ \frac{kip}{beam}$$

Torsion due to the eccentricity of CL of Bearing and CL of Bent without Impact

$$TorsionLL_n = \max\left(FBmLLRxn_n, BBmLLRxn_n\right)\ e_{brg} \qquad TorsionLL_n = 51.305\ \frac{kft}{beam}$$

CENTRIFUGAL FORCE: CF (*AASHTO LRFD 3.6.3*)

Skew Angle of Bridge, $\theta = 0\ \text{deg}$

Design Speed $v = 45\ \text{mph}$

$(f\quad g) = \left(\frac{4}{3}\quad 32.2\ \frac{\text{ft}}{\text{sec}^2}\right)$

Degree of Curve, $\phi_c = 0.00001\ \text{deg}$ (Input 4° curve or 0.00001° for 0° curve)

Radius of Curvature, $R_c = \frac{(360\ \text{deg})\ 100\ \text{ft}}{2\ \pi\ \phi_c}$

$R_c = 572957795.131\ \text{ft}\ (R_c = \infty)$

Centri. Force Factor, $C = f\ \frac{v^2}{R_c\ g}$ (AASHTO LRFD EQ 3.6.3 − 1)

$C = 0$

$P_{cf} = C\ \text{TruckT}\ (\text{NofLane})\ (m)$

$P_{cf} = 0\ \text{kip}$

Centrifugal force **parallel** to bent (X-direction)

$CF_X = \left(\frac{P_{cf}\ \cos(\theta)}{\text{NofBm}}\right)$

$CF_X = 0\ \frac{\text{kip}}{\text{beam}}$

Centrifugal force **normal** to bent (Z-direction)

$CF_Z = \left(\frac{P_{cf}\ \sin(\theta)}{\text{NofBm}}\right)$

$CF_Z = 0\ \frac{\text{kip}}{\text{beam}}$

Moments at cg of the Bent Cap due to Centrifugal Force

$M_{CF\_X} = CF_Z\left(h_6 + \frac{\text{hCap}}{2}\right)$

$M_{CF\_X} = 0\ \frac{\text{kft}}{\text{beam}}$

$M_{CF\_Z} = CF_X\left(h_6 + \frac{\text{hCap}}{2}\right)$

$M_{CF\_Z} = 0\ \frac{\text{kft}}{\text{beam}}$

BRAKING FORCE: BR (*AASHTO LRFD 3.6.4*)

The braking force shall be taken as maximum of 5% of the Resultant Truck plus lane load OR 5% of the Design Tandem plus Lane Load or 25% of the design truck.

$P_{br1} = 5\%\ (\text{Lane} + \text{TruckT})\ (\text{NofLane})\ (m)$ (Truck + Lane)

$P_{br1} = 14.892\ \text{kip}$

$P_{br2} = 5\%\ (\text{Lane} + 50\ \text{kip})\ (\text{NofLane})\ (m)$ (Tandem + Lane)

$P_{br2} = 12.087\ \text{kip}$

$P_{br3} = 25\%\ (\text{TruckT})\ (\text{NofLane})\ (m)$ (DesignTruck)

$P_{br3} = 45.9\ \text{kip}$

$P_{br} = \max(P_{br1}, P_{br2}, P_{br3})$

$P_{br} = 45.9\ \text{kip}$

Braking force **parallel** to bent (X-direction)

$BR_X = \frac{P_{br}\ \sin(\theta)}{\text{NofBm}}$

$BR_X = 0\ \frac{\text{kip}}{\text{beam}}$

Braking force **normal** to bent (Z-direction)

$$BR_Z = \frac{P_{br}\cos(\theta)}{NofBm} \qquad BR_Z = 3.825\frac{kip}{beam}$$

Moments at cg of the Bent Cap due to Braking Force

$$M_{BR\_X} = BR_Z\left(h_6 + \frac{hCap}{2}\right) \qquad M_{BR\_X} = 44.848\frac{kft}{beam}$$

$$M_{BR\_Z} = BR_X\left(h_6 + \frac{hCap}{2}\right) \qquad M_{BR\_Z} = 0\frac{kft}{beam}$$

WATER LOADS: WA (*AASHTO LRFD 3.7*)

Note : To be applied <u>only</u> on bridge components below design high water surface.

<u>Substructure:</u>

$V = 0\frac{ft}{sec}$ (Design Stream Velocity) Specific Weight, $\gamma_{water} = 62.4\ pcf$

<u>Longitudinal Stream Pressure:</u> *AASHTO LRFD 3.7.3.1*

AASHTO LRFD Table 3.7.3.1-1 for Drag Coefficient, C_D

semicircular-nosed pier	0.7
square-ended pier	1.4
debris lodged against the pier	1.4
wedged-nosed pier with nose angle 90 deg or less	0.8

Columns and Drilled Shafts: Longitudinal Drag Force Coefficient for Column, $C_{D\_col} = 1.4$

Longitudinal Drag Force Coefficient for Drilled Shaft, $C_{D\_ds} = 0.7$

$$p_T = C_D\frac{V^2}{2\ g}\gamma_{water}$$ (Longitudinal stream pressure) AASHTO LRFD EQ (C3.7.3.1-1)

$$p_{T\_col} = C_{D\_col}\frac{V^2}{2\ g}\gamma_{water} \qquad p_{T\_col} = 0\ ksf$$

$$p_{T\_ds} = C_{D\_ds}\frac{V^2}{2\ g}\gamma_{water} \qquad p_{T\_ds} = 0\ ksf$$

<u>Lateral Stream Pressure:</u> AASHTO LRFD 3.7.3.2

AASHTO LRFD Table 3.7.3.2-1 for Lateral Drag Coefficient, C_L

Angle,θ, between direction of flow and longitudinal axis of the pile	CL
0deg	0
5deg	0.5
10deg	0.7
20deg	0.9
>30deg	1

Lateral Drag Force Coefficient, $C_L = 0.0$

Lateral stream pressure, $p_L = C_L \frac{V^2}{2\,g} \gamma_{water}$ $p_L = 0$ ksf

Bent Cap: Longitudinal stream pressure $C_L = 1.4$

$p_{Tcap} = C_L \frac{V^2}{2\,g} \gamma_{water}$ $p_{Tcap} = 0$ ksf

WA on Columns

Water force on column **parallel** to bent (X-direction)

$WA_{col\_X} = wCol\ p_{T\_col}$ $WA_{col\_X} = 0 \frac{kip}{ft}$

If angle between direction of flow and longitudinal axis of pile = 0 then apply load at one exterior column only; otherwise apply it on all columns. WA at all columns will be distributed uniformly rather than triangular distribution on column height.

Water force on column **normal** to bent (Z-direction)

$WA_{col\_Z} = bCol\ p_L$ $WA_{col\_Z} = 0 \frac{kip}{ft}$

WA on Drilled Shafts

Water force on drilled shaft **parallel** to bent (X-direction)

$WA_{dshaft\_X} = DsDia\ p_{T\_ds}$ $WA_{dshaft\_X} = 0 \frac{kip}{ft}$

Water force on drilled shaft **normal** to bent (Z-direction)

$WA_{dshaft\_Z} = DsDia\ p_L$ $WA_{dshaft\_Z} = 0 \frac{kip}{ft}$

WA on Bent Cap (input as a punctual load)

Water force on bent cap **parallel** to bent (X-direction)

$WA_{cap\_X} = wCap\ hCap\ (p_{Tcap})$ (If design HW is below cap then input zero) $WA_{cap\_X} = 0$ kip

Water force on bent cap **normal** to bent (Z-direction)

$WA_{cap\_Z} = hCap\ p_L$ (If design HW is below cap then input zero) $WA_{cap\_Z} = 0 \frac{kip}{ft}$

WIND ON SUPERSTRUCTURE: WS (*AASHTO LRFD 3.8.1.2.2*)

Note : Wind Loads to be applied only on bridge exposed components above water surface

AASHTO LRFD Table 3.8.1.2.2-1 specifies the wind load components for various angles of attack. In order to simplify the analysis, this calculation considers as default values those for girders which generate the maximum effect on structure. The results can be considered as conservative. For a superstructure other than a girder type and/or for a more detailed analysis, use the proper values as specified in the above mentioned table.

AASHTO LRFD table 3.8.1.2.2-1 (modified)

Skew Angle	Girders	
	Lateral	Longitudinal
Degrees	(Ksf)	(Ksf)
0	0.05	0
15	0.044	0.006
30	0.041	0.012
45	0.033	0.016
60	0.017	0.019

If the bridge is approximately 30' high and local wind velocities are known to be less than 100 mph, wind load for this bridge should be from AASHTO LRFD TABLE 3.8.2.2-1. Otherwise use AASHTO LRFD EQ 3.8.1.2.1-1 as mentioned above.

$p_{tsup} = 0.05ksf$ Normal to superstructure (conservative suggested value 0.050 ksf)

$p_{lsup} = 0.012ksf$ Along Superstructure (conservative suggested value 0.019 ksf)

$WS_{chk} = \text{if}\left(p_{tsup}\ hsup \geq 0.3\ klf, \text{"OK"}, \text{"N.G."}\right)$ $WS_{chk} = \text{"OK"}$

$$Wsup_{Long} = \frac{p_{lsup}\ hsup\ TribuLength}{NofBm} \qquad Wsup_{Long} = 0.439\ \frac{kip}{beam}$$

$$Wsup_{Trans} = \frac{p_{tsup}\ hsup\ TribuLength}{NofBm} \qquad Wsup_{Trans} = 1.828\ \frac{kip}{beam}$$

Wind force on superstructure **parallel** to bent (X-direction)

$$WS_{super\_X} = Wsup_{Long}\sin(\theta) + Wsup_{Trans}\cos(\theta) \qquad WS_{super\_X} = 1.828\ \frac{kip}{beam}$$

Wind force on superstructure **normal** to bent (Z-direction)

$$WS_{super\_Z} = Wsup_{Long}\cos(\theta) + Wsup_{Trans}\sin(\theta) \qquad WS_{super\_Z} = 0.439\ \frac{kip}{beam}$$

Moments at cg of the Bent Cap due to Wind load on superstructure

$$M_{super\_X} = WS_{super\_Z}\left(\frac{hCap}{2} + BrgTh + \frac{hsup}{2}\right) \qquad M_{super\_X} = 2.38\ \frac{kft}{beam}$$

$$M_{super\_Z} = WS_{super\_X}\left(\frac{hCap}{2} + BrgTh + \frac{hsup}{2}\right) \qquad M_{super\_Z} = 9.916\ \frac{kft}{beam}$$

WIND ON SUBSTRUCTURE: WS (*AASHTO LRFD 3.8.1.2.3*)

Base Wind pressure, $p_{sub} = 0.04$ ksf will be applied on exposed substructure both transverse & longitudinal direction

Wind on Columns

Wind force on columns **parallel** to bent (X-direction)

$$WS_{col\_X} = \left[p_{sub}\left(bCol\cos(\theta) + wCol\sin(\theta)\right)\right] \qquad WS_{col\_X} = 0.14\ \frac{kip}{ft}$$

Apply WS loads at all columns even with zero degree attack angle.

Wind force on columns **normal** to bent (Z-direction)

$$WS_{col\_Z} = \left[p_{sub}\,(bCol\,\sin(\theta) + wCol\,\cos(\theta))\right] \qquad WS_{col\_Z} = 0.14\,\frac{kip}{ft}$$

Wind on Bent Cap & Ear Wall

$$WS_{ew\_X} = p_{sub}\,hEarWall\,(wEarWall\,\sin(\theta) + wCap\,\cos(\theta)) \qquad WS_{ew\_X} = 0\,kip$$

$$WS_{ew\_Z} = p_{sub}\,hEarWall\,(wEarWall\,\cos(\theta) + wCap\,\sin(\theta)) \qquad WS_{ew\_Z} = 0\,kip$$

Wind force on bent cap **parallel** to bent (X-direction)

$$WS_{cap\_X} = \left[p_{sub}\,hCap\,(CapL\,\sin(\theta) + wCap\,\cos(\theta))\right] + WS_{ew\_X}$$ (punctual load) $$WS_{cap\_X} = 0.64\,kip$$

Wind force on bent cap **normal** to bent (Z-direction)

$$WS_{cap\_Z} = \frac{\left[p_{sub}\,hCap\,(CapL\,\cos(\theta) + wCap\,\sin(\theta))\right] + WS_{ew\_Z}}{CapL} \qquad WS_{cap\_Z} = 0.16\,\frac{kip}{ft}$$

WIND ON VEHICLES: WL (*AASHTO LRFD 3.8.1.3*)

AASHTO LRFD Table 3.8.1.3-1 specifies the wind on live load components for various angles of attack. In order to simplify the analysis, this calculation considers as default values the maximum wind components as defined in the above mentioned table. The results can be considered conservative. For a more detailed analysis, use the proper skew angle according to the table.

AASHTO LRFD table 3.8.1.3-1

Skew Angle	Normal Componen	Parallel Component
Degrees	(Klf)	(Klf)
0	0.1	0
15	0.088	0.012
30	0.082	0.024
45	0.066	0.032
60	0.034	0.038

(suggested value 0.1 kip/ft) $$p_{WLt} = 0.1\,\frac{kip}{ft}$$

(suggested value 0.038 kip/ft) $$p_{WLl} = 0.04\,\frac{kip}{ft}$$

$$WL_{Par} = \frac{p_{WLl}\,TribuLength}{NofBm} \qquad WL_{Par} = 0.233\,\frac{kip}{beam}$$

$$WL_{Nor} = \frac{p_{WLt}\,TribuLength}{NofBm} \qquad WL_{Nor} = 0.583\,\frac{kip}{beam}$$

Wind force on live load **parallel** to bent (X-direction)

$$WL_X = WL_{Nor}\,\cos(\theta) + WL_{Par}\,\sin(\theta) \qquad WL_X = 0.583\,\frac{kip}{beam}$$

Wind force on live load **normal** to bent (Z-direction)

$$WL_Z = WL_{Nor}\,\sin(\theta) + WL_{Par}\,\cos(\theta) \qquad WL_Z = 0.233\,\frac{kip}{beam}$$

Moments at cg of the Bent Cap due to Wind load on Live Load

$$M_{WL\_X} = WL_Z \left(h_6 + \frac{hCap}{2}\right) \qquad M_{WL\_X} = 2.736 \, \frac{kft}{beam}$$

$$M_{WL\_Z} = WL_X \left(h_6 + \frac{hCap}{2}\right) \qquad M_{WL\_Z} = 6.84 \, \frac{kft}{beam}$$

Vertical Wind Pressure: *(AASHTO LRFD 3.8.2)*

$DeckWidth = FDeckW$ Bridge deck width including parapet and sidewalk

$P_{uplift} = -(0.02ksf)\ DeckWidth\ TribuLength$ (Acts upword Y-direction) $P_{uplift} = -66.033\ kip$

Applied at the windward quarter-point of the deck width.

Note: Applied only for Strength III and for Service IV limit states only when the direction of wind is perpendicular to the longitudinal axis of the bridge. (*AASHTO LRFD table 3.4,1-2*, factors for permanent loads)

Load Combinations: using AASHTO LRFD Table 3.4.1-1

STRENGTH_I **=** 1.25 DC + 1.5 DW + 1.75 (LL + BR + CF) + 1.0 WA

STRENGTH_IA **=** 0.9 DC + 0.65 DW + 1.75 (LL + BR + CF) + 1.0 WA

STRENGTH_III **=** 1.25 DC + 1.5 DW + 1.4 WS + 1.0 WA + 1.4 P_{uplift}

STRENGTH_IIIA **=** 0.9 DC + 0.65 DW + 1.4 WS + 1.0 WA + 1.4 P_{uplift}

STRENGTH_V **=** 1.25 DC + 1.5 DW + 1.35 (LL + BR + CF) + 0.4 WS + 1.0 WA + 1.0 WL

STRENGTH_VA **=** 0.9 DC + 0.65 DW + 1.35 (LL + BR + CF) + 0.4 WS + 1.0 WA + 1.0 WL

SERVICE_I **=** 1.0 DC + 1.0 DW + 1.0 $\left(LL_{no\_Impact} + BR + CF\right)$ + 0.3 WS + 1.0 WA + 1.0 WL

All these loadings as computed above such as DC, DW, LL, WL, WA, WS, etc., are placed on the bent frame composed of bent cap and columns and drilled shafts. The frame is analyzed in RISA using load combinations as stated above. Output Loadings for various load combinations for column and drilled shaft are used to run PCA Column program to design the columns. It is found that **3'-6"X3'-6" Column with 12~#11 bars** is sufficient for the loadings. Drilled shaft and other foundation shall be designed for appropriate loads.

Total Vertical Foundation Load at Service I Limit State:

Forward Span Superstructure DC (F_{FDC}) & DW (F_{FDW}):

$$F_{FDC} = (FNofBm - 2)\ FSuperDC_{Int} + 2\ FSuperDC_{Ext} \qquad F_{FDC} = 259.607\ kip$$

$$F_{FDW} = (FNofBm)\ FSuperDW \qquad F_{FDW} = 38.5\ kip$$

Backward Span Superstructure DC (F_{BDC}) & DW (F_{BDW}):

$F_{BDC} = (BNofBm - 2)\ BSuperDC_{Int} + 2\ BSuperDC_{Ext}$ $\qquad F_{BDC} = 259.607\ kip$

$F_{BDW} = (BNofBm)\ BSuperDW$ $\qquad F_{BDW} = 38.5\ kip$

Total Cap Dead Load Weight (TCapDC):

$TCapDC = (CapDC)\ (CapL) + (NofBm)\ (BrgSeatDC) + EarWallDC$ $\qquad TCapDC = 112.8\ kip$

Total DL on columns including Cap weight (F_{DC}):

$F_{DL} = \left(F_{FDC} + F_{FDW}\right) + \left(F_{BDC} + F_{BDW}\right) + TCapDC$ $\qquad F_{DL} = 709.015\ kip$

Column & Drilled Shaft Self Weight:

DShaft Length, $DsHt = 0\ ft$ $\qquad$ if Rounded Col, $ColDia = 0\ ft$

$ColDC = if\left[ColDia > 0ft, \frac{\pi}{4}\ (ColDia)^2\ (HCol)\ \gamma_c, wCol\ bCol\ HCol\ \gamma_c\right]$ $\qquad$ Column Wt, $ColDC = 40.425\ kip$

$DsDC = \frac{\pi}{4}\ (DsDia)^2\ (DsHt)\ \gamma_c$ $\qquad$ Dr Shaft Wt, $DsDC = 0\ kip$

Total Dead Load on Drilled Shaft (DL_on_DShaft):

$DL\_on\_DShaft = F_{DL} + (NofCol)\ (ColDC) + (NofDs)\ (DsDC)$ $\qquad DL\_on\_DShaft = 789.865\ kip$

Live Load on Drilled Shaft:

$m = 0.85$ (Multile Presence Factors for 3 Lanes) $\qquad$ (AASHTO LRFD Table 3.6.1.1.2 − 1)

$R_{LL} = (Lane + Truck)\ (NofLane)\ (m)$ (Total LLRxn without Impact) $\qquad R_{LL} = 277.44\ kip$

Total Load, DL+LL per Drilled Shaft of Intermediate Bent:

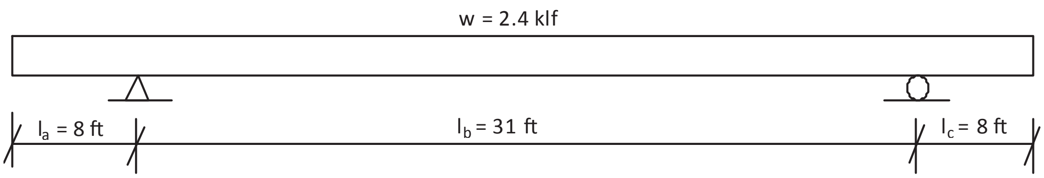
$$Load\_on\_DShaft = \frac{DL\_on\_DShaft + R_{LL}}{NofDs} \qquad Load\_on\_DShaft = 266.8\ ton$$

5. PRECAST COMPONENT DESIGN

Precast Cap Construction and Handling:

$w = b\ h\ \gamma_c$ (Cap selfweight) $\qquad w = 2.4\ \text{klf}$

Due to the location of girder bolts on cap, pickup points at 8' from both ends. Indeed, we can model cap lifting points as simply supported beam under self weight supported at 8' and 39' respectively from very end.



$l_a = 8\ \text{ft} \qquad l_b = 31\ \text{ft} \qquad l_c = 8\text{ft}$

Construction factor:

$\lambda_{cons} = 1.25 \qquad \lambda_{cons} = 1.25$

Maximum Positive Moment (M_{maxP}) & Negative Moment (M_{maxN}):

$$M_{maxP} = \frac{w\ CapL}{2}\left(\frac{CapL}{4} - l_a\right) \qquad M_{maxP} = 211.5\ \text{kft}$$

$$M_{maxN} = \frac{w\ l_a^2}{2} \qquad M_{maxN} = 76.8\ \text{kft}$$

Factored Maximum Positive Moment (M_{uP}) & Negative Moment (M_{uN}):

$$M_{uP} = \lambda_{cons}\ M_{maxP} \qquad M_{uP} = 264.375\ \text{kft}$$

$$M_{uN} = \lambda_{cons}\ M_{maxN} \qquad M_{uN} = 96\ \text{kft}$$

$$S = \frac{b\ h^2}{6} \quad \text{(Cap Section Modulus)} \qquad S = 18432\ \text{in}^3$$

Maximum Positive Stress (f_{tP}) & Negative Stress (f_{tN}):

$$f_{tP} = \frac{M_{uP}}{S} \qquad f_{tP} = 172.119\ \text{psi}$$

$$f_{tN} = \frac{M_{uN}}{S} \qquad f_{tN} = 62.5\ \text{psi}$$

Modulus of Rupture: According PCI hand book 6th edition modulus of rupture, fr = 7.5√f'c is divided by a safety factor 1.5 in order to design a member without cracking

$f'_c = 5\ \text{ksi}$ (Compressive Strength of Concrete)

Unit weight factor, $\lambda = 1$

$f_r = 5\ \lambda \sqrt{f'_c\ \text{psi}}$ (PCI EQ 5.3.3.2)

$f_r = 353.553\ \text{psi}$

$f_{r\_check} = \text{if}\left[\left(f_r > f_{tP}\right)\left(f_r > f_{tN}\right), \text{"OK"}, \text{"N.G."}\right]$

$f_{r\_check} = \text{"OK"}$

Precast Column Construction and Handling:

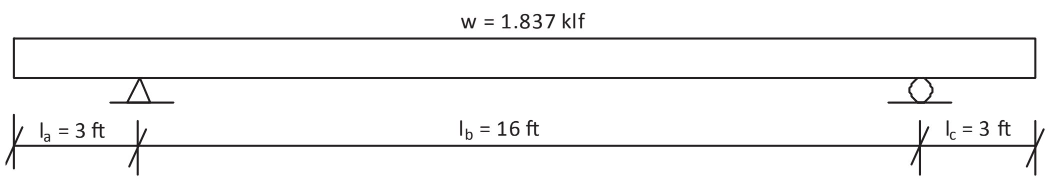
$wCol = 3.5\ \text{ft}$ (Column width)

Column breadth, $bCol = 3.5\ \text{ft}$

$w_{col} = wCol\ bCol\ \gamma_c$ (Column self weight)

$w_{col} = 1.837\ \text{klf}$

Due to the location of girder bolts on column, pick up points at 3' from both ends. Indeed, we can model column lifting points as simply supported beam under self weight supported at 3' and 19' respectively from very end.



$l_a = 3\ \text{ft}$

$l_b = 16\ \text{ft}$

$l_c = 3\ \text{ft}$

Maximum Positive Moment (M_{maxP}) & Negative Moment (M_{maxN}):

$$M_{maxP} = \frac{w_{col}\ HCol}{2}\left(\frac{HCol}{4} - l_a\right)$$

$M_{maxP} = 50.531\ \text{kft}$

$$M_{maxN} = \frac{w_{col}\ l_a^2}{2}$$

$M_{maxN} = 8.269\ \text{kft}$

Factored Maximum Positive Moment (M_{uP}) & Negative Moment (M_{uN}):

$$M_{uP} = \lambda_{cons}\ M_{maxP}$$

$M_{uP} = 63.164\ \text{kft}$

$$M_{uN} = \lambda_{cons}\ M_{maxN}$$

$M_{uN} = 10.336\ \text{kft}$

$$S_{col} = \frac{wCol\ bCol^2}{6}$$ (Column Section Modulus)

$S_{col} = 12348\ \text{in}^3$

Maximum Positive Stress (f_{tP}) & Negative Stress (f_{tN}):

$$f_{tP} = \frac{M_{uP}}{S_{col}} \qquad f_{tP} = 61.384\ \text{psi}$$

$$f_{tN} = \frac{M_{uN}}{S_{col}} \qquad f_{tN} = 10.045\ \text{psi}$$

Modulus of Rupture: According PCI hand book 6th edition modulus of rupture, fr = 7.5√f'c is divided by a safety factor 1.5 in order to design a member without cracking

$f'_c = 5\ \text{ksi}$ (Compressive Strength of Concrete) Unit weight factor, $\lambda = 1$

$$f_r = 5\ \lambda \sqrt{f'_c\ \text{psi}} \quad \text{(PCI EQ 5.3.3.2)} \qquad f_r = 353.553\ \text{psi}$$

$$f_{r\_check} = \text{if}\left[\left(f_r > f_{tP}\right)\left(f_r > f_{tN}\right), \text{"OK"}, \text{"N.G."}\right] \qquad f_{r\_check} = \text{"OK"}$$

DEVELOPMENT LENGTH: AASHTO LRFD 5.11

$A_b = 1.56\ \text{in}^2$ (Area of Bar) $d_b = 1.41\ \text{in}$ (Diameter of Bar) $f'_c = 5\ \text{ksi}$

Modification Factor: According to AASHTO LRFD 5.11.2.1.2, the basic development length, l_{db} is required to multiply by the modification factor to obtain the development length l_d for tension or compression.

$\lambda_{mod} = 1.0$

Basic Tension Development: AASHTO LRFD 5.11.2.1 for bars upto #11

$$l_{db} = \max\left[1.25\left(\frac{A_b}{\text{in}}\right)\frac{f_y}{\sqrt{f'_c\ \text{ksi}}}, 0.4\ d_b\ \frac{f_y}{\text{ksi}}, 12\ \text{in}\right] \quad \text{(AASHTO LRFD 5.11.2.1.1)} \qquad l_{db} = 52.324\ \text{in}$$

$$l_d = \left(\lambda_{mod}\right) l_{db} \qquad l_d = 4.36\ \text{ft}$$

Basic Compression Development: AASHTO LRFD 5.11.2.2

$$l_{db} = \max\left(\frac{0.63\ d_b\ f_y}{\sqrt{f'_c\ \text{ksi}}}, 0.3\ d_b\ \frac{f_y}{\text{ksi}}, 8\ \text{in}\right) \quad \text{AASHTO LRFD EQ (5.11.2.2.1} - 1, 2) \qquad l_{db} = 25.38\ \text{in}$$

$$l_d = \left(\lambda_{mod}\right) l_{db} \qquad l_d = 2.115\ \text{ft}$$

APPENDIX G

Recommended ABC Design Specifications

5.14.6—Provisions for Design of Prefabricated Systems for Accelerated Bridge Construction

5.14.6.1—General

The design of most modular systems for rapid renewal follows traditional LRFD design specifications. The requirements specified herein shall supplement the requirements of other sections of the LRFD Design Specifications for the design of prefabricated modular systems for rapid renewal. These requirements apply to precast concrete components and prefabricated composite steel girder systems.

The design of bridges built using large-scale prefabrication is not specifically covered in the AASHTO *LRFD Bridge Design Specifications.* When lifting prefabricated components, the location of the support points need to be identified and accounted for in the design, including dynamic effects.

5.14.6.2—Design Objectives

5.14.6.2.1—Rideability

The provisions of *LRFD 2.5.2.4–Rideability* shall be applicable with the following additions:

Construction tolerances, with regard to the profile <u>and crossslope</u> of the finished deck, shall be indicated on the plans or in the specifications or special provisions.

Where concrete decks without an initial overlay are used, consideration should be given to providing an additional <u>minimum</u> thickness of 0.5 in. to permit correction of the deck profile by grinding, and to compensate for thickness loss due to abrasion. <u>For precast decked concrete girder bridges, where the deck is part of the initial precast section, consideration should be given to either increasing this allowance or providing a variable thickness deck to permit correction of the deck profile due to effects of camber.</u>

5.14.6.2.2—Deformations

Stresses and deflections shall be computed to control the integrity of the modular components during lifting and transportation. The Engineer of Record (EOR) shall define deformation controls suitable for each span.

For steel or prestressed concrete modular systems, for the purposes of monitoring the structure under fabrication, lifting, transportation and setting in the final location, it is recommended that the EOR determine the anticipated deflection profile for the following conditions when spanning the temporary supports or pick points:

– Under the self-weight (and prestress) of the composite beams and diaphragms.
– Of the composite superstructure and with addition of superimposed dead load from barriers, parapets, medians or sidewalks.

The above deflection conditions can be calculated using any appropriate calculation technique based upon elastic analysis. For all the above, for precast prestressed or post-tensioned beams, take into account the age of the concrete at the time the operation is assumed to take place.

Under the initial lift condition, ensure that the anticipated flexural tensile stress induced in the top of the structural concrete slab for the assumed support locations is no greater than 0.125 ksi or 0.19÷f′cm (ksi) where f'_{cm} = anticipated strength of concrete at the time of the initial lift operation. If the above conditions cannot be satisfied, then it is recommended that the assumed locations of the lifting points be revised.

5.14.6.3—Loads and Load Factors

5.14.6.3.1—Definitions

- *Camber Leveling Force*—A vertically applied force used to equalize differential camber between prefabricated elements in a prefabricated modular structural system prior to establishing continuity or connectivity between the elements.
- *Dynamic Dead Load Allowance*—An increase or decrease in the self-weight of components to account for inertial effects during handling and transportation of prefabricated elements.

5.14.6.3.2—Load and Load Designation

CL = Camber leveling force (kip)
C = Locked-in force effects due to load applied to erected prefabricated elements to correct misalignment due to differential camber prior to establishing continuity

5.14.6.3.3—Load Factors and Combinations

When camber leveling forces, *CL,* are considered and they increase the critical effect in the design of the member, the load factor in all Service Load Combinations shall be taken as specified for *DC* in Table 3.4.1.1-1. Where camber leveling forces act to reduce the critical effect being considered, the load factor shall be taken as 0.0.

5.14.6.3.4—Load Factors for Construction Loads

This AASHTO LRFD Section 3.4.2 addresses the Strength Limit State and Service Limit State checks for construction loads.

The following additional requirements for LRFD Section 3.4.2 are extended to apply to prefabricated elements and modular systems (concrete and steel composite). These additional requirements are invoked to guard against damage or permanent distortions to the modular system during handling and placement.

1. The Designer shall analyze spans on the assumed temporary/lifting supports based on the Strength I Limit State with a load factor equal to 1.25.

2. When investigating Strength Load Combinations I during construction, load factors for the weight of the structure and appurtenances, *DC* and *DW,* as well as applied camber leveling load, *CL,* shall not be taken to be less than 1.25.
3. When evaluating prefabricated components or individual elements of modular systems during construction, a dynamic dead load allowance of 15 percent, acting up or down, shall be applied to all dead load present at the time of handling and transportation. A reduced value may be used at the discretion of the Owner or when measures are taken to minimize inertial effects during transportation.
4. The Designer shall also check the spans to be brought into service for displacements based on Service I Limit State. Service stresses in the span while being handled and placed shall have a service load factor on dead load of 1.30 (handling impact factor). If a rigorous structural analysis allowing for the three-dimensional effects of inadvertent twist during transportation is undertaken and included, the service load handling impact factor may be reduced to 1.05. No factored loads shall be used for deflection calculations.
5. No permanent distortion (twist) as a result of handling and placement will be allowed.
6. Contract Documents shall include a completed table of "anticipated deflections" as discussed in LRFD Section 3.4.2.2.
7. Plan notes for construction loads shall include "the magnitude and location" of construction loads considered in the design as outlined in LRFD Section C3.4.2.1.
8. The bridge is not subject to Service III limit state while under construction. Bridges analyzed carrying construction equipment shall utilize Service I with a 5% impact factor.

5.14.6.4—Analysis

LRFD Section 4.5, Mathematical Modeling, provides general guidance for mathematical modeling of bridges. The following additional requirements are extended to apply to prefabricated concrete and steel composite modular systems:

1. Prefabricated elements and modular systems are to be analyzed based on elastic behavior for handling and placement. Inelastic analysis will not be permitted.
2. The analysis may consider the influence of continuous composite precast barriers and rails on the behavior of modular systems during handling and placement.
3. Analysis of modular systems may be based on approximate or refined methods in accordance with AASHTO LRFD Bridge Design Specifications.
4. Contract Plans shall state that all formwork for the deck shall be supported from the longitudinal girders similar to conventional construction methods. Shored construction shall not be assumed. Decked girder systems shall be designed to accommodate future deck replacement without the use of shoring during deck removal and replacement operations.

5.14.6.5—Control of Cracking (Non-Prestressed Components)

LRFD Section 5.7.3.4, Control of Cracking by Distribution of Reinforcement, addresses requirements for all reinforced concrete members. It is extended to apply to prefabricated elements and systems.

1. Provisions specified in LRFD Article 5.7.3.4 for the distribution of tension reinforcement to control flexural cracking shall apply to all prefabricated elements and systems at the Service I Limit State.
2. The longitudinal reinforcement in the deck and superimposed attached items like sidewalks, parapets, and traffic railings shall be analyzed.

5.14.6.6—Lifting and Handling Stresses (Non-Prestressed Components)

Specify maximum tensile stress in non-prestressed precast concrete components during transportation, handling and erection under the Service I load combination. A 30% handling impact factor on dead loads shall be assumed. As an alternate, we can specify that precast components be handled in a manner that restricts the crack widths to acceptable limits.

The lifting inserts should be so arranged that when the element is lifted it remains stable and the bottom edge remains horizontal. The positions of lifting inserts are calculated to limit lifting stresses and to ensure that the precast element hangs in the correct orientation during lifting. Check the potential for lateral instability during transportation and erection.

Analysis of lifting and handling stresses shall be based on the recommended lifting points shown on the plans. The minimum concrete strength at which precast elements can be lifted should be specified on the plans.

5.14.6.7—Prestressed Components

Requirements of LRFD Section 5.9.4, Stress Limits for Concrete, shall be modified as follows for modular systems:

Minimum compressive strength at time of handling f'_{cm} should be specified on the plans.

5.9.4.1—For Temporary Stresses Before Losses—Fully Prestressed Components

5.9.4.1.2—Tension Stresses

Modify second bullet of Table 5.9.4.1.2-1 for "Other Than Segmentally Constructed Bridges":

	1. In areas other than the precompressed tensile zone and without bonded reinforcement, <u>and in top flanges of noncomposite prestressed components that will serve as the riding surface in the finished bridge</u>	

Add to Table 5.9.4.1.2-1 for "Other Than Segmentally Constructed Bridges":

	2. For handling stresses in the top flange of noncomposite prestressed components that will serve as the riding surface in the finished bridge	$0.24 \div f'_{cm}$ (ksi)

5.9.4.2—For Stresses at Service Limit State After Losses—Fully Prestressed Components

5.9.4.2.1—Compression Stresses

This section addresses compression stresses in Prestressed Concrete members. It is extended to apply to prefabricated elements and systems.

LRFD Table 5.9.4.2.1-1, the third bullet shall apply to prestressed girder elements and modular systems during shipping and handling with a $\Phi w = 1.0$.

5.9.4.2.2—Tension Stresses

This section addresses tension stresses in prestressed concrete. It is extended to apply to prefabricated elements and systems.

Prestressing losses may be calculated by either the Approximate or Refined method in AASHTO LRFD Articles 5.9.5.3 and 5.9.5.4.

Service III is for tension limits subject to normal anticipated highway "traffic loading." These loadings do not include nor do they apply to construction vehicles.

Use Service I for construction loadings. During design, the actual scheduling of construction is not known. Since the age of the members can have a significant effect on the stresses early on, conservative assumptions must be made to ensure that the design stresses are for the worst case scenario.

Add to Table 5.9.4.2.2-1 for "Other Than Segmentally Constructed Bridges":

	3. For components subjected to locked-in effects due to application of camber leveling forces	No tension

5.11.5.3.1—Lap Splices in Tension

This section specifies a minimum of 12″ length for lap splices in tension. The minimum length requirement may be waived if demonstrated by test results on a specimen representing the proposed joint design using UHPC. An experimentally determined development length may be used as the basis for the joint design.

5.14.6.8—Design of the Grouted Splice Coupler

The AASHTO *LRFD Bridge Design Specifications* Article 5.11.5.2.2 requires that all mechanical reinforcing splice devices develop 125% of the specified yield strength of the bar. Several manufacturers produce grouted splice couplers that can meet and exceed this requirement. If this requirement is met, the coupler can be treated the same as a reinforcing lap splice.

5.14.6.9—Provisions for Joints

The following sections modify applicable section of Section 5 of the *LRFD Bridge Design Specifications:*

5.14.4.3.3d—Longitudinal Construction Joints

For longitudinal joints designed as shear-flexure joints without transverse post-tensioning that are also required to resist forces due to differential camber between adjacent components, the key shall be filled with an approved concrete. Minimum compressive strength and time required to attain the minimum compressive strength shall be specified on the plans. The applied camber leveling force shall not be removed until the joint is capable of resisting shear due to differential camber. Grinding for profile or cross-slope correction shall not begin until the concrete has attained the specified minimum compressive strength.

5.14.4.3.3e—Cast-in-Place Closure Joint

Concrete in the closure joint should have strength comparable to that of the precast components. The width of the longitudinal joint shall be large enough to accommodate development of reinforcement in the joint. Where development sufficient for anchorage of the reinforcement can be demonstrated by test results on a specimen representing the proposed joint design, the width of the joint can be based upon an experimentally determined development length plus a clear distance between the joint reinforcement and the nearest concrete surface adequate for concrete placement in the joint. Otherwise, the joint width shall not be less than 12.0 in.

5.14.6.10—Provisions for Steel Composite Systems

This AASHTO sub-section addressing requirements for the design of composite steel modular systems. The following sections modify applicable section of Section 6, Steel Structures, of the *LRFD Bridge Design Specifications:*

6.7.4.1—Diaphragms and Cross Frames

This section addresses the location of diaphragms and cross frames in steel structures. The following additional requirements are extended to apply to prefabricated elements and systems.

1. In interior lift points for composite modular system shall be considered an interior support.
2. At interior supports provide either a diaphragm or a cross-frame with necessary stiffeners as appropriate for bracing, connections and local bearing. The designer should address suitable diaphragm or cross-frame details to provide the necessary compression flange stability under temporary handling conditions.
3. Investigation shall include the stability of compression flanges during handling and placement. Diaphragms or cross-frames required for the construction condition may be specified to be temporary bracing.

6.10.1.1.1a—Sequence of Loading

This section addresses loads applied to a steel structure. The following additional requirements are extended to apply to prefabricated steel modular systems.

1. Shored construction as allowed in the last sentence of this section is not allowed for spans assembled using steel modular systems.
2. Contract Plans shall state that forming and shoring of the deck shall be supported from the longitudinal girders similar to conventional construction methods.

APPENDIX H

Recommended ABC Construction Specifications

XX Special Requirements for Prefabricated Elements and Systems for Accelerated Bridge Construction

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XX.1 General

XX.1.1 Description

This specification for prefabricated elements and modular systems for Accelerated Bridge Construction (ABC) supplements the requirements of the LRFD Construction Specifications. The work addressed in this section consists of manufacturing, storing, transporting, and assembling prefabricated substructure and superstructure elements and modular systems, specifically intended for accelerated bridge construction applications, including decked precast prestressed girders, decked steel girder modules, abutments and wings, pier columns and caps, and precast concrete bridge barriers herein referred to as elements or modular systems in accordance with the contract plans.

C XX.1.1 Commentary

Accelerated Bridge Construction is a project classification in which prefabricated bridge elements and modular systems are used to accelerate bridge construction. Bridge elements that have traditionally been cast-in-place or erected in pieces are either manufactured off-site and/or subassembled and erected as a unit to facilitate faster construction on-site and reduce related impacts to traffic. Prefabricated bridge elements for substructures typically consist of precast concrete elements connected in the field to create a homogeneous unit, and superstructure modules typically consist of concrete or steel girder pairs prefabricated with composite concrete deck slabs.

The fabrication of bridge elements and modular systems is performed off-site (or on-site away from traffic) under controlled conditions. Following fabrication, the bridge elements or modules are transported to the work site for rapid field installation.

XX.1.2 Benefits

Accelerated Bridge Construction structure types are intended to minimize field construction time, simplify field construction operations and improve quality control (i.e., quality and durability of structure). Utilizing Accelerated Bridge Construction structure types can increase construction zone safety through reduced exposure time, minimize traffic impacts due to construction operations, minimize construction environmental impacts, and streamline overall construction operations.

By replacing typical cast-in-place concrete construction with factory-produced precast elements (both stand-alone substructure elements and girder/deck superstructure modules), several benefits are realized. Controlled conditions associated with factory production of prefabricated bridge elements result in higher-quality precast elements with less variability. Mass production can yield significant time savings for bridges requiring similar elements.

XX.2 Responsibilities

XX.2.1 Design

Similar to a traditional bridge project, the engineer-of-record is responsible for the final design of the bridge. As such, design of all the bridge elements and systems is the responsibility of the engineer-of-record. Design of the prefabricated bridge elements should not only consider the final in-service condition (typical design condition) but should also consider construction loading, including a feasible means of construction. Special design consideration should therefore be given to loading, due to construction conditions such as transportation, support on blocking, and unique (one-time) demands during erection.

Projects designated as Accelerated Bridge Construction should include plan details corresponding to the anticipated accelerated construction methods. Basic schematic graphics illustrating the anticipated construction methods (suggested erection sequence) as well as details to facilitate the anticipated construction methods (such as lifting lugs or similar) should be provided in the details.

C XX.2.1 Commentary

Projects intended to utilize Accelerated Bridge Construction design concepts should be directly designated as such. Plans and special provisions shall impose construction time restrictions and mandate shortened construction schedules. To ensure consistency in receipt of construction bids, bridge type designation as Accelerated Bridge Construction should not be left solely to the contractor. Value engineering studies could also afford opportunities to redesign a "conventional" bridge type using ABC design concepts to achieve shortened construction schedules.

Assurance should be provided to verify that the design assumptions and planned construction activities are consistent since the design details are highly dependent upon the assumed construction methods. One method to achieve this assurance would be to require (per plan or specification) that the contractor submit the proposed construction methods (i.e. module picking locations, temporary support locations, etc.) to the engineer-of-record for approval prior to beginning construction.

XX.2.2 Construction

The contractor shall be responsible for the safe construction of the bridge. This responsibility includes the design and construction of any temporary structures, falsework, or specialized equipment required to construct the bridge.

In addition, the contractor shall be responsible for producing the proposed bridge in an undamaged condition with correct geometry to industry standard with built-in dead load stresses and erection stresses which are consistent with the design assumptions.

The contractor shall be responsible for performing all construction operations with applicable project guidelines. The contractor shall be responsible for hiring a competent engineer with the requisite qualifications to design the temporary works or complete the proposed construction engineering in accordance with his defined means and methods. The requirement for a qualified construction engineer working on behalf of the contractor shall be clearly identified in the contract documents at the direction of the Contracting Authority and the Engineer of Record.

C XX.2.2 Commentary

The bid plans should be sufficiently developed with regard to construction loadings and allowable erection stresses on elements and components as design assumptions are generally not made part of bid documents. The bid plans should also include one feasible method of erection. Such measures are needed to assure contractors will have a set of constructable plans that can built in the designated time frames specified in the contract documents at bid time.

XX.2.3 Inspection

The owner or the owner's representative is responsible for inspection of the bridge construction as the owner deems appropriate.

Two phases of inspection should be implemented for Accelerated Bridge Construction projects. Fabrication inspection should monitor the fabrication operations in the shop and/or at the site casting facility to verify the quality of the physical pieces to be used in the bridge construction. Materials, quality of workmanship, shop operations, and geometry are issues that should be addressed for the fabrication inspection process. Field inspection should verify that the proposed erection methods are executed in the field and that the final in-place bridge elements meet provisions per plans and special provisions. Specific contractor means-and-methods should be reviewed to ensure the contractor's methodology conforms to the assumptions made during design and/or addresses concerns that may arise if deviating from the original design intent.

XX.3 Materials

XX.3.1 Description

The materials used for prefabricated elements and systems, closure pours, and connections shall conform to the requirements of the LRFD Bridge Construction Specifications, the other articles in this section, and the project special provisions.

XX.3.2 Concrete

High Performance Concrete (HPC) for prefabricated elements shall conform to the requirements of Section 8 of the LRFD Bridge Construction Specifications and the project special provisions.

XX.3.3 Steel

Structural steel, reinforcing steel, and prestressing steel for prefabricated elements shall conform to the requirements of the LRFD Bridge Construction Specifications and the project special provisions.

XX.3.4 Closure Pours

High early strength Self-Consolidating Concrete (SCC) mix designs for substructure closure pours and pile pockets, as shown on the plans, shall comply with the requirements of the project special provisions.

1. A high early strength Ultra High Performance Concrete (UHPC) mix design for superstructure closure pours, as shown on the plans, shall comply with the requirements as specified below and the requirements of the project special provisions.

   MATERIAL
   Ultra High Performance Concrete (UHPC)
   The material shall be Ultra High Performance Concrete consisting of the following components all supplied by one manufacturer:

   - Fine aggregate;
   - Cementitious material;
   - Superplasticizer;
   - Accelerator; and
   - Steel fibers, specifically made for steel reinforcement with a minimum tensile strength 360,000 psi (2,500 MPa).
   - Water that is potable or free from foreign materials in amounts harmful to concrete and embedded steel.

   Qualification Testing. The contractor shall complete the qualification testing of the UHPC 2 months before placement of the joint. The minimum concrete compressive strength shall be 10,000 psi at 48 hours and 24,000 psi at 28 days. The minimum flexural strength at 28 days shall be 5,000 psi. The compressive strength shall be measured by ASTM C39. Concrete flexural strength shall be according to ASTM C78. Only a concrete mix design that passes these tests may be used to form the joint.

XX.3.5 Grout

A structural nonshrink grout shall be applied at all pier column joints to ensure uniform bearing, as shown on the plans. Nonshrink grout shall be high-performance structural non-shrink grout that has low-permeability; quick-setting, rapid strength gain; and high-bond strength. Mix grout just prior to use according to the manufacturer's instructions. Follow manufacturer's recommendation for dosage of corrosion inhibitor admixture. Use structural nonshrink grout that meets a minimum compressive strength of 4,000 psi within 24 hours when tested as specified in AASHTO T106. The grout shall be prepackaged, commercially available, and approved prior to use.

XX.3.6 Couplers

Where shown on the plans, use grouted splice couplers to join precast substructure elements. Provide couplers that use cementitious grout placed inside a steel casting. Use grouted splice couplers that can provide 100% of the specified minimum tensile strength of the connecting Grade 60 reinforcing bar. This equates to 90 ksi for reinforcement conforming to ASTM A615 and 80 ksi for reinforcing conforming to ASTM A706.

XX.4 Fabrication

XX.4.1 Qualifications of the Fabricator

The elements shall be provided by a fabricator with experience in the manufacture of similar products, satisfactory to the Contracting Authority, and shall provide documentation demonstrating adequate staff, appropriate forms, experienced personnel, and a quality control plan.

XX.4.2 Fabrication Plants

All manufacturing plants/casting facilities shall satisfy the following minimum requirements:

1. Plant Casting
   The precast concrete manufacturing plant used for the prefabrication of prestressed concrete elements shall be certified by the Prestressed Concrete Institute Plant Certification Program. All precast products used in the bridge system shall be fabricated by the same precast plant. The Fabricator shall submit proof of certification prior to the start of production.

   Certification shall be as follows:

   - For deck panels, certification shall be category B2 or higher. For straight strand members, certification shall be category B3 or higher. For draped strand members, certification shall be in category B4.
   - Site-casting shall conform to the Alternate Site Casting provisions listed herein and prequalified by the Engineer.

2. Site Casting

If the contractor elects to fabricate the non-prestressed bridge elements at a temporary casting facility, the casting shall comply with the provisions listed below:

A. Equipment

Use equipment meeting the following requirements:

1. Casting Beds

For precast concrete, use casting beds rigidly constructed and supported so that under the weight (mass) of the concrete and vertical reactions of hold-ups and hold-downs there will be no vertical deformation of the bed.

2. Forms

Use forms for precast true to the dimensions as shown in the contract documents, true to line, mortar tight, and of sufficient rigidity to not sag or bulge out of shape under placement and vibration of concrete. Ensure inside surfaces are smooth and free of any projections, indentations, or offsets that might restrict differential movements of forms and concrete.

3. Curing

a) Use a method of curing that prevents loss of moisture and maintains an internal concrete temperature at least 40°F (4°C) during the curing period. Obtain Engineer's approval for this method.
b) When using accelerated heat curing, do so under a suitable enclosure. Use equipment and procedures that will ensure uniform control and distribution of heat and prevent local overheating. Ensure the curing process is under the direct supervision and control of competent operators.
c) When accelerated heat is used to obtain temperatures above 100°F, record the temperature of the interior of the concrete using a system capable of automatically producing a temperature record at intervals of no more than 15 minutes during the curing period. Space the systems at a minimum of one location per 100 feet of length per unit or fraction thereof, with a maximum of three locations along each line of units being cured. Ensure all units, when calibrated individually, are accurate within ±5°F (3°C). Do not artificially raise the temperature of the concrete above 100°F for a minimum of 2 hours after the units have been cast. After the 2-hour period, the temperature of the concrete may be raised to a maximum temperature of 160°F (71°C) at a rate not to exceed 25°F (15°C) per hour. Lower the temperature of the concrete at a rate not to exceed 40°F (22°C) per hour by reducing the amount of heat applied until the interior of the concrete has reached the temperature of the surrounding air.
d) In all cases, cover the concrete and leave covered until curing is completed. Do not under any circumstances remove units from the casting bed until the strength requirements are met.

4. Removal of Forms

If forms are removed before the concrete has attained the strength which will permit the units to be moved, immediately replace the protection and resume curing after the forms are removed. Do not remove protection any time before the units attain the specified compressive strength when the surrounding air temperature is below 20°F (−7°C).

5. Tolerances

Fabrication tolerances shall conform with Section 4.4 of these specifications.

6. Surface Finish
   Finish as surfaces which will be exposed in the finished structure as provided in Section 8.10 of the LRFD Bridge Construction Specifications.

XX.4.3 Fabrication Requirements

Do not place concrete in the forms until the engineer has inspected the form and has approved all materials in the precast elements and the placement of the materials in the form.

Provide the Engineer a tentative casting schedule at least 2 weeks in advance to make inspection and testing arrangements. A similar notification is required for the shipment of precast elements to the job site.

Obtain a minimum compressive strength of 500 psi prior to stripping the form. Minimum compressive strength prior to moving unit shall be 4,500 psi or as provided in the project plans or specifications. The precast elements will have a minimum cure of 14 days prior to placement.

Supply test data such as slump, air voids, or unit weight for the fresh concrete and compressive strengths for the hardened concrete after 7, 14, and 28 days, if applicable.

Finish the precast elements according to Section 8.10 of the LRFD Bridge Construction Specifications.

Decked girder systems shall be supported at the bearing points during deck casting operations and storage. Shored construction is not allowed. Contract Documents shall include a completed table of "anticipated deflections." The deflection control shall be checked prior to pouring and monitored throughout the pouring process.

The prefabricated superstructure span shall be preassembled to ensure a proper match between modules to the satisfaction of the Engineer before shipping to the job site. The procedure for leveling any differential camber shall be established during the preassembly and approved by the engineer. The modules shall be matched as closely as possible for camber and match-marked. Dimensions shall be provided to the contractor for setting precast substructure elevations.

The modules should be measured for sweep and the bearing anchor bolt locations reconfigured as needed. Anchor bolts may be cast into the precast pier cap or, at the Contractor's option, drilled and grouted into the precast pier cap, at no additional cost to the Contracting Authority.

XX.4.4 Fabrication Tolerances

Fabrication tolerances shall be according to standard precast practice. PCI MNL-116 Manual for Quality Control for Plants and Production of Precast and Prestressed Concrete Production or PCI MNL-135-00 Tolerance Manual for Precast and Prestressed Concrete Construction shall be consulted for more detailed tolerances for precast elements. Tolerances for project-specific requirements shall be detailed in the project plans and specifications.

Construct modules to the following minimum tolerances unless noted otherwise:

- Deck surfaces must meet a 1/8 inch in 10-foot straightedge requirement in longitudinal and transverse directions.
- Control of camber during fabrication is required to achieve ride quality. Differences in camber between adjacent modules shall not exceed ¼ inch at the time of erection. Establish the differential camber by preassembling the modules as required herein.
- Ensure beam seat bearing areas are flat and perpendicular transversely to the vertical axis of the beam.

XX.4.5 Yard Assembly

Contractor should ensure that the prefabricated elements will fit-up and align properly before shipping from the precast facility. Assembling each superstructure and substructure composed of prefabricated elements in the yard prior to shipping the elements to the project site would be a suitable way for performing such verification. If assembled in the yard, use blocking to simulate the support of the elements and the spacing between the elements. Verify the construction of all elements in compliance with all plan requirements. All connections shall be dry fit in the fabrication yard prior to installation of the elements at the bridge site.

XX.5 Submittals

The submittals requiring written approval from the Engineer are as follows:

XX.5.1 Shop Drawings

The Contractor shall prepare and submit shop details and all other necessary working drawings for approval in accordance with the requirements of project specifications. The Contractor shall submit six copies of the shop drawings for approval. Fabrication shall not begin until written approval of the submitted shop drawings has been received from the Engineer. Deviation from the approved shop drawings will not be permitted without written order or approval of the Engineer.

Prepare shop drawings under the seal of a licensed Professional Engineer. Submit xx sets for approval 28 days before fabrication.

The Shop Drawings shall include, but not necessarily be limited to, the following:

- Show all lifting inserts, hardware, or devices and locations on the shop drawings for engineer's approval.
- Description of method of curing, handling, storing, transporting, and erecting the sections.
- Show locations and details of the lifting devices and lifting holes, including supporting calculations, type, and amount of any additional precast concrete reinforcing required for lifting.
- Show any leveling inserts in the deck and include the leveling procedure for modules.
- Show details of vertical elevation adjusting hardware.
- Show minimum compressive strength attained for precast concrete deck and concrete traffic rail prior to handling the modules.
- Show details of structural steel, shear connectors, and bearing assemblies as well as elastomeric bearing pads.
- Quantities for each section (concrete volume, reinforcing steel weight, and total section weight).

Do not order materials or begin work until receiving final approval of the shop drawings. The Contracting Authority will reject any module fabricated before receiving written approval or outside of specified tolerances, subject to the review of the engineer. The Contractor shall be responsible for costs incurred due to faulty detailing or fabrication.

XX.5.2 Assembly Plan

Prepare the assembly plan under the seal of a licensed Professional Engineer. Submit xx sets for approval 28 days before fabrication.

The assembly plan shall include, but not necessarily be limited to, the following:

- A work area plan, depicting known utilities overhead and below the work area, drainage inlet structures, protective measures, etc.
- Details of all equipment that will be employed for the assembly of the superstructure, substructure, and approach slabs.
- Details of all equipment to be used to lift modules including cranes, excavators, lifting slings, sling hooks, jacks, etc. Include crane locations, operation radii, lifting calculations, etc.
- Computations to indicate the magnitude of stress in the prefabricated components during erection are within allowable limits and to demonstrate that all of the erection equipment has adequate capacity for the work to be performed.
- Detailed sequence of construction and a CPM schedule for all operations. Account for setting and cure time for any grouts and concrete closure pours, splice couplers, and fill of pile pockets.
- Methods of providing temporary support of the elements. Include methods of adjusting, bracing, and securing the element after placement.
- Procedures for controlling tolerance limits.
- Methods for leveling any differential camber between adjacent modules prior to placing closure pour.
- Methods of forming closure pours, fill concrete, and sealing lifting holes.
- Methods for curing grout, closure pour, and lifting hole concrete.
- Method for diamond grinding to achieve deck profile and transverse or longitudinal grooving. Method of verification of deck smoothness.
- A list of personnel that will be responsible for the grouting of the reinforcing splice couplers. Include proof of completion of two successful installations within the last 2 years. Training of new personnel within 3 months of installation by a manufacturer's technical representative is an acceptable substitution for this experience. In this case, provide proof of training.

XX.6 Quality Assurance

1. When precast members are manufactured in established casting yards, the manufacturer shall be responsible for the continuous monitoring of the quality of all materials and concrete strengths. Tests shall be performed in accordance with AASHTO or ASTM methods. The Engineer shall be allowed to observe all sampling and testing, and the results of all tests shall be made available to the engineer.
2. An owner representative will inspect the fabrication of the members for quality assurance. This inspection will include the examination of materials, work procedures, and the final fabricated product. At least fourteen (14) days prior to the scheduled start of casting on any member or test section, the Fabricator shall contact the owner to provide notice of the scheduled start date. The Inspector shall have the authority to reject any material or workmanship that does not meet the requirements of the contract documents. The inspector shall affix an acceptance stamp to members ready for shipment. The Inspector's acceptance implies that, in the opinion of the Inspector, the members were fabricated from accepted materials and processes and loaded for shipment in accordance with the contract requirements. The Inspector's stamp of acceptance for shipment does not imply that the members will not be rejected by the Engineer if subsequently found to be defective. The Fabricator shall fully cooperate with the Inspector in the inspection of the work in progress. The Fabricator shall allow the Inspector unrestricted access to the necessary areas of the shop or site casting yard during work hours.
3. Permanently mark each module with date of fabrication, supplier identification, and module identification. Stamp markings in fresh concrete.

4. Prevent cracking or damage of precast components during handling and storage.
5. Replace defects and breakage of precast concrete deck and concrete traffic rail according to the following:
   - Modules that sustain concrete damage or surface defects during fabrication, handling, storage, hauling, or erection are subject to review or rejection.
   - Obtain approval before performing concrete repairs.
   - Concrete repair work must reestablish the module's structural integrity, durability, and aesthetics to the satisfaction of the Engineer.
   - Determine the cause when damage occurs and take corrective action.
   - Failure to take corrective action, leading to similar repetitive damage, can be cause for rejection of the damaged module.
   - Cracks that extend to the nearest reinforcement plane and fine surface cracks that do not extend to the nearest reinforcement plane but are numerous or extensive are subject to review and rejection.
6. Modules will be rejected for any of the following reasons:
   - Fabrication not in conformance with the contract documents.
   - Full-depth cracking of concrete and concrete breakage that is not repairable to 100% conformance to the actual product is cause for rejection.
   - Camber that does not meet the requirements required by the plans or shop drawings.
   - Honeycombed texture.
   - Dimensions not within the allowable tolerances specified in the contract documents.
   - Defects that indicate concrete proportioning, mixing, and molding not conforming to the contract documents.
   - Damaged ends, preventing satisfactory joints.
   - Damage during transportation, erection, or construction determined to be significant by the Engineer.
7. The plant (or fabricator) will document all test results for structural concrete. The quality control file will contain at least the following information:
   - Module identification
   - Date and time of fabrication of concrete pour
   - Concrete cylinder test results
   - Quantity of used concrete and the batch printout
   - Form-stripping date and repairs if applicable
   - Location/number of blockouts and lifting inserts
   - Temperature and moisture of curing period
   - Document lifting device details, requirements, and inserts

XX.7 Handling, Storing, and Transportation

1. Damage/Cracking

Prevent cracking or damage of prefabricated elements and modules during handling and storage and transportation is central to the success of an ABC project as each component is an integral part of the finished structure.

Modules damaged during handling, storage, or transportation will be repaired or replaced at the Contract Authority's direction at no cost to the Contract Authority. The Prime Contractor will be liable for repairing or replacing the damaged modules to the satisfaction of the Engineer, irrespective of the source of the damage.

The *PCI New England Region Bridge Member Repair Guidelines,* Report PCINER-01-BMRG, shall be used in conjunction with this specification to identify defects that may occur during the fabrication and handling of bridge elements determine the consequences of the defects, appropriate repair procedure if warranted and making decisions on acceptance/repair or rejection.

2. Precast Element Sizes

The size of precast elements should be finalized by the precaster and the contractor with consideration for shipping restrictions, equipment availability, and site constraints. The final element sizes will be shown on the assembly plan.

3. Lifting Devices

The design and detailing of the lifting devices is the responsibility of the fabricator. Lifting devices shall be used in a manner that does not cause damage, cracking, or torsional forces. The Contractor will provide the spacing and location of the lifting devices on the shop drawings and calculate handling stresses.

Lifting devices should be placed to avoid being visible once the prefabricated element is placed or should be detailed with recessed pockets that can be patched after installation.

4. Safety

The contractor shall be responsible for the safety and stability of prefabricated elements during all stages of handling, transportation, and construction.

5. Handling and Storing

Beams shall be stored horizontal, in an upright position, supported at their designated bearing points.

Follow Chapter 5 of the *PCI Design Handbook* for handling and erection bracing requirements.

The angle between the top surface of the precast element and the lifting line shall not be less than sixty degrees, when measured from the top surface of the precast elements to the lifting line. If two cranes are used, the lifting lines should be vertical.

Modules shall be lifted at the designated points by approved lifting devices properly attached to the module and proper hoisting procedures. The Contractor is responsible for handling stresses in the modules. The Contractor will provide the spacing and location of the lifting devices on the shop drawings and calculate handling stresses. The Contractor shall include all necessary precast element modifications to resist handling stresses on the shop drawings. The locations of the lifting points shall be chosen so that the anticipated flexural tensile stress induced in the top of the structural concrete slab for the assumed support locations is no greater than the allowable stress. The Contracting Authority may institute an instrumentation program to monitor handling and erection stresses in the modules. The contractor shall provide the necessary cooperation for the instrumentation program.

Storage areas shall be smooth and well compacted to prevent damage due to differential settlement.

Precast elements shall be stored in such a manner that adequate support is provided to prevent cracking or creep-induced deformation (sagging) during storage for long periods of time. Precast elements shall be checked at least once per month to ensure that creep-induced deformation does not occur.

Modules shall be protected from freezing temperatures (0°C, 32°F) for 5 days or until precast concrete attains design compressive strength detailed on the plans, whichever comes first. Do not remove protection any time before the units attain the specified compressive strength when the surrounding air temperature is below 20°F.

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

The modules shall not be subject to damaging torsional, dynamic, or impact stresses. Care should be taken during handling, storage, and transportation to prevent cracking or damage. Units damaged by improper storage or handling shall be replaced or repaired to the satisfaction of the owner at the Contractor's expense. Contractor will be responsible for any schedule delays due to rejected elements.

6. Transportation

Minimum compressive strength prior to moving unit shall be 4,500 psi or as provided in the project plans or specifications.

A 48-hour notice of the loading and shipping schedule shall be provided to the Contracting Authority.

Transport modules horizontal with beams on the bottom side for support. Support the modules at approximately the same points they will be supported when installed.

Material, quality, and condition after shipment will be inspected after delivery to the construction site, with this and any previous inspections constituting only partial acceptance.

XX.8 Geometry Control

XX.8.1 General

Construction geometry control for differential camber, skewness, and cross slope are key to ensuring proper fit-up of prefabricated elements and systems.

The contractor shall check the elevations and alignment of the structure at every stage of construction to ensure proper erection of the structure to the final grade shown on the design plans. Use vertical adjustment devices to provide grade adjustment to meet the elevation tolerances shown on the substructure elevation plans. Pier columns and pier cap elevations can be adjusted with shim stacks contained in the grouted joints. Girder seat elevations at the erected abutments and piers shall not deviate from the plan elevations by more than ± ¼ inch. Corrections and adjustments for grade shall be done only when approved by the engineer.

Bridge cross slope up to 4 degrees can be accommodated by tilting the superstructure modules with respect to plumb. The slope of the bridge seat shall conform to the bridge cross slope. Corrections for grade by shimming or neoprene pads shall be done only when approved by the engineer.

XX.8.2 Camber and Deflection

Differential camber of prestressed girders can lead to dimensional problems with the connections. Control of camber during fabrication is required to achieve ride quality. Schedule fabrication so that camber differences between adjacent deck sections are minimized. Differences in camber between adjacent modules shall not exceed ⅛ inch at the time of erection. Establish the differential camber by preassembling the modules as required herein.

XX.8.3 Equalizing Differential Camber

Differential camber in prestressed girders is a common occurrence. Several steps can be taken during the fabrication and storage stages of the girder to minimize the potential for differential camber in girders that will be placed adjacent to each other in the bridge. In general, all aspects of the fabrication process should be as uniform as possible for each girder. Mix design and concrete batch quality should be carefully monitored. Cure time should not vary, which may inadvertently occur if only some of the girders are permitted an extended curing period. Location of temporary supports for girders in fabrication yard should be uniform. Exposure to sunlight should also be uniform.

Estimates of girder camber should be made with the recognition that girder camber is inherently variable due to the many parameters that influence it. Allowances should therefore be made in tolerances in the project to permit a reasonable level of deviation not exceeding ¼ inch of actual camber from predicted values.

Skews cause special problems with decked girders that are not present in cast-in-place systems. When the ends of the girders are skewed, the corners of the deck will have different elevations because one corner is farther "up" the camber curve than the other corner. Consequently, for a skewed girder, the top elevation of the deck at the obtuse corner is higher than at the acute corner. A method to eliminate the saw tooth effect is to increase the bearing elevation of each adjacent girder as you move from the acute corner of the deck to the obtuse corner.

For steel composite modular systems, dead load deflections for the steel beam and diaphragms alone and for the weight of the deck, back wall, and barriers shall be shown on the plans at every tenth point. Differences in camber between adjacent modules shall not exceed ⅛ inch at the time of erection. Establish the differential camber by preassembling the modules as required herein.

Equip all deck sections with leveling inserts for field adjustment or equalizing of differential camber. The inserts with threaded ferrules are cast in the deck, centered over the beam's web. A minimum tension capacity of 5,500 lb is required for the inserts. After all adjustments are complete and the deck sections are in their final position, fill all leveling insert holes with a nonshrink epoxy grout.

Have available a leveling beam and suitable jacking assemblies for attachment to the leveling inserts of adjacent beams. Adjust the deck sections to the tolerances required. More than one leveling beam may be necessary.

If the prescribed adjustment tolerance between deck sections cannot be attained by use of the approved leveling system, shimming the bearings of the deck sections may be necessary.

C XX.8.3 Commentary

One important consideration in ABC is eliminating the differential camber between the precast girders. It is important to develop an adequate means of removing the differential camber between the girders on site. Differential camber in prefabricated elements could lead to fit-up problems and riding surface issues. If the differential camber is excessive, dead load can be applied to the high beam to bring it within the connection tolerance.

LRFD Article 2.5.2.4, Rideability, requires the deck of the bridge shall be designed to permit the smooth movement of traffic. Construction tolerances, with regard to the profile of the finished deck, shall be indicated on the plans or in the specifications or special provisions. The number of deck joints shall be kept to a practical minimum. Where concrete decks without an initial overlay are used, consideration should be given to providing an additional thickness of 0.5 in. to permit correction of the deck profile by grinding and to compensate for thickness loss due to abrasion.

While the application of an overlay helps overcome finite geometric tolerances, it also requires another significant critical path activity prior to opening a structure to traffic. Today's availability of low-permeability concretes and corrosion-resistant reinforcing steels allows owners to forego the use of overlays on bridge decks.

With prefabricated superstructure construction, the objective is to develop methods that achieve the final ride surface without the use of overlays. Control of cambers during fabrication and equalizing cambers or leveling in the field is intended to achieve the required ride quality.

An attractive option is diamond grinding decks with sacrificial cover to obtain the desired surface profile. Such a method can be faster and more cost-effective.

Accurate predictions of the deflections and camber are difficult to determine since modulus of elasticity of concrete, E_c*, varies with stress and age of concrete. The effects of creep on deflections are difficult to estimate. An accuracy of 10% to 20% is often sufficient.*

Three methods typically employed to level girders are:

Jacking – A cross beam and portable hydraulic jack are used to apply counteracting forces to the tops of girders to adjust the elevations of the girder surfaces to a level condition.

Surcharging – Heavy weights are loaded onto the tops of girders to reduce differential camber. Surcharging will likely only work for minor differential camber, as the differential camber leveling forces can be significant.

Crane-Assisted Leveling – A crane is used to lift one end of the girder to bring the connectors near the middle of the girder into vertical alignment with the adjacent girder's connectors. Welds are made or clamps are installed and the crane incrementally lowers the lifted end to progressively bring further connectors along the longitudinal joint into vertical alignment.

XX.8.4 Finishing of Bridge Deck

XX.8.4.1 Diamond Grind Bridge Deck

Diamond grind the bridge deck for profile improvement as required by the plans, to a maximum depth of ½ inch, in conformance with the LRFD Construction Specifications. An additional thickness of ½ inch (minimum) should be incorporated in the deck to permit correction of the deck profile by grinding. Diamond grinding of the bridge deck shall not begin until the UHPC closure pour concrete has reached the specified minimum compressive strength of 10 ksi.

XX.8.4.2 Saw Cut Groove Texture Finish

Saw cut longitudinal grooves into top-of-bridge deck using a mechanical cutting device after diamond grinding. Saw cutting grooves shall conform to Section 8 of the LRFD Bridge Construction Specifications.

XX.9 Connections

XX.9.1 Requirements for UHPC Joints in the Deck

Prior to the initial placement of the UHPC, the contractor shall arrange for an on-site meeting with the materials supplier representative and the Engineer. The contractor's staff shall attend the site meeting. The objective of the meeting will be to clearly outline the procedures for mixing, transporting, finishing, and curing of the UHPC material.

Mock-ups of each UHPC pour shall be performed prior to actual UHPC construction and conducted per the requirements of the special provisions and the recommendation of the materials supplier representative. The mock-up process shall be observed by the materials supplier representative.

Forming, batching, placing, and curing shall be in accordance with the procedures recommended by the materials supplier and as submitted and accepted by the Materials Engineer.

All the forms for UHPC shall be constructed from plywood. Use top and bottom forms for UHPC joints.

Two portable batching units will be used for mixing of the UHPC. The contractor shall follow the batching sequence as specified by the materials supplier and approved by the District Materials Engineer.

Each UHPC placement shall be cast using one continuous pour. No cold joints are permitted.

An epoxy bonding coat shall be applied to the HPC deck interface with the UHPC joint. Surface preparation for the joint interface shall be as required in the project special provisions.

The concrete in the form shall be cured according to materials supplier recommendations at minimum temperature of 60°F to attain the design strength.

XX.9.2 Requirements for Mechanical Grouted Splices

A template will be required for accurate mechanical splice placement during element fabrication and/or field cast conditions to ensure fit-up between joined elements. Placement tolerances should be as recommended by the manufacturer. The grouting process should follow the manufacturer's recommendations for materials and equipment. All connections between precast elements be dry fit in the fabrication yard prior to installation of the elements at the bridge site.

GROUTED SPLICE COUPLERS

Submit xx copies of an independent test report confirming the compliance of the coupler, for each supplied coupler size, with the following requirements:

- Develop 100% of the specified minimum tensile strength of the attached Grade 60 reinforcing bar. This equates to 90 ksi bar stress for an ASTM A615 bar and 80 ksi bar stress for an ASTM A706 bar.
- Determine through testing the amount of time required to provide 100% of the specified minimum yield strength of the attached reinforcing bar. Use this value to develop the assembly plan timing.

Submit the specification requirements for the grout, including required strength gain to develop the specified minimum yield strength of the connected reinforcing bar.

XX.9.3 Requirements for Posttensioned (PT) Connections

Requirements for posttensioning in the LRFD Specifications shall apply for PT connections.

PT connections can be used between precast concrete elements. Common types of PT connections are between pieces in a segmental box girder bridge, in pier columns and pier caps, and in precast concrete bridge decks. PT has been combined with grouted shear keys to connect deck elements here the PT is run in the longitudinal direction on typical stringer bridges. The PT systems typically include multiple grouted strands in ducts and grouted high strength thread bars.

XX.9.4 Requirements for Bolted Connections

Requirements for bolted connections in LRFD Specifications shall apply for bolted connections between prefabricated steel elements and modules.

XX.10 Erection Methods

It shall be the contractor's responsibility to employ methods and equipment which will produce satisfactory work under the site conditions encountered and project time constraints.

C XX.10 Commentary

Erection of bridge elements and modules may be done using land-based cranes or using specialized equipment supported by the permanent bridge or by temporary beams. Some suggested erection methods suitable for rapid replacement applications are as follows:

C XX.10.1 Conventional Erection Methods

Conventional erection methods refer to the typical construction methods that are employed in most bridge construction applications. Bridge element erection is done using cranes (rubber-tire or crawler). Cranes may be land-based or barge-mounted.

Advantages of this type of erection method include the following:

- *Conventional cranes are readily available for purchase or rental.*
- *Construction crews are familiar with working with conventional cranes.*
- *Conventional cranes can be used to erect bridge elements with a variety of geometric configurations.*
- *Operation is relatively simple using charts provided by the crane manufacturer which show allowable capacity for particular crane geometry and load radius.*

Disadvantages of this type of erection method include the following:

- *Required crane sizes increase with increased load and pick radius.*
- *Cranes require substantial space and foundation base for operation. Positioning and operation often require traffic disruptions.*
- *Access to erect structure may be challenging based on site conditions (adjacent rivers, steep grades, existing structures, or other geometric constraints, etc.)*

C XX.10.2 Specialized Erection Methods

C XX.10.2.1 Straddle Carriers

A straddle carrier is a self-propelled frame system in which the supported load is located within the central portion of the frame. Commonly used in the precast concrete industry to transport long and heavy precast beams, these commercially available rolling gantry cranes can be used in bridge construction in certain situations.

For bridge superstructure erection/demolition applications, the straddle carrier would be supported by either the permanent bridge or by temporary beams.

Straddle carriers typically support the load and their own self-weight on two bases (either rubber-tire or crane rail) with fixed transverse dimensions between wheels. Due to heavy wheel loads, concrete bridge decks are typically insufficient to support straddle carriers at areas away from the supporting girders. As such, straddle carriers are generally limited to use in applications with parallel supporting elements (temporary beams or permanent girders).

Potential advantages include eliminating the need for a crane (especially advantageous in high-elevation or over-waterway construction applications) and potentially avoiding traffic disruptions on the intersected roadway.

Potential disadvantages include limited availability and limited use based on fixed dimensions and existing bridge condition.

C XX.10.2.2 Specialty Erection Trusses

Specialty erection trusses can be utilized to facilitate rapid and repetitive construction operations. Steel trusses are fabricated in modules which allow shipping in pieces and assembly at the work site. Following assembly, the erection trusses are positioned to support a rolling gantry crane used to erect the new prefabricated bridge elements.

One type of specialty erection truss is referred to as Above Deck Driven Carrier (ADDC). Following assembly on site, these trusses are rolled into position on the existing bridge, temporarily supported on blocking at the piers, and used to support the rolling gantry system.

Another type of erection truss is referred to as Launched Temporary Truss Bridge (LTTB). Following assembly on site, these trusses are moved into position by launching them parallel to the bridge while support is provided on temporary falsework. These trusses are used to support the rolling gantry system.

Potential advantages include eliminating the need for a crane (especially advantageous in high-elevation or over-waterway construction applications) and potentially avoiding traffic disruptions on the intersected roadway.

Potential disadvantages include required custom design and fabrication as well as limited use based on field conditions.

C XX.10.2.3 Self-Propelled Modular Transporters

There are families of high-capacity, highly maneuverable transport trailers called Self-Propelled Modular Transporters (SPMTs) that are being used in ABC applications to transport and erect prefabricated elements, modular systems or complete spans. SPMTs have been particularly favored for removing the existing span moving the prefabricated superstructure from the staging area to its final position. SPMTs can also be adapted to install prefabricated deck and superstructure elements and modules from above where the use of land-based cranes is not feasible.

The term "modular" in the title describes the ability to connect the trailers in various configurations to form a larger transporter. The SPMTs are highly maneuverable and can be moved and rotated in all three dimensional axes. The FHWA document entitled Manual on Use of Self-Propelled Modular Transporters to Remove and Replace Bridges is recommended for more information on these machines.

XX.11 Erection Procedures

XX.11.1 General Requirements for Installation of Precast Elements and Systems

1. Dry fit adjacent precast elements in the yard prior to shipping to the site.
2. Establish working points, working lines, and benchmark elevations prior to placement of all precast elements.
3. Place precast elements in the sequence and according to the methods outlined in the assembly plan. Adjust the height of each precast element by means of leveling devices or shims.
4. Use personnel that are familiar with installation and grouting of splice couplers that have completed at least two successful projects in the last 2 years. Training of new personnel within 3 months of installation by a manufacturer's technical representative is an acceptable substitution for this experience.
5. Keep bonding surfaces free from laitance, dirt, dust, paint, grease oil, or any contaminants other than water.
6. Follow the recommendations of the manufacturer for the installation and grouting of the couplers.

XX.11.2 General Procedure for Superstructure Modules

1. Do not place modules on precast substructure until the compressive test result of the cylinders for the precast substructure connection concrete has reached the specified minimum values.
2. Survey the top elevation of the precast concrete substructures. Establish working points, working lines, and benchmark elevations prior to placement of all modules.
3. Clean bearing surface before modules are erected.
4. Lift and erect modules using lifting devices as shown on the shop drawings in conformance with the assembly plans.
5. Set module in the proper location. Survey the top elevation of the modules. Check for proper alignment and grade within specified tolerances. Approved shims may be used between the bearing and the girder to compensate for minor differences in elevation between modules and approach elevations. Follow match-marks.

6. Temporarily support, anchor, and brace all erected modules as necessary for stability and to resist wind or other loads until they are permanently secured to the structure. Support, anchor, and brace all modules as detailed in the assembly plan.
7. Differences in camber between adjacent modules shipped to the site shall not exceed the prescribed limits. If there is a differential camber, the contractor shall apply dead load to the high beam to bring it within the connection tolerance. A leveling beam can also be used to equalize camber. The leveling procedure shall be demonstrated during the preassembly process prior to shipping to the site. The assembly plan shall indicate the leveling process to be applied in the field. If a leveling beam is to be used, have available a leveling beam and suitable jacking assemblies for attachment to the leveling inserts of adjacent modules. Equip all modules with leveling inserts for field adjustment or equalizing of differential camber. The inserts with threaded ferrules are cast in the deck, centered over the beam's web. A minimum tension capacity of 5,500 lb is required for the inserts.
8. Saturate surface dry (SSD) all closure pour surfaces prior to connecting the modules. Apply an epoxy bonding coat as required by the project specifications.
9. Form closure pours and seal lifting holes as required by the approved assembly plan. The closure pour forms and the sealed lifting holes shall be free of any material such as oil, grease, or dirt that may prevent bonding of the joint. Apply epoxy bonding coat where required by plans or specifications.
10. Cast UHPC closure pours and fill lifting holes with UHPC as shown on the plans. Cure closure pours and lifting holes.
11. Remaining concrete defects and holes for inserts shall be repaired as required by the Engineer.
12. Do not apply superimposed dead loads or construction live loads to the prefabricated superstructure until the compressive test result of the cylinders for the UHPC closure pour concrete has reached the specified minimum compressive strength of 10 ksi.

XX.11.3 General Procedure for Pier Columns and Caps

1. Lift the precast element as shown in the assembly plan using lifting devices as shown on the shop drawings.
2. Survey the elevation of the completed structure directly below the element. Provide shims to bring the bottom of the element to the required elevation.
3. Set the element in the proper horizontal location. Check for proper horizontal and vertical alignment within specified tolerances. Remove and adjust the shims and reset the element if it is not within tolerance.
4. Check the grouted splice couplers between adjacent elements that will support common precast elements in future stages of construction. Set the element and install the couplers once the connection geometry is established and checked.
5. Install temporary bracing if specified in the assembly plan.
6. Allow the grout in the coupler to cure until the coupler can resist 100% of the specified minimum yield strength of the bar prior to removal of bracing and proceeding with installation of elements above the element.

XX.11.4 General Procedure for Abutment Stem and Wingwalls (supported on piles)

1. Lift abutment stem precast element or wingwall precast element as shown in the assembly plan using lifting devices as shown on the shop drawings.
2. Set the precast element in the proper horizontal location. Check for proper alignment within specified tolerances.

3. Adjust the devices prior to full release from the crane if vertical leveling devices are used. This will reduce the amount of torque required to turn the bolts in the leveling devices. Check for proper grade within specified tolerances.
4. Place high early strength self-consolidating concrete around pile tops as shown on the plans. Allow concrete to flow partially under the precast element. The entire underside of the precast element need not be filled with concrete.
5. Do not remove the installation bolts (if used) or proceed with the installation of additional precast elements above until the compressive test result of the cylinders for the pile connection concrete has reached the specified minimum values.

Related SHRP 2 Research

Geotechnical Solutions for Soil Improvement, Rapid Embankment Construction, and Stabilization of the Pavement Working Platform (R02)

Nondestructive Testing to Identify Concrete Bridge Deck Deterioration (R06A)

Bridges for Service Life Beyond 100 Years: Innovative Systems, Subsystems, and Components (R19A)