SHRP 2

STRATEGIC HIGHWAY RESEARCH PROGRAM

Accelerating solutions for highway safety, renewal, reliability, and capacity

Innovative Bridge Designs for Rapid Renewal

ABC Toolkit

S2-R04-RR-2

TRANSPORTATION RESEARCH BOARD

OF THE NATIONAL ACADEMIES

TRANSPORTATION RESEARCH BOARD 2013 EXECUTIVE COMMITTEE*

OFFICERS

Chair: Deborah H. Butler, *Executive Vice President, Planning, and CIO, Norfolk Southern Corporation, Norfolk, Virginia*
Vice Chair: Kirk T. Steudle, *Director, Michigan Department of Transportation, Lansing*
Executive Director: Robert E. Skinner, Jr., *Transportation Research Board*

MEMBERS

Victoria A. Arroyo, *Executive Director, Georgetown Climate Center, and Visiting Professor, Georgetown University Law Center, Washington, D.C.*
Scott E. Bennett, *Director, Arkansas State Highway and Transportation Department, Little Rock*
William A. V. Clark, *Professor of Geography (emeritus) and Professor of Statistics (emeritus), Department of Geography, University of California, Los Angeles*
James M. Crites, *Executive Vice President of Operations, Dallas–Fort Worth International Airport, Texas*
Paula J. C. Hammond, *Secretary, Washington State Department of Transportation, Olympia*
John S. Halikowski, *Director, Arizona Department of Transportation, Phoenix*
Michael W. Hancock, *Secretary, Kentucky Transportation Cabinet, Frankfort*
Susan Hanson, *Distinguished University Professor Emerita, School of Geography, Clark University, Worcester, Massachusetts*
Steve Heminger, *Executive Director, Metropolitan Transportation Commission, Oakland*
Chris T. Hendrickson, *Duquesne Light Professor of Engineering, Carnegie Mellon University, Pittsburgh, Pennsylvania*
Jeffrey D. Holt, *Managing Director, Bank of Montreal Capital Markets, and Chairman, Utah Transportation Commission, Huntsville, Utah*
Kevin L. Keith, *Missouri Department of Transportation, Jefferson City*
Gary P. LaGrange, *President and CEO, Port of New Orleans, Louisiana*
Michael P. Lewis, *Director, Rhode Island Department of Transportation, Providence*
Joan McDonald, *Commissioner, New York State Department of Transportation, Albany*
Donald A. Osterberg, *Senior Vice President, Safety and Security, Schneider National, Inc., Green Bay, Wisconsin*
Steve Palmer, *Vice President of Transportation, Lowe's Companies, Inc., Mooresville, North Carolina*
Sandra Rosenbloom, *Director, Innovation in Infrastructure, The Urban Institute, Washington, D.C. (Past Chair, 2012)*
Henry G. (Gerry) Schwartz, Jr., *Chairman (retired), Jacobs/Sverdrup Civil, Inc., St. Louis, Missouri*
Kumares C. Sinha, *Olson Distinguished Professor of Civil Engineering, Purdue University, West Lafayette, Indiana*
Daniel Sperling, *Professor of Civil Engineering and Environmental Science and Policy; Director, Institute of Transportation Studies; University of California, Davis*
Gary C. Thomas, *President and Executive Director, Dallas Area Rapid Transit, Dallas, Texas*
Phillip A. Washington, *General Manager, Regional Transportation District, Denver, Colorado*

EX OFFICIO MEMBERS

Rebecca M. Brewster, *President and COO, American Transportation Research Institute, Smyrna, Georgia*
Anne S. Ferro, *Administrator, Federal Motor Carrier Safety Administration, U.S. Department of Transportation*
LeRoy Gishi, *Chief, Division of Transportation, Bureau of Indian Affairs, U.S. Department of the Interior, Washington, D.C.*
John T. Gray II, *Senior Vice President, Policy and Economics, Association of American Railroads, Washington, D.C.*
Michael P. Huerta, *Administrator, Federal Aviation Administration, U.S. Department of Transportation*
Joung Ho Lee, *Associate Director for Finance and Business Development, American Association of State Highway and Transportation Officials, Washington, D.C.*
David T. Matsuda, *Administrator, Maritime Administration, U.S. Department of Transportation*
Michael P. Melaniphy, *President and CEO, American Public Transportation Association, Washington, D.C.*
Victor M. Mendez, *Administrator, Federal Highway Administration, U.S. Department of Transportation*
Robert J. Papp *(Adm., U.S. Coast Guard), Commandant, U.S. Coast Guard, U.S. Department of Homeland Security*
Cynthia L. Quarterman, *Administrator, Pipeline and Hazardous Materials Safety Administration, U.S. Department of Transportation*
Peter M. Rogoff, *Administrator, Federal Transit Administration, U.S. Department of Transportation*
David L. Strickland, *Administrator, National Highway Traffic Safety Administration, U.S. Department of Transportation*
Joseph C. Szabo, *Administrator, Federal Railroad Administration, U.S. Department of Transportation*
Polly Trottenberg, *Under Secretary for Policy, U.S. Department of Transportation*
Robert L. Van Antwerp *(Lt. General, U.S. Army), Chief of Engineers and Commanding General, U.S. Army Corps of Engineers, Washington, D.C.*
Barry R. Wallerstein, *Executive Officer, South Coast Air Quality Management District, Diamond Bar, California*
Gregory D. Winfree, *Acting Administrator, Research and Innovative Technology Administration, U.S. Department of Transportation*
Frederick G. (Bud) Wright, *Executive Director, American Association of State Highway and Transportation Officials, Washington, D.C.*

* *Membership as of February 2013.*

THE SECOND STRATEGIC HIGHWAY RESEARCH PROGRAM

Innovative Bridge Designs for Rapid Renewal: ABC Toolkit

SHRP 2 Report S2-R04-RR-2

HNTB Corporation
Genesis Structures, Inc.
Structural Engineering Associates
Iowa State University

TRANSPORTATION RESEARCH BOARD

Washington, D.C.
2013
www.TRB.org

SUBSCRIBER CATEGORIES

Bridges and Other Structures
Construction
Design
Highways

THE SECOND STRATEGIC HIGHWAY RESEARCH PROGRAM

America's highway system is critical to meeting the mobility and economic needs of local communities, regions, and the nation. Developments in research and technology—such as advanced materials, communications technology, new data collection technologies, and human factors science—offer a new opportunity to improve the safety and reliability of this important national resource. Breakthrough resolution of significant transportation problems, however, requires concentrated resources over a short time frame. Reflecting this need, the second Strategic Highway Research Program (SHRP 2) has an intense, large-scale focus, integrates multiple fields of research and technology, and is fundamentally different from the broad, mission-oriented, discipline-based research programs that have been the mainstay of the highway research industry for half a century.

The need for SHRP 2 was identified in *TRB Special Report 260: Strategic Highway Research: Saving Lives, Reducing Congestion, Improving Quality of Life,* published in 2001 and based on a study sponsored by Congress through the Transportation Equity Act for the 21st Century (TEA-21). SHRP 2, modeled after the first Strategic Highway Research Program, is a focused, time-constrained, management-driven program designed to complement existing highway research programs. SHRP 2 focuses on applied research in four areas: Safety, to prevent or reduce the severity of highway crashes by understanding driver behavior; Renewal, to address the aging infrastructure through rapid design and construction methods that cause minimal disruptions and produce lasting facilities; Reliability, to reduce congestion through incident reduction, management, response, and mitigation; and Capacity, to integrate mobility, economic, environmental, and community needs in the planning and designing of new transportation capacity.

SHRP 2 was authorized in August 2005 as part of the Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (SAFETEA-LU). The program is managed by the Transportation Research Board (TRB) on behalf of the National Research Council (NRC). SHRP 2 is conducted under a memorandum of understanding among the American Association of State Highway and Transportation Officials (AASHTO), the Federal Highway Administration (FHWA), and the National Academy of Sciences, parent organization of TRB and NRC. The program provides for competitive, merit-based selection of research contractors; independent research project oversight; and dissemination of research results.

SHRP 2 Report S2-R04-RR-2
ISBN: 978-0-309-12952-7
Library of Congress Control Number: 2013931860
© 2013 National Academy of Sciences. All rights reserved.

COPYRIGHT INFORMATION

Authors herein are responsible for the authenticity of their materials and for obtaining written permissions from publishers or persons who own the copyright to any previously published or copyrighted material used herein.

The second Strategic Highway Research Program grants permission to reproduce material in this publication for classroom and not-for-profit purposes. Permission is given with the understanding that none of the material will be used to imply TRB, AASHTO, or FHWA endorsement of a particular product, method, or practice. It is expected that those reproducing material in this document for educational and not-for-profit purposes will give appropriate acknowledgment of the source of any reprinted or reproduced material. For other uses of the material, request permission from SHRP 2.

Note: SHRP 2 report numbers convey the program, focus area, project number, and publication format. Report numbers ending in "w" are published as web documents only.

NOTICE

The project that is the subject of this report was a part of the second Strategic Highway Research Program, conducted by the Transportation Research Board with the approval of the Governing Board of the National Research Council.

The members of the technical committee selected to monitor this project and to review this report were chosen for their special competencies and with regard for appropriate balance. The report was reviewed by the technical committee and accepted for publication according to procedures established and overseen by the Transportation Research Board and approved by the Governing Board of the National Research Council.

The opinions and conclusions expressed or implied in this report are those of the researchers who performed the research and are not necessarily those of the Transportation Research Board, the National Research Council, or the program sponsors.

The Transportation Research Board of the National Academies, the National Research Council, and the sponsors of the second Strategic Highway Research Program do not endorse products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the object of the report.

SHRP 2 REPORTS

Available by subscription and through the TRB online bookstore: www.TRB.org/bookstore

Contact the TRB Business Office: 202.334.3213

More information about SHRP 2: www.TRB.org/SHRP2

THE NATIONAL ACADEMIES

Advisers to the Nation on Science, Engineering, and Medicine

The **National Academy of Sciences** is a private, nonprofit, self-perpetuating society of distinguished scholars engaged in scientific and engineering research, dedicated to the furtherance of science and technology and to their use for the general welfare. On the authority of the charter granted to it by the Congress in 1863, the Academy has a mandate that requires it to advise the federal government on scientific and technical matters. Dr. Ralph J. Cicerone is president of the National Academy of Sciences.

The **National Academy of Engineering** was established in 1964, under the charter of the National Academy of Sciences, as a parallel organization of outstanding engineers. It is autonomous in its administration and in the selection of its members, sharing with the National Academy of Sciences the responsibility for advising the federal government. The National Academy of Engineering also sponsors engineering programs aimed at meeting national needs, encourages education and research, and recognizes the superior achievements of engineers. Dr. Charles M. Vest is president of the National Academy of Engineering.

The **Institute of Medicine** was established in 1970 by the National Academy of Sciences to secure the services of eminent members of appropriate professions in the examination of policy matters pertaining to the health of the public. The Institute acts under the responsibility given to the National Academy of Sciences by its congressional charter to be an adviser to the federal government and, upon its own initiative, to identify issues of medical care, research, and education. Dr. Harvey V. Fineberg is president of the Institute of Medicine.

The **National Research Council** was organized by the National Academy of Sciences in 1916 to associate the broad community of science and technology with the Academy's purposes of furthering knowledge and advising the federal government. Functioning in accordance with general policies determined by the Academy, the Council has become the principal operating agency of both the National Academy of Sciences and the National Academy of Engineering in providing services to the government, the public, and the scientific and engineering communities. The Council is administered jointly by both Academies and the Institute of Medicine. Dr. Ralph J. Cicerone and Dr. Charles M. Vest are chair and vice chair, respectively, of the National Research Council.

The **Transportation Research Board** is one of six major divisions of the National Research Council. The mission of the Transportation Research Board is to provide leadership in transportation innovation and progress through research and information exchange, conducted within a setting that is objective, interdisciplinary, and multimodal. The Board's varied activities annually engage about 7,000 engineers, scientists, and other transportation researchers and practitioners from the public and private sectors and academia, all of whom contribute their expertise in the public interest. The program is supported by state transportation departments, federal agencies, including the component administrations of the U.S. Department of Transportation, and other organizations and individuals interested in the development of transportation. **www.TRB.org**

www.national-academies.org

SHRP 2 STAFF

Ann M. Brach, *Director*
Stephen J. Andrle, *Deputy Director*
Neil J. Pedersen, *Deputy Director, Implementation and Communications*
James Bryant, *Senior Program Officer, Renewal*
Kenneth Campbell, *Chief Program Officer, Safety*
JoAnn Coleman, *Senior Program Assistant, Capacity and Reliability*
Eduardo Cusicanqui, *Financial Officer*
Walter Diewald, *Senior Program Officer, Safety*
Jerry DiMaggio, *Implementation Coordinator*
Shantia Douglas, *Senior Financial Assistant*
Charles Fay, *Senior Program Officer, Safety*
Carol Ford, *Senior Program Assistant, Renewal and Safety*
Elizabeth Forney, *Assistant Editor*
Jo Allen Gause, *Senior Program Officer, Capacity*
Rosalind Gomes, *Accounting/Financial Assistant*
Abdelmename Hedhli, *Visiting Professional*
James Hedlund, *Special Consultant, Safety Coordination*
Alyssa Hernandez, *Reports Coordinator*
Ralph Hessian, *Special Consultant, Capacity and Reliability*
Andy Horosko, *Special Consultant, Safety Field Data Collection*
William Hyman, *Senior Program Officer, Reliability*
Michael Marazzi, *Senior Editorial Assistant*
Linda Mason, *Communications Officer*
Reena Mathews, *Senior Program Officer, Capacity and Reliability*
Matthew Miller, *Program Officer, Capacity and Reliability*
Michael Miller, *Senior Program Assistant, Capacity and Reliability*
David Plazak, *Senior Program Officer, Capacity*
Monica Starnes, *Senior Program Officer, Renewal*
Onno Tool, *Visiting Professional*
Dean Trackman, *Managing Editor*
Connie Woldu, *Administrative Coordinator*
Patrick Zelinski, *Communications/Media Associate*

ACKNOWLEDGMENTS

This work was sponsored by the Federal Highway Administration in cooperation with the American Association of State Highway and Transportation Officials. It was conducted in the second Strategic Highway Research Program, which is administered by the Transportation Research Board of the National Academies. The project was managed by Monica A. Starnes, Ph.D., Senior Program Officer for SHRP 2 Renewal.

The research reported on herein was performed by HNTB Corp. with Kenneth Price, P.E., as principal investigator and Bala Sivakumar, P.E., as co-principal investigator. Also providing support were Genesis Structures (Kansas City), Structural Engineering Associates (San Antonio), and Iowa State University (Ames). 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 provided important information and documentation for this project.

FOREWORD

Monica A. Starnes, PhD
SHRP 2 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 in Utah.

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.

At its inception, the project focused on identifying and evaluating the historical 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 itself to develop new structural concepts by incrementally improving proven and accepted bridge systems, components, and details. Structural evaluations, analyses, designs, and laboratory testing provided the tools to achieve the sought improvements.

This *ABC Toolkit* (the *Toolkit*) was produced with bridge practitioners in mind. It provides a series of design and construction concepts for prefabricated elements and their connections. Based on the scope of work, the *Toolkit* also provides proposed language for AASHTO design and construction specifications.

Since the initiation of this research project, other ABC-related programs either have matured (e.g., Utah DOT's ABC program) or have been established (e.g., FHWA's Every Day Counts [EDC]) in parallel. While the *Toolkit* provides concepts for designing and building complete bridges, it is not meant to be a complete manual 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.

The *Toolkit* is being published as an interim publication with the understanding that additional work will be completed by SHRP 2 to include lateral sliding concepts for bridges in a future version of the *Toolkit*. Additional work will be undertaken by others to bring the terminology of the *Toolkit* into agreement with that used in the FHWA-EDC program.

CONTENTS

Online version of this report:
www.trb.org/Main/Blurbs/168046.aspx.

1 INTRODUCTION

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

ABC applications in the United States have developed two different approaches: accelerated construction of bridges in place using prefabricated bridge elements and systems and the use of bridge movement technology and equipment to move completed superstructures from an off-alignment location into the final position. Rapid construction of bridges in place offers the promise of limited closures, maybe days or weeks at the 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.

Despite the gradual lowering of costs, departments of transportation (DOTs) are hesitant about using ABC techniques because of their perceived risks and higher initial costs. Rather than custom engineering 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. A key objective of the SHRP 2 Renewal Project R04 was to develop “standardized approaches to designing and constructing complete bridge systems for rapid renewals.” The aim therefore was 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 and erection. In this regard the R04 team has focused on specific strategies for ABC systems, as follows:

1. As light as possible:
   - Is sized in a manner to be manageable for transportation and installation,
   - Simplifies transportation and erection of bridge components, and
   - Could improve the load rating of existing piers/foundations;
2. As simple as possible:
   - Fewer girders,
   - Fewer field splices,
   - Fewer bracing systems, and
   - No temporary bracing to be removed;
3. As simple to erect as possible:
   - Fewer workers on-site,
   - Fewer cast-in-place operations,
   - No falsework structures required for prefabricated elements and systems, and
   - Simpler geometry.

The ABC design concepts have been classified into five tiers based on implementation duration 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; and
- Tier 5: Overall project schedule is significantly reduced by months to years.

Modular systems allow a more versatile option for ABC not limited by space availability at the bridge site. Modular bridge systems are particularly suited to be used as Tier 2 concepts for weekend bridge replacements or as Tier 3 concepts where the entire bridge may be scheduled to be replaced within 1 to 2 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 placement using self-propelled modular transporters (SPMTs). Tier 5 involves accelerating a statewide bridge renewal program by months or years by applying ABC technologies included in the other tiers.

Project R04 was composed of three distinct phases over a time period of 4 years, beginning in 2008. Phase I 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 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 can be advanced to ABC standard concepts in Phase III. Work on Phase III commenced on January 1, 2011, and was completed in March 2012. Phase III also included the construction of the first ABC demonstration project utilizing the modular ABC systems covered in the standard concepts.

OVERVIEW OF THE *ABC TOOLKIT*

Prefabricated bridge elements and systems (PBES) are structural components of a bridge that are built either off-site or adjacent to the site, in a manner to reduce the on-site construction and mobility impact times that can adversely affect the traveling public. Because of their versatility, PBES can be used to address many common site and constructability issues. Use of PBES has demonstrated proven benefits to agency owners, contractors, and the traveling public. Compared to conventional construction methods it is faster and safer, lowers mobility impacts, provides better quality, lowers cost, and is easily adaptable to many site conditions.

Use of PBES has demonstrated proven benefits to agency owners, contractors, and the traveling public.

Overcoming impediments to the greater use of PBES was a key focus of this research. The research team developed pre-engineered designs optimized for modular construction and ABC. Standardizing ABC systems will bring about greater familiarity with ABC technologies and concepts and also foster more widespread use of ABC. The *ABC Toolkit* (the *Toolkit*) developed for prefabricated elements and modular systems in the R04 project is composed of the following:

1. ABC standard concepts;
2. ABC sample design calculations;
3. Recommended ABC design specifications (load and resistance factor design [LRFD]); and
4. Recommended ABC construction specifications (LRFD).

Standard concepts 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 and construction technologies as outlined below:

- 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:
    - — Decked steel stringer systems,
    - — Concrete deck bulb tees, and
    - — Concrete deck double tees;
- ABC bridge erection systems:
    - — Erection using cranes,
    - — Above-deck driven carriers, and
    - — Launched temporary truss bridges.

The development of detailed sample design calculations for use by future designers provides a step-by-step guidance on the overall structural design of the prefabricated bridge elements and systems. The sample design calculations pertain to the same standard bridge configurations for steel and concrete used in the ABC standard concepts. The intent was to provide sample design calculations that could be used in conjunction with the ABC standard concepts so that the practitioner new to ABC would get a comprehensive look at how ABC designs are carried out and translated into design drawings and details.

LRFD Bridge Design Specifications do not explicitly deal with the unique aspects of large-scale prefabrication such as element interconnection, system strength, and behavior of rapid deployment systems during construction. The work in this project also entailed the identification of any shortcomings in the current LRFD Bridge Design Specifications that may be limiting their use for ABC designs and making recommendations for addressing these limitations. Recommended LRFD specifications for ABC bridge design are also included in the *Toolkit*. The user should note that these are recommendations that have not been formally adopted by AASHTO.

Recommended LRFD construction specifications for prefabricated elements and modular systems include best practices that were compiled by the research team with the intent that they would be used in conjunction with the standard concepts for steel and concrete modular systems. As such, these specifications for rapid replacement focus heavily on means and methods requirements for rapid construction using prefabricated modular systems.

These tools have been included in the appendices to this report as follows:

- Appendix A, ABC Standard Concepts;
- Appendix B, ABC Sample Design Calculations;
- Appendix C, Recommended ABC Design Specifications; and
- Appendix D, Recommended ABC Construction Specifications.

OBJECTIVES FOR THE *ABC TOOLKIT*

An objective of this project was to identify impediments and obstacles to greater use of ABC and 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 the gradual lowering of costs and life-cycle cost savings, bridge owners are hesitant about using ABC techniques because of their higher initial costs and perceived risks. Another impediment to the rapid delivery of projects is the slow engineering process of custom engineering every solution. Rather than custom engineering every solution, 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.

Use of pre-engineered standards in bridge engineering is commonplace. Many states have decided to make best use of their program dollars by greatly standardizing design through development of pre-engineered systems, plans, etc., encompassing entire bridge systems including even the quantity takeoff for various standard configurations. These are guideline drawings that can reduce engineering calculations and details because the bulk of the calculations and details can be used for different site conditions. Use of pre-engineered bridge systems can lead to low in-place constructed costs and improved quality. A transition of the pre-engineered but stick-built systems to pre-engineered and prefabricated ABC systems is a worthy objective of this project.

Standardized designs geared for erection using conventional crane-based erection will allow repetitive use of modular superstructure systems.

Standardized designs geared for erection using conventional crane-based erection 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 to bring overall costs in line with cast-in-place construction. Where site conditions make crane-based erection difficult, overhead erection using ABC construction technologies provides an attractive alternative. Both these options are addressed in the ABC standards.

Typical ABC details for superstructure and substructure systems for routine bridges that are suitable for a range of spans are included in the *Toolkit*. Bridge designers are well versed in sizing beams and designing reinforcing steel for conventional construction for a specific site, and it would be appropriate for the engineer of record (EOR) to perform these functions for ABC projects as well. A single set of ABC designs for national use would also not be practical, as there are state-specific modifications to LRFD Bridge Design Specifications, including loads, design permit vehicle for Strength II, and performance criteria for service-limit states. The EOR, guided by the standard concepts and details and the accompanying set of ABC sample design calculations, will be able to easily complete an ABC design for a routine bridge replacement project. The standard concepts will need to be customized by the EOR to fit the specific site in terms of the bridge geometry, span configuration, member sizes, and foundations. The overall configurations of the modules, their assembly, connection details, tolerances, and finishing would remain unchanged from site to site. The ABC designs should also be reviewed for compliance with state-specific LRFD design criteria.

Repeated use of the same system will allow the continuous refinement of the ABC concept, thereby reducing risks and lowering costs. The standard concepts provide substantially complete details pertaining to the ABC aspects of the project. Much of the remaining work in preparing design plans is not particularly ABC related but more bridge- and site-specific customization. Specific instructions to designers are covered through general information sheets, plan notes, and instructions so that all the key design and construction issues in ABC projects are adequately addressed. The standard concepts, used in conjunction with the ABC sample design calculations and design specifications, will provide the “training wheels” that designers are looking for until they get comfortable with ABC.

USE OF THE *ABC TOOLKIT*

This *Toolkit* is not meant to be a comprehensive manual on all aspects of ABC. It is focused on the design and assembly of routine bridges using ABC techniques that would be of value to engineers, owners, and contractors new to ABC. It complements other publications on ABC, which should be consulted for more specific information on topics outside the scope of this *Toolkit*. The SHRP 2 R04 report is also a valuable reference for practical ABC technologies.

2 STANDARD CONCEPTS AND DETAILS FOR ABC

INTRODUCTION

Standard concepts have been developed for the most useful ABC technologies that can be deployed on a large scale in bridge replacement applications. The technologies incorporated into the standard concepts 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 these standards 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.

To get maximum advantage from the on-site construction speed possible 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 document provides standard concepts 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 as outlined in Chapter 1. Modular deck segments for concrete and steel bridge superstructures up to 130-ft spans that can be transported and erected in one piece provide the ideal building blocks for accelerated bridge construction. By standardizing the designs for these typical span ranges for routine or workhorse bridges, their availability through local or regional fabricators will be greatly increased. This will reduce lead time and cost.

Erection methods for large-scale prefabricated systems may not be well understood by those new to ABC. To assist the owners and engineers with their implementation of ABC, a goal was to develop a set of standard conceptual details demonstrating the possibilities and limits of ABC erection technologies. Where possible, crane-based erection would be the most cost effective. Guidelines are also provided for conventional erection of ABC systems using cranes. The erection concepts presented in the drawings are intended to assist the owner, the designer, and the contractor in selecting suitable erection equipment for the handling and assembly of prefabricated modular systems reflected in the ABC design standards.

Another task entailed the identification of any shortcomings in the current LRFD Bridge Design Specifications that may be limiting their use for ABC designs and construction 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. These recommendations have been included in the *Toolkit* for use in conjunction with the plans and sample design calculations.

DESIGN CONSIDERATIONS FOR ABC STANDARD CONCEPTS FOR MODULAR SYSTEMS

Although most agencies are aware of ABC, very few practice it on a large scale. 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 development of the *Toolkit* was aimed at making the use of ABC commonplace.

Findings from the outreach efforts of owner and contractor concerns and impediments to ABC implementation served as a starting point for the R04 team to explore ABC solutions, specifically design and construction concepts that could be further developed and refined for implementation and incorporated into the standard concepts:

- The largest impediment to increased use of ABC appears to be the higher initial costs. Reducing cost was a priority for most owners, as well as an overarching objective for this project.
- Owners have concerns about long-term durability of joints and connections in precast elements.
- 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.
- Lack of familiarity with ABC methods is a concern, particularly for designers. States are looking for design standards and other aids that could help them to design and implement ABC. The *Toolkit* is geared to fill this need.
- There is a need for ABC design criteria for structures and components 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. The majority of the contractors are not receptive to owners requiring a specific method of construction to be used in ABC contracts.

- Lack of access for equipment or the need for large staging areas unavailable in urban locations is a hindrance to large-scale prefabrication. The use of precast elements for substructure 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 could overcome access issues.
- Standardizing components is good 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 by states and prefabricators will be one goal.
- Contractors would be more willing to make equipment purchases if ABC became more standardized or industrialized and was based around certain methods of erection to speed the assembly. This increases the prospects for repetitive use of the same equipment.

The availability of ABC standards will promote the use of rapid renewal technologies, increase efficiency, and reduce costs over time.

The availability of ABC standards will promote the use of rapid renewal technologies, increase efficiency, and reduce costs over time. Sufficient repetition is needed to make precast components more economical and their construction more efficient and faster. To this end, standardized ABC designs applied over several projects provide a way to build this capability and improve overall efficiency within the local contractors and prefabricators.

Design considerations for the ABC standard concepts for modular systems developed in this project are as follows:

- ABC designs for routine bridges that can be used for most sites with minimal bridge-specific adjustments.
- Standardized designs for modular systems, which cover span ranges from 40 to 130 ft, 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.
- Substructure modules that can accommodate deep or shallow foundations based on site requirements.
- ABC designs and specifications that allow the contractor to self-perform the prefabrication of nonprestressed components.
- Prefabricated modules designed to be quickly assembled in the field with full moment connections. Joint details that allow rapid assembly in the field.
- 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.
- Ability 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.

- Use of high-performance materials: high-performance concrete or ultra-high-performance concrete (UHPC), high-performance steel, or A588 weathering steel.
- Segments that can be installed without the need for cross frames or diaphragms between adjacent segments. Improves the speed of construction and reduces costs. Use of diaphragms is optional based on owner preference.
- Control of camber for longer spans, which is 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.
- An integral wearing surface used in lieu of a field-applied overlay to expedite field construction.
- Prefabricated approach slabs to expedite the approach work. Explore methods such as flooded backfill to reduce time for backfilling operations.

Prefabricated components can provide a 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. Prefabricated elements can be used with or without overlays. Moving toward elements with overlays will allow larger vertical tolerances without the need for grinding.

Posttensioning is an acceptable alternative that is well established for ABC that the designers can find information on from other sources. This *Toolkit* focuses on more 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. In addition to flooded backfill to reduce backfilling operations, expanded polystyrene (EPS) geofoam can be used for rapid embankment placement. Refer to the FHWA ABC website for information on EPS geofoam.

DESIGN CONSIDERATIONS FOR ABC CONNECTIONS

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, the 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, because 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 is crucial to both the speed of construction and the overall durability and long-term maintenance of the final structure. The use of cast-in-place 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 the 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 in-filled with grout or match cast with epoxied joints if precise tolerances can be maintained. Posttensioning (PT) requires an additional step and complexity during on-site construction. Bridge owners could provide alternates for ABC connections including posttensioning with epoxy joints or closure pours that use rapid-set low-permeability concrete mixes based on performance specifications.

Full moment connections between modular substructure components were utilized in this project to emulate cast-in-place construction. The closure pours were constructed using self-consolidating concrete that can be completed quickly and results in the highest-quality durable connection. Self-consolidating concrete, also known as self-compacting concrete (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 attractive benefits while maintaining all of concrete's customary mechanical and durability characteristics.

Superstructure joints have perhaps been the most challenging aspect of ABC projects. Design considerations for connections between deck segments include the following:

- Full moment connections that are practical to build quickly.
- Durability at least equal to that of the precast deck.
- Joint details suitable for heavy truck traffic sites.
- Acceptable ride quality (similar to cast-in-place [CIP] decks).
- No requirement for the use of overlays for durability. An integral wearing surface consisting of an extra thickness of monolithic concrete slab may be provided. ABC systems with and without overlay can be advanced as effective ABC solutions. Using overlays will allow larger tolerances in fabrication. ABC systems shown in this *Toolkit* are designed to work without overlay; but the owners may choose to provide an overlay such as latex-modified concrete, polymer concrete, or asphalt with membrane overlays consistent with their long-term preservation practices. This may be done after the ABC period.

- No requirement for posttensioning in the field. It should be noted that PT is a viable option for ABC. The ABC concepts developed for this *Toolkit* have used joints without PT.
- Details that can accommodate slight differential camber between adjacent modules.
- Rapid strength gain so that the bridge can be opened to traffic quickly.

Investigations of superstructure joint types and material options have identified full moment connections using UHPC joints as the connection type for modular superstructure systems to satisfy the criteria for rapid constructability, structural behavior, and durability. The properties of UHPC make it possible to create small-width, full-depth, full moment closure pour connections between modular components. These connections may be significantly reduced in size as compared to conventional concrete construction practice and could include greatly simplified reinforcement designs. A lab testing program was carried out at Iowa State University in this project to further evaluate the performance of UHPC in ABC applications. The tests evaluated the constructability of UHPC joints within an ABC approach and assessed the strength and serviceability of transverse UHPC joints under simulated live loads. The Iowa ABC demonstration project completed in 2011 under this project was the first in the United States to use UHPC to provide a full, moment-resisting transverse joint in the superstructure at the piers. By late 2010, field-cast UHPC longitudinal connections between prefabricated bridge components had been implemented in at least nine bridges in Canada and two in the United States. The disadvantages of using UHPC include federal restrictions for sole source materials and the Buy America provision that will apply for the steel fibers.

ABC STANDARD CONCEPTS AND DETAILS

The use of modular systems is a proven method of accelerating bridge construction.

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 site conditions (alignment, span length, width) to the standard. The use of modular systems 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, etc. Contractors have also developed various proprietary systems and concepts to accelerate bridge construction, and ABC designs should be open to such innovations as well. The ABC solutions contained in the *Toolkit* should be enhanced with other technologies in the future as they evolve and become market ready for widespread implementation. The standard concepts are contained in Appendix A.

The details presented in the plans are intended to serve as general guidance in the development of designs suitable for accelerated bridge construction. The details should not be perceived as standards that are ready to be inserted into contract plans.

Overview of ABC Standard Concepts

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

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

The superstructure cross section and module widths have been shown for a typical two-lane bridge with shoulders as shown in Figure 2.1. Although the bridge cross section was chosen to represent a routine bridge structure (having the same width as the Iowa demonstration bridge), the design concepts, details, fabrication, and assembly are equally applicable to other bridge widths. The close stringer spacings were chosen to accommodate the module size and weight requirements for highway transport. Where shipping requirements for module widths are relaxed, or when the modules are fabricated adjacent to the site, wider girder spacings may be more economical. These designs can be applied to spans less than 40 ft as well.

Standardized designs for superstructure systems cover spans to 130 ft, as these are spans that can be transported and erected in one piece at many sites. In the span range up to 130 ft, the precast designs utilize 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 spliced girders can be used to simplify girder shipping because the girder can be fabricated in two or three pieces and spliced together in the field. Many of the details included on the standards can be used for these 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. Useful references for posttensioned spliced girder design would be the *Precast Bulb Tee Girder Manual* published by Utah DOT (2010b) and PCI's *State-of-the-Art of Precast/Prestressed Concrete Spliced Girder Bridges* (1992).

Substructure construction takes up a significant portion of the total on-site construction time. Reducing the duration to complete substructure work is critical for all rapid renewal projects. With this goal in mind, ABC standards are provided for abutments, wingwalls, and complete precast piers that are commonly used in routine bridge replacements. These substructure systems could be support on deep foundations or

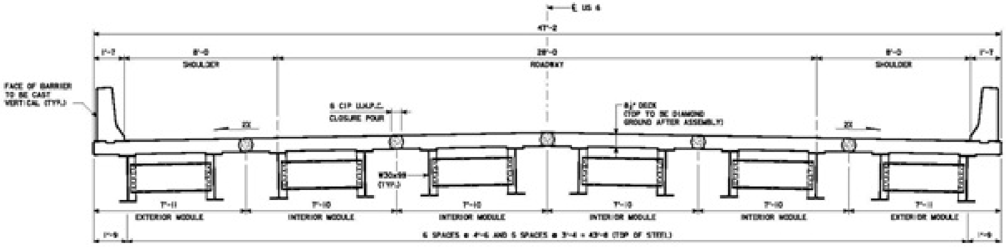


Figure 2.1. *Decked steel stringer system.*

on spread footings, depending on soil conditions at the site. All substructure modules have dimensions and weights suitable for highway transportation and erection using conventional equipment. It should be noted that the ABC standard concepts are intended for low to moderate seismic regions only.

Organization of ABC Standard Concepts

The systems presented in these ABC standard concepts consist of the following sheets that detail the ABC concepts noted:

1. Sheets G1 through G3:
   — General information sheets;
2. Sheets A1 through A12:
   — Semi-integral abutments,
   — Integral abutments,
   — Wingwalls,
   — Pile foundations and spread footings, and
   — Precast approach slabs;
3. Sheets P1 through P9:
   — Precast conventional pier,
   — Precast straddle bent, and
   — Drilled shaft and spread footing option;
4. Sheets S1 through S8:
   — Decked steel girder interior module,
   — Decked steel girder exterior module, and
   — Bearing and connection details;
5. Sheets C1 through C11:
   — Prestressed deck bulb-tee interior module,
   — Prestressed deck bulb-tee exterior module,
   — Prestressed double-tee module, and
   — Bearing and connection details.

General Information Sheets

Three sheets (Table 2.1) containing general information and instructions on the use of the ABC standard concepts 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 site-specific customization.

TABLE 2.1. GENERAL INFORMATION SHEETS

Sheet No.	Description
G1	Standard Prefabricated Substructure I
G2	Standard Prefabricated Substructure II
G3	Standard Prefabricated Superstructure

The general information sheets introduce the intent and scope of the standard concepts. They note that the intent of these ABC standard concepts is to provide information that applies to the design, detailing, fabrication, handling, and assembly of prefabricated components used in accelerated bridge construction, designed in accordance with the AASHTO *LRFD Bridge Design Specifications*.

The instructions note that the details presented should not be perceived as standards that are ready to be inserted into contract plans. Their implementation should warrant a complete design by the EOR in accordance with the requirements for the project site and state DOT standards and specifications. The standards were developed to comply with the AASHTO *LRFD Bridge Design Specifications*, 5th ed., and will need to consider subsequent editions and interims. The designer should 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.

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 a 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 nonseismic or low-seismic areas—Seismic Zones 1 and 2.

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 plans;

- Fabrication tolerances;
- Site casting requirements;
- Geometry control;
- Mechanical grouted splices;
- Element sizes; and
- General procedure for installation of modules.

ABC Standard Concepts for Abutments

Reducing the duration of the substructure work is critical for all rapid renewal projects. With this goal in mind, ABC standards are provided for abutments, wingwalls, and approach slabs that are commonly used in conventional bridge replacements. Details are based on a pile driving tolerance of ±3 in. The size of the corrugated metal pipe (CMP) void can be increased if difficult driving conditions are anticipated.

Reducing the duration of the substructure work is critical for all rapid renewal projects.

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 wingwalls are usually combined with the precast abutment barrel to form a complete system. Precast modular abutments have been constructed in several states. They consist of the following:

- Integral abutments; and
- Semi-integral abutments.

Integral connection of the superstructure to the substructure will be a preferred type for ABC construction due to its fast assembly. Since not all states employ the use of integral abutments, or foundation issues may limit their use, standards have been created for both integral and nonintegral 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 to reduce shipping weights, allowing 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 with low-shrinkage properties for enhanced long-term durability.

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. Continuity and elimination of joints, in addition to providing a more maintenance-free durable structure, can lead the way to more innovative and aesthetically pleasing solutions to bridge design. Providing a joint-free and maintenance-free bridge should be an important goal of rapid renewals. Use of integral or semi-integral abutments allows the joints to be moved beyond the bridge. Integral abutment bridges have proved 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 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 term “integral bridges” or “integral abutment bridges” is 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 provides the greatest flexibility and, hence, offers 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 is a variant of the integral abutment design. It is defined as a structure where 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.

Benefits of Using Jointless Construction for ABC

ABC seeks to reduce on-site construction time and 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, meaning 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 of jointless construction for ABC projects may be summarized as follows:

- Tolerance problems are reduced. The close tolerances required when utilizing expansion bearings and joints are eliminated with the use of integral abutments.
- Rapid construction. With integral abutments, only one row of vertical (not battered) piles is used and fewer piles are needed. The entire end diaphragm or backwall can be cast simultaneously and with less forming. Integral abutment bridges are more quickly erected than jointed bridges.
- 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, diminishes vehicular impact loads, and reduces 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. Furthermore, the use of integral abutments eliminates loss of girder support, the most common cause of damage to bridges in seismic events.

Precast Approach Slabs for ABC

Most bridge replacement projects require an approach slab at each end to prevent live load–induced compaction of the backfill, which eventually leads to a bump at the backwalls. Use of cast-in-place construction for the approach slabs could take up a significant portion of the total on-site construction time. Placing, finishing, and curing ground-supported slabs are slow operations, which under optimal conditions could take several days of on-site construction, even with rapid-set concrete mixes. It is therefore imperative that much of this construction be moved off-site so that the approach slabs are off the critical path for the ABC period. The ABC standard concepts (Table 2.2) show details for prefabricated approach slabs in easy-to-transport panels (size and weight) that are then connected in the field with a UHPC joint to form full moment connections. A precast sleeper slab is used as end support for the approach slabs and also a location for the expansion joint. By this approach the schedule that would have taken several days at best to complete using conventional methods is compressed into a single day—for the field assembly of the precast slabs and sleeper slabs and the casting of the UHPC joints. Posttensioning with epoxy joints or rapid-set concrete mixes may be used as alternatives for UHPC connections if the owners choose.

TABLE 2.2. ABC STANDARDS SHEETS FOR PRECAST ABUTMENTS AND APPROACH SLABS

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

ABC Standard Concepts for Piers

Precast Complete Piers

Precast complete piers are also composed of separate components fabricated off-site or off-line, shipped and assembled in the field into a complete bridge pier. Piers with single- 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 are usually 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. Table 2.3 lists standards sheets for precast piers.

Some states, specifically those in high seismic regions, employ the use of integral pier caps. However, the standards were developed only for nonintegral piers in this project, which is 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 using precast concrete. This connection reinforcement detail is often quite 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 nonintegral pier cap the superstructure and deck will be continuous and jointless over the piers. Also, nonintegral piers would be easier to reuse under a superstructure replacement.

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 practicable. Precast spread footings, where feasible, 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 are 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 utilized.

TABLE 2.3. ABC STANDARDS SHEETS FOR PRECAST PIERS

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)

ABC Standard Concepts for Steel Girder Superstructures

Modular Superstructure Systems

Modular superstructure systems composed of both steel and concrete girders have been included in the pre-engineered standards (see Table 2.4). 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 include the following steel and concrete systems:

Each modular system is expected to see a 75- to 100-year service life.

- Decked steel stringer system;
- Concrete deck bulb tees; and
- Deck double tees.

Decked Steel Stringer System

The decked steel stringer system is a proven concept shown to be quite economical and rapidly constructible. 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 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 made using conventional cranes. 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 the site and transportation requirements. It should be noted that steel products are subject to Buy America provisions on federally funded projects.

Many states are familiar with the "Inverset" system or variations of this system. 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 of this modular concept. As for the precast deck girders, the recommended connection will be the full moment connection for all the same reasons previously discussed. The deck will be cast with the steel girders supported at their

TABLE 2.4. ABC STANDARDS SHEETS FOR 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

permanent bearing locations. All formwork for the deck will be supported from the longitudinal girders similar to conventional deck construction (shored construction will not be assumed). This ensures that future deck replacements can be carried out without shoring. An integral wearing surface, typically 1½ to 2 in., can be built monolithic with the deck slab. In the future, the wearing surface concrete can be removed and replaced while preserving the structural deck slab.

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 UHPC joints as the preferred connection type for modular superstructure systems (steel and concrete) to satisfy the criteria for constructability, structural behavior, and durability as noted above. The term "ultra-high-performance concrete" refers to a class of advanced cementitious materials that display significantly enhanced material properties considered very beneficial to ABC. 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. Conventional materials and 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 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 to conventional concrete construction practice and could include greatly simplified reinforcement designs. Posttensioning with epoxy joints can be an alternate to UHPC if preferred by owner or when UHPC is not available.

The UHPC joint detail used had a 6-in. joint width with #5 U bars. UHPC has a strength gain of at least 10 ksi in 48 h when deck grinding can begin, where specified. It is suitable for Tier 2 projects using modular systems. (The R04 team has been informed by the supplier that new UHPC mixes are available for bridges requiring 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. New York State DOT has built a few bridges with this detail using straight bars. Use of straight bars in UHPC joints is planned for the second ABC demonstration project in New York under the R04 project. FHWA tests have validated the strength and serviceability of such UHPC joints for modular construction. FHWA publications on UHPC should be consulted for more information (Graybeal 2006, 2010, 2011, 2012).

Rapid-set concrete mixes may be used in such cases when traffic needs to be allowed in a few hours. This *Toolkit* focuses on the innovative approaches studied and developed under SHRP 2 R04; the *Toolkit* is not intended to be all encompassing for all ABC techniques, materials, and technologies available (other ABC resources should be consulted). Other ABC techniques and materials are mentioned in the *Toolkit* as potential alternatives, but will not be thoroughly discussed.

ABC Standard Concepts for Concrete Girder Superstructures

Modular superstructure systems for concrete girders have been included in the pre-engineered standards. 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 (see Table 2.5) include the following concrete systems:

- 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 over 50 years. There is wide acceptance among owners and contractors because they are easy and economical to build and to maintain. In most cases the girders are used with a CIP deck built on-site. For ABC applications the key difference lies in the fact that the girders will have an integral deck, thus eliminating the need for a CIP deck. The precast deck bulb tee (DBT) girders and double tee 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 bulb tee and double tee girders are proven systems, having been standardized for use by several states, such as Utah, Washington, and Idaho. The NEXT beam, a variation of the double tee, has been developed by PCI Northeast to serve the ABC market. The structure depth is also advantageous for sites with underclearance issues. We expect the deck girder costs will be very competitive when compared with the girder and CIP deck systems and may come in even lower for sites where there may be constraints

TABLE 2.5. ABC STANDARDS SHEETS FOR 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

to deck casting operations. 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 the site and transportation requirements.

Joints Between Modules

Similar to the decked steel modular systems, the concrete girder flanges will be joined using the UHPC joint detail, which has a 6-in. joint width with #5 U bars. 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 requires a thickening of the flanges for DBT girders from 6 in. to 9 in. Use of straight bars in the joints would be preferable for DBT bridges to minimize the flange thickness and shipping weights. 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.

Camber and Riding Surface Issues

LRFD Article 2.5.2.4, Rideability, requires the deck of the bridge to be designed to permit the smooth movement of traffic. 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. 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.

Differential camber in prefabricated elements could lead to fit-up and riding surface issues. To the traveling public, the smoothness of the riding surface is a significant riding comfort issue. It is important to develop an adequate means of controlling or removing the differential camber between the girders on-site. Although 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. An integral wearing surface may be an alternative to address differential camber issues. 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 are intended to achieve the required ride quality. Fabrication should be scheduled so that camber differences between adjacent deck sections are minimized at the time of erection. One option is diamond grinding decks with sacrificial cover to obtain the desired surface profile. If the differential camber is excessive, the contractors could apply dead load to the high beam to bring it within the connection tolerance. A leveling beam and jacks may also be used to equalize camber. 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.

Standard Conceptual Details for ABC Construction Technologies

The modular systems discussed in the previous sections may be erected using conventional construction techniques when site conditions permit. Given the proper project criteria, 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 upon 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 into the following two categories.

Bridge Movement Systems

Bridge movement systems include technologies in which the erection equipment is designed specifically to lift and transport large complete or partial segments of preassembled structures. SPMTs, lateral sliding, and launching would be good examples of these technologies. If the best option for a site is a complete preassembly of the structure that is then moved to its final position, there are several excellent published references on bridge movement technologies that can guide designers and owners (e.g., FHWA 2001, Utah DOT 2008). Movement of preassembled complete structures is a well-developed technology in the United States, with several specialty firms that provide this service nationally. Phase IV of this project involves designing a bridge replacement using a lateral slide and will develop design standards for such systems.

Bridge Erection Systems

Bridge erection systems include technologies in which the erection equipment is designed to deliver individual components of a proposed structure in a span-by-span process. These technologies are intended to be easily transportable, lightweight, and modular systems. The use of this type of equipment to deliver fully preassembled structures is not practical.

Because the ABC design standards developed in this research are for modular superstructure and substructure systems, the conceptual details for ABC construction technologies focus on bridge erection systems that are intended specifically to deliver and assemble modular systems. Rapid bridge renewal projects using modular systems can be categorized into one of the project types as follows:

1. ABC bridge designs built with “conventional” construction; or
2. ABC bridge designs built with ABC construction technologies.

The designer should ascertain whether its 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. The use of ABC construction technology compels the owners and consultants to consider the following variables:

1. Bridge project type;
2. Site and traffic constraints;
3. Available space (if any, where and condition) for construction staging areas;
4. Environment surrounding the project site; and
5. Project construction time period.

The development of the 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 series of questions for owners and designers, as shown on sheet CC2 (Appendix A), will 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 and is dependent on a number of factors, including the number of bridges to be built, convenience of crane support on the ground or by other means, span lengths, condition of the existing bridge to support crane loads, and site restrictions. General selection guidelines are included in the construction concept drawings and are shown below.

Rapid Bridge Demolition

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.

For rapid renewal applications the existing bridge must be demolished in a rapid process to allow the erection of the replacement structure.

Overview of ABC Construction Technologies

To assist the owners and engineers with their implementation of an ABC construction technology, a goal was to develop a set of standard conceptual details defining terminology and demonstrating the possibilities and limits of each ABC construction technology. Guidelines are also provided for conventional erection of ABC systems 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 also foster more constructible designs. Once a construction technology 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. 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. Planning and designing specific temporary structures and specific contractor operations are performed by the contractor and its engineer. 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 of up to 130 ft may be feasible in many 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 could be safely handled using conventional erection. Keeping the maximum weight less than 80 tons will generally allow greater ease of erection. Components up to 125 tons may be used where needed for longer spans or wider bridge widths after careful consideration of site conditions. Substructure units tend to 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

The above-deck carriers and launched temporary trusses are technologies that allow rapid replacement of structures where 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 such that traffic disruptions are 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 structure.

Current ADDCs exist in two forms and both perform a similar function. An ADDC rides over an existing bridge structure and then delivers components of the new bridge spans 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, 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 moving loads.

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 where 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 on both 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 using 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 of the existing bridge structure during the delivery of the bridge components.

This ABC construction technology would be applicable where an existing bridge or a set of twin bridges is to be widened and where 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 are as follows:

1. Minimize disruption to traffic and the environment at the lower level of the bridge project;
2. Can be used where conventional crane access is limited by site constraints;
3. Allow for faster rates of erection due to simplified delivery approach of components;
4. Minimize disruptions at the lower level of the project site because component delivery occurs at the end of the existing bridge;
5. Reduce need to work around existing traffic and reduce need to reduce lanes, shift lanes, and detour lanes, which in turn improves safety for both the workers and the traveling public; and
6. Can be used to deliver prefabricated, modularized components of ABC-type substructures and superstructures.

Launched Temporary Truss Bridge

A launched temporary truss bridge (LTTB) is designed to deliver individual components of a proposed structure in a span-by-span process with minimal disruption to activities and environment below structure.

Currently LTTBs exist in many forms; however, the basic principle of the technology is the same for each. The LTTBs are launched across or lifted over a span or set of spans and then, 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 based on sets of criteria that vary from project to project. The equipment could be quite specialized based on the needs of the project and could require extensive modifications from project to project based on changes in span lengths and component weights.

The idea behind a modified LTTB would be 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 across new spans to deliver components for a new bridge structure. This construction technology would be multifunctional, would be easily transportable on both 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 using 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. Once the modified LTTB had "bridged" the new span, it would be stabilized and supported at each pier and abutment substructure unit.

This ABC construction technology would be applicable where new bridge structures are to be erected and could also be applicable where an existing bridge or a set of twin bridges is to be widened.

Advantages of LTTBs are as follows:

1. Minimize disruption to traffic and the environment at the lower level of the bridge project;
2. Can be used where conventional crane access is limited by site constraints;
3. Minimize disruptions at the lower level of the project site because component delivery occurs at the end of the existing bridge;
4. Reduce need to work around existing traffic and reduce need to reduce lanes, shift lanes, and detour lanes, which in turn improves safety for both the workers and the traveling public;
5. Increase 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;
6. Allow work to proceed on multiple fronts (i.e., where multiple-span LTTBs are used, girders can be set while the next girder is delivered);
7. Allow for temporary loads to be introduced directly into piers, minimizing the need for falsework; and
8. Can be used to deliver prefabricated, modularized components of ABC-type substructures and superstructures.

Organization of Conceptual Details for ABC Construction Technologies

The erection concepts presented in the drawings are intended to assist the owner, the designer, and the contractor 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 2.6 and 2.7.

Erection concepts presented in the drawings group the bridges into short-span and long-span categories using the following criteria:

- Short span: Bridges with span lengths up to 70 ft and maximum prefabricated bridge module weight equal to 90,000 lb; and
- Long span: Bridges with span lengths greater than 70 ft up to 130 ft and maximum prefabricated bridge module weight equal to 250,000 lb.

TABLE 2.6. OVERVIEW OF DRAWINGS FOR ABC CONSTRUCTION TECHNOLOGIES

Drawing	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.

TABLE 2.7. 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

(continued on next page)

TABLE 2.7. ABC CONSTRUCTION TECHNOLOGIES SHEETS (CONTINUED)

Sheet No.	Description
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

3

SAMPLE DESIGN CALCULATIONS AND SPECIFICATIONS FOR ABC

INTRODUCTION

The challenge to future deployment of ABC systems lies partly in the area of being able 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 these ever-longer and more-slender beams will experience is that which occurs during shipping and handling prior to final erection.

At the current time, under a design–bid–build delivery method, the engineering 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 an innovative erection technique can be used. When design–build procurement is used, there is greater alignment between design and construction that 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 will be a worthwhile goal for this project and other ongoing projects related to rapid renewal.

Guidance has been developed for engineers alerting them of 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 accomplish this, the design community needs guidance or minimum analysis requirements for various erection methods for modular construction.

The design community needs guidance or minimum analysis requirements for various erection methods for modular construction.

RECOMMENDED LRFD DESIGN SPECIFICATIONS FOR ABC

Design criteria proposed for the ABC standards are in accordance with the AASHTO LRFD Bridge Design Specifications. The "Design Life—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 using large-scale prefabrication is not specifically covered in the LRFD Bridge Design Specifications.

The work in this project entailed the identification of any shortcomings in the current LRFD Bridge Design Specifications that may be limiting 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? Determine which loads associated with support conditions during fabrication may differ from 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, etc.
- Limit states and load factors during construction. What are the applicable limit states and load factors during construction, including limit state for checking of construction vehicles and equipment? Check critical stability effects as the component is fabricated, 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 lifting and erection stresses in prefabricated components. To what extent is cracking allowed in a prefabricated system 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? Need for temporary supports during erection.
- 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 achieve savings in weight and cost. Additional bracings for temporary support points during construction. The designer should consider the impact on live load distribution from any reductions in the use of cross frames or diaphragms.
- 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.

Implementing the recommended ABC design provisions into the existing sections of the LRFD Bridge Design Specifications would be difficult because the 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 C for the Recommended LRFD Design Specifications for ABC.

ABC SAMPLE DESIGN EXAMPLES

The sample design calculations will be instructive in highlighting the differences between CIP construction and modular prefabricated construction and the advantages of modular systems. Currently, economical design using CIP construction requires simplified fabrication and minimizing girder lines, with less emphasis on weight reduction. However, for ABC, shipping weights have to be minimized for economy and constructability. Shop labor is then easier to control quality. Shop-fabricated modular elements also increase the speed of construction. Stability of the shape must be ensured for all stages of construction per LRFD. Unlike CIP construction, girder stability during construction is less of an issue for predecked modular construction. This will allow more efficient designs of steel modular systems to minimize material and fabrication expense while ensuring adequate strength, stiffness, and stability.

Prefabricated modular steel bridges compete favorably with other materials when considering the greater use of shop labor in comparison to field labor, the speed at which they 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.

While important, in ABC design it is the careful determination of span arrangement and module dimensions for shipping and erection that can add significant savings.

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 important, in ABC design it is the careful determination of span arrangement and module dimensions for shipping and erection that can add significant savings. In fact, for CIP construction it is the cost of the substructure, particularly intermediate piers, for each design that usually determines the most economical span arrangement. Conventional rules of design used for economical span arrangement may not apply to modular systems, with cost of shipping and erection taking on additional significance in the overall economics of ABC projects. 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 sample design calculations developed in this project will serve as training tools to increase familiarity about ABC design issues and design criteria among engineers. Three sample design calculations are provided in Appendix B to illustrate the ABC design process for the following prefabricated modular systems:

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

The sample design calculations pertain to the same standard bridge configurations for steel and concrete used in the ABC standard concepts. The intent was to have sample design calculations that could be used in conjunction with the ABC standard concepts so that the practitioner will get a comprehensive view of how ABC designs are performed and translated into design drawings and details. The sample design calculations focus on the design checks for the modules for each stage of construction and the design of the connection details. Additional features of the sample design calculations include demonstration of any special LRFD loadings during construction and in the final condition; load combinations for design; stress and strength checks; deformations; and lifting and handling stresses. The sample design calculations have extensive documentation describing the design criteria, the design steps executed, the design philosophy adopted, and the design specifications checks performed. All sample design calculations 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. Two separate designs are illustrated for the precast pier—one for a straddle bent and one for a conventional pier. The examples are organized in a logical sequence to make them easy to follow. Each example has a table of contents at the beginning (as given below) to guide the reader and allow easier navigation. The sample design calculations are contained in Appendix B.

Sample Design 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 Design 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 Design Calculation 3a: Precast Pier Design for ABC (70-ft 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 Design Calculation 3b: Precast Pier Design for ABC (70-ft 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

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 standards concepts for steel and concrete modular systems developed in SHRP 2 R04. As such, these specifications for rapid replacement focus heavily on means and methods requirements for rapid construction using prefabricated modular systems. The specification also identifies responsibilities for design, construction, and inspection during an ABC project. It also identifies two phases of inspection—fabrication inspection and field inspection—that are the responsibility of the owner. Quality control and geometry control of components are identified as key parts of ABC construction. Adherence to prescribed tolerances and verification of fit-up in the yard are identified as the basis for successful field assembly within a tight ABC window. Requirements for various connection types commonly used in ABC, including UHPC joints, are defined so that they may be selected to fit the needs of specific projects and component types. Much of these provisions reflects a compilation of best practices for ABC construction that will need to be continually reviewed and updated as new information and lessons learned are accumulated from future ABC projects.

Implementing ABC concepts into the existing sections of the *LRFD Bridge Construction Specifications* would be difficult because 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*. A table of contents (as given below) is provided to guide the reader and allow easier navigation. See Appendix D for the Recommended LRFD Construction Specifications for ABC. A bridge owner using these specifications as a guide could develop its own special provisions for an ABC project.

Table of Contents

1 General
- 1.1 Description
- 1.2 Benefits

2 Responsibilities
- 2.1 Design
- 2.2 Construction
- 2.3 Inspection

3 Materials
- 3.1 Description
- 3.2 Concrete
- 3.3 Steel
- 3.4 Closure Pours
- 3.5 Grout
- 3.6 Couplers

4 Fabrication
- 4.1 Qualifications of the Fabricator
- 4.2 Fabrication Plants
- 4.3 Fabrication Requirements
- 4.4 Fabrication Tolerances
- 4.5 Yard Assembly

5 Submittals
- 5.1 Shop Drawings
- 5.2 Assembly Plan

6 Quality Assurance

7 Handling, Storing, and Transportation

8 Geometry Control
- 8.1 General
- 8.2 Camber and Deflection
- 8.3 Equalizing Differential Camber
- 8.4 Finishing of Bridge Deck
  - 8.4.1 Diamond Grind Bridge Deck
  - 8.4.2 Saw Cut Groove Texture Finish

SELECTED BIBLIOGRAPHY

AASHTO. 2010. *AASHTO LRFD Bridge Design Specifications*. 5th ed. Washington, D.C.

FHWA. 2001. *Manual on Use of Self-Propelled Modular Transporters to Remove and Replace Bridges*. U.S. Department of Transportation, Washington, D.C.

FHWA. 2005. *Framework for Prefabricated Bridge Elements and Systems (PBES) Decision Making*. U.S. Department of Transportation, Washington, D.C.

FHWA. 2009. *Connection Details for Prefabricated Bridge Elements and Systems*. U.S. Department of Transportation, Washington, D.C.

FHWA. 2010. *Accelerated Bridge Construction (ABC) Decision Making and Economic Modeling Tool*. Quarterly Reports, Transportation Pooled Fund Project TPF-5(221). U.S. Department of Transportation, Washington, D.C.

FHWA. 2011. *Accelerated Bridge Construction Manual*. Publication HIF-12-013. U.S. Department of Transportation, Washington, D.C.

Graybeal, B. 2006. *Material Property Characterization of Ultra-High Performance Concrete*. Report FHWA-HRT-06-103. FHWA, Washington, D.C.

Graybeal, B. 2010. *Behavior of Field-Cast Ultra-High Performance Concrete Bridge Deck Connections Under Cyclic and Static Structural Loading*. NTIS Report PB2011-101995. National Technical Information Service, Springfield, Va.

Graybeal, B. 2011. *Ultra-High Performance Concrete*. Report FHWA-HRT-11-038. FHWA, Washington, D.C.

Graybeal, B. 2012. *Construction of Field-Cast Ultra-High Performance Concrete Connections*. Report FHWA-HRT-12-038. FHWA, Washington, D.C.

Precast/Prestressed Concrete Institute. 1992. *State-of-the-Art of Precast/Prestressed Concrete Spliced Girder Bridges*. PCI Committee on Bridges, PCI, Chicago, Ill.

Utah DOT. 2008. *Manual for the Moving of Utah Bridges Using Self Propelled Modular Transporters (SPMTs)*. Utah Department of Transportation, Salt Lake City.

Utah DOT. 2010a. *Precast Approach Slab Manual*. Utah Department of Transportation, Salt Lake City.

Utah DOT. 2010b. *Precast Bulb Tee Girder Manual*. Utah Department of Transportation, Salt Lake City.

Utah DOT. 2010c. *Precast Substructure Elements Manual*. Utah Department of Transportation, Salt Lake City.

ABC STANDARD CONCEPTS

STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

STANDARD CONCEPTS FOR ABC MODULAR SYSTEMS

INDEX OF DRAWINGS

ABC DESIGN CONCEPTS

GENERAL INFORMATION

SHEET NO.	DESCRIPTION
G1	STANDARD PREFABRICATED SUBSTRUCTURE I
G2	STANDARD PREFABRICATED SUBSTRUCTURE II
G3	STANDARD PREFABRICATED SUPERSTRUCTURE

ABUTMENT

SHEET NO.	DESCRIPTION
A1	GENERAL NOTES AND INDEX OF DRAWINGS
A2	SEMI-INTEGRAL ABUTMENT PLAN & ELEVATION
A3	ABUTMENT REINFORCEMENT DETAILS
A4	WINGWALL REINFORCEMENT DETAILS 1
A5	WINGWALL REINFORCEMENT DETAILS 2
A6	SEMI-INTEGRAL ABUTMENT SECTION
A7	INTEGRAL PLAN & 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 SECTION

PIER

SHEET NO.	DESCRIPTION
P1	GENERAL NOTES
P2	PRECAST PIER ELEV. & DETAILS (CONVENTIONAL PIER)
P3	PRECAST PIER CAP DETAILS (CONVENTIONAL PIER)
P4	PRECAST COLUMN DETAILS (CONVENTIONAL PIER)
P5	PRECAST PIER ELEV. & 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)

STEEL GIRDER SUPERSTRUCTURE

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

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

HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012

SHEET NUMBER 01

STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

STANDARD CONCEPTS FOR ABC MODULAR SYSTEMS

INDEX OF DRAWINGS

ABC CONSTRUCTION CONCEPTS

HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012

SHEET NUMBER 02

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 LOW TO MODERATE SEISMIC REGIONS.

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.

SKEW:

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
OWNER SPECIFIED DESIGN PERMIT LOADS ARE NOT INCLUDED IN THESE DESIGNS
FUTURE WEARING SURFACE = 25 PSF
LIVE LOAD SURCHARGE PER LRFD. APPROACH SLAB SUPPORT LENGTH = 20FT

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 PREFABRICATED COMPONENTS 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.

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 80 TONS, HOWEVER HIGHER WEIGHTS MAY BE ACCEPTABLE FOR CERTAIN SUBSTRUCTURE ELEMENTS.

LIGHTWEIGHT CONCRETE

LIGHTWEIGHT HIGH PERFORMANCE CONCRETE (LWHPC) IS A PROVEN TECHNOLOGY THAT CAN BE USED TO REDUCE THE WEIGHT OF PRECAST ELEMENTS.

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

STANDARD PREFABRICATED SUBSTRUCTURE
GENERAL INFORMATION SHEET 1

HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012

SHEET NUMBER G1

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. VERIFY WITH THE COUPLER MANUFACTURER THE MINIMUM GROUT COMPRESSIVE STRENGTH NEEDED TO ENSURE 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 11

HNTB
SEA / ISU / GENESIS

OCTOBER 2011
REV: SEPTEMBER 2012

SHEET NUMBER G2

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, A DOUBLE TEE GIRDER AND A COMPOSITE DECKED STEEL STRINGER MODULE. ALL SYSTEMS INCLUDE A FULL DEPTH FLANGE THAT SERVES AS THE RIDING SURFACE.

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
OWNER-SPECIFIED DESIGN PERMIT VEHICLES ARE NOT INCLUDED IN THESE DESIGNS
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.

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 STRIPPED DURING FABRICATION, USE A HIGH-PRESSURE STREAM OF WATER TO ROUGHEN THE BEVELED FACES AT ALL CLOSURE JOINTS TO AN AMPLITUDE OF ¼ 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 IN ACCORDANCE WITH SUPPLIER RECOMMENDATIONS.

DIAMOND GRIND BRIDGE DECK:

AN ADDITIONAL THICKNESS OF ½ INCH HAS BEEN INCORPORATED IN THE DECK TO PERMIT CORRECTION OF THE DECK PROFILE BY DIAMOND GRINDING. DIAMOND GRINDING CAN BE ELIMINATED IF AN OVERLAY IS USED.

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.

LIGHTWEIGHT CONCRETE

LIGHTWEIGHT HIGH PERFORMANCE CONCRETE (LWHPC) IS A PROVEN TECHNOLOGY THAT CAN BE USED TO REDUCE THE WEIGHT OF PRECAST ELEMENTS

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

STANDARD PREFABRICATED GIRDER SUPERSTRUCTURE

GENERAL INFORMATION SHEET III

HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012

SHEET NUMBER G3

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. (AASHTO LRFD CONSTRUCTION SPECIFICATIONS 8.13) 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

REINFORCING STEEL: GRADE 60

CONCRETE 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.

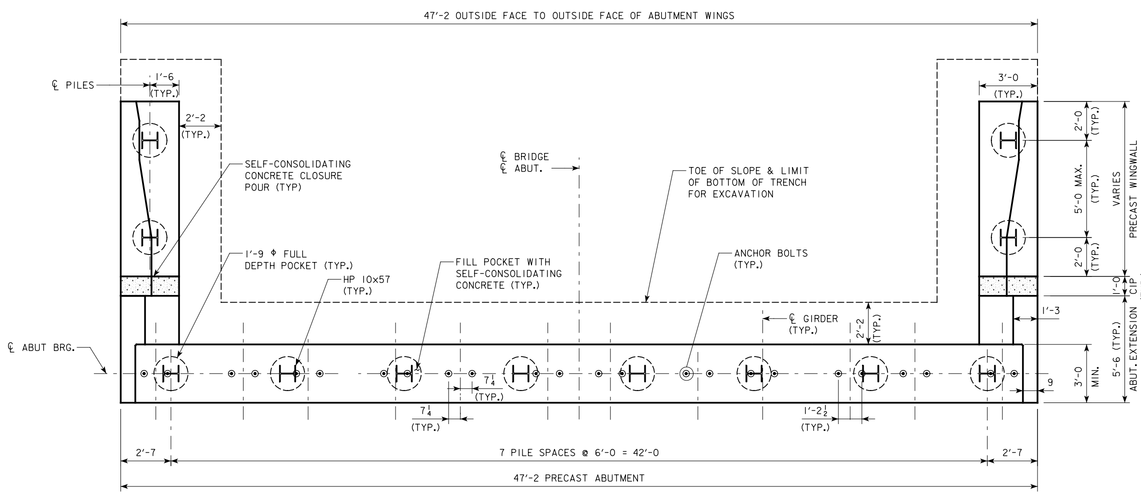
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

STANDARD PREFABRICATED
CONCRETE SUBSTRUCTURE

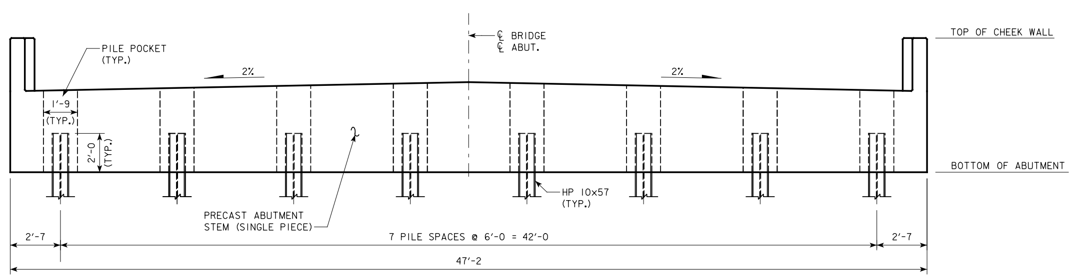
GENERAL NOTES 1

HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012

SHEET NUMBER A1



ABUTMENT PLAN AND PILE LAYOUT



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. 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.
8. ANCHOR BOLTS MAY BE OMITTED BASED ON OWNER PRACTICES.

FOR INFORMATIONAL PURPOSES ONLY

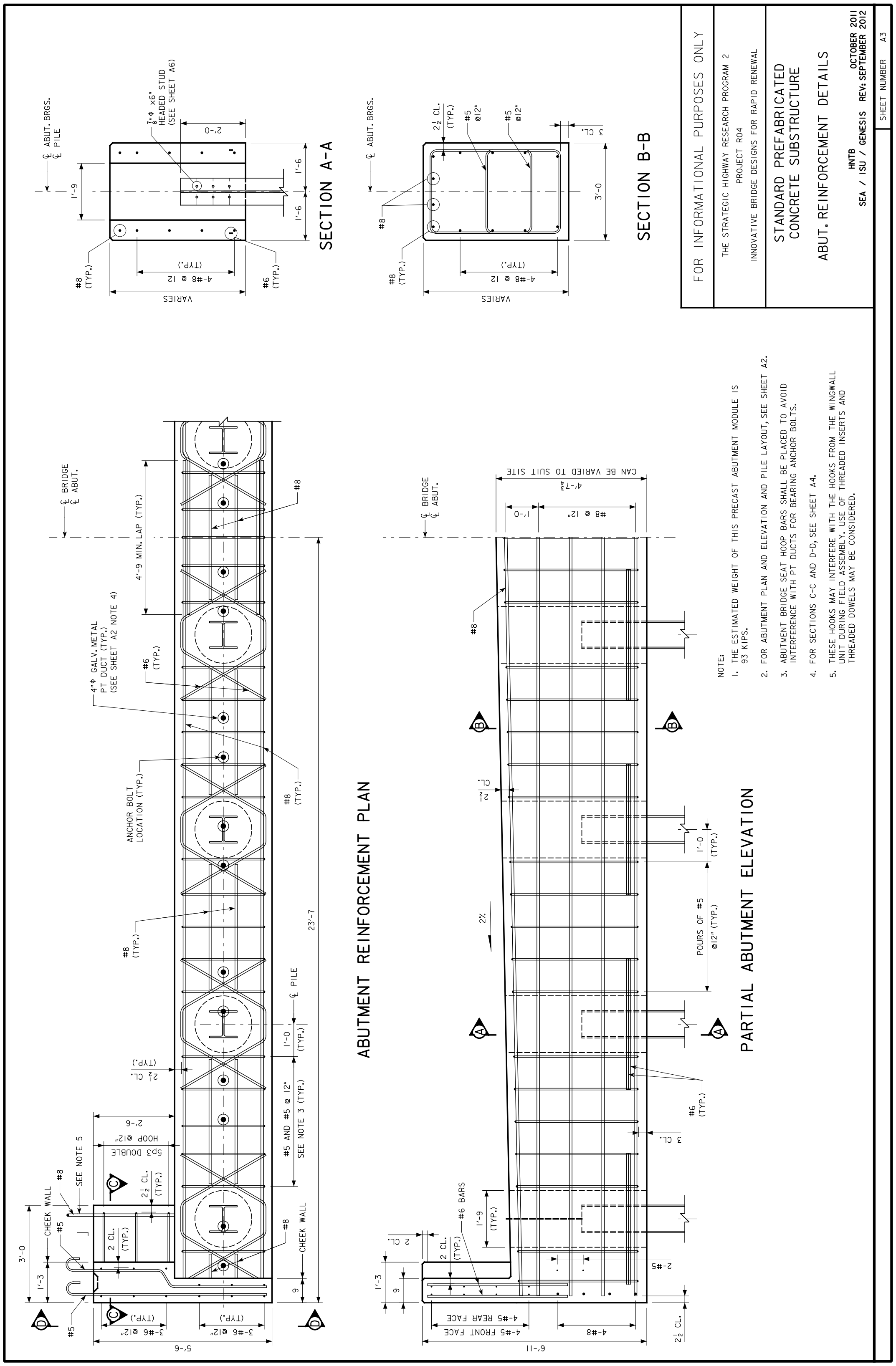
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

STANDARD PREFABRICATED CONCRETE SUBSTRUCTURE

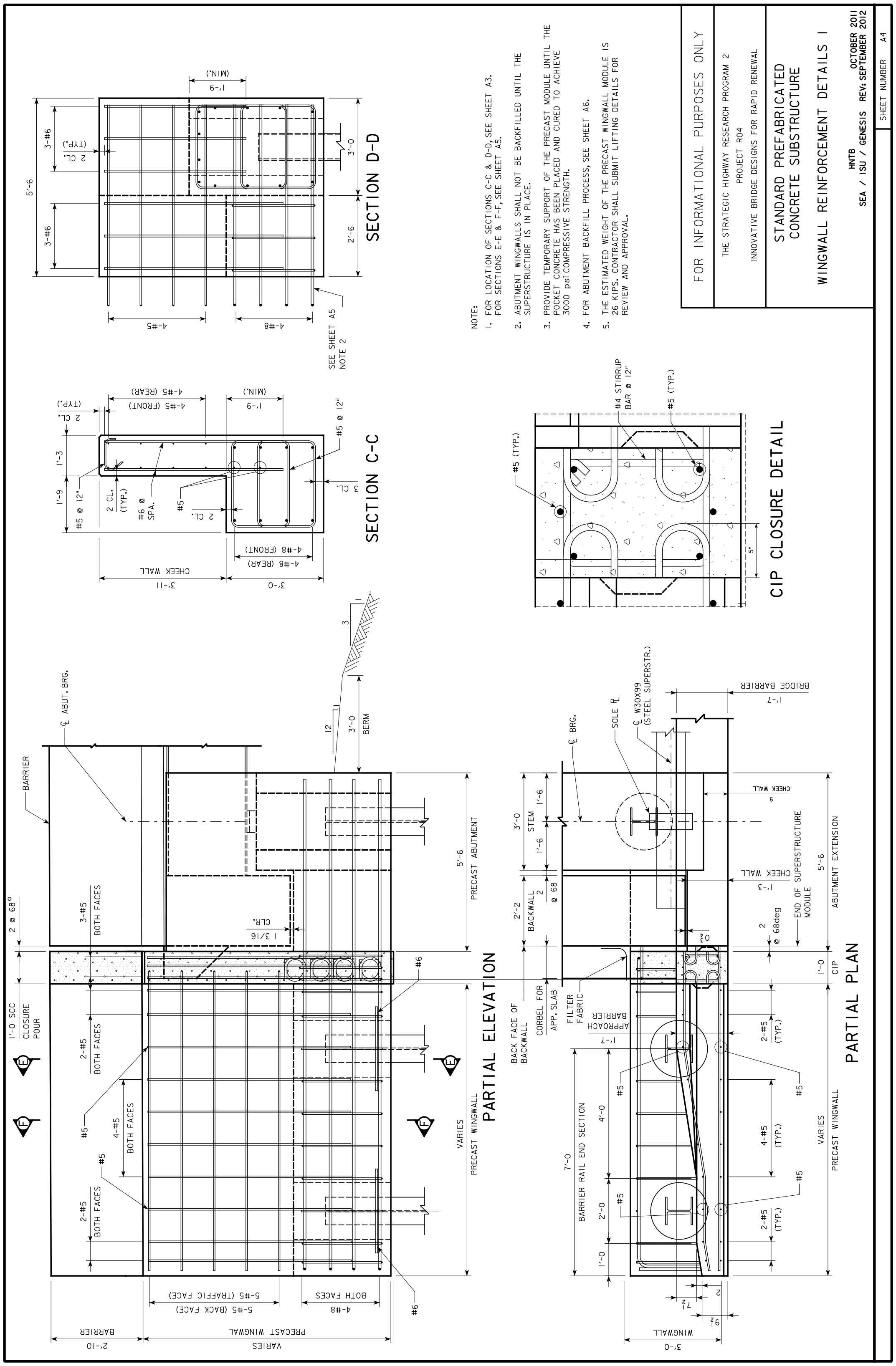
SEMI-INTEGRAL ABUTMENT PLAN & ELEVATION

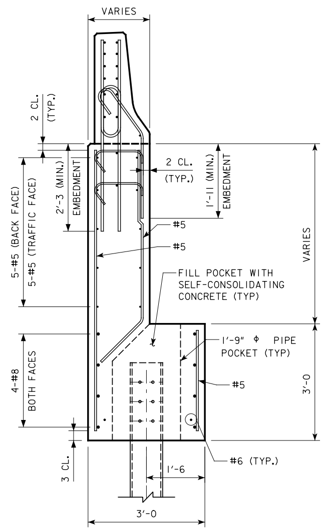
HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012

SHEET NUMBER A2

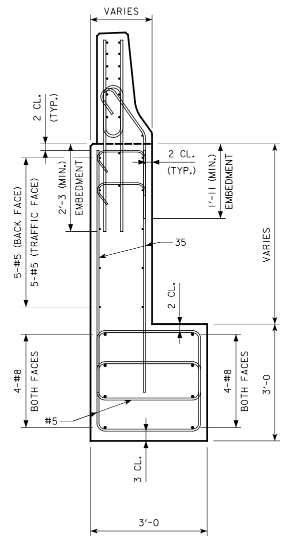


FOR INFORMATIONAL PURPOSES ONLY
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL
STANDARD PREFABRICATED CONCRETE SUBSTRUCTURE
ABUT. REINFORCEMENT DETAILS
HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012
SHEET NUMBER A3
ABUTMENT REINFORCEMENT PLAN
PARTIAL ABUTMENT ELEVATION
SECTION A-A
SECTION B-B
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.
4"Φ GALV. METAL PT DUCT (TYP.) (SEE SHEET A2 NOTE 4)
ANCHOR BOLT LOCATION (TYP.)
4'-9 MIN. LAP (TYP.)
#5 AND #5 @ 12" SEE NOTE 3 (TYP.)
5p3 DOUBLE HOOP @12"
CHEEK WALL
SEE NOTE 5
CAN BE VARIED TO SUIT SITE
POURS OF #5 @12" (TYP.)
4-#5 REAR FACE
4-#5 FRONT FACE
7/8"Φ x6" HEADED STUD (SEE SHEET A6)
℄ ABUT. BRGS.
℄ PILE
℄ BRIDGE ABUT.
23'-7

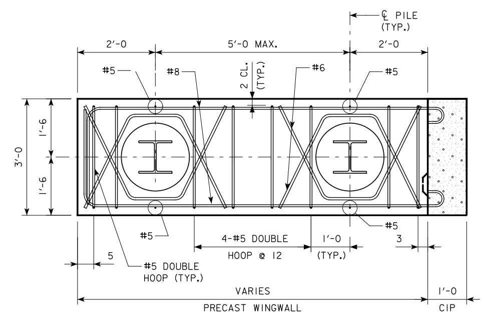

FOR INFORMATIONAL PURPOSES ONLY
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL
STANDARD PREFABRICATED CONCRETE SUBSTRUCTURE
WINGWALL REINFORCEMENT DETAILS I
HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012
SHEET NUMBER A4
NOTE:
1. FOR LOCATION 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.
SECTION D-D
SECTION C-C
CIP CLOSURE DETAIL
PARTIAL ELEVATION
PARTIAL PLAN



SECTION E-E



SECTION F-F



WINGWALL REINFORCEMENT PLAN

NOTE:

1. FOR LOCATION OF SECTION E-E & F-F, SEE SHEET A4.

FOR INFORMATIONAL PURPOSES ONLY

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

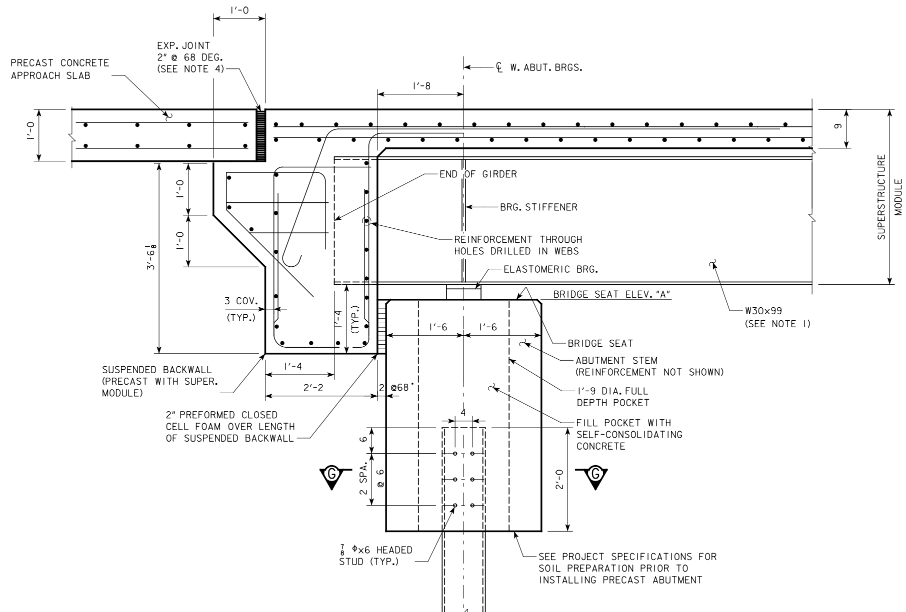
STANDARD PREFABRICATED
CONCRETE SUBSTRUCTURE

WINGWALL REINFORCEMENT DETAILS 2

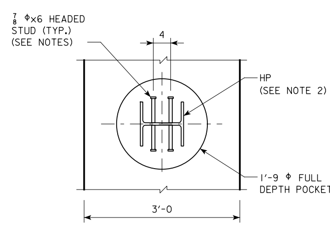
HNTB
SEA / ISU / GENESIS

OCTOBER 2011
REV: SEPTEMBER 2012

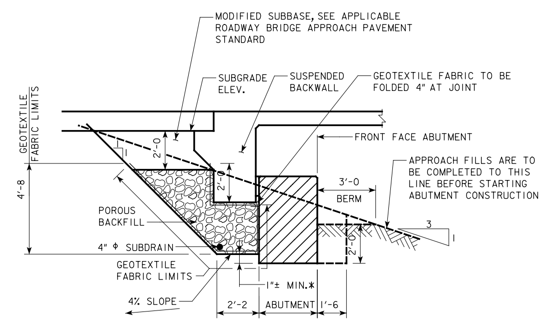
SHEET NUMBER A5



TYPICAL SEMI-INTEGRAL ABUTMENT SECTION WITH STEEL SUPERSTRUCTURE
(SUSPENDED BACKWALL)



SECTION G-G



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.

FOR INFORMATIONAL PURPOSES ONLY

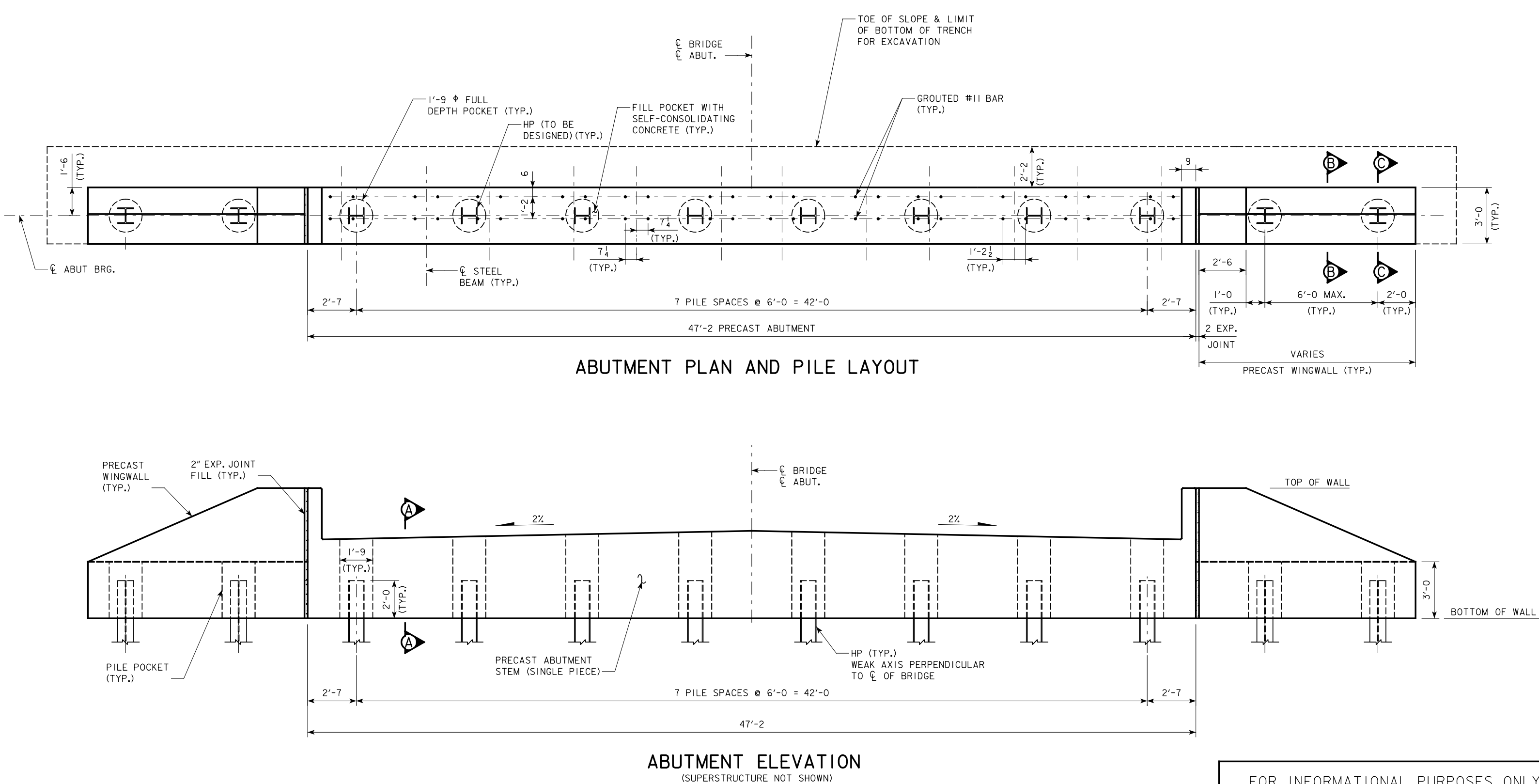
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

STANDARD PREFABRICATED
CONCRETE SUBSTRUCTURE

SEMI-INTEGRAL
ABUTMENT SECTION

HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012

SHEET NUMBER A6



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.

FOR INFORMATIONAL PURPOSES ONLY

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

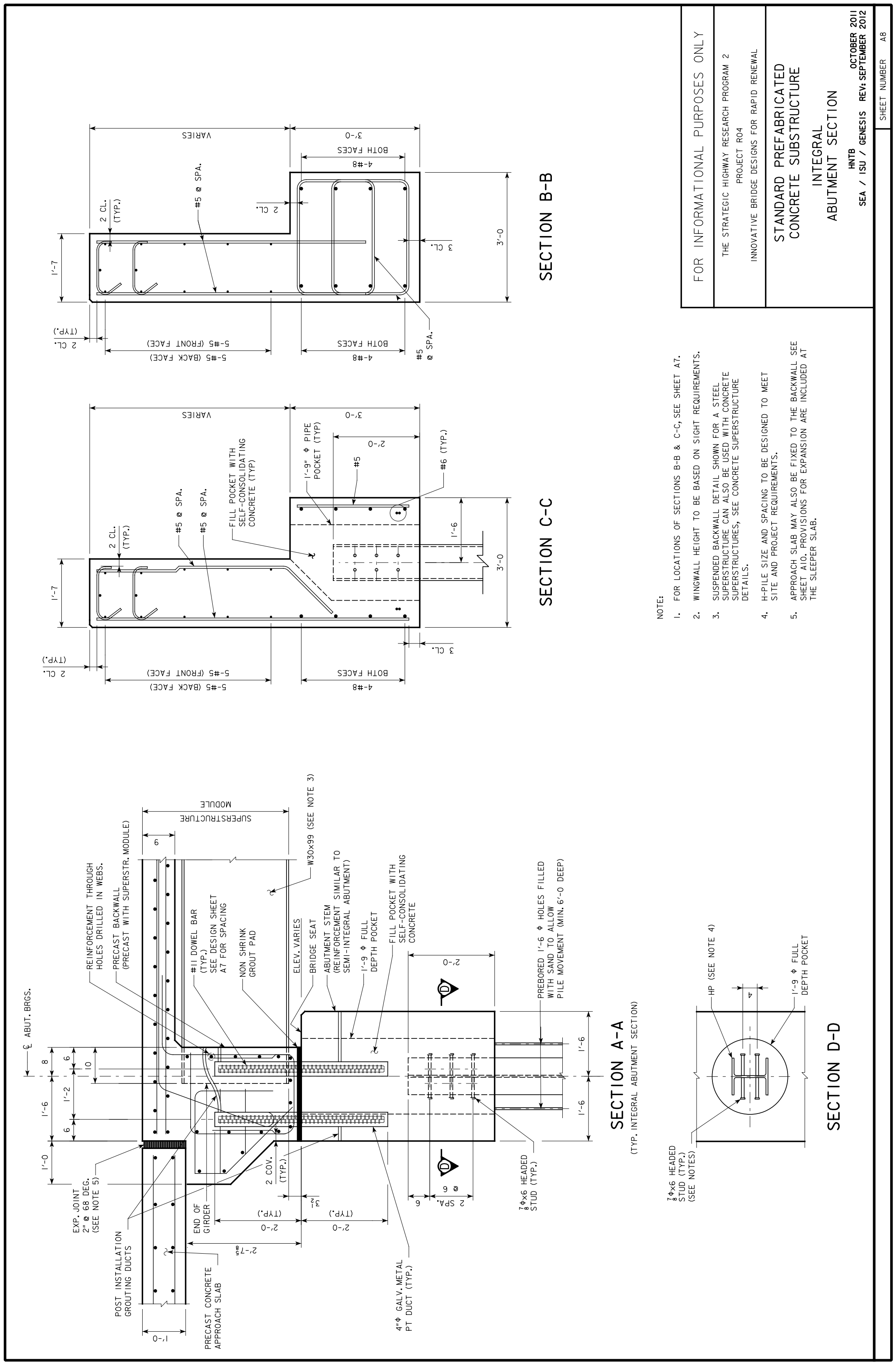
STANDARD PREFABRICATED CONCRETE SUBSTRUCTURE

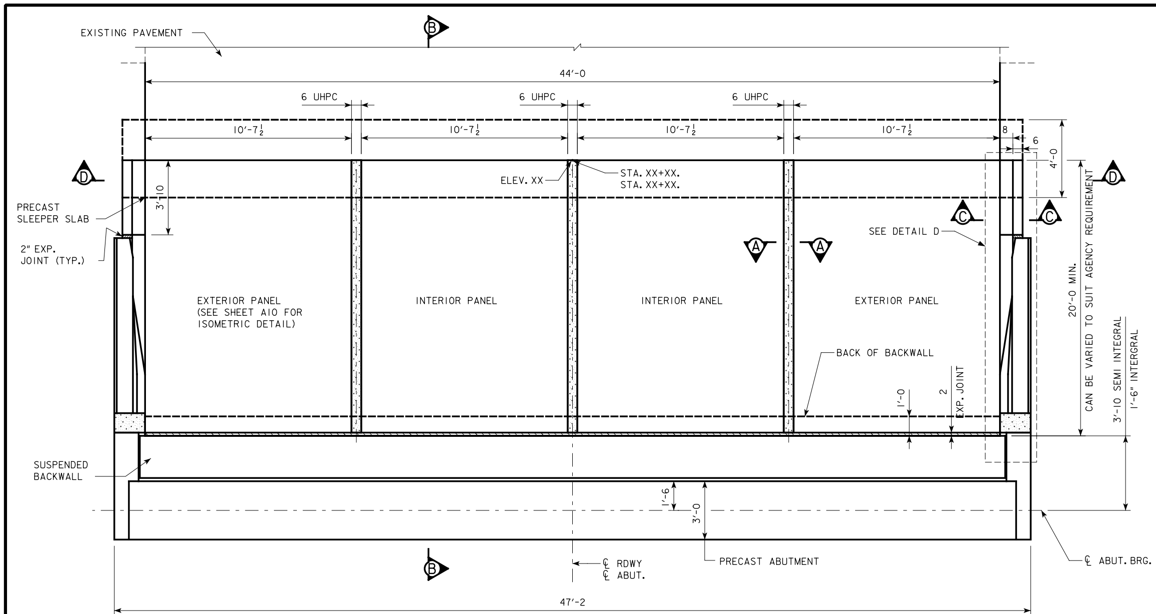
INTEGRAL ABUTMENT PLAN & ELEVATION

HNTB
SEA / ISU / GENESIS

OCTOBER 2011
REV: SEPTEMBER 2012

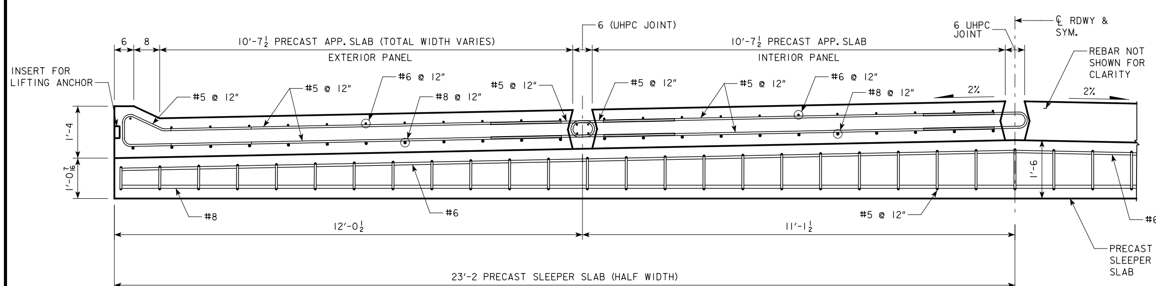
SHEET NUMBER A7


FOR INFORMATIONAL PURPOSES ONLY
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL
STANDARD PREFABRICATED CONCRETE SUBSTRUCTURE
INTEGRAL ABUTMENT SECTION
HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012
SHEET NUMBER A8
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.
SECTION A-A
(TYP. INTEGRAL ABUTMENT SECTION)
SECTION B-B
SECTION C-C
SECTION D-D
℄ ABUT. BRGS.
REINFORCEMENT THROUGH HOLES DRILLED IN WEBS.
PRECAST BACKWALL (PRECAST WITH SUPERSTR. MODULE)
#11 DOWEL BAR (TYP.) SEE DESIGN SHEET A7 FOR SPACING
NON SHRINK GROUT PAD
ELEV. VARIES
BRIDGE SEAT
ABUTMENT STEM (REINFORCEMENT SIMILAR TO SEMI-INTEGRAL ABUTMENT)
1'-9 Φ FULL DEPTH POCKET
FILL POCKET WITH SELF-CONSOLIDATING CONCRETE
W30x99 (SEE NOTE 3)
SUPERSTRUCTURE MODULE
PREBORED 1'-6 Φ HOLES FILLED WITH SAND TO ALLOW PILE MOVEMENT (MIN. 6'-0 DEEP)
7/8 Φx6 HEADED STUD (TYP.) (SEE NOTES)
HP (SEE NOTE 4)
EXP. JOINT 2" @ 68 DEG. (SEE NOTE 5)
POST INSTALLATION GROUTING DUCTS
PRECAST CONCRETE APPROACH SLAB
END OF GIRDER
2 COV. (TYP.)
4" Φ GALV. METAL PT DUCT (TYP.)
1'-9" Φ PIPE POCKET (TYP)
FILL POCKET WITH SELF-CONSOLIDATING CONCRETE (TYP)
#5 @ SPA.
#6 (TYP.)
5-#5 (FRONT FACE)
5-#5 (BACK FACE)
4-#8 BOTH FACES
2 CL. (TYP.)
3 CL.
VARIES
3'-0
1'-7



APPROACH SLAB PLAN

(SEMI-INTEGRAL ABUTMENT SHOWN, INTEGRAL ABUTMENT SIMILAR)



HALF-SECTION D-D

TOLERANCES FOR APPROACH SLABS

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
VARIATION FROM SPECIFIED CAMBER	+/- 1/8" PER 10 FEET +/- 1/4" 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.

FOR INFORMATIONAL PURPOSES ONLY

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

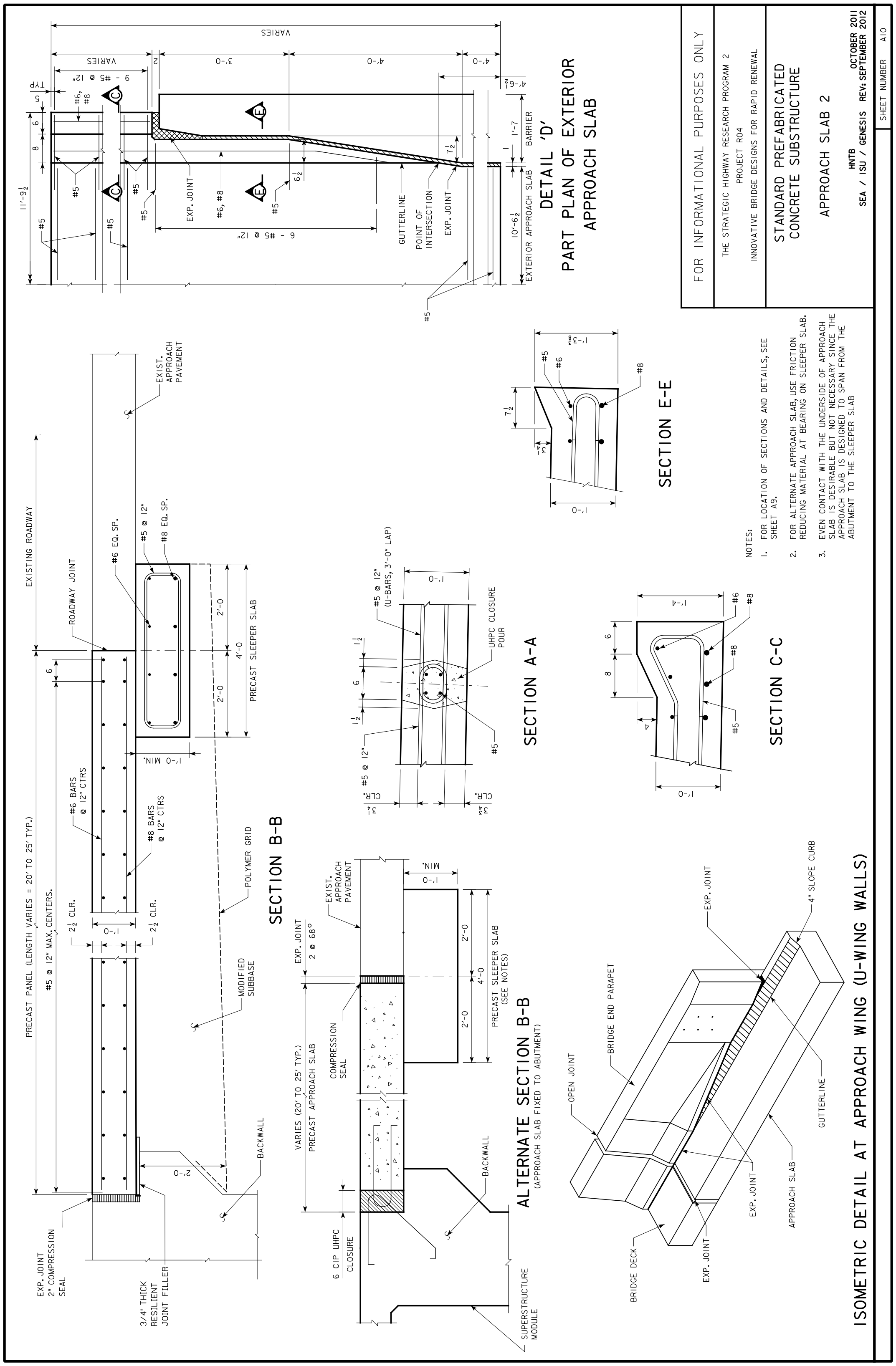
STANDARD PREFABRICATED
CONCRETE SUBSTRUCTURE

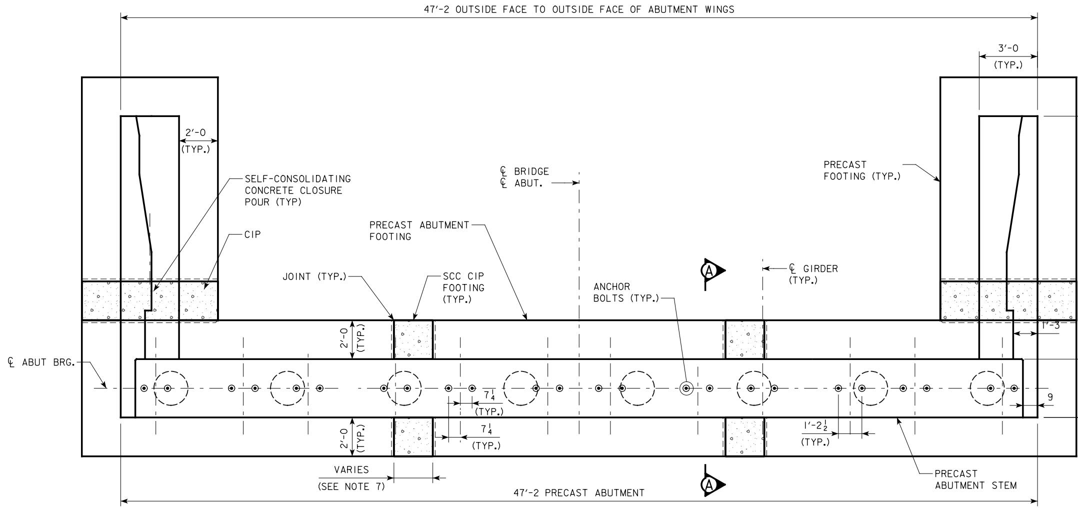
APPROACH SLAB 1

HNTB
SEA / ISU / GENESIS

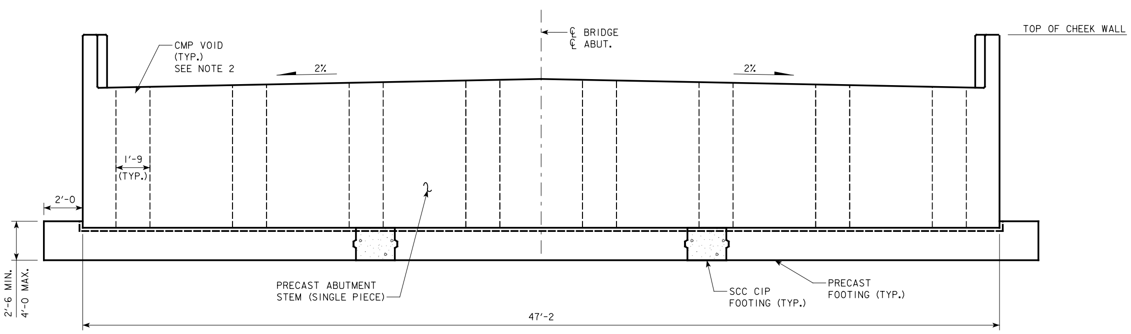
OCTOBER 2011
REV: SEPTEMBER 2012

SHEET NUMBER A9


DETAIL 'D'
PART PLAN OF EXTERIOR APPROACH SLAB
SECTION E-E
SECTION C-C
SECTION A-A
SECTION B-B
ALTERNATE SECTION B-B
(APPROACH SLAB FIXED TO ABUTMENT)
ISOMETRIC DETAIL AT APPROACH WING (U-WING WALLS)
NOTES:
1. FOR LOCATION OF SECTIONS AND DETAILS, SEE SHEET A9.
2. FOR ALTERNATE APPROACH SLAB, USE FRICTION REDUCING MATERIAL AT BEARING ON SLEEPER SLAB.
3. EVEN CONTACT WITH THE UNDERSIDE OF APPROACH SLAB IS DESIRABLE BUT NOT NECESSARY SINCE THE APPROACH SLAB IS DESIGNED TO SPAN FROM THE ABUTMENT TO THE SLEEPER SLAB
FOR INFORMATIONAL PURPOSES ONLY
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL
STANDARD PREFABRICATED CONCRETE SUBSTRUCTURE
APPROACH SLAB 2
HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012
SHEET NUMBER
A10



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.

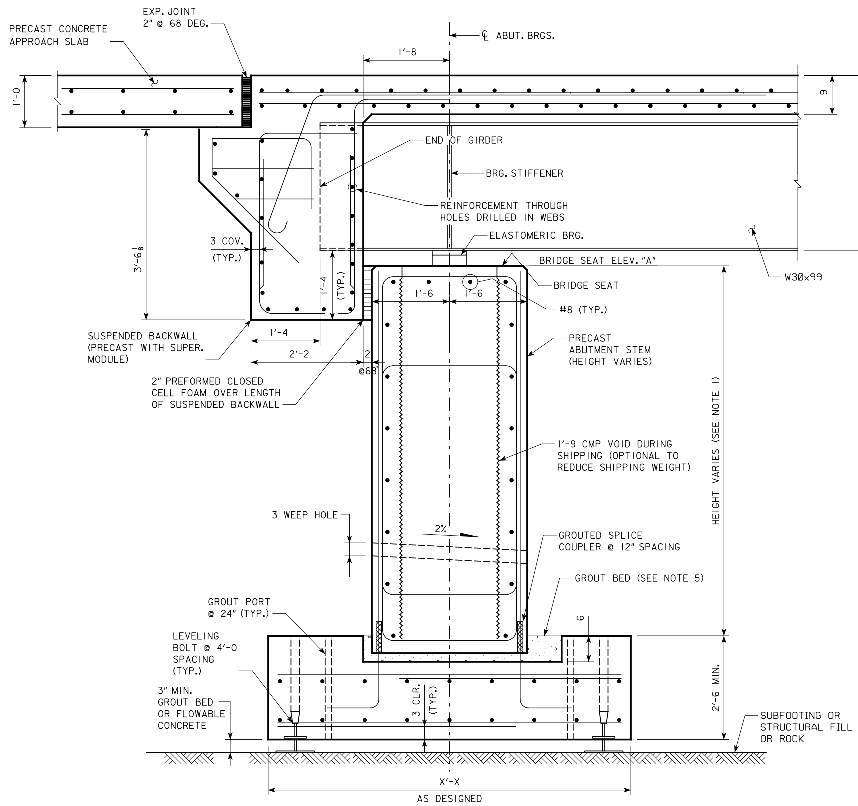
FOR INFORMATIONAL PURPOSES ONLY

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

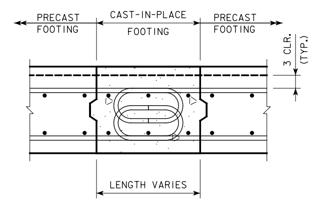
HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012

SHEET NUMBER A11



TYPICAL SEMI-INTEGRAL ABUTMENT SECTION WITH STEEL SUPERSTRUCTURE

(SUSPENDED BACKWALL)



CIP FOOTING SECTION DETAIL

NOTES:

1. THIS IS A CONCEPT DRAWING FOR AN ABUTMENT OF ANY HEIGHT. 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.
5. PROVIDE STEEL SHIM PACK AT SPACING NOT TO EXCEED 4 FT.

FOR INFORMATIONAL PURPOSES ONLY

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

STANDARD PREFABRICATED CONCRETE SUBSTRUCTURE
SPREAD FOOTING OPTION SECTION

HNTB
SEA / ISU / GENESIS

OCTOBER 2011
REV: SEPTEMBER 2012

SHEET NUMBER A12

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 $\frac{1}{2}$ 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 LOW TO MODERATE SEISMIC REGIONS.

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

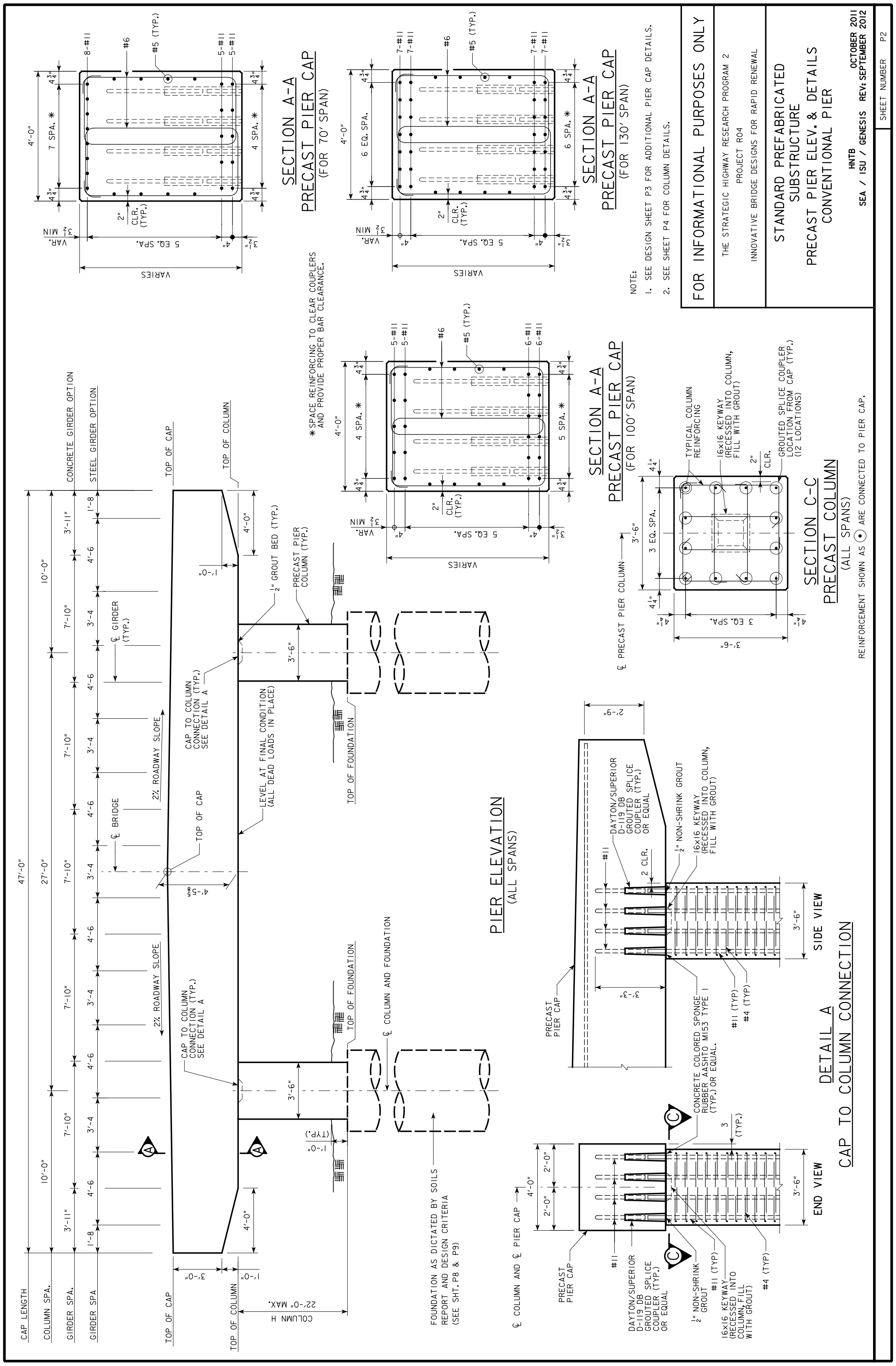
ALTERNATIVELY, THE DESIGNER MAY CHOOSE A PRESTRESSED/POST-TENSIONED DESIGN FOR THE PIER CAP TO ACHIEVE A SECTION OF REDUCED SIZE AND WIEGHT WHERE SUCH CONSIDERATIONS ARE DEEMED CRITICAL FOR CONSTRUCTABILITY.

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

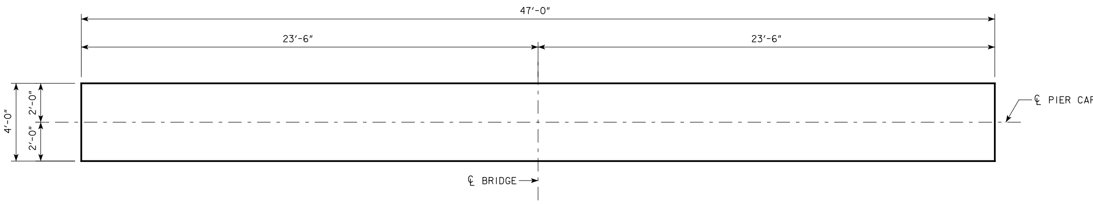
STANDARD PREFABRICATED
SUBSTRUCTURE
GENERAL NOTES

HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012

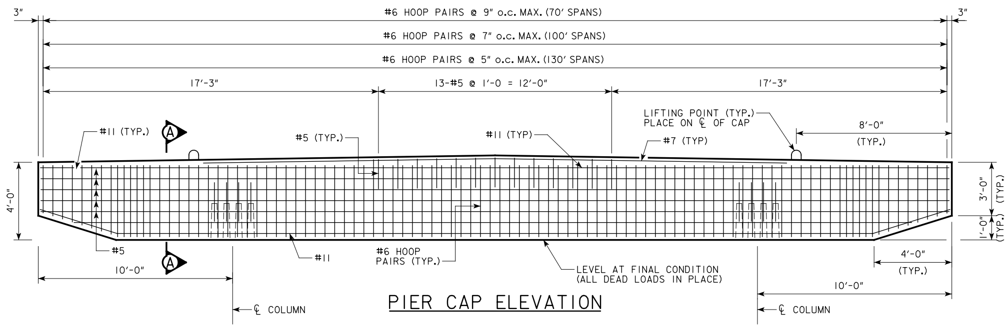
SHEET NUMBER P1



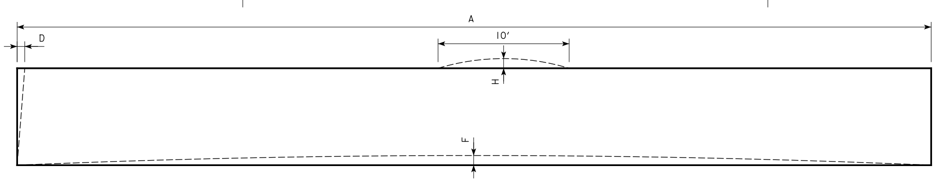
FOR INFORMATIONAL PURPOSES ONLY
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL
STANDARD PREFABRICATED SUBSTRUCTURE
PRECAST PIER ELEV. & DETAILS
CONVENTIONAL PIER
HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012
SHEET NUMBER P2
NOTE:
1. SEE DESIGN SHEET P3 FOR ADDITIONAL PIER CAP DETAILS.
2. SEE SHEET P4 FOR COLUMN DETAILS.
SECTION A-A
PRECAST PIER CAP
(FOR 70' SPAN)
SECTION A-A
PRECAST PIER CAP
(FOR 130' SPAN)
SECTION A-A
PRECAST PIER CAP
(FOR 100' SPAN)
* SPACE REINFORCING TO CLEAR COUPLERS AND PROVIDE PROPER BAR CLEARANCE.
SECTION C-C
PRECAST COLUMN
(ALL SPANS)
REINFORCEMENT SHOWN AS ⊙ ARE CONNECTED TO PIER CAP.
PIER ELEVATION
(ALL SPANS)
DETAIL A
CAP TO COLUMN CONNECTION
END VIEW
SIDE VIEW



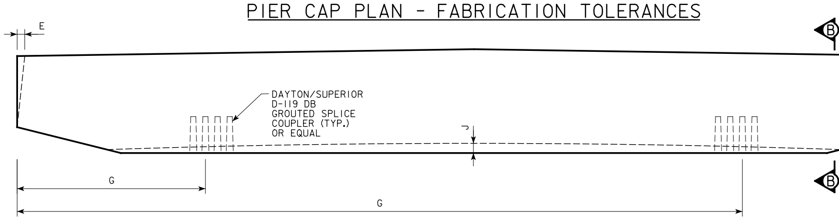
PIER CAP PLAN



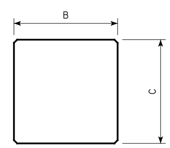
PIER CAP ELEVATION



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.

FOR INFORMATIONAL PURPOSES ONLY

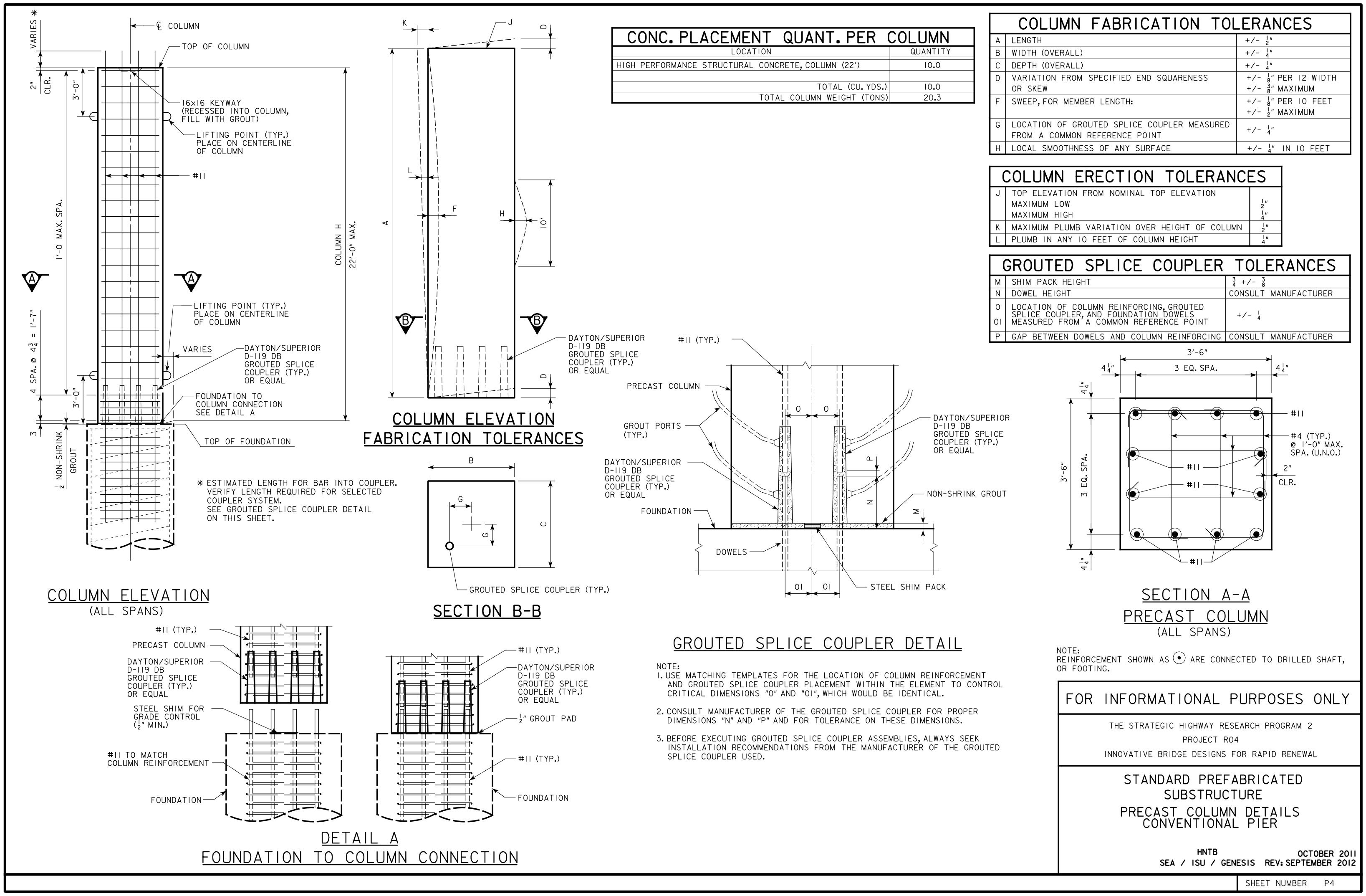
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

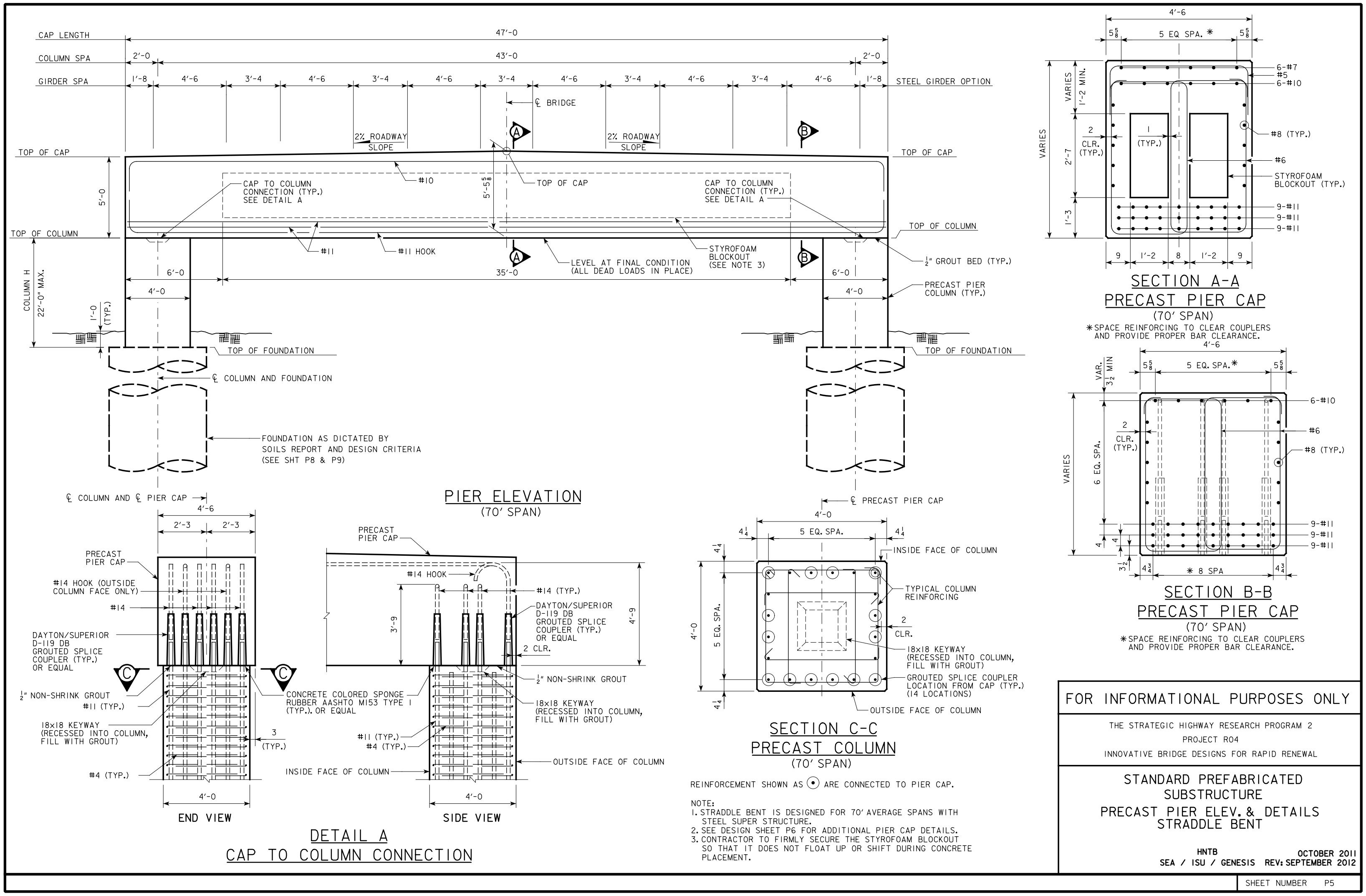
STANDARD PREFABRICATED
SUBSTRUCTURE
PRECAST PIER CAP DETAILS
CONVENTIONAL PIER

HNTB
SEA / ISU / GENESIS

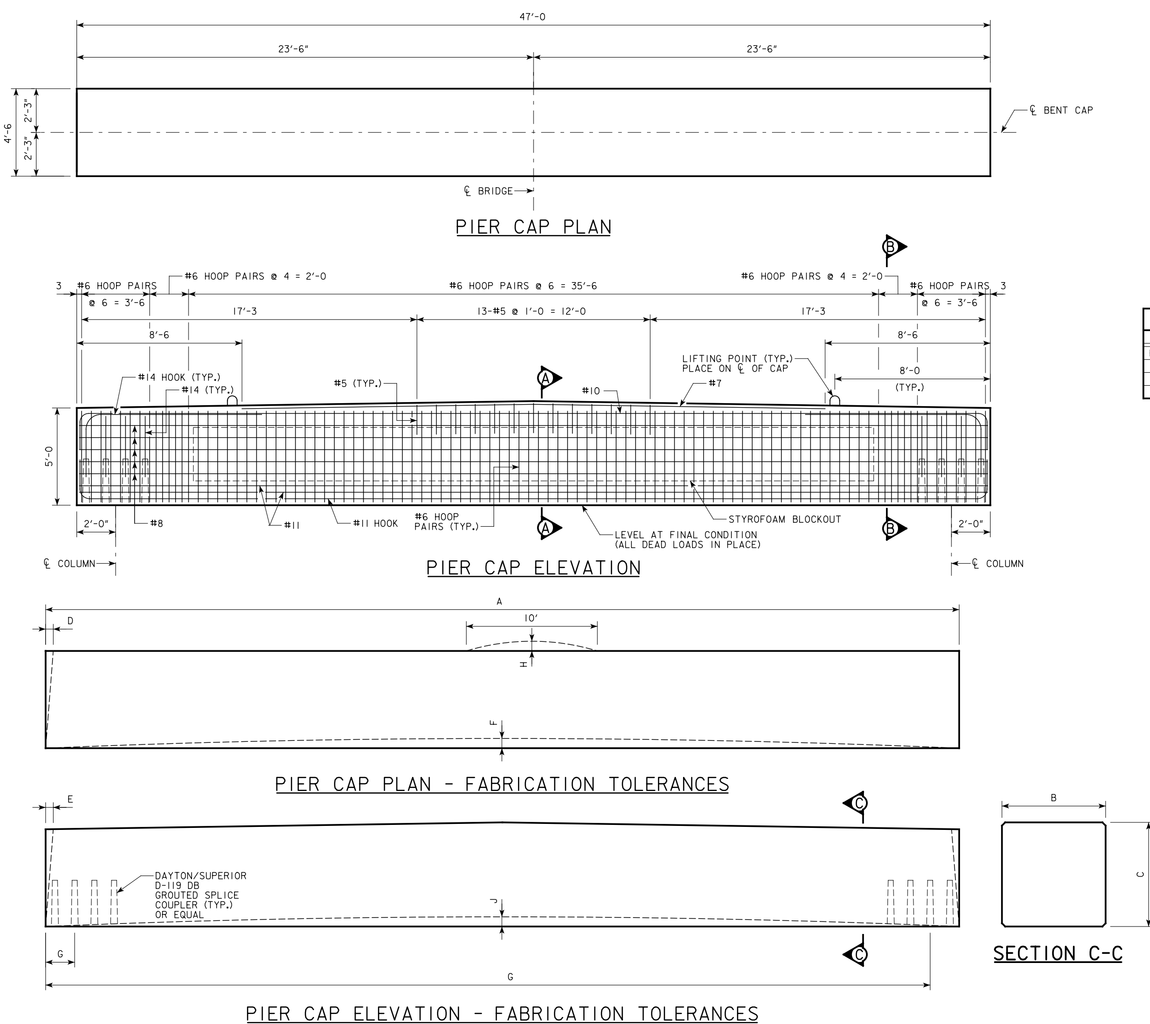
OCTOBER 2011
REV: SEPTEMBER 2012

SHEET NUMBER P3


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 +/- 1/2"
B WIDTH (OVERALL) +/- 1/4"
C DEPTH (OVERALL) +/- 1/4"
D VARIATION FROM SPECIFIED END SQUARENESS OR SKEW +/- 1/8" PER 12 WIDTH +/- 3/8" MAXIMUM
F SWEEP, FOR MEMBER LENGTH: +/- 1/8" PER 10 FEET +/- 1/2" MAXIMUM
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
COLUMN ERECTION TOLERANCES
J TOP ELEVATION FROM NOMINAL TOP ELEVATION MAXIMUM LOW 1/2" MAXIMUM HIGH 1/4"
K MAXIMUM PLUMB VARIATION OVER HEIGHT OF COLUMN 1/2"
L PLUMB IN ANY 10 FEET OF COLUMN HEIGHT 1/4"
GROUTED SPLICE COUPLER TOLERANCES
M SHIM PACK HEIGHT 3/4 +/- 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 +/- 1/4
P GAP BETWEEN DOWELS AND COLUMN REINFORCING CONSULT MANUFACTURER
COLUMN ELEVATION
(ALL SPANS)
℄ COLUMN
TOP OF COLUMN
16x16 KEYWAY (RECESSED INTO COLUMN, FILL WITH GROUT)
LIFTING POINT (TYP.) PLACE ON CENTERLINE OF COLUMN
#11
VARIES
DAYTON/SUPERIOR D-119 DB GROUTED SPLICE COUPLER (TYP.) OR EQUAL
FOUNDATION TO COLUMN CONNECTION SEE DETAIL A
TOP OF FOUNDATION
COLUMN H 22'-0" MAX.
VARIES *
2" CLR.
3'-0"
1'-0 MAX. SPA.
4 SPA. @ 4 1/4 = 1'-7"
1/2 NON-SHRINK GROUT
* 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
GROUTED SPLICE COUPLER (TYP.)
GROUTED SPLICE COUPLER DETAIL
#11 (TYP.)
PRECAST COLUMN
GROUT PORTS (TYP.)
FOUNDATION
DOWELS
NON-SHRINK GROUT
STEEL SHIM PACK
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)
3'-6"
3 EQ. SPA.
4 1/4"
#4 (TYP.) @ 1'-0" MAX. SPA. (U.N.O.)
2" CLR.
NOTE:
REINFORCEMENT SHOWN AS ⊙ ARE CONNECTED TO DRILLED SHAFT, OR FOOTING.
DETAIL A
FOUNDATION TO COLUMN CONNECTION
STEEL SHIM FOR GRADE CONTROL (1/2" MIN.)
#11 TO MATCH COLUMN REINFORCEMENT
1/2" GROUT PAD
FOR INFORMATIONAL PURPOSES ONLY
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL
STANDARD PREFABRICATED SUBSTRUCTURE
PRECAST COLUMN DETAILS CONVENTIONAL PIER
HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012
SHEET NUMBER P4



PIER ELEVATION
(70' SPAN)
CAP LENGTH
COLUMN SPA
GIRDER SPA
STEEL GIRDER OPTION
TOP OF CAP
TOP OF COLUMN
TOP OF FOUNDATION
℄ BRIDGE
2% ROADWAY SLOPE
CAP TO COLUMN CONNECTION (TYP.) SEE DETAIL A
STYROFOAM BLOCKOUT (SEE NOTE 3)
LEVEL AT FINAL CONDITION (ALL DEAD LOADS IN PLACE)
1/2" GROUT BED (TYP.)
PRECAST PIER COLUMN (TYP.)
COLUMN H 22'-0" MAX.
℄ 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
PRECAST PIER CAP
#14 HOOK (OUTSIDE COLUMN FACE ONLY)
DAYTON/SUPERIOR D-119 DB GROUTED SPLICE COUPLER (TYP.) OR EQUAL
1/2" NON-SHRINK GROUT
18x18 KEYWAY (RECESSED INTO COLUMN, FILL WITH GROUT)
CONCRETE COLORED SPONGE RUBBER AASHTO M153 TYPE I (TYP.), OR EQUAL
INSIDE FACE OF COLUMN
OUTSIDE FACE OF COLUMN
SECTION C-C
PRECAST COLUMN
(70' SPAN)
TYPICAL COLUMN REINFORCING
GROUTED SPLICE COUPLER LOCATION FROM CAP (TYP.) (14 LOCATIONS)
REINFORCEMENT SHOWN AS ⊙ ARE CONNECTED TO PIER CAP.
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.
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.
FOR INFORMATIONAL PURPOSES ONLY
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL
STANDARD PREFABRICATED SUBSTRUCTURE
PRECAST PIER ELEV. & DETAILS STRADDLE BENT
HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012
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.

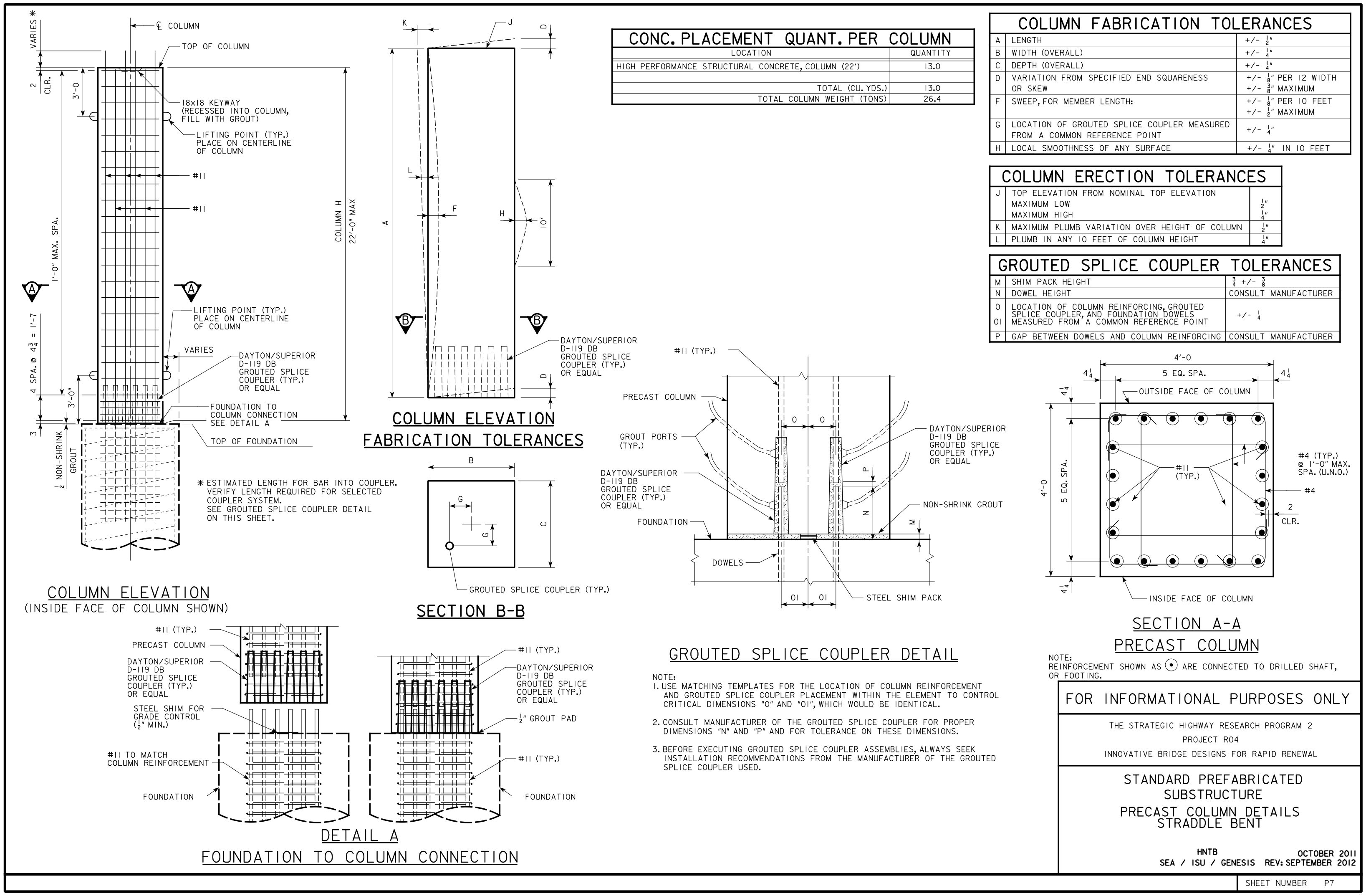
FOR INFORMATIONAL PURPOSES ONLY

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

STANDARD PREFABRICATED SUBSTRUCTURE
PRECAST PIER CAP DETAILS
STRADDLE BENT

HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012

SHEET NUMBER P6



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 OI	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

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.

NOTE:
REINFORCEMENT SHOWN AS ⊙ ARE CONNECTED TO DRILLED SHAFT, OR FOOTING.

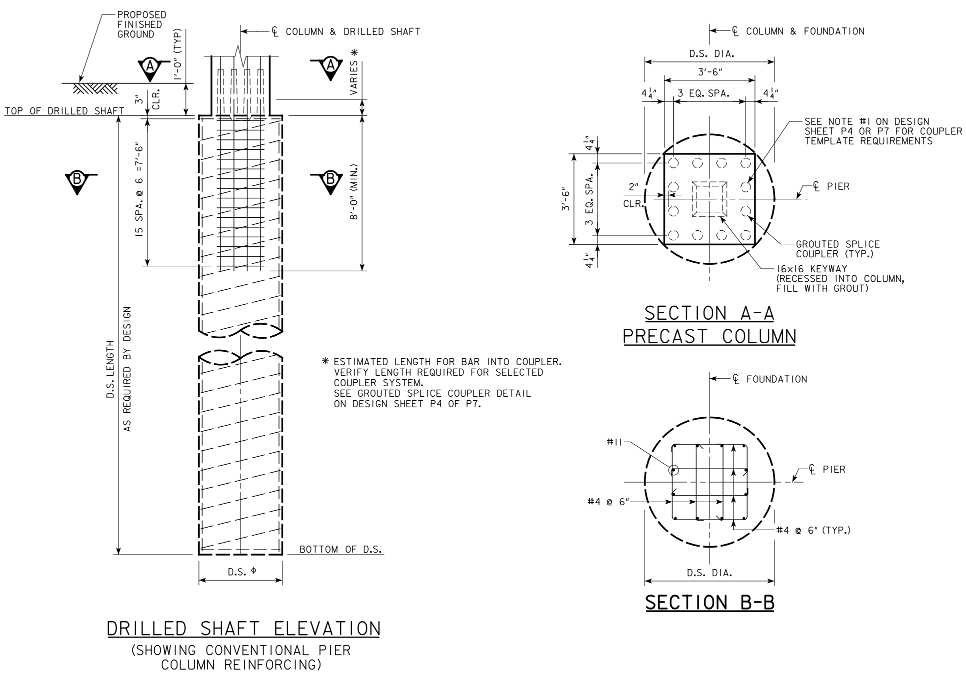
FOR INFORMATIONAL PURPOSES ONLY

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

STANDARD PREFABRICATED SUBSTRUCTURE
PRECAST COLUMN DETAILS
STRADDLE BENT

HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012

SHEET NUMBER P7



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.

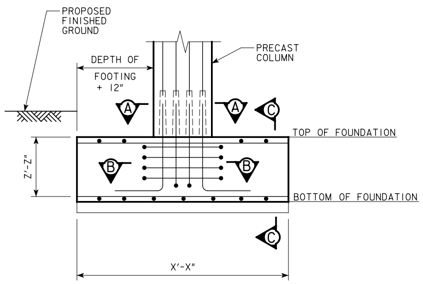
FOR INFORMATIONAL PURPOSES ONLY

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

STANDARD PREFABRICATED SUBSTRUCTURE
FOUNDATION DETAILS
DRILLED SHAFT

HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012

SHEET NUMBER P8

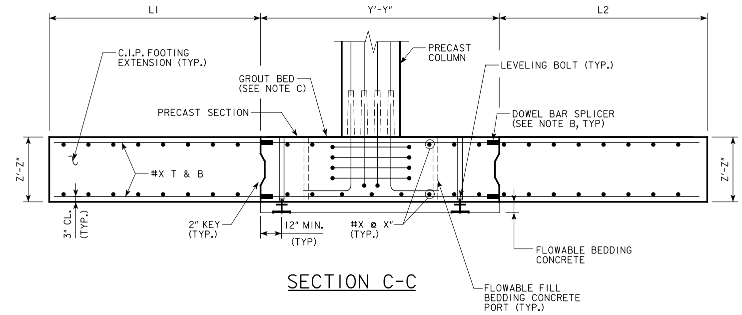


TYPICAL COLUMN PIER FRONT ELEVATION

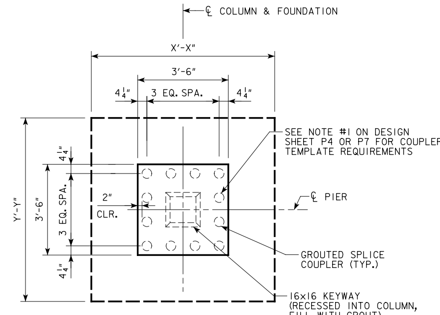
(SHOWING CONVENTIONAL PIER REINFORCING)

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 C-C



SECTION A-A
PRECAST COLUMN

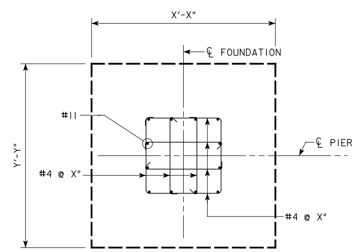
PRECAST FOOTING NOTES:

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

FOR INFORMATIONAL PURPOSES ONLY

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

STANDARD PREFABRICATED SUBSTRUCTURE
FOUNDATION DETAILS
PRECAST FOOTING

HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012

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

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

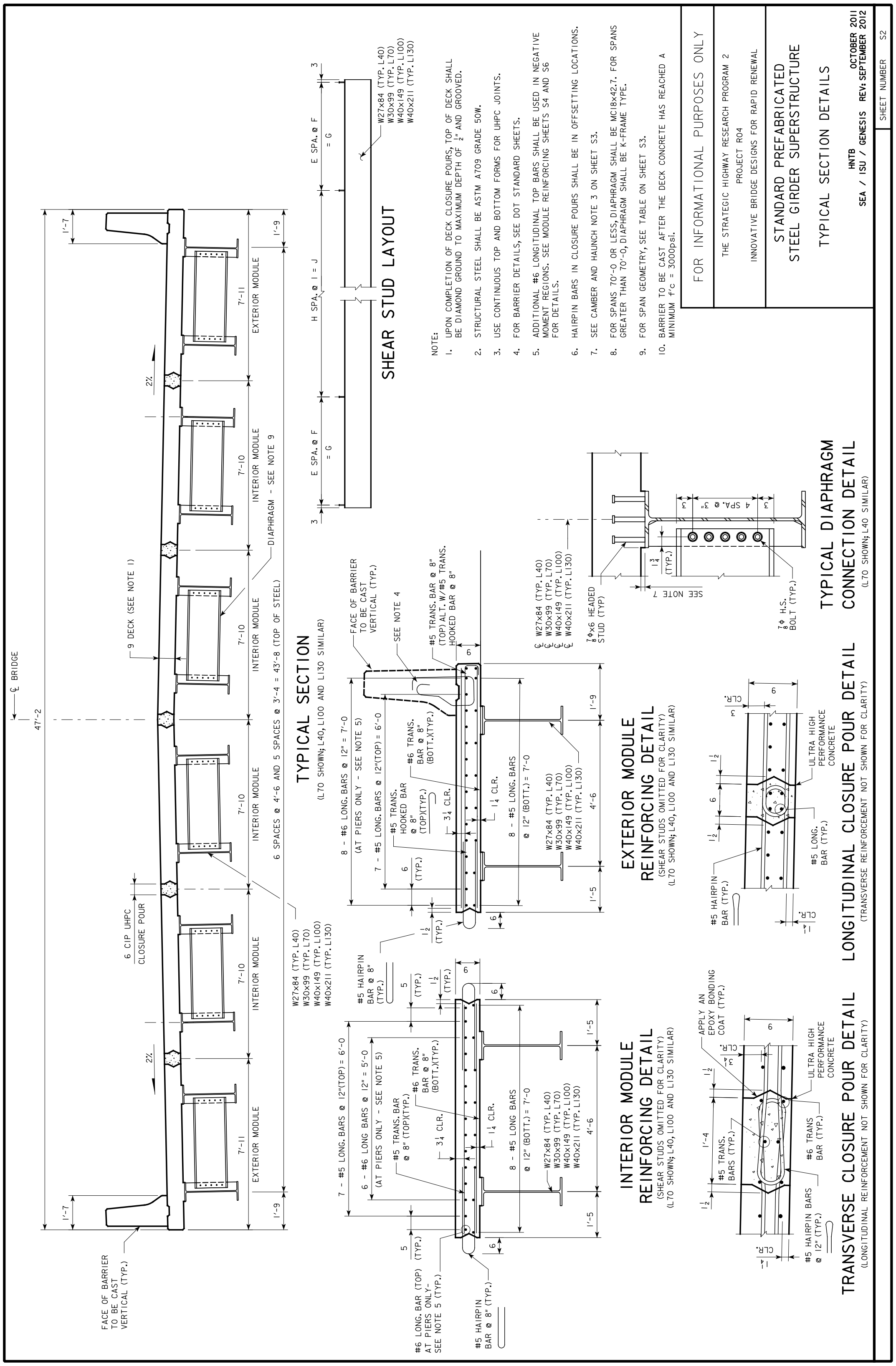
STANDARD PREFABRICATED
STEEL GIRDER SUPERSTRUCTURE

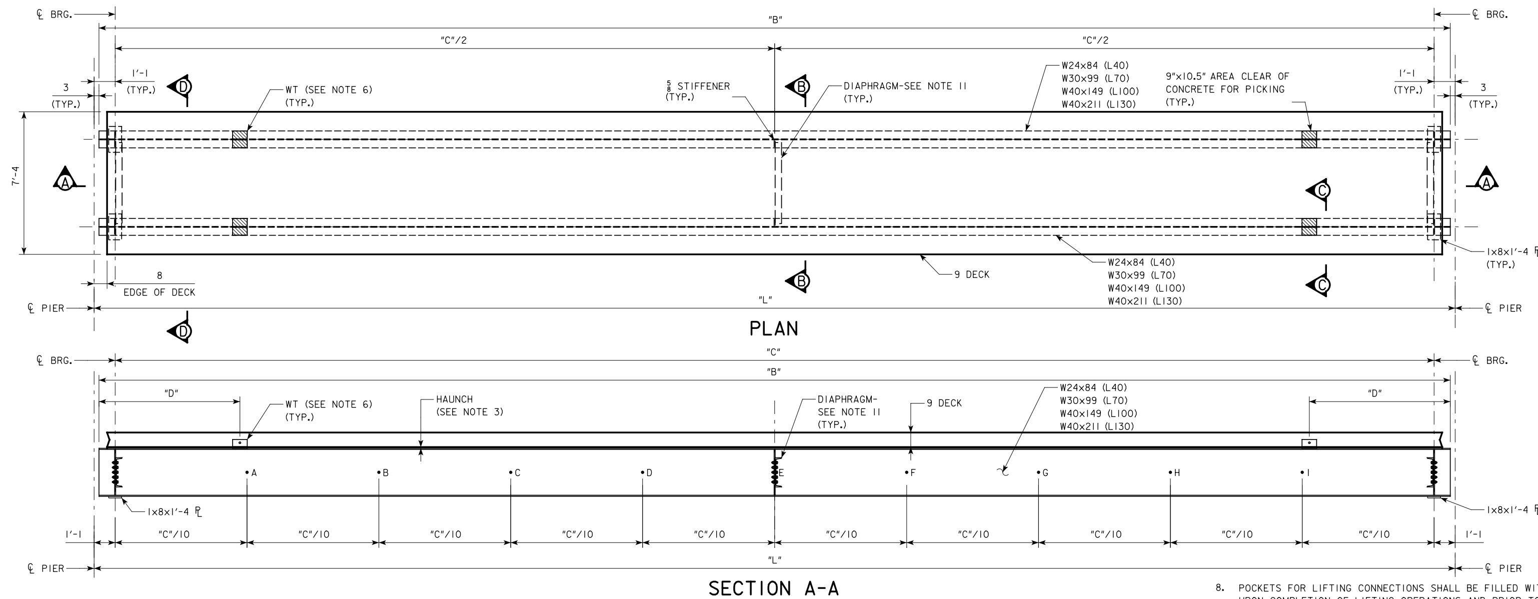
GENERAL NOTES AND
INDEX OF DRAWINGS

HNTB
SEA / ISU / GENESIS

OCTOBER 2011
REV: SEPTEMBER 2012

SHEET NUMBER S1


TYPICAL SECTION
(L70 SHOWN; L40, L100 AND L130 SIMILAR)
SHEAR STUD LAYOUT
INTERIOR MODULE REINFORCING DETAIL
(SHEAR STUDS OMITTED FOR CLARITY)
(L70 SHOWN; L40, L100 AND L130 SIMILAR)
EXTERIOR MODULE REINFORCING DETAIL
(SHEAR STUDS OMITTED FOR CLARITY)
(L70 SHOWN; L40, L100 AND L130 SIMILAR)
TRANSVERSE CLOSURE POUR DETAIL
(LONGITUDINAL REINFORCEMENT NOT SHOWN FOR CLARITY)
LONGITUDINAL CLOSURE POUR DETAIL
(TRANSVERSE REINFORCEMENT NOT SHOWN FOR CLARITY)
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 1/2" 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.
FOR INFORMATIONAL PURPOSES ONLY
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL
STANDARD PREFABRICATED STEEL GIRDER SUPERSTRUCTURE
TYPICAL SECTION DETAILS
HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012
SHEET NUMBER S2



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

FOR INFORMATIONAL PURPOSES ONLY

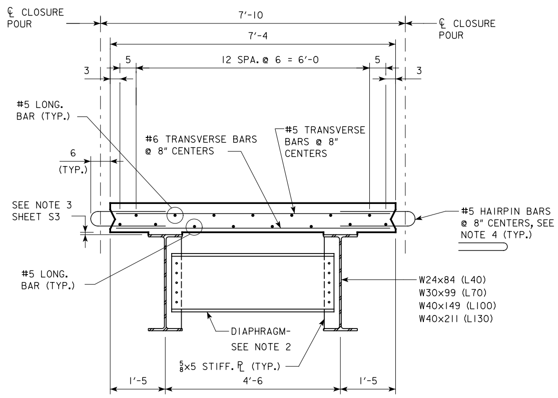
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

STANDARD PREFABRICATED
STEEL GIRDER SUPERSTRUCTURE

INTERIOR MODULE
(INTERIOR SPAN)

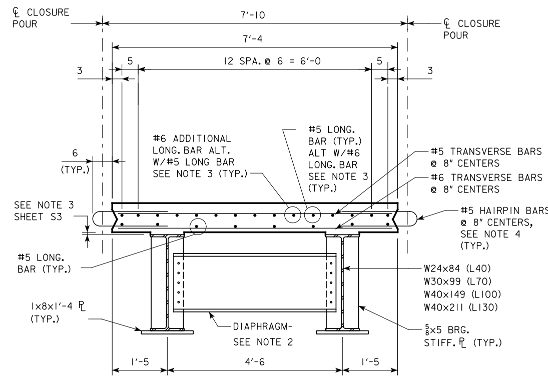
HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012

SHEET NUMBER S3



SECTION B-B

(L40 & L70 SHOWN; L100 & L130 SIMILAR)



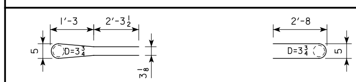
SECTION D-D

(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

FOR INFORMATIONAL PURPOSES ONLY

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

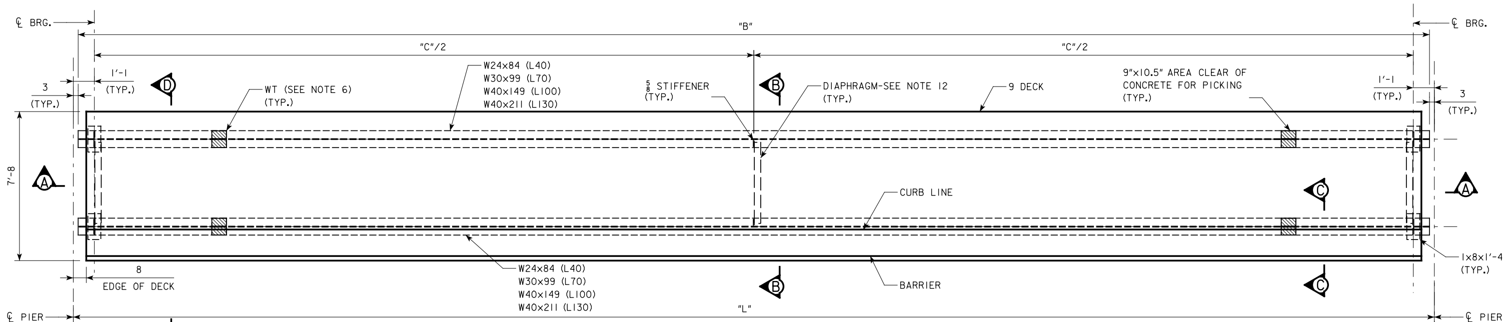
STANDARD PREFABRICATED
STEEL GIRDER SUPERSTRUCTURE

INTERIOR MODULE REINF.

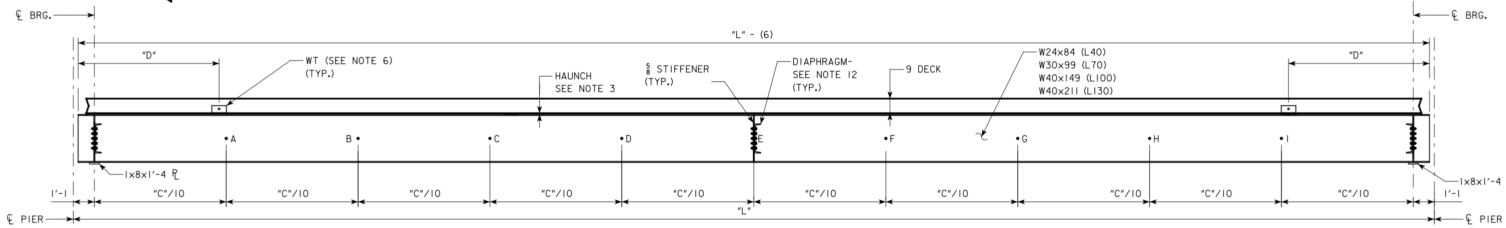
HNTB
SEA / ISU / GENESIS

OCTOBER 2011
REV: SEPTEMBER 2012

SHEET NUMBER S4



PLAN

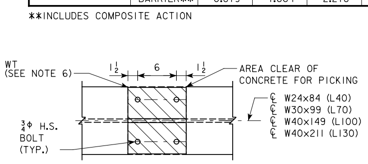


SECTION A-A

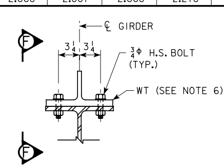
SAMPLE DEAD LOAD DEFLECTIONS - EXTERIOR MODULE

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

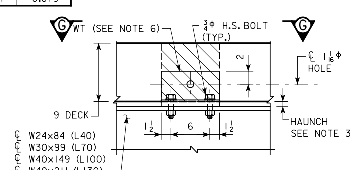


SECTION G-G



SECTION C-C

(DECK NOT SHOWN FOR CLARITY)



SECTION F-F

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.

FOR INFORMATIONAL PURPOSES ONLY

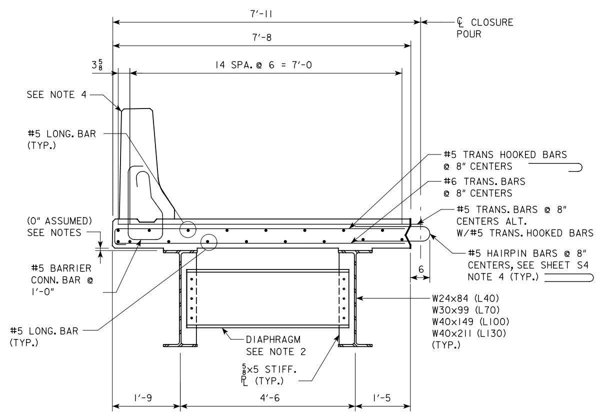
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

STANDARD PREFABRICATED
STEEL GIRDER SUPERSTRUCTURE

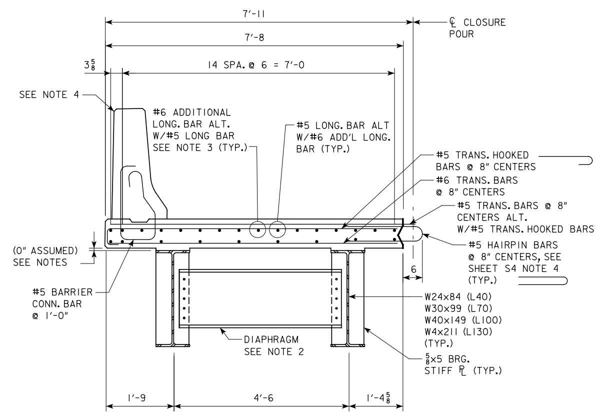
EXTERIOR MODULE
(INTERIOR SPAN)

HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012

SHEET NUMBER S5

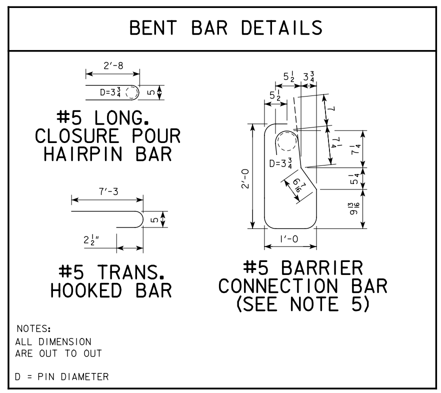


SECTION B-B
(L40 & L70 SHOWN; L100 & L130 SIMILAR)



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



FOR INFORMATIONAL PURPOSES ONLY

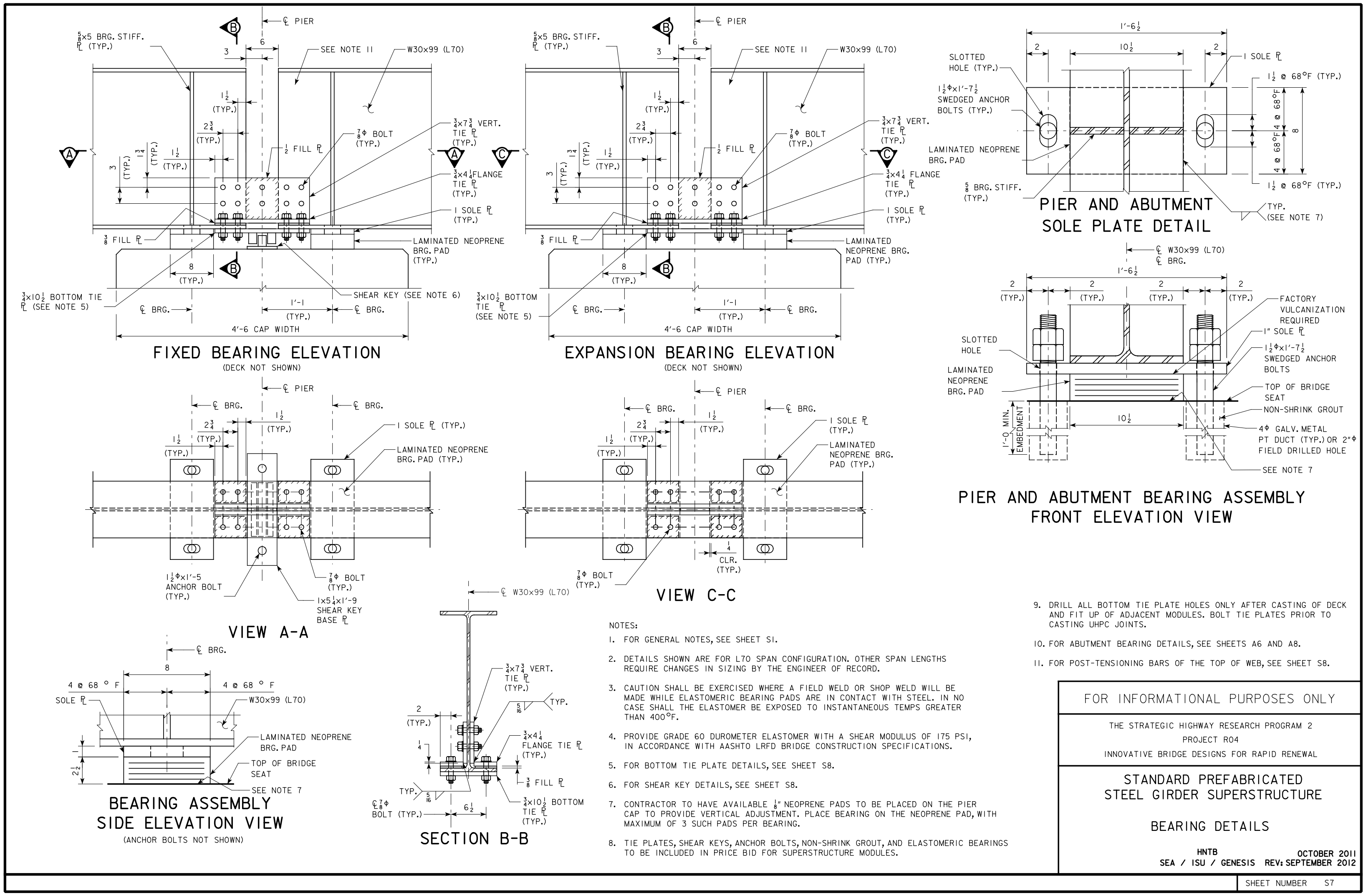
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

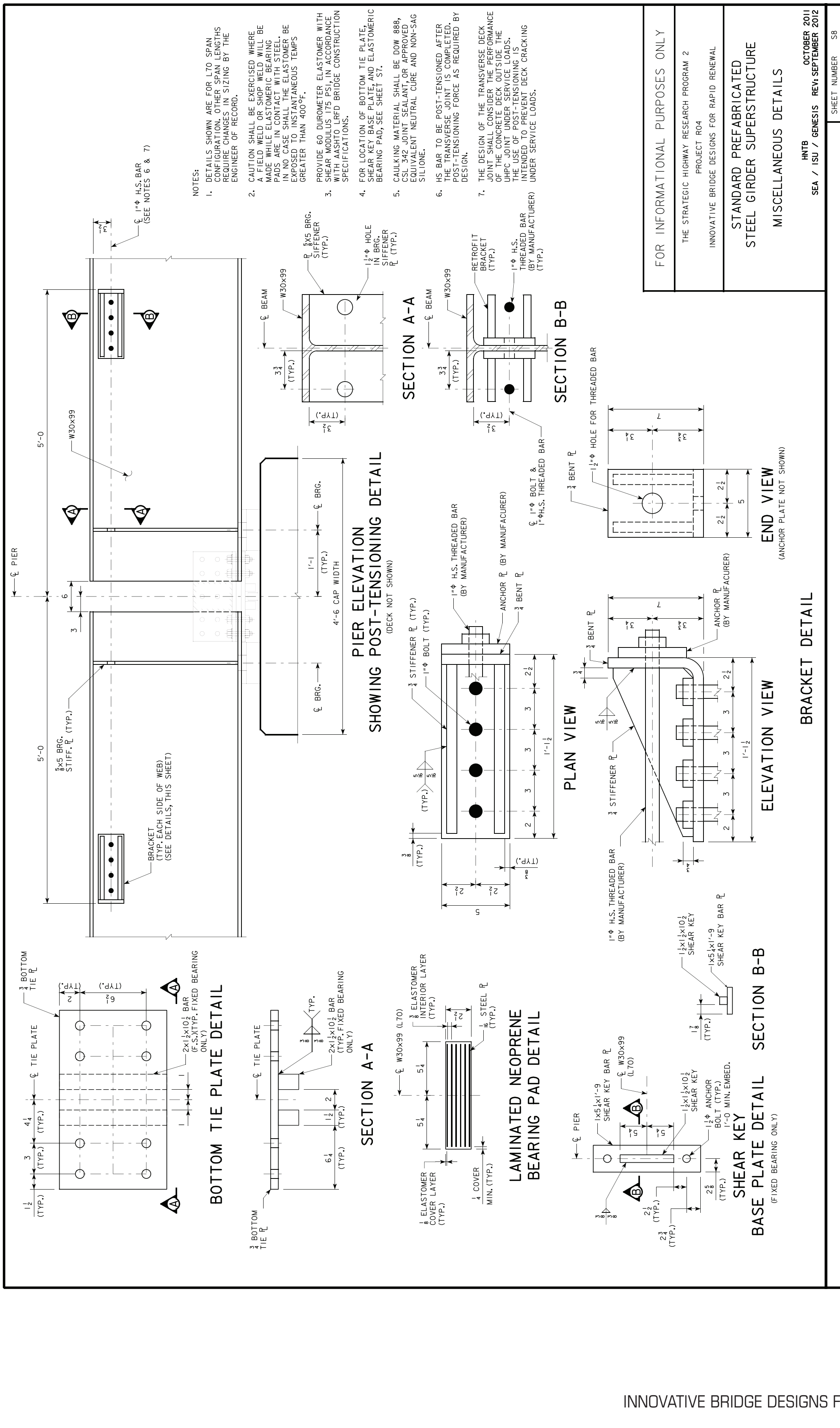
STANDARD PREFABRICATED
STEEL GIRDER SUPERSTRUCTURE

EXT. MODULE 2 REINF.

HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012

SHEET NUMBER S6


FIXED BEARING ELEVATION
(DECK NOT SHOWN)
EXPANSION BEARING ELEVATION
(DECK NOT SHOWN)
PIER AND ABUTMENT SOLE PLATE DETAIL
PIER AND ABUTMENT BEARING ASSEMBLY FRONT ELEVATION VIEW
VIEW A-A
VIEW C-C
BEARING ASSEMBLY SIDE ELEVATION VIEW
(ANCHOR BOLTS NOT SHOWN)
SECTION B-B
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 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.
FOR INFORMATIONAL PURPOSES ONLY
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL
STANDARD PREFABRICATED
STEEL GIRDER SUPERSTRUCTURE
BEARING DETAILS
HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012
SHEET NUMBER S7


BOTTOM TIE PLATE DETAIL
SECTION A-A
LAMINATED NEOPRENE BEARING PAD DETAIL
SHEAR KEY BASE PLATE DETAIL
(FIXED BEARING ONLY)
SECTION B-B
PIER ELEVATION
SHOWING POST-TENSIONING DETAIL
(DECK NOT SHOWN)
PLAN VIEW
ELEVATION VIEW
END VIEW
(ANCHOR PLATE NOT SHOWN)
BRACKET DETAIL
SECTION A-A
SECTION B-B
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.
3. PROVIDE 60 DUROMETER ELASTOMER WITH 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.
FOR INFORMATIONAL PURPOSES ONLY
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL
STANDARD PREFABRICATED
STEEL GIRDER SUPERSTRUCTURE
MISCELLANEOUS DETAILS
HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012
SHEET NUMBER S8

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 AS REQUIRED BY SPECIFICATIONS.

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

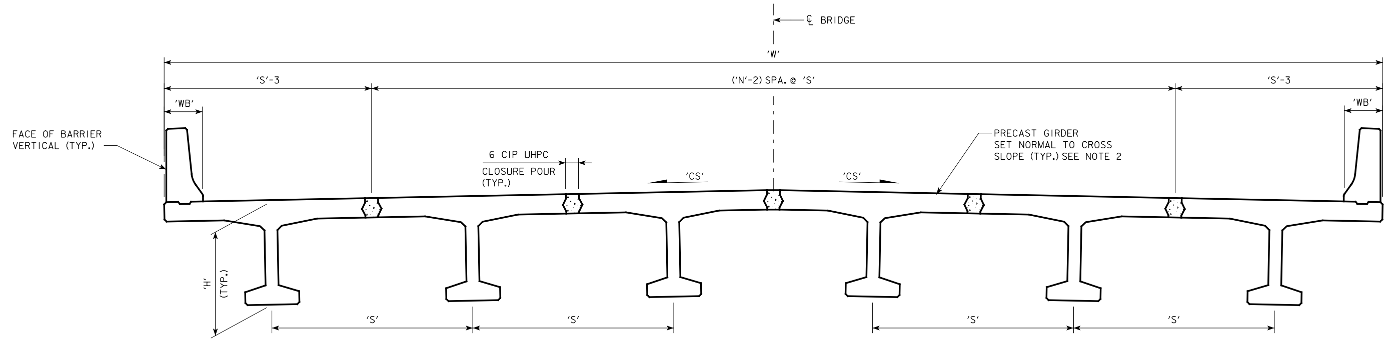
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

STANDARD PRECAST PRESTRESSED DECKED GIRDER SUPERSTRUCTURE

GENERAL NOTES AND INDEX OF DRAWINGS

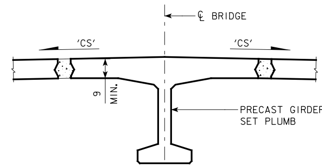
HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012

SHEET NUMBER C1



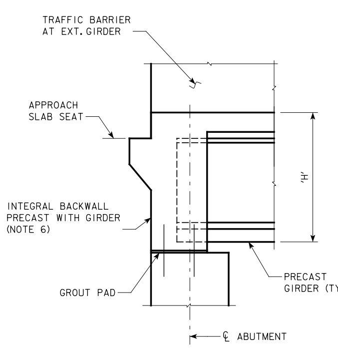
TYPICAL SECTION

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 %

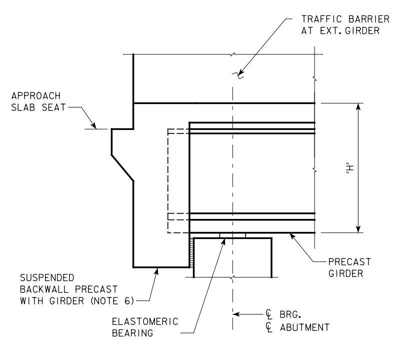


CENTRAL GIRDER

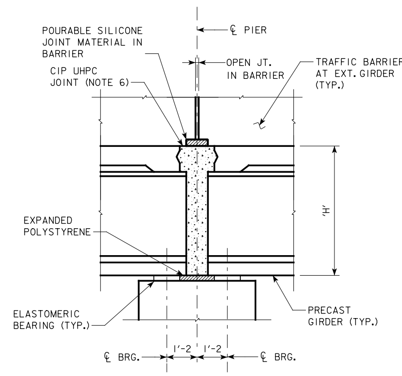
(SEE NOTE 2)



ELEVATION AT INTEGRAL ABUTMENT



ELEVATION AT SEMI-INTEGRAL ABUTMENT



ELEVATION AT INTERMEDIATE PIER

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.

FOR INFORMATIONAL PURPOSES ONLY

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

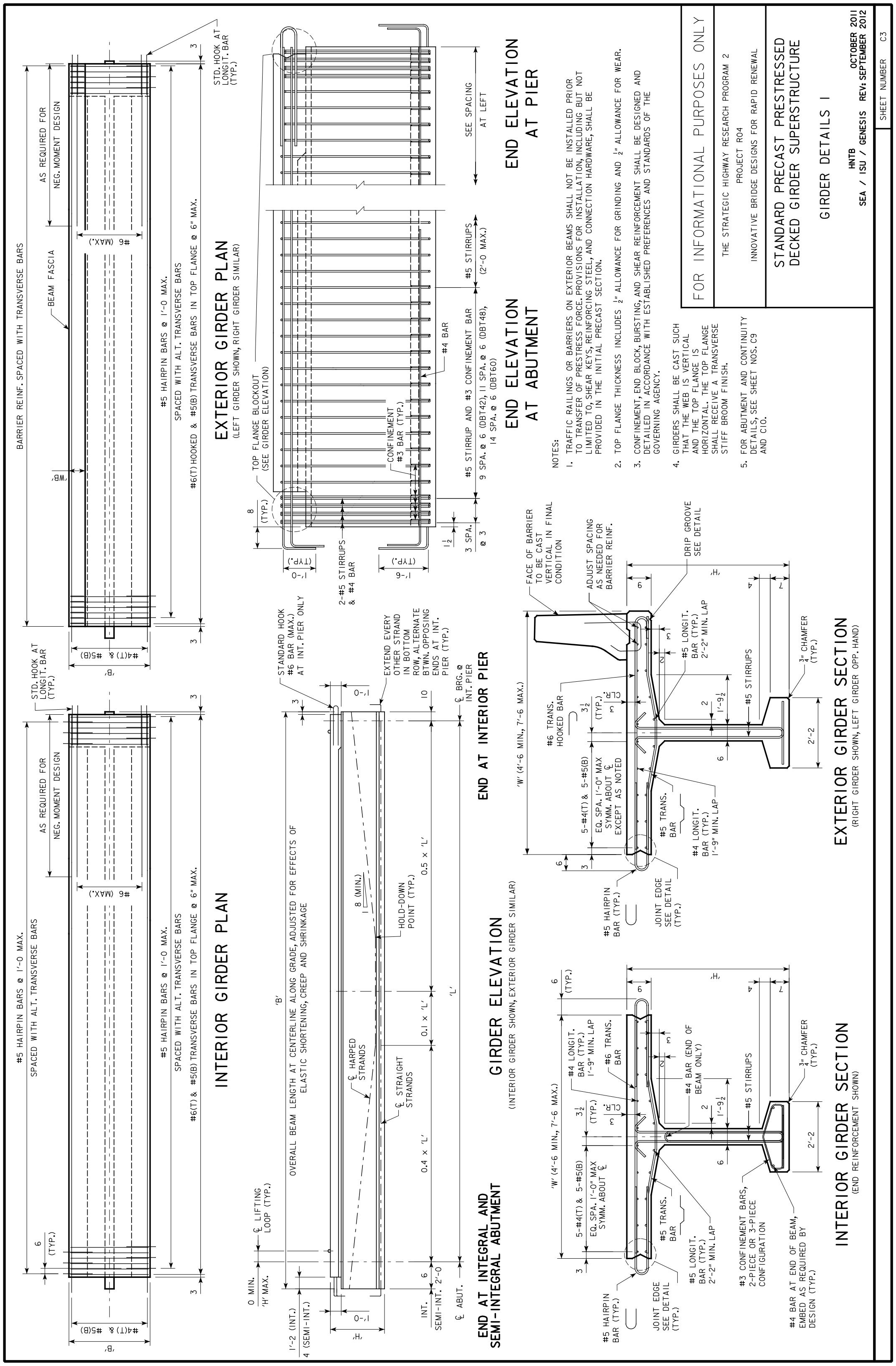
STANDARD PRECAST PRESTRESSED DECKED GIRDER SUPERSTRUCTURE

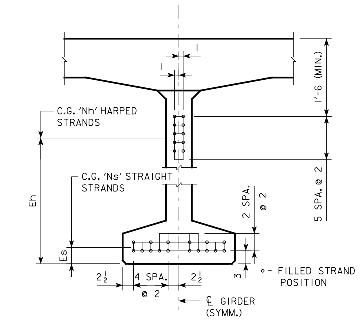
TYPICAL SECTION

HNTB
SEA / ISU / GENESIS

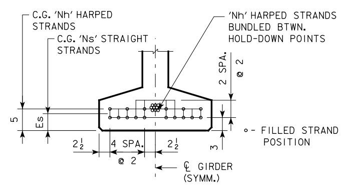
OCTOBER 2011
REV: SEPTEMBER 2012

SHEET NUMBER C2

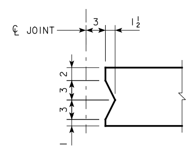

INTERIOR GIRDER PLAN
EXTERIOR GIRDER PLAN
(LEFT GIRDER SHOWN, RIGHT GIRDER SIMILAR)
GIRDER ELEVATION
(INTERIOR GIRDER SHOWN, EXTERIOR GIRDER SIMILAR)
END AT INTEGRAL AND SEMI-INTEGRAL ABUTMENT
END AT INTERIOR PIER
END ELEVATION AT ABUTMENT
END ELEVATION AT PIER
INTERIOR GIRDER SECTION
(END REINFORCEMENT 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 1/2" ALLOWANCE FOR GRINDING AND 1/2" 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.
FOR INFORMATIONAL PURPOSES ONLY
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL
STANDARD PRECAST PRESTRESSED DECKED GIRDER SUPERSTRUCTURE
GIRDER DETAILS I
HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012
SHEET NUMBER C3



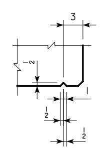
STRAND PATTERN AT ℄ BEARING
(EXAMPLE PATTERN SHOWN, SEE TABLE)



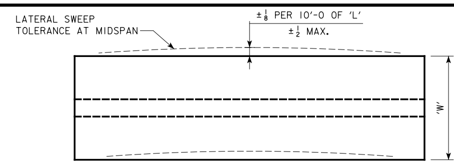
STRAND PATTERN AT MIDSPAN
(EXAMPLE PATTERN SHOWN, SEE TABLE)



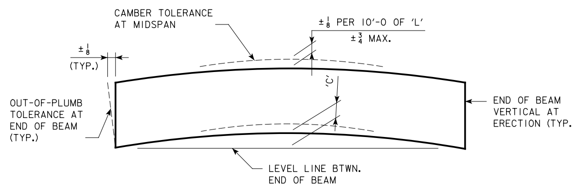
JOINT EDGE DETAIL



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

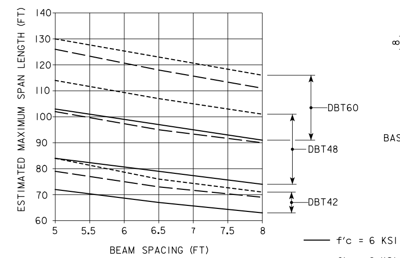
BEAM TYPE	'L' (FT)	'S' (FT)	f'c (KSI)	STRAND SIZE (IN)	STRAIGHT 'Ns'	STRAIGHT 'Es' (IN)	HARPED 'Nh'	HARPED 'Eh' (IN)	TOTAL 'N'	TOTAL 'E' (BRG.) (IN)	TOTAL 'E' (MID.) (IN)
DBT42	40	5	8	0.5	6	3.00	4	23.00	10	11.00	3.80
		8	8	0.5	6	3.00	6	22.00	12	12.50	4.00
	70	5	8	0.5	12	3.33	10	20.00	22	10.91	4.09
		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

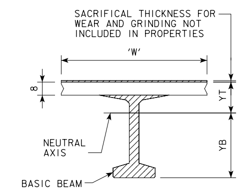
SECTION PROPERITES FOR BEAM DESIGN

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)	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.

FOR INFORMATIONAL PURPOSES ONLY

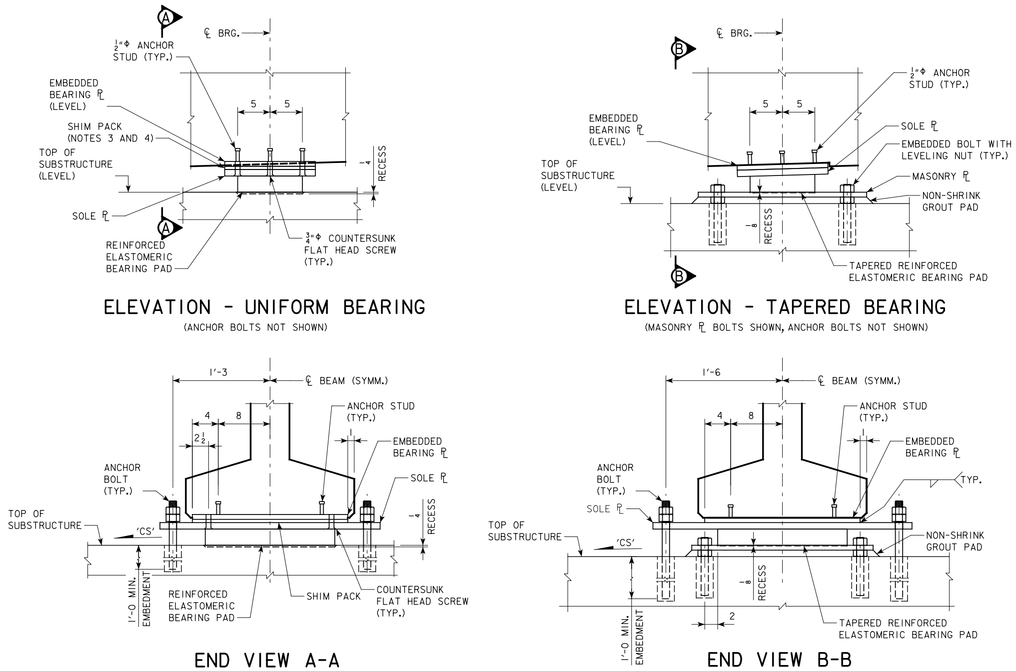
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

STANDARD PRECAST PRESTRESSED
DECKED GIRDER SUPERSTRUCTURE

GIRDER DETAILS 2

HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012

SHEET NUMBER C4



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.

FOR INFORMATIONAL PURPOSES ONLY

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

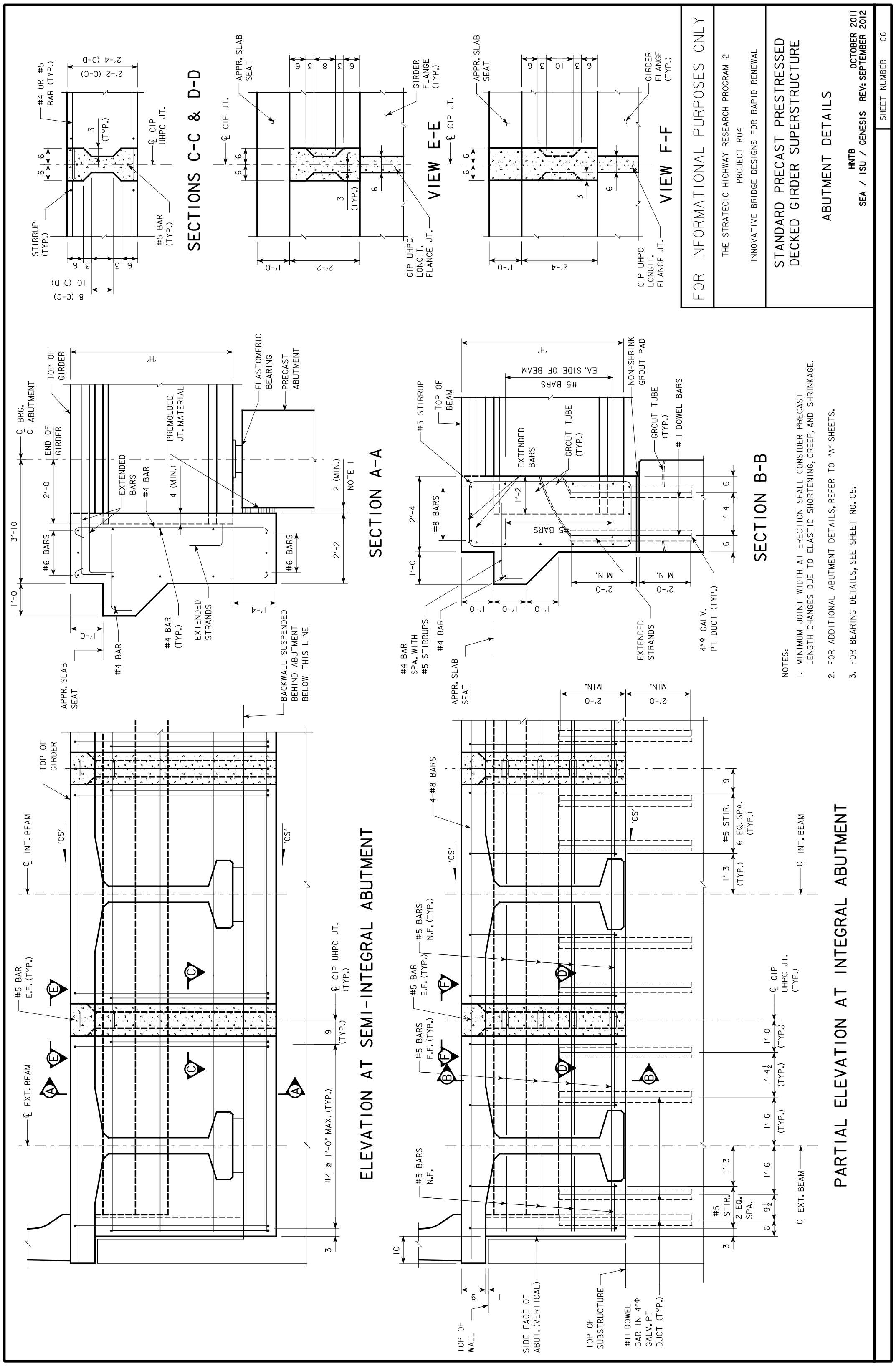
STANDARD PRECAST PRESTRESSED DECKED GIRDER SUPERSTRUCTURE

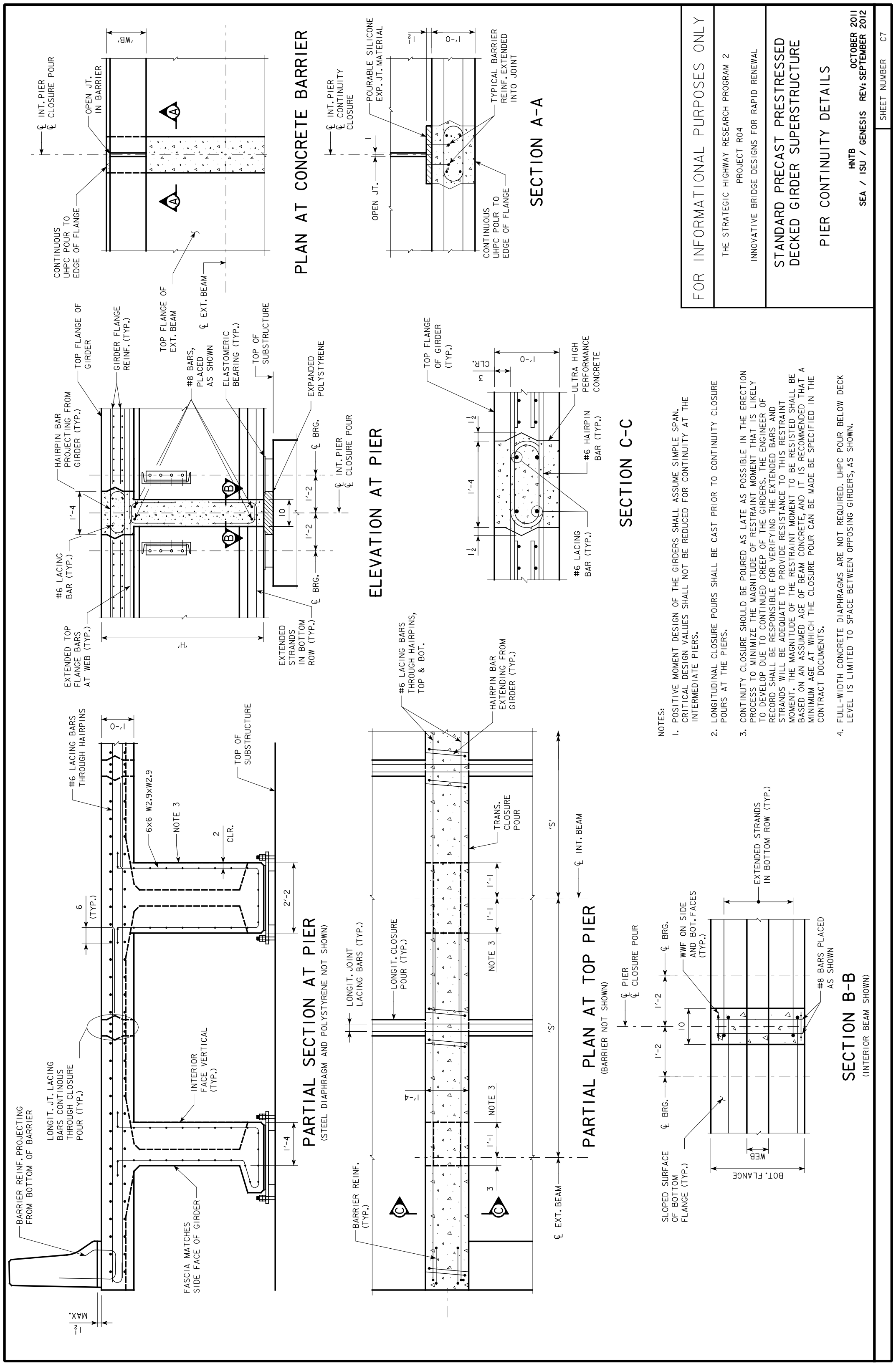
BEARING DETAILS

HNTB
SEA / ISU / GENESIS

OCTOBER 2011
REV: SEPTEMBER 2012

SHEET NUMBER C5


ELEVATION AT SEMI-INTEGRAL ABUTMENT
PARTIAL ELEVATION AT INTEGRAL ABUTMENT
SECTION A-A
SECTION B-B
SECTIONS C-C & D-D
VIEW E-E
VIEW F-F
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.
FOR INFORMATIONAL PURPOSES ONLY
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL
STANDARD PRECAST PRESTRESSED DECKED GIRDER SUPERSTRUCTURE
ABUTMENT DETAILS
HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012
SHEET NUMBER C6


PARTIAL SECTION AT PIER
(STEEL DIAPHRAGM AND POLYSTYRENE NOT SHOWN)
PARTIAL PLAN AT TOP PIER
(BARRIER NOT SHOWN)
SECTION B-B
(INTERIOR BEAM SHOWN)
ELEVATION AT PIER
SECTION C-C
PLAN AT CONCRETE BARRIER
SECTION A-A
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.
FOR INFORMATIONAL PURPOSES ONLY
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL
STANDARD PRECAST PRESTRESSED
DECKED GIRDER SUPERSTRUCTURE
PIER CONTINUITY DETAILS
HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012
SHEET NUMBER C7

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 MINIMIZE 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 ¼".
7. THE MAXIMUM PERMISSIBLE RELATIVE VERTICAL DISTANCE BETWEEN FLANGE TIPS OF ADJACENT BEAMS IS ¾". 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

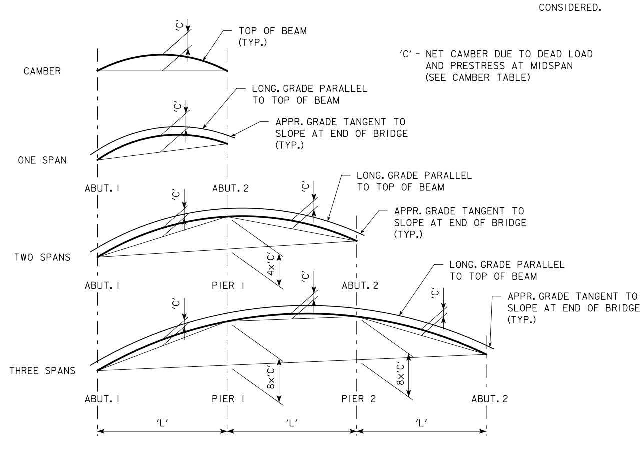
1. ERECT ALL BEAMS WITHIN A SPAN PRIOR TO CAMBER EQUALIZATION.
2. ONCE ALL BEAMS WITHIN A SPAN ARE ERECTED, IDENTIFY THE BEAM WITH THE MOST CAMBER (HIGH BEAM) AND THE BEAM WITH THE LEAST CAMBER (LOW BEAM).
3. WHERE THE FLANGE TIP OF THE HIGH BEAM IS MORE THAN ¼" ABOVE THE FLANGE TIP OF AN ADJACENT BEAM, 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 ¼" 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.

FOR INFORMATIONAL PURPOSES ONLY

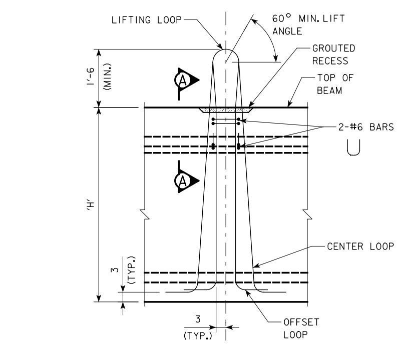
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

STANDARD PRECAST PRESTRESSED DECKED GIRDER SUPERSTRUCTURE

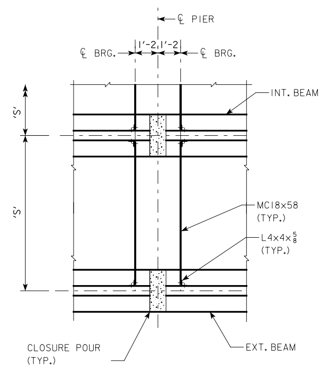
CAMBER AND PLACEMENT NOTES

HNTB — OCTOBER 2011
SEA / ISU / GENESIS — REV: SEPTEMBER 2012

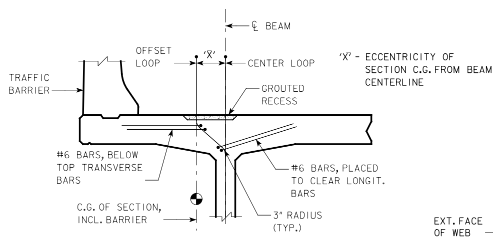
SHEET NUMBER C8



LIFTING LOOP DETAIL

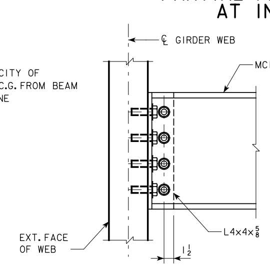


PARTIAL FRAMING PLAN AT INT. PIER

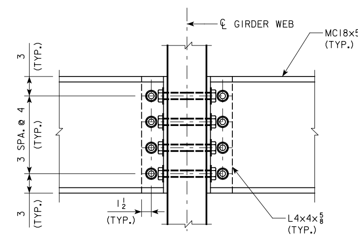


SECTION A-A

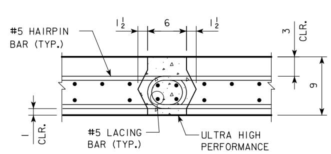
(EXT. BEAM SHOWN, OFFSET LOOP MAY BE OMITTED FOR INT. BEAM)



DIAPHRAGM CONNECTION AT EXTERIOR GIRDER



DIAPHRAGM CONNECTION AT INTERIOR GIRDER



LONGITUDINAL CLOSURE POUR DETAIL

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.

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.

FOR INFORMATIONAL PURPOSES ONLY

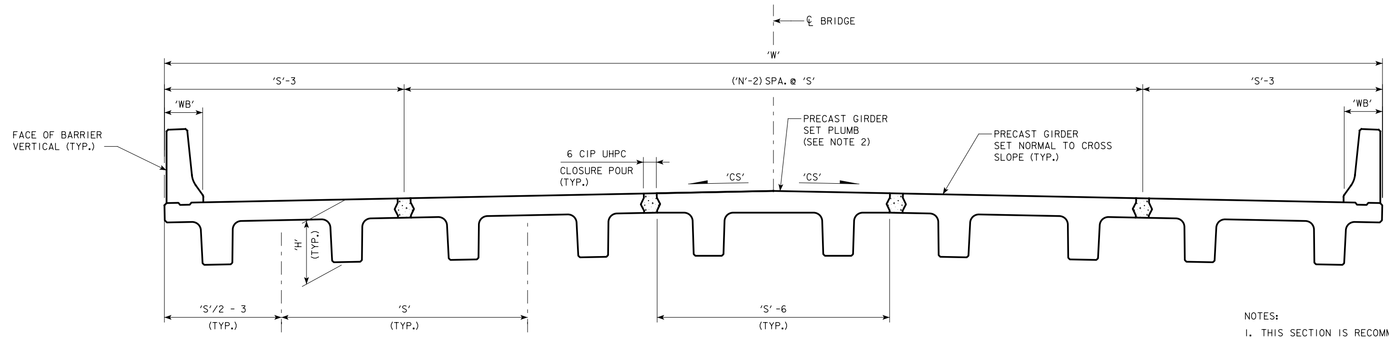
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

STANDARD PRECAST PRESTRESSED DECKED GIRDER SUPERSTRUCTURE

MISCELLANEOUS DETAILS

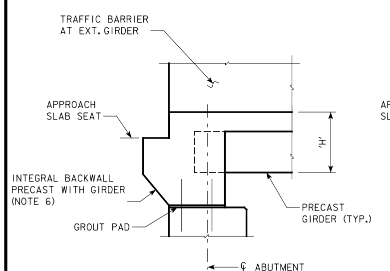
HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012

SHEET NUMBER C9

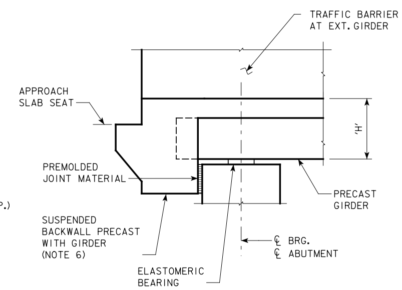


TYPICAL SECTION

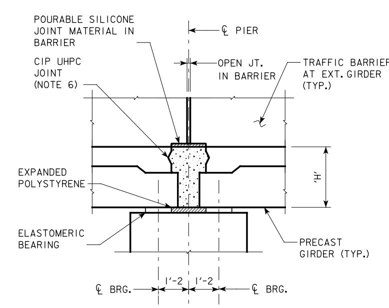
TYPICAL SECTION DIMENSIONS		
DESCRIPTION	VARIABLE	VALUE
OUT-OUT WIDTH	'W'	47'-2
BARRIER WIDTH	'WB'	1'-7
NO. OF GIRDERS	'N'	5
GIRDER SPACING	'S'	9'-6$\frac{3}{8}$
GIRDER HEIGHT	'H'	2'-4
CROSS SLOPE	'CS'	2 %



ELEVATION AT
INTEGRAL ABUTMENT



ELEVATION AT
SEMI-INTEGRAL ABUTMENT



ELEVATION AT
INTERMEDIATE PIER

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.

FOR INFORMATIONAL PURPOSES ONLY

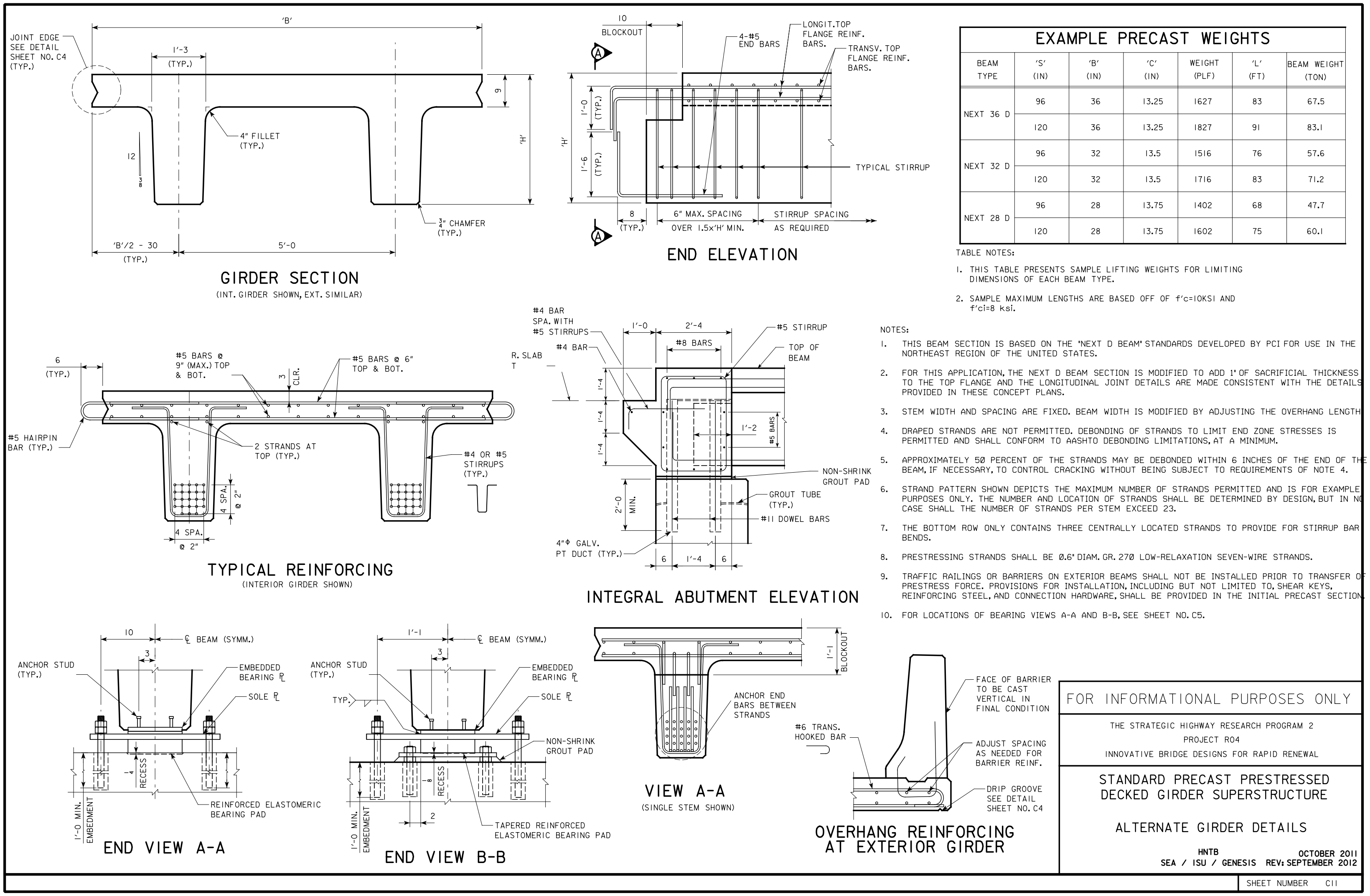
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

STANDARD PRECAST PRESTRESSED
DECKED GIRDER SUPERSTRUCTURE

ALTERNATE TYPICAL SECTION

HNTB OCTOBER 2011
SEA / ISU / GENESIS REV: SEPTEMBER 2012

SHEET NUMBER C10



EXAMPLE PRECAST WEIGHTS

BEAM TYPE	'S' (IN)	'B' (IN)	'C' (IN)	WEIGHT (PLF)	'L' (FT)	BEAM WEIGHT (TON)
NEXT 36 D	96	36	13.25	1627	83	67.5
	120	36	13.25	1827	91	83.1
NEXT 32 D	96	32	13.5	1516	76	57.6
	120	32	13.5	1716	83	71.2
NEXT 28 D	96	28	13.75	1402	68	47.7
	120	28	13.75	1602	75	60.1

TABLE NOTES:

1. THIS TABLE PRESENTS SAMPLE LIFTING WEIGHTS FOR LIMITING DIMENSIONS OF EACH BEAM TYPE.
2. SAMPLE MAXIMUM LENGTHS ARE BASED OFF OF f'c=10KSI AND f'ci=8 ksi.

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 4.
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.

FOR INFORMATIONAL PURPOSES ONLY

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

STANDARD PRECAST PRESTRESSED DECKED GIRDER SUPERSTRUCTURE

ALTERNATE GIRDER DETAILS

HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012

SHEET NUMBER C11

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

SHEET	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 & 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

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

FOR INFORMATIONAL PURPOSES ONLY

THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

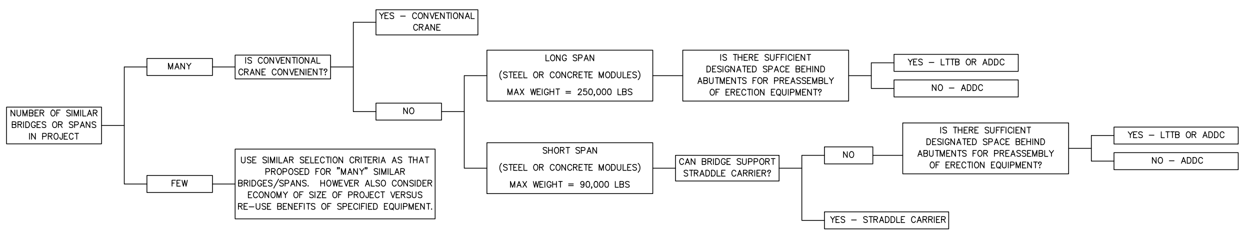
GENERAL NOTES I

HNTB OCTOBER 2011
SEA / ISU / GENESIS REV: SEPTEMBER 2012

SHEET NUMBER CC1

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.

FOR INFORMATIONAL PURPOSES ONLY

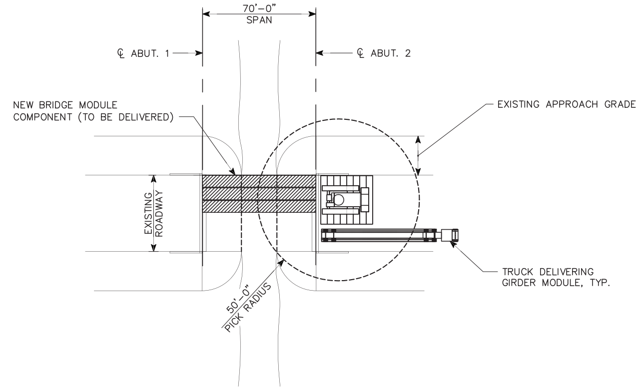
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

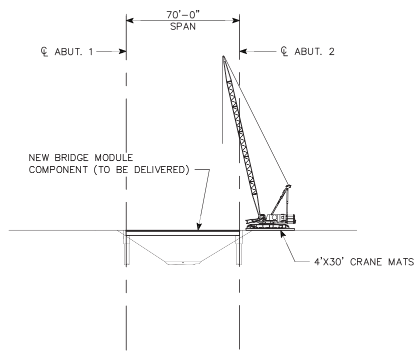
GENERAL NOTES II

HNTB — OCTOBER 2011
SEA / ISU / GENESIS — REV: SEPTEMBER 2012

SHEET NUMBER CC2



REPLACEMENT SINGLE SHORT SPAN BRIDGE – PLAN VIEW



REPLACEMENT SINGLE SHORT SPAN BRIDGE – ELEVATION VIEW

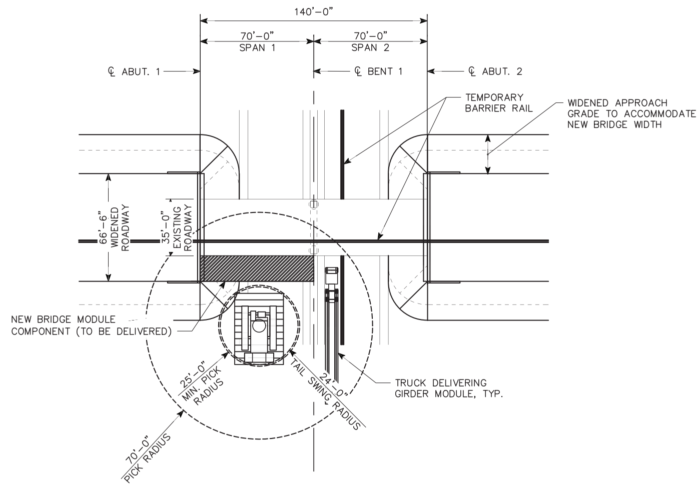
TRAFFIC DISRUPTION

EXISTING ELEVATED ROADWAY CLOSED DURING BRIDGE REPLACEMENT CONSTRUCTION OPERATIONS.
FOR ROADWAY CROSSING, LOWER ROADWAY CLOSED DURING DEMOLITION AND ERECTION OPERATIONS.

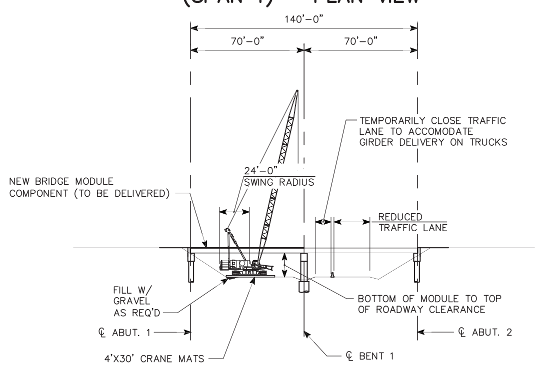
NOTES:

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.

FOR INFORMATIONAL PURPOSES ONLY
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2 PROJECT R04 INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL
ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS **CONVENTIONAL ERECTION REPLACEMENT SINGLE SHORT SPAN BRIDGE** HNTB OCTOBER 2011 SEA / ISU / GENESIS REV: SEPTEMBER 2012
SHEET NUMBER CC3



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.

FOR INFORMATIONAL PURPOSES ONLY

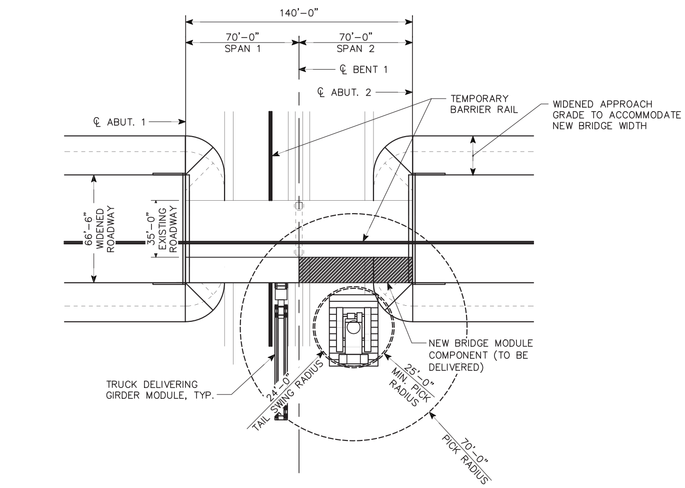
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

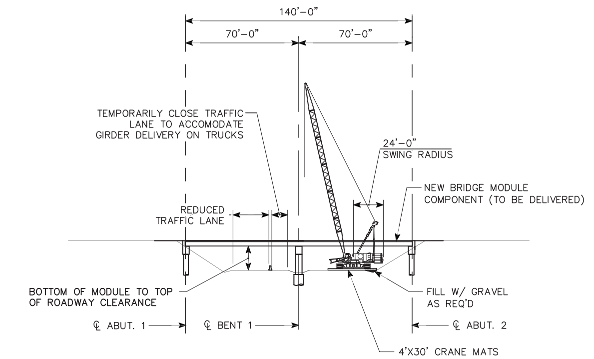
CONVENTIONAL ERECTION WIDEN SHORT SPAN BRIDGE OVER ROADWAY

HNTB — OCTOBER 2011
SEA / ISU / GENESIS — REV: SEPTEMBER 2012

SHEET NUMBER CC4



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.

FOR INFORMATIONAL PURPOSES ONLY

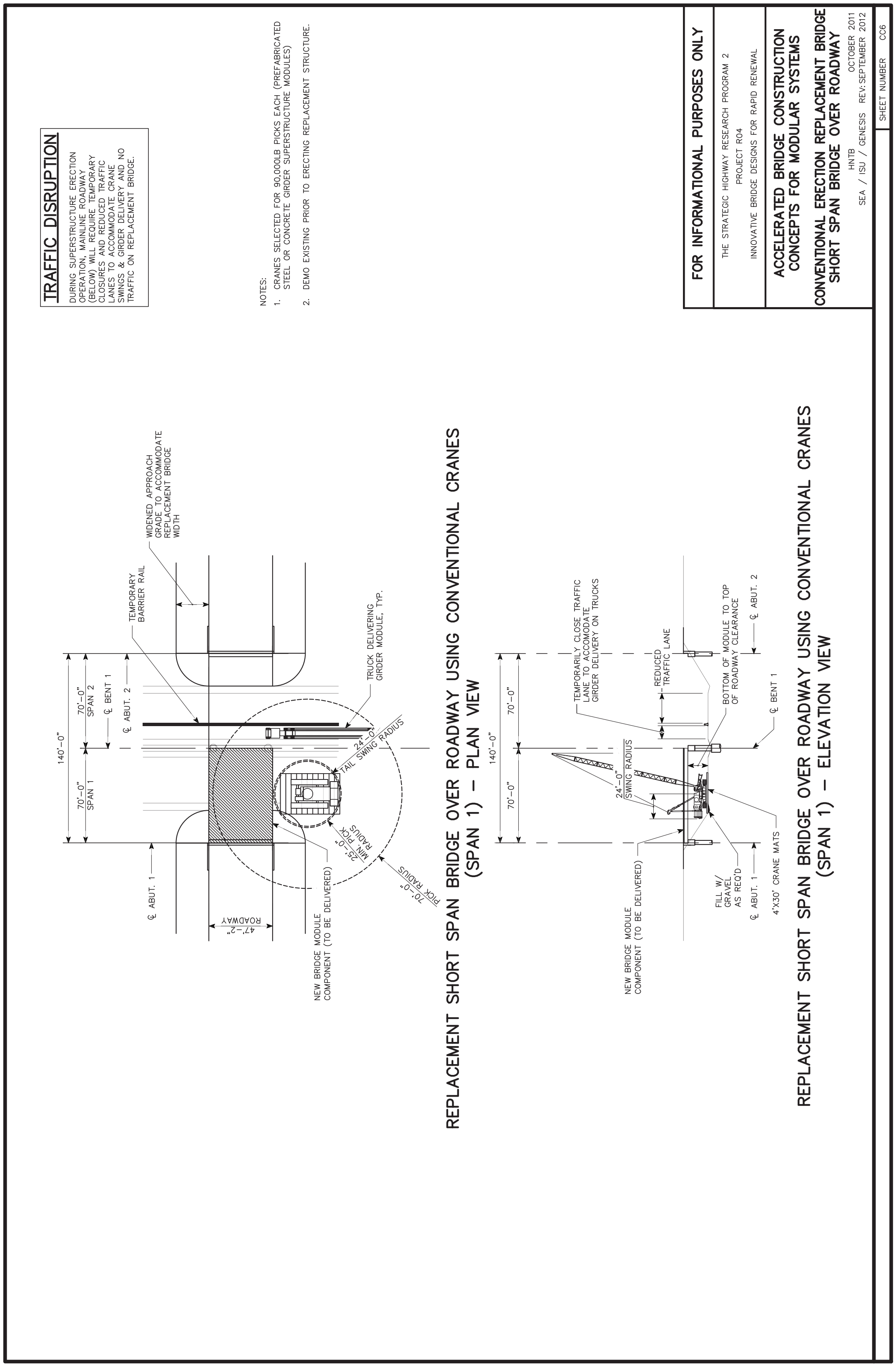
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

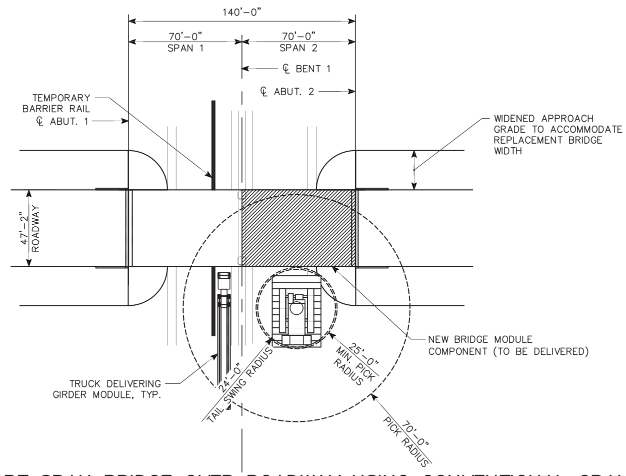
CONVENTIONAL ERECTION WIDEN SHORT SPAN BRIDGE OVER ROADWAY

HNTB OCTOBER 2011
SEA / ISU / GENESIS REV: SEPTEMBER 2012

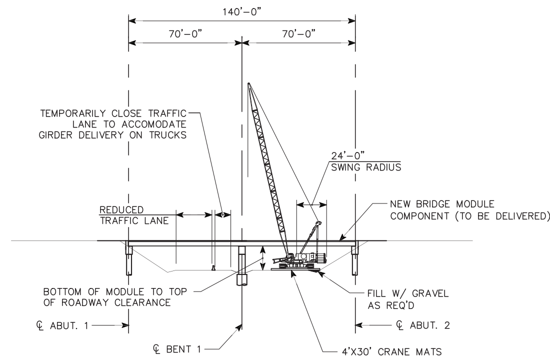
SHEET NUMBER CC5



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.
140'-0"
70'-0"
SPAN 1
70'-0"
SPAN 2
℄ BENT 1
℄ ABUT. 2
℄ ABUT. 1
TEMPORARY BARRIER RAIL
WIDENED APPROACH GRADE TO ACCOMMODATE REPLACEMENT BRIDGE WIDTH
TRUCK DELIVERING GIRDER MODULE, TYP.
24'-0"
TAIL SWING RADIUS
25'-0"
MIN. PICK RADIUS
70'-0"
PICK RADIUS
47'-2"
ROADWAY
NEW BRIDGE MODULE COMPONENT (TO BE DELIVERED)
REPLACEMENT SHORT SPAN BRIDGE OVER ROADWAY USING CONVENTIONAL CRANES (SPAN 1) – PLAN VIEW
140'-0"
70'-0"
70'-0"
24'-0"
SWING RADIUS
TEMPORARILY CLOSE TRAFFIC LANE TO ACCOMODATE GIRDER DELIVERY ON TRUCKS
REDUCED TRAFFIC LANE
BOTTOM OF MODULE TO TOP OF ROADWAY CLEARANCE
℄ ABUT. 2
℄ BENT 1
NEW BRIDGE MODULE COMPONENT (TO BE DELIVERED)
FILL W/ GRAVEL AS REQ'D
℄ ABUT. 1
4'X30' CRANE MATS
REPLACEMENT SHORT SPAN BRIDGE OVER ROADWAY USING CONVENTIONAL CRANES (SPAN 1) – ELEVATION VIEW
FOR INFORMATIONAL PURPOSES ONLY
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 ROADWAY
HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012
SHEET NUMBER CC6



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.

FOR INFORMATIONAL PURPOSES ONLY

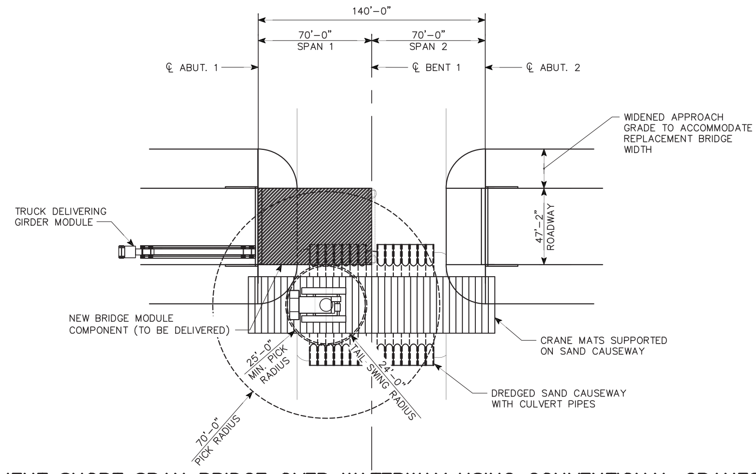
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

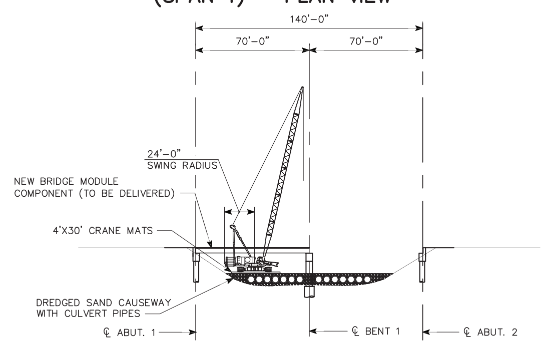
CONVENTIONAL ERECTION REPLACEMENT BRIDGE SHORT SPAN BRIDGE OVER ROADWAY

HNTB OCTOBER 2011
SEA / ISU / GENESIS REV: SEPTEMBER 2012

SHEET NUMBER CC7



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

TRAFFIC DISRUPTION

EXISTING ROADWAY CLOSED DURING BRIDGE REPLACEMENT CONSTRUCTION OPERATIONS.

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.

FOR INFORMATIONAL PURPOSES ONLY

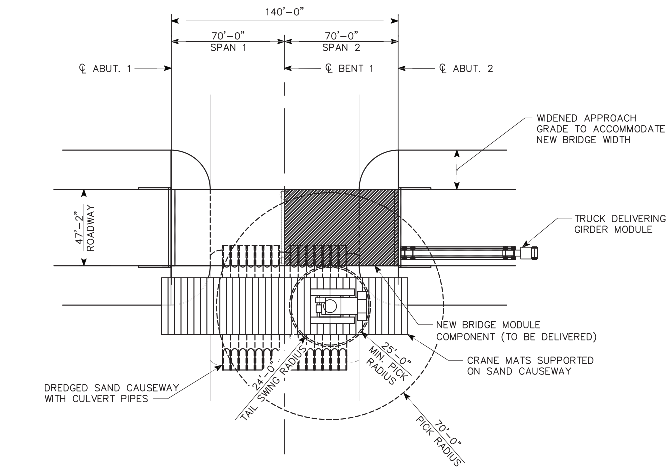
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

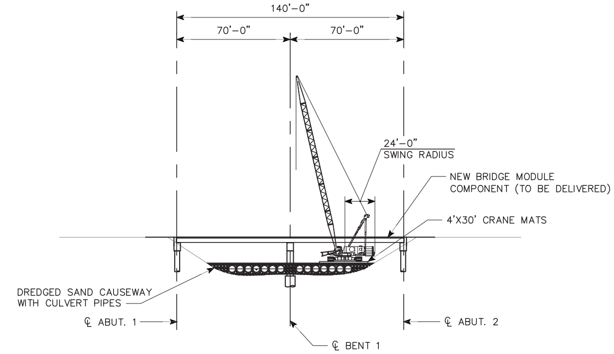
CONVENTIONAL ERECTION REPLACEMENT BRIDGE SHORT SPAN BRIDGE OVER WATERWAY (OPT 1)

HNTB OCTOBER 2011
SEA / ISU / GENESIS REV: SEPTEMBER 2012

SHEET NUMBER CC8



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

TRAFFIC DISRUPTION

EXISTING ROADWAY CLOSED DURING BRIDGE REPLACEMENT CONSTRUCTION OPERATIONS.

FOR INFORMATIONAL PURPOSES ONLY

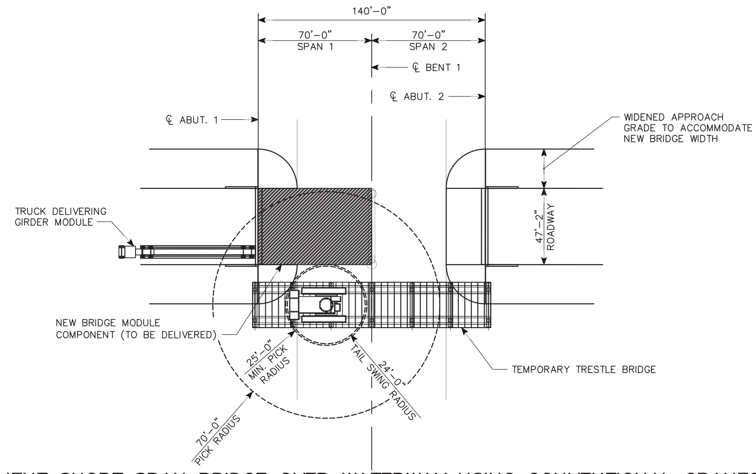
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

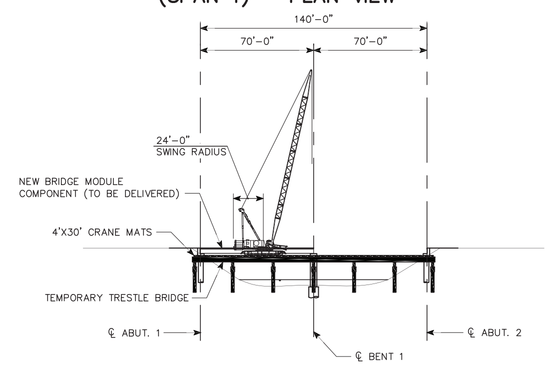
CONVENTIONAL ERECTION REPLACEMENT BRIDGE SHORT SPAN BRIDGE OVER WATERWAY (OPT 1)

HNTB OCTOBER 2011
SEA / ISU / GENESIS REV: SEPTEMBER 2012

SHEET NUMBER CC9



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

TRAFFIC DISRUPTION

EXISTING ROADWAY CLOSED DURING BRIDGE REPLACEMENT CONSTRUCTION OPERATIONS.

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.

FOR INFORMATIONAL PURPOSES ONLY

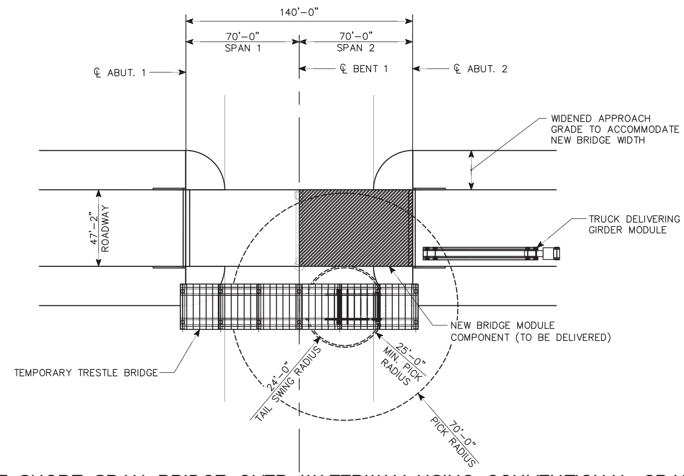
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

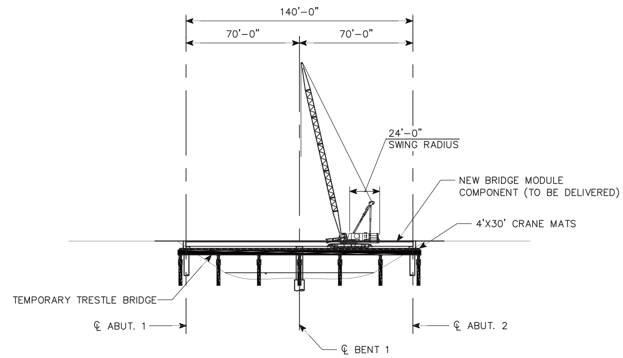
CONVENTIONAL ERECTION REPLACEMENT BRIDGE SHORT SPAN BRIDGE OVER WATERWAY (OPT 2)

HNTB OCTOBER 2011
SEA / ISU / GENESIS REV: SEPTEMBER 2012

SHEET NUMBER CC10



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

TRAFFIC DISRUPTION

EXISTING ROADWAY CLOSED DURING BRIDGE REPLACEMENT CONSTRUCTION OPERATIONS.

FOR INFORMATIONAL PURPOSES ONLY

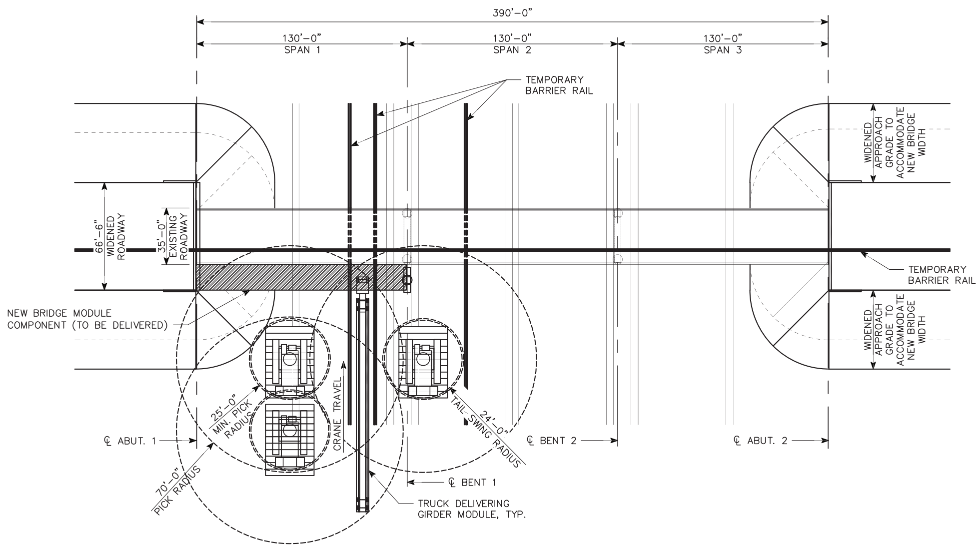
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

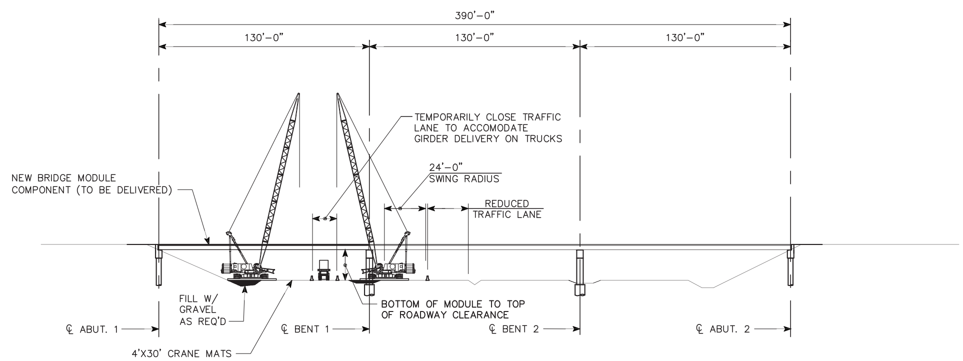
CONVENTIONAL ERECTION REPLACEMENT BRIDGE SHORT SPAN BRIDGE OVER WATERWAY (OPT 2)

HNTB OCTOBER 2011
SEA / ISU / GENESIS REV: SEPTEMBER 2012

SHEET NUMBER CC11



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.

FOR INFORMATIONAL PURPOSES ONLY

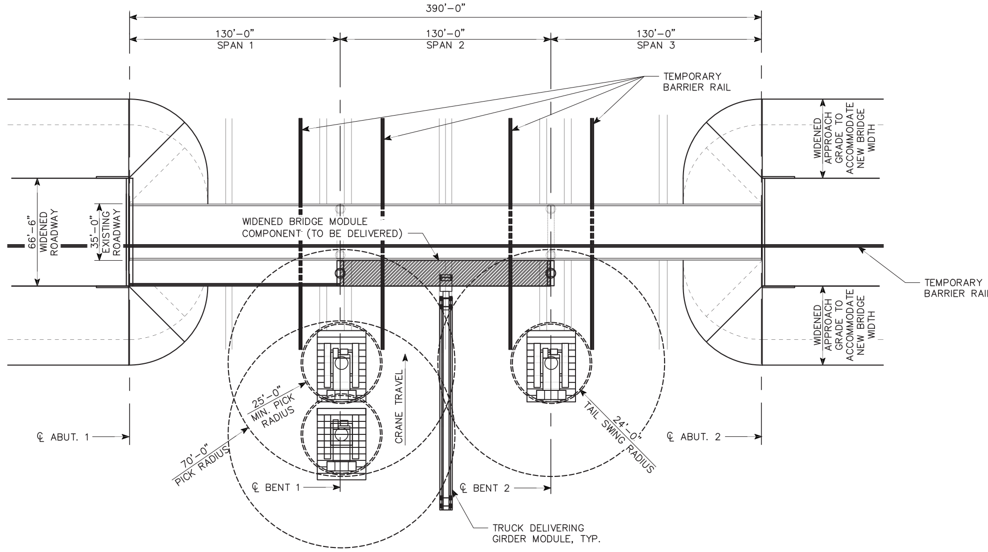
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

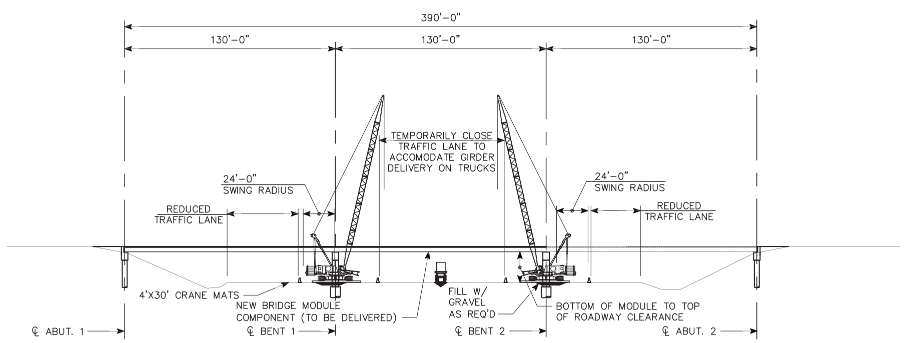
CONVENTIONAL ERECTION WIDEN LONG SPAN BRIDGE OVER ROADWAY

HNTB OCTOBER 2011
SEA / ISU / GENESIS REV: SEPTEMBER 2012

SHEET NUMBER CC12



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

FOR INFORMATIONAL PURPOSES ONLY

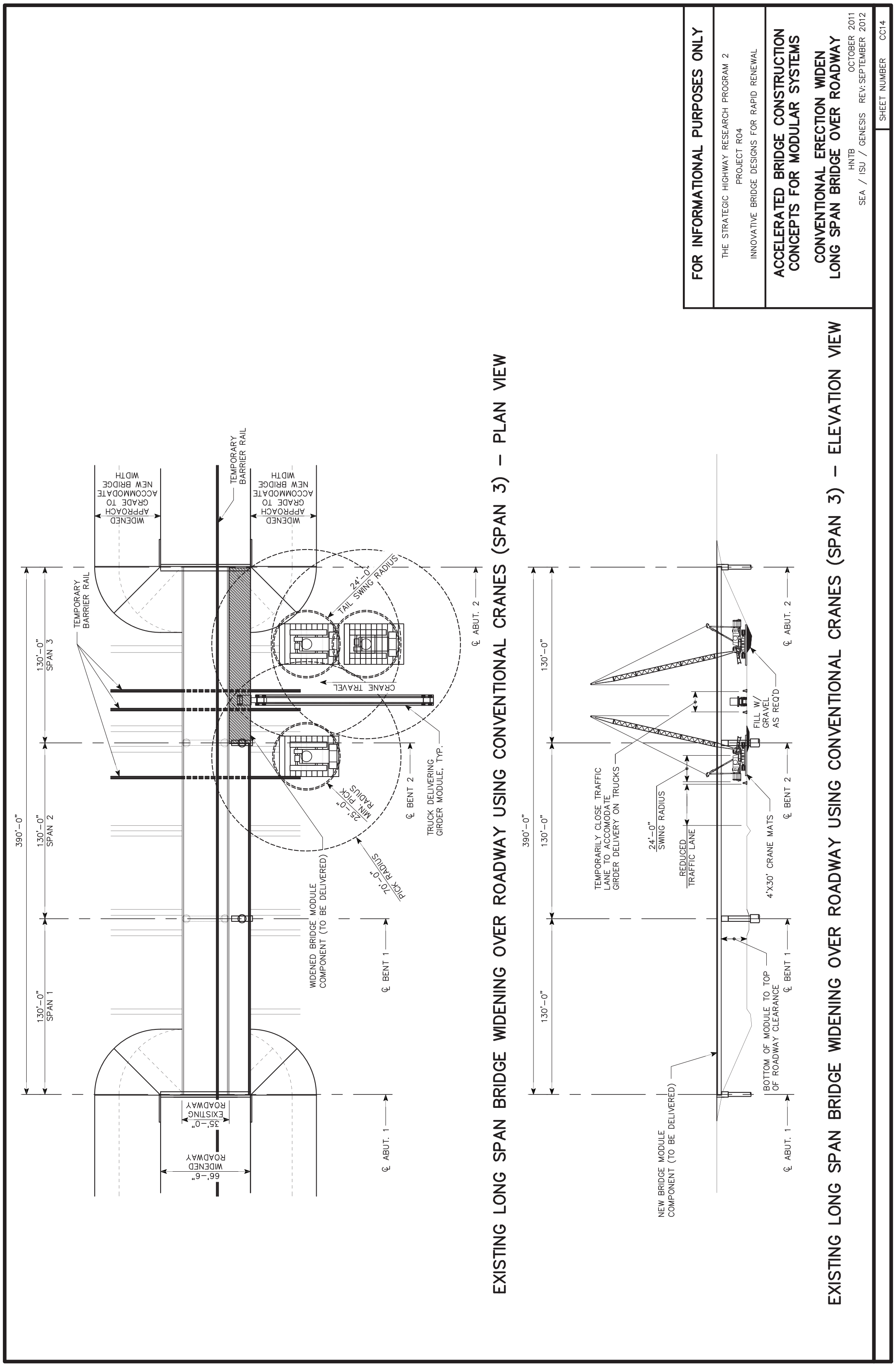
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

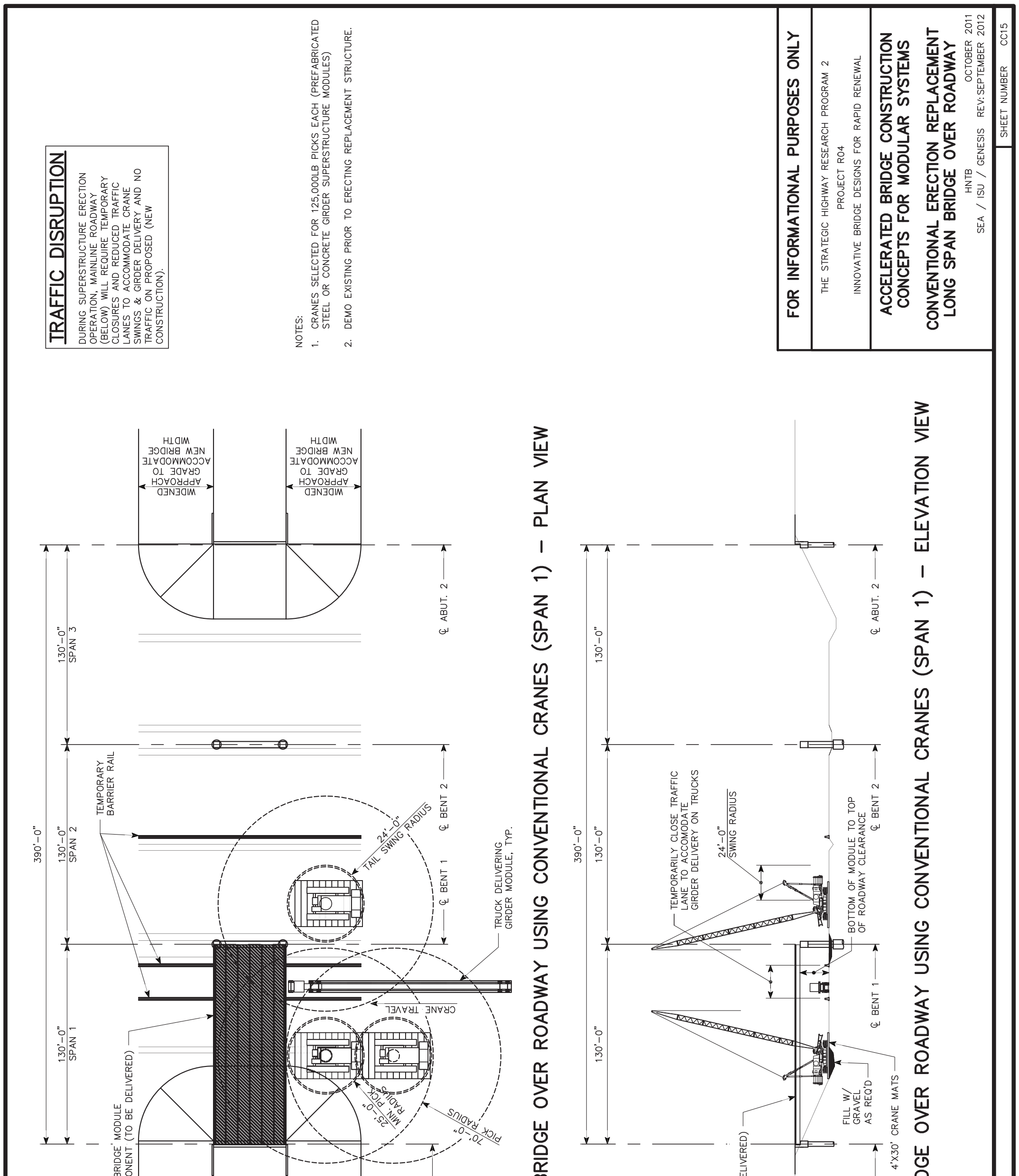
ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

CONVENTIONAL ERECTION WIDEN LONG SPAN BRIDGE OVER ROADWAY

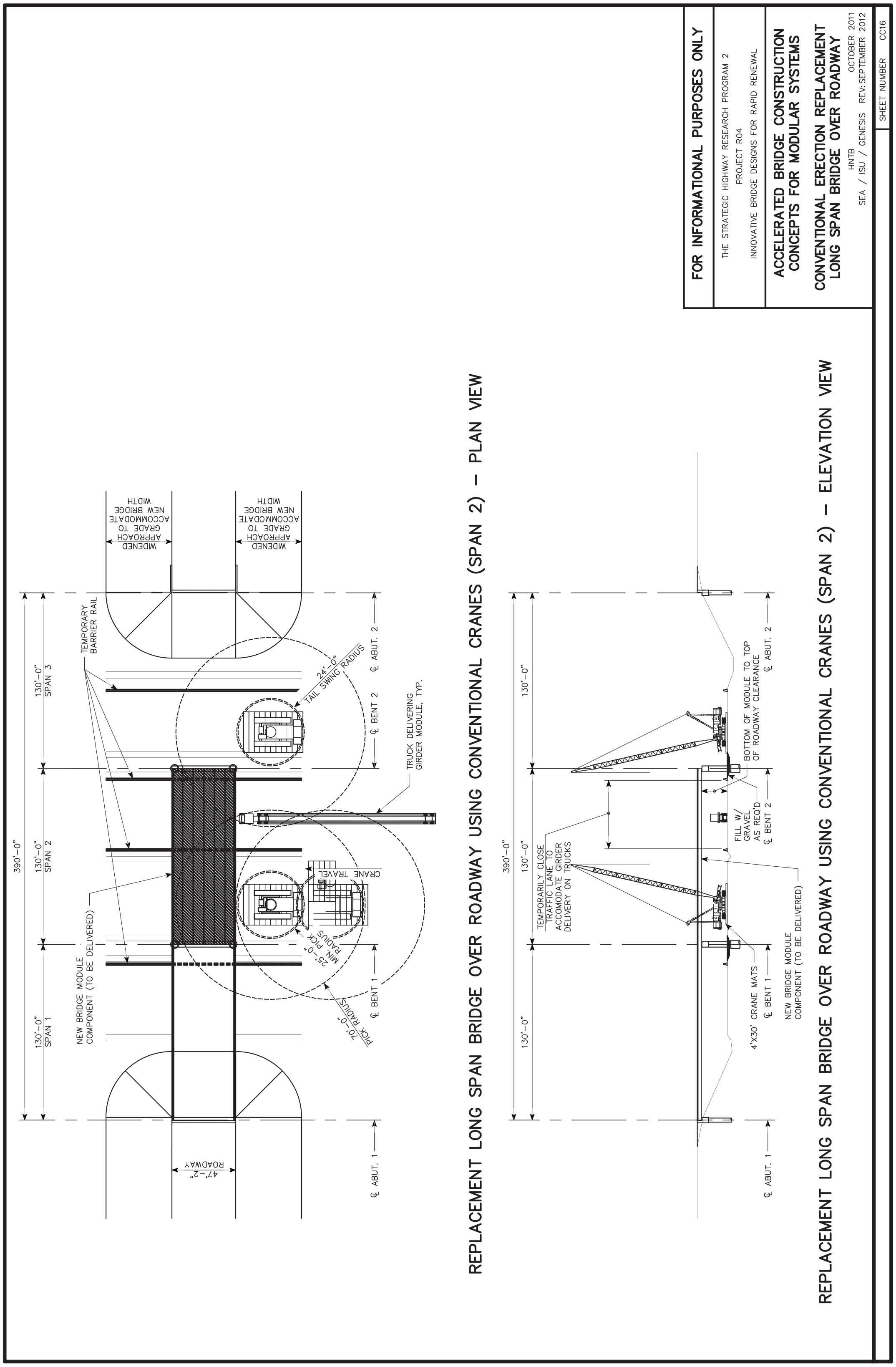
HNTB OCTOBER 2011
SEA / ISU / GENESIS REV: SEPTEMBER 2012

SHEET NUMBER CC13

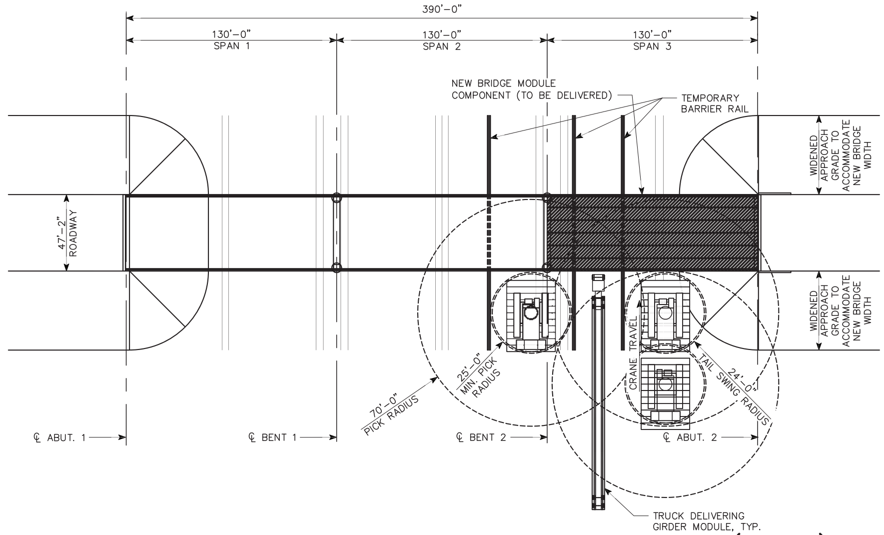

FOR INFORMATIONAL PURPOSES ONLY
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL
ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS
CONVENTIONAL ERECTION WIDEN LONG SPAN BRIDGE OVER ROADWAY
HNTB
OCTOBER 2011
SEA / ISU / GENESIS REV: SEPTEMBER 2012
SHEET NUMBER CC14
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
390'-0"
130'-0" SPAN 1
130'-0" SPAN 2
130'-0" SPAN 3
130'-0"
TEMPORARY BARRIER RAIL
WIDENED APPROACH GRADE TO ACCOMMODATE NEW BRIDGE WIDTH
24'-0" TAIL SWING RADIUS
CRANE TRAVEL
25'-0" MIN. PICK RADIUS
70'-0" PICK RADIUS
TRUCK DELIVERING GIRDER MODULE, TYP.
WIDENED BRIDGE MODULE COMPONENT (TO BE DELIVERED)
35'-0" EXISTING ROADWAY
66'-6" WIDENED ROADWAY
℄ ABUT. 1
℄ BENT 1
℄ BENT 2
℄ ABUT. 2
TEMPORARILY CLOSE TRAFFIC LANE TO ACCOMODATE GIRDER DELIVERY ON TRUCKS
24'-0" SWING RADIUS
REDUCED TRAFFIC LANE
4'X30' CRANE MATS
FILL W/ GRAVEL AS REQ'D
NEW BRIDGE MODULE COMPONENT (TO BE DELIVERED)
BOTTOM OF MODULE TO TOP OF ROADWAY CLEARANCE



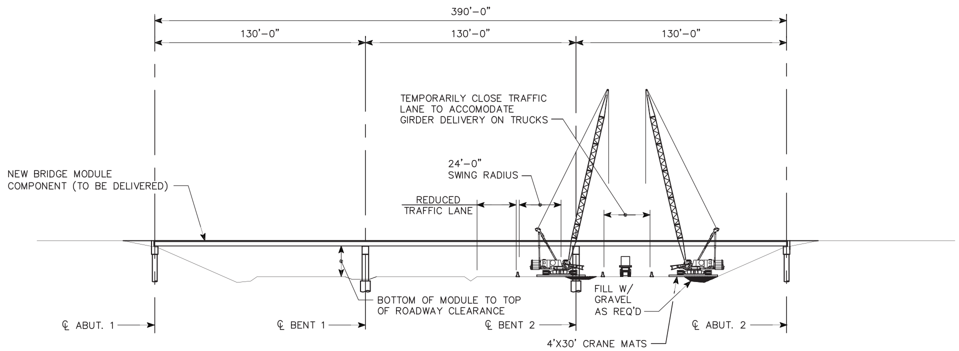
TRAFFIC DISRUPTION
DURING SUPERSTRUCTURE ERECTION OPERATION, MAINLINE ROADWAY (BELOW) WILL REQUIRE TEMPORARY CLOSURES AND REDUCED TRAFFIC LANES TO ACCOMMODATE CRANE SWINGS & GIRDER DELIVERY AND NO TRAFFIC ON PROPOSED (NEW CONSTRUCTION).
NOTES:
1. CRANES SELECTED FOR 125,000LB PICKS EACH (PREFABRICATED STEEL OR CONCRETE GIRDER SUPERSTRUCTURE MODULES)
2. DEMO EXISTING PRIOR TO ERECTING REPLACEMENT STRUCTURE.
390'-0"
130'-0" SPAN 1
130'-0" SPAN 2
130'-0" SPAN 3
TEMPORARY BARRIER RAIL
NEW BRIDGE MODULE COMPONENT (TO BE DELIVERED)
47'-2" ROADWAY
℄ ABUT. 1
℄ BENT 1
℄ BENT 2
℄ ABUT. 2
25'-0" MIN. PICK RADIUS
70'-0" PICK RADIUS
CRANE TRAVEL
24'-0" TAIL SWING RADIUS
TRUCK DELIVERING GIRDER MODULE, TYP.
WIDENED APPROACH GRADE TO ACCOMMODATE NEW BRIDGE WIDTH
REPLACEMENT LONG SPAN BRIDGE OVER ROADWAY USING CONVENTIONAL CRANES (SPAN 1) – PLAN VIEW
390'-0"
130'-0"
130'-0"
130'-0"
TEMPORARILY CLOSE TRAFFIC LANE TO ACCOMODATE GIRDER DELIVERY ON TRUCKS
24'-0" SWING RADIUS
BOTTOM OF MODULE TO TOP OF ROADWAY CLEARANCE
FILL W/ GRAVEL AS REQ'D
4'X30' CRANE MATS
NEW BRIDGE MODULE COMPONENT (TO BE DELIVERED)
℄ ABUT. 1
℄ BENT 1
℄ BENT 2
℄ ABUT. 2
REPLACEMENT LONG SPAN BRIDGE OVER ROADWAY USING CONVENTIONAL CRANES (SPAN 1) – ELEVATION VIEW
FOR INFORMATIONAL PURPOSES ONLY
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL
ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS
CONVENTIONAL ERECTION REPLACEMENT LONG SPAN BRIDGE OVER ROADWAY
HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012
SHEET NUMBER CC15



FOR INFORMATIONAL PURPOSES ONLY
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL
ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS
CONVENTIONAL ERECTION REPLACEMENT LONG SPAN BRIDGE OVER ROADWAY
HNTB OCTOBER 2011
SEA / ISU / GENESIS REV: SEPTEMBER 2012
SHEET NUMBER CC16
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
390'–0"
130'–0" SPAN 1
130'–0" SPAN 2
130'–0" SPAN 3
130'–0"
NEW BRIDGE MODULE COMPONENT (TO BE DELIVERED)
TEMPORARY BARRIER RAIL
WIDENED APPROACH GRADE TO ACCOMMODATE NEW BRIDGE WIDTH
24'–0" TAIL SWING RADIUS
TRUCK DELIVERING GIRDER MODULE, TYP.
CRANE TRAVEL
25'–0" MIN. PICK RADIUS
70'–0" PICK RADIUS
47'–2" ROADWAY
℄ ABUT. 1
℄ BENT 1
℄ BENT 2
℄ ABUT. 2
TEMPORARILY CLOSE TRAFFIC LANE TO ACCOMODATE GIRDER DELIVERY ON TRUCKS
BOTTOM OF MODULE TO TOP OF ROADWAY CLEARANCE
FILL W/ GRAVEL AS REQ'D
4'X30' CRANE MATS



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

FOR INFORMATIONAL PURPOSES ONLY

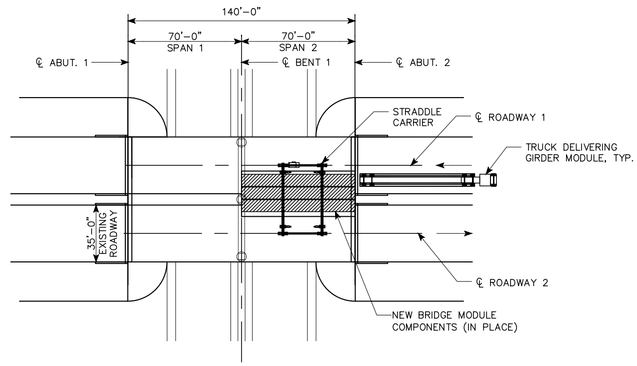
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

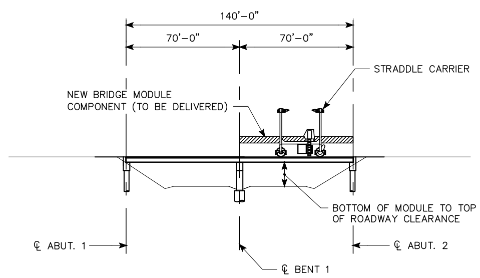
CONVENTIONAL ERECTION REPLACEMENT LONG SPAN BRIDGE OVER ROADWAY

HNTB OCTOBER 2011
SEA / ISU / GENESIS REV: SEPTEMBER 2012

SHEET NUMBER CC17



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.

FOR INFORMATIONAL PURPOSES ONLY

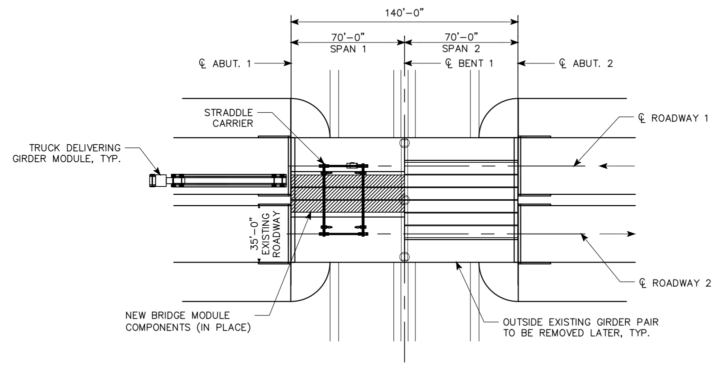
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

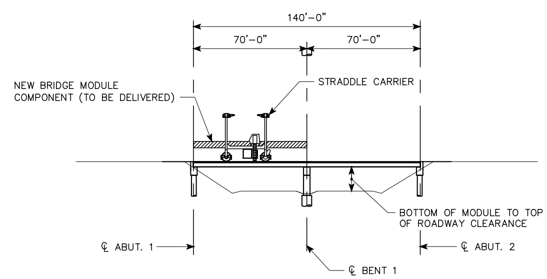
STRADDLE CARRIER ON PERMANENT BRIDGE SHORT SPAN BRIDGE

HNTB OCTOBER 2011
SEA / ISU / GENESIS REV: SEPTEMBER 2012

SHEET NUMBER CC18



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.

FOR INFORMATIONAL PURPOSES ONLY

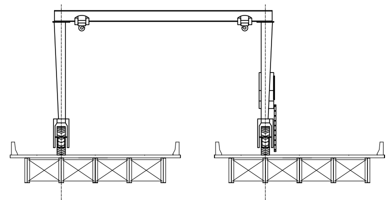
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

STRADDLE CARRIER ON PERMANENT BRIDGE SHORT SPAN BRIDGE

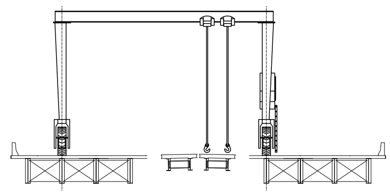
HNTB OCTOBER 2011
SEA / ISU / GENESIS REV: SEPTEMBER 2012

SHEET NUMBER CC19



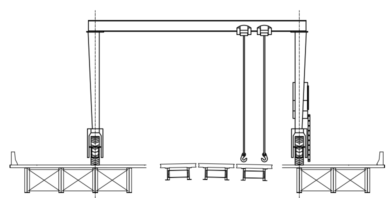
STAGE 1:

1. POSITION STRADDLE CARRIER AT ℄ EXISTING GIRDERS.



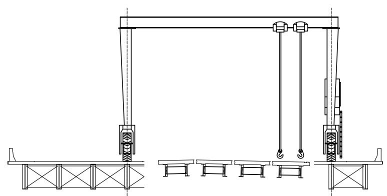
STAGE 2:

1. REMOVE EXISTING INSIDE GIRDERS AND SLAB.
2. ERECT NEW PREFABRICATED SUPERSTRUCTURE MODULES.



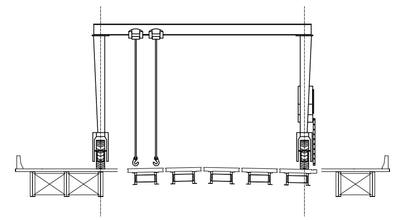
STAGE 3:

1. SHIFT STRADDLE CARRIER.
2. REMOVE EXISTING GIRDER AND REPLACE WITH NEW PREFABRICATED SUPERSTRUCTURE MODULES.



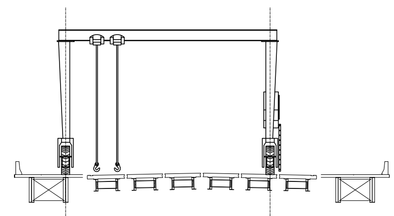
STAGE 4:

1. SHIFT STRADDLE CARRIER.
2. REMOVE EXISTING GIRDER AND REPLACE WITH NEW PREFABRICATED SUPERSTRUCTURE MODULES.



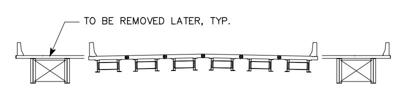
STAGE 5:

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.

FOR INFORMATIONAL PURPOSES ONLY

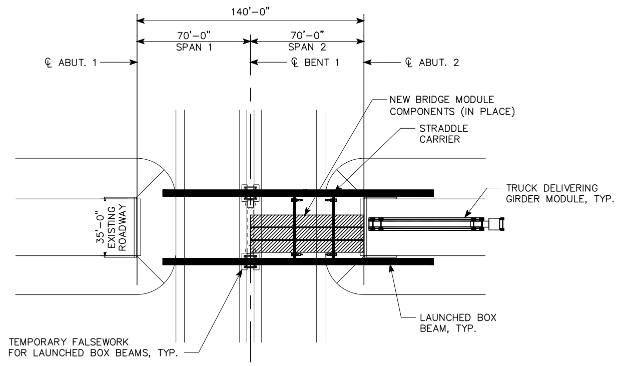
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

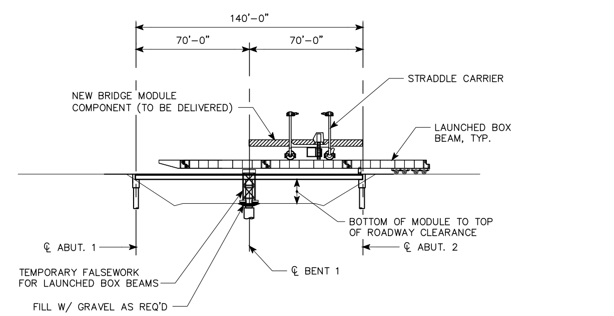
STRADDLE CARRIER ON PERMANENT BRIDGE STAGED CONSTRUCTION

HNTB OCTOBER 2011
SEA / ISU / GENESIS REV: SEPTEMBER 2012

SHEET NUMBER CC20



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.

FOR INFORMATIONAL PURPOSES ONLY

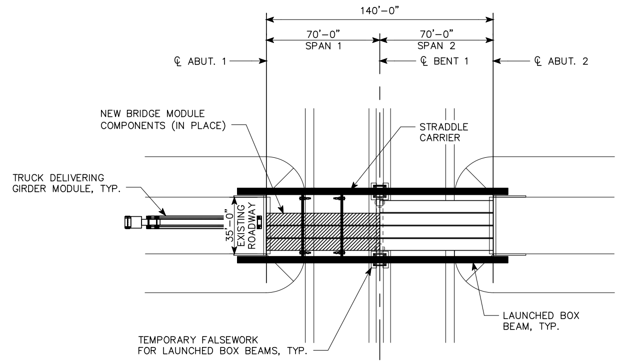
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

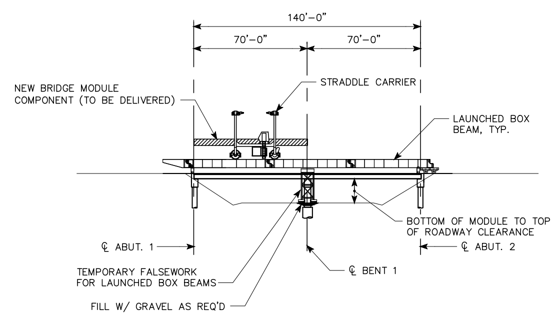
STRADDLE CARRIER ON LAUNCH BEAMS
SHORT SPAN BRIDGE

HNTB OCTOBER 2011
SEA / ISU / GENESIS REV: SEPTEMBER 2012

SHEET NUMBER CC21



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.

FOR INFORMATIONAL PURPOSES ONLY

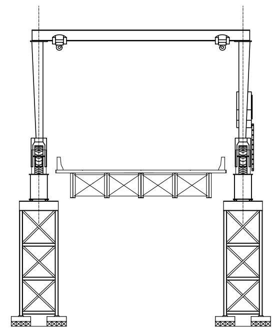
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

STRADDLE CARRIER ON LAUNCH BEAMS SHORT SPAN BRIDGE

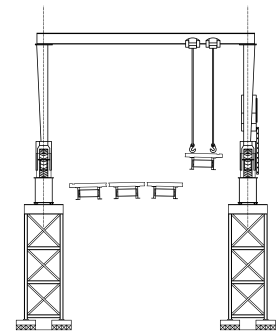
HNTB — OCTOBER 2011
SEA / ISU / GENESIS — REV: SEPTEMBER 2012

SHEET NUMBER CC22



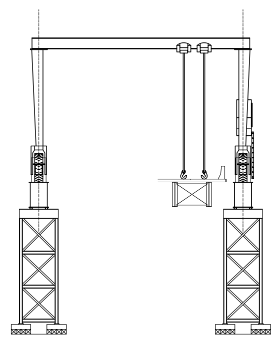
STAGE 1:

1. LAUNCH BOX BEAMS AND POSITION STRADDLE CARRIER.



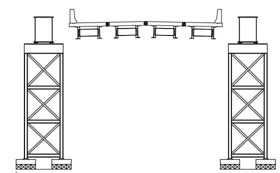
STAGE 3:

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.

FOR INFORMATIONAL PURPOSES ONLY

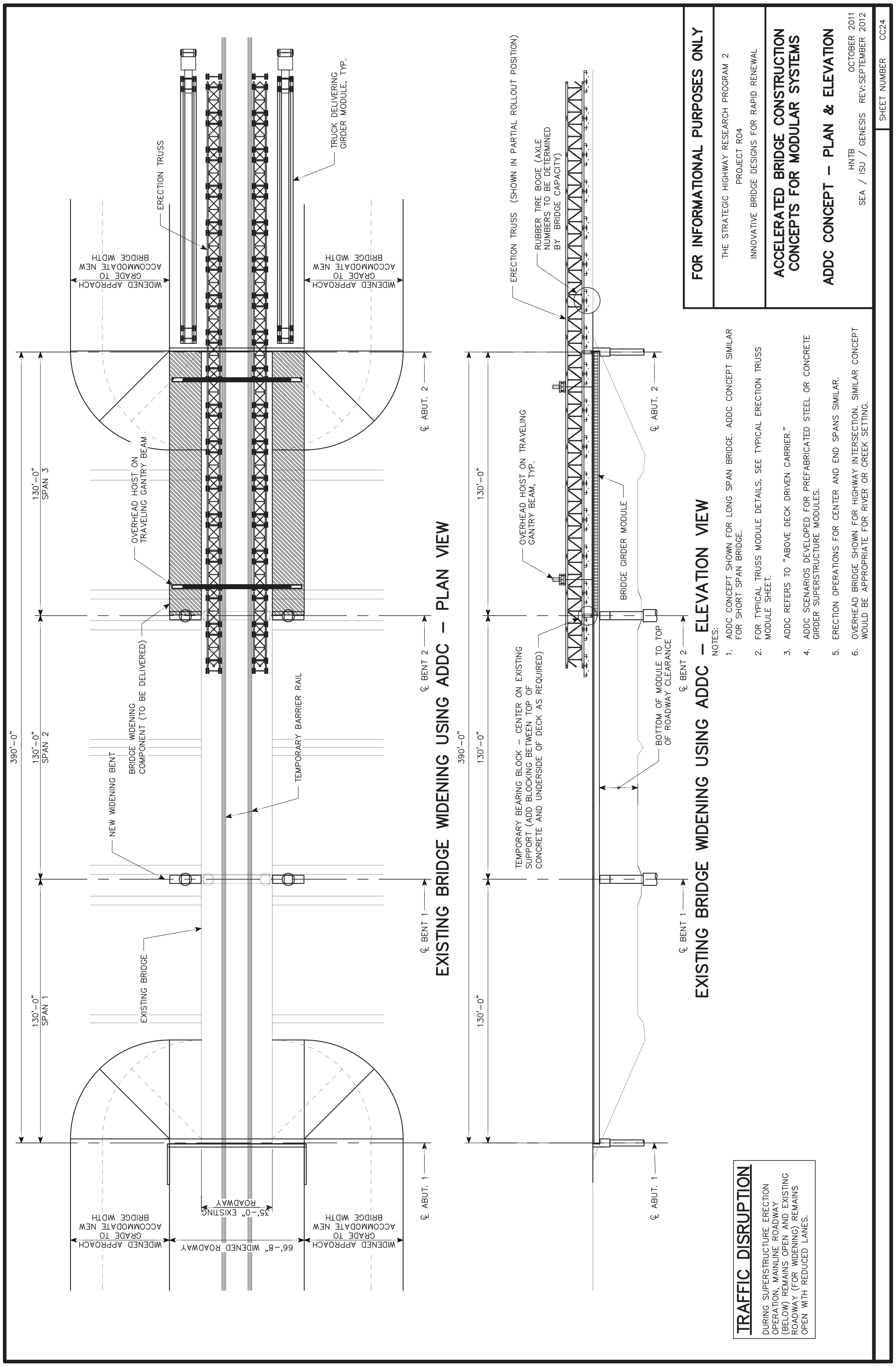
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

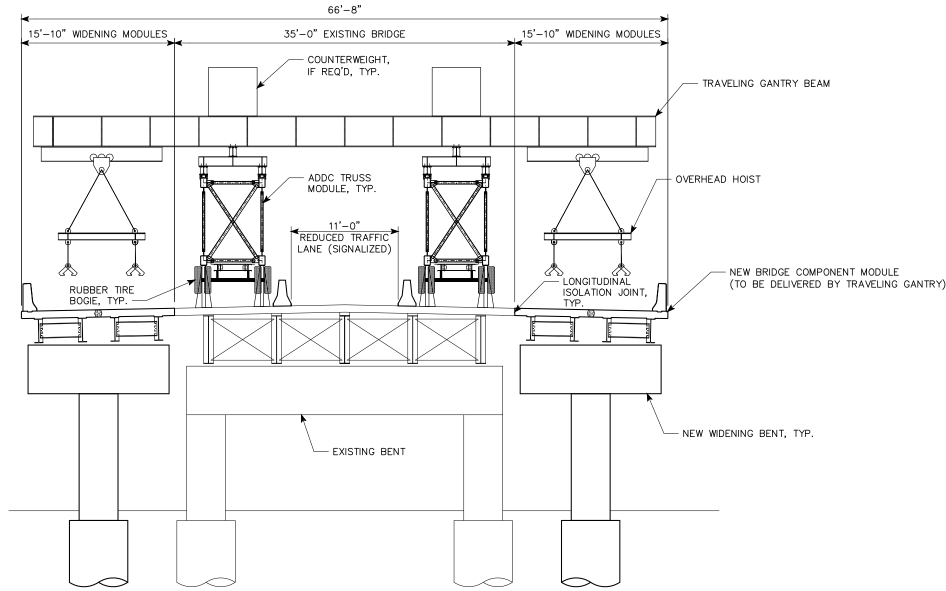
STRADDLE CARRIER ON LAUNCH BEAM STAGED CONSTRUCTION

HNTB OCTOBER 2011
SEA / ISU / GENESIS REV: SEPTEMBER 2012

SHEET NUMBER CC23



FOR INFORMATIONAL PURPOSES ONLY
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
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012
SHEET NUMBER CC24
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.
390'-0"
130'-0" SPAN 1
130'-0" SPAN 2
130'-0" SPAN 3
130'-0"
℄ ABUT. 1
℄ BENT 1
℄ BENT 2
℄ ABUT. 2
WIDENED APPROACH GRADE TO ACCOMMODATE NEW BRIDGE WIDTH
66'-8" WIDENED ROADWAY
35'-0" EXISTING ROADWAY
EXISTING BRIDGE
NEW WIDENING BENT
BRIDGE WIDENING COMPONENT (TO BE DELIVERED)
TEMPORARY BARRIER RAIL
OVERHEAD HOIST ON TRAVELING GANTRY BEAM
ERECTION TRUSS
TRUCK DELIVERING GIRDER MODULE, TYP.
ERECTION TRUSS (SHOWN IN PARTIAL ROLLOUT POSITION)
RUBBER TIRE BOGIE (AXLE NUMBERS TO BE DETERMINED BY BRIDGE CAPACITY)
OVERHEAD HOIST ON TRAVELING GANTRY BEAM, TYP.
BRIDGE GIRDER MODULE
TEMPORARY BEARING BLOCK – CENTER ON EXISTING SUPPORT (ADD BLOCKING BETWEEN TOP OF CONCRETE AND UNDERSIDE OF DECK AS REQUIRED)
BOTTOM OF MODULE TO TOP OF ROADWAY CLEARANCE



EXISTING BRIDGE WIDENING USING ADDC – TYPICAL CROSS SECTION

FOR INFORMATIONAL PURPOSES ONLY

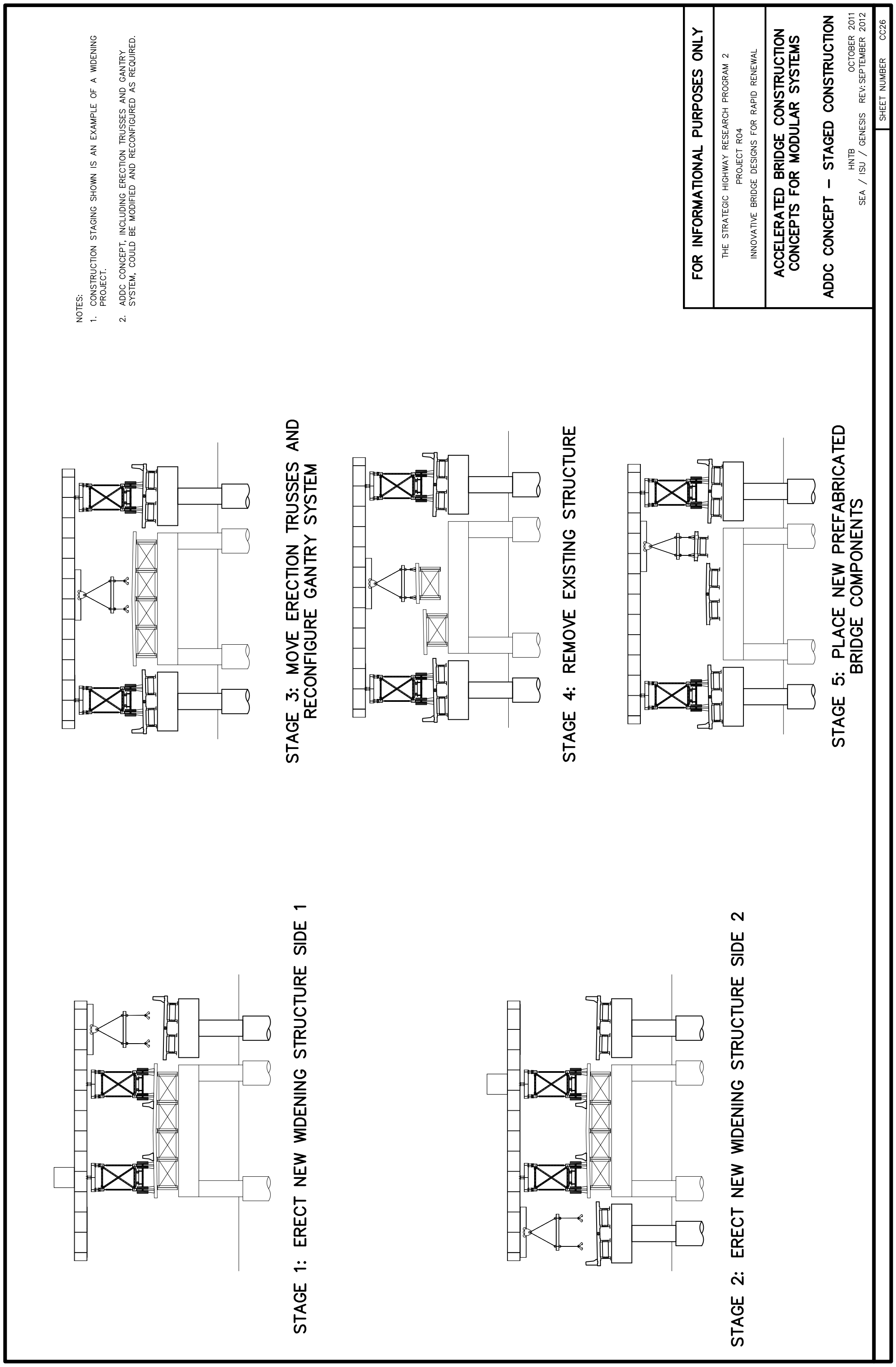
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

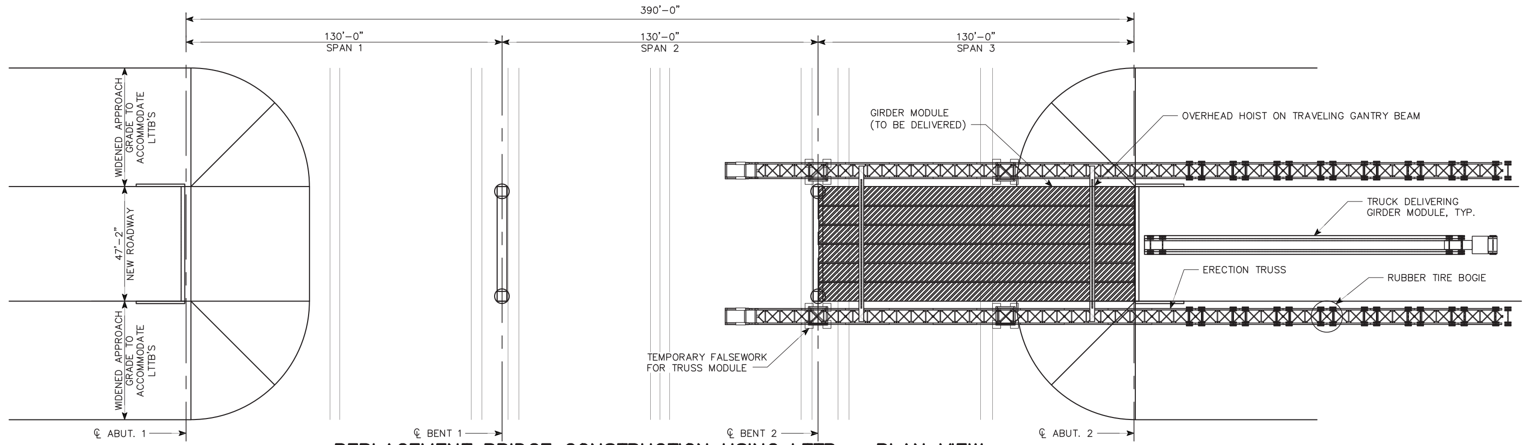
ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

ADDC CONCEPT – TYPICAL CROSS SECTION

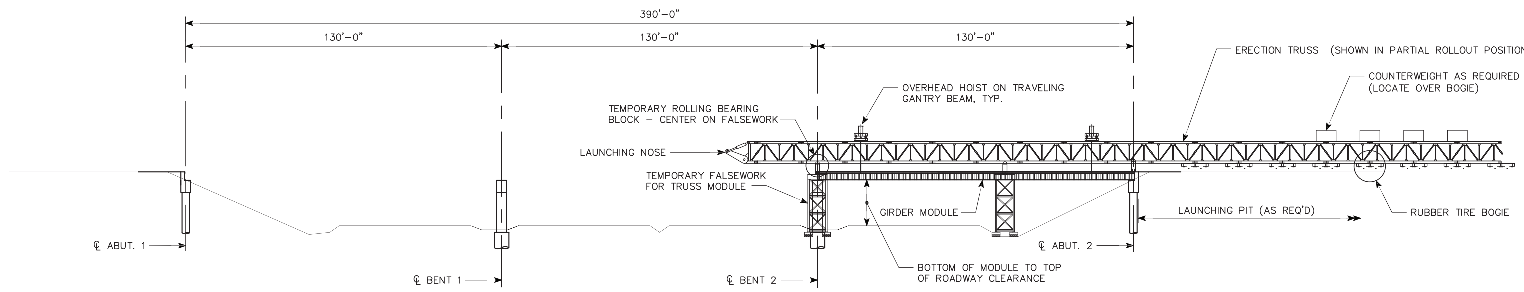
HNTB OCTOBER 2011
SEA / ISU / GENESIS REV: SEPTEMBER 2012

SHEET NUMBER CC25


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.
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
FOR INFORMATIONAL PURPOSES ONLY
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
HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012
SHEET NUMBER
CC26



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.

FOR INFORMATIONAL PURPOSES ONLY

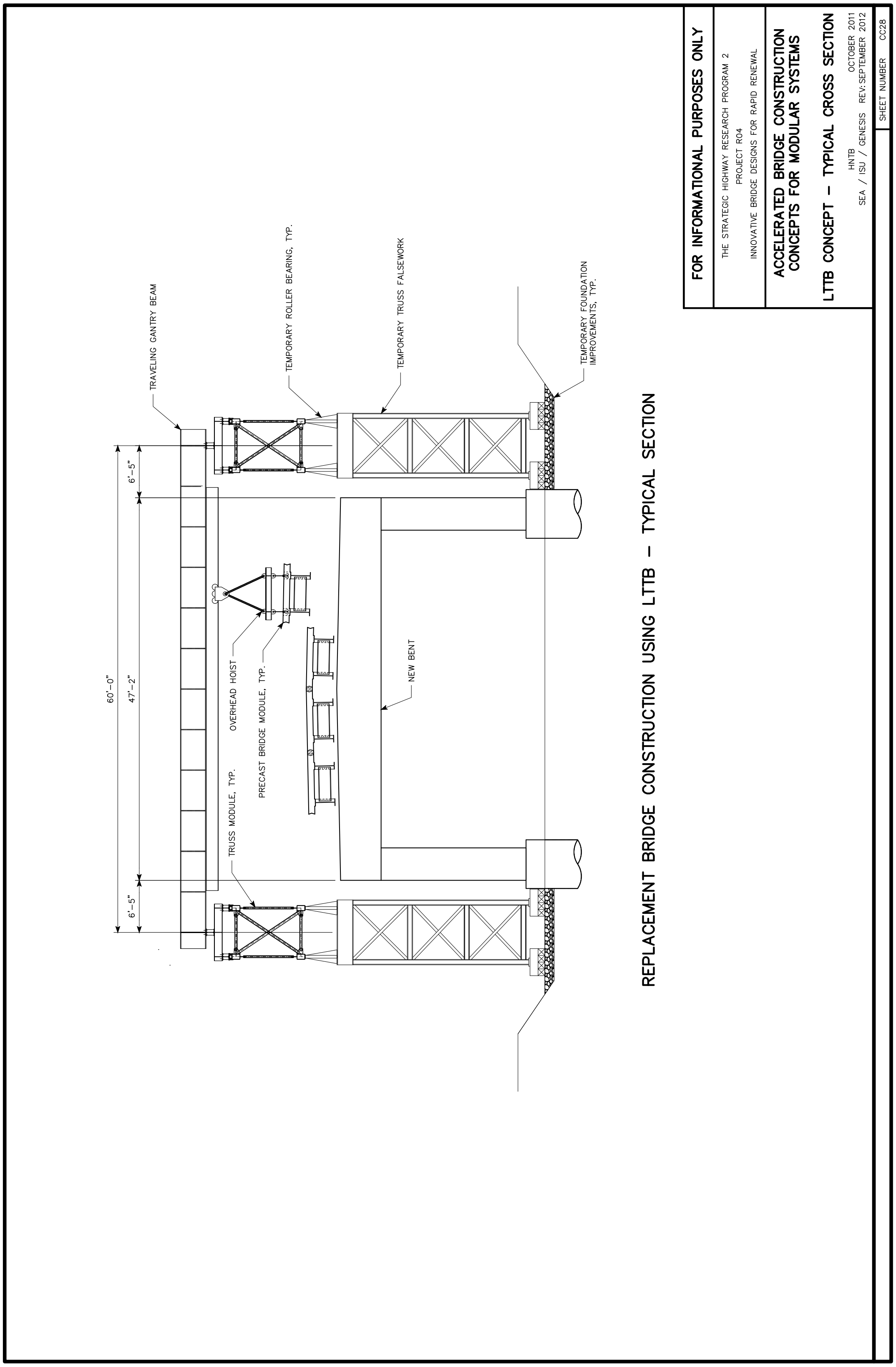
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

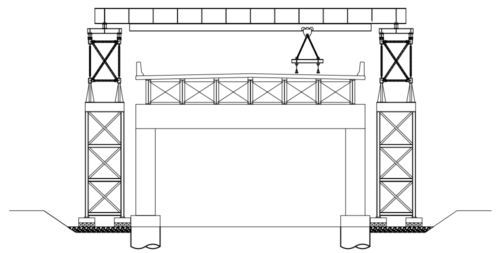
ACCELERATED BRIDGE CONSTRUCTION CONCEPTS FOR MODULAR SYSTEMS

LTTB CONCEPT – PLAN & ELEVATION

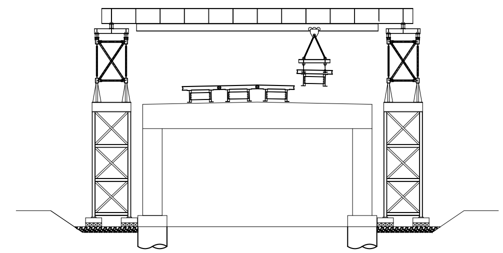
HNTB — OCTOBER 2011
SEA / ISU / GENESIS — REV: SEPTEMBER 2012

SHEET NUMBER CC27

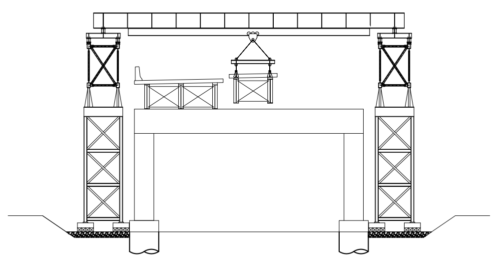

FOR INFORMATIONAL PURPOSES ONLY
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL
ACCELERATED BRIDGE CONSTRUCTION
CONCEPTS FOR MODULAR SYSTEMS
LTTB CONCEPT – TYPICAL CROSS SECTION
HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012
SHEET NUMBER
CC28
REPLACEMENT BRIDGE CONSTRUCTION USING LTTB – TYPICAL SECTION
TRAVELING GANTRY BEAM
TEMPORARY ROLLER BEARING, TYP.
TEMPORARY TRUSS FALSEWORK
TEMPORARY FOUNDATION
IMPROVEMENTS, TYP.
NEW BENT
OVERHEAD HOIST
PRECAST BRIDGE MODULE, TYP.
TRUSS MODULE, TYP.
60'-0"
6'-5"
47'-2"
6'-5"



STAGE 1: LAUNCH ERECTION TRUSS & ASSEMBLE ROLLING GANTRY SYSTEM



STAGE 3: REPLACE WITH NEW STRUCTURE



STAGE 2: REMOVE EXISTING STRUCTURE

FOR INFORMATIONAL PURPOSES ONLY

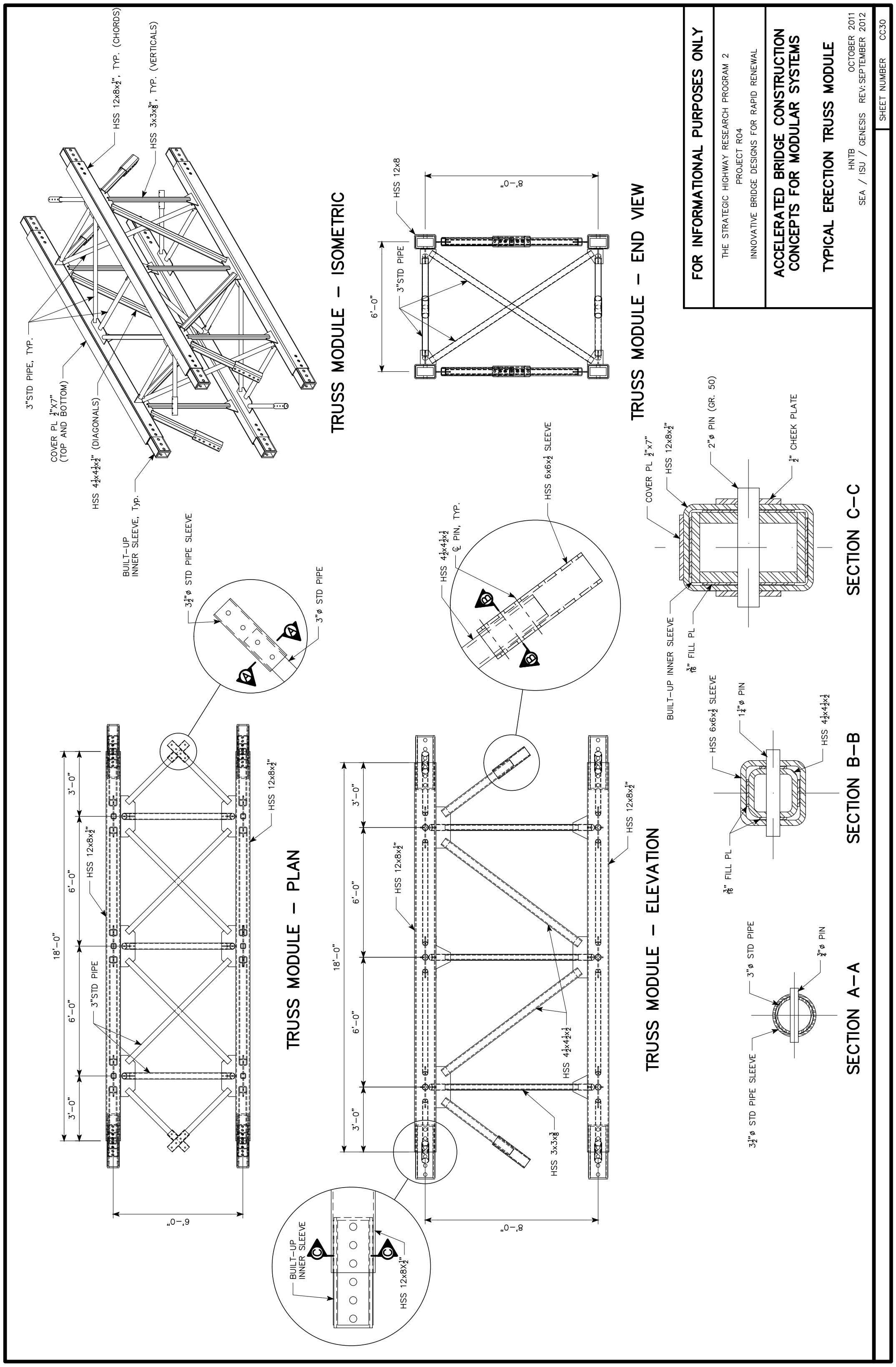
THE STRATEGIC HIGHWAY RESEARCH PROGRAM 2
PROJECT R04
INNOVATIVE BRIDGE DESIGNS FOR RAPID RENEWAL

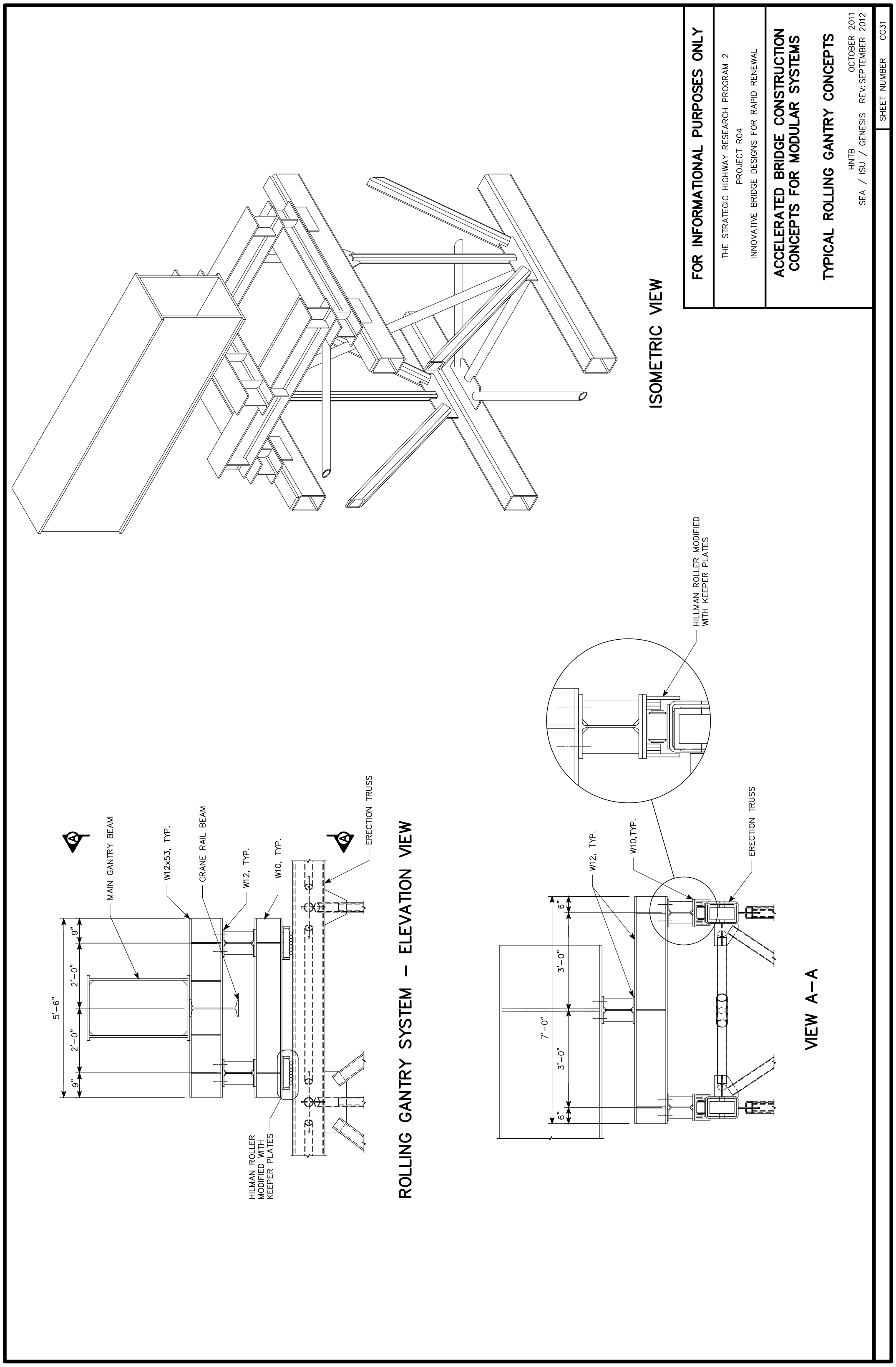
ACCELERATED BRIDGE CONSTRUCTION
CONCEPTS FOR MODULAR SYSTEMS

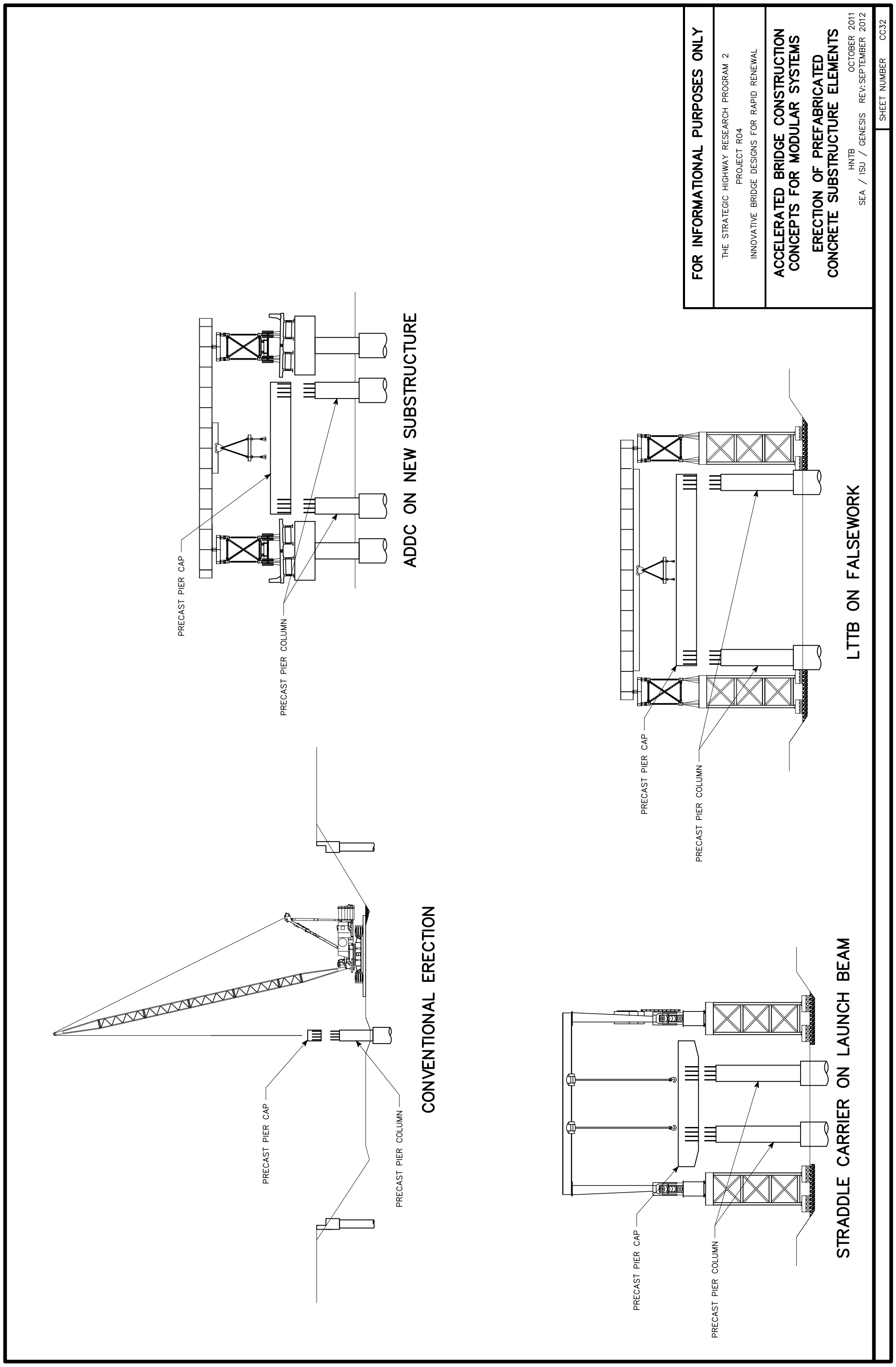
LTTB CONCEPT – STAGED CONSTRUCTION

HNTB OCTOBER 2011
SEA / ISU / GENESIS REV: SEPTEMBER 2012

SHEET NUMBER CC29


FOR INFORMATIONAL PURPOSES ONLY
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
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012
SHEET NUMBER
CC30
TRUSS MODULE – ISOMETRIC
HSS 12x8x½", TYP. (CHORDS)
HSS 3x3x⅜", TYP. (VERTICALS)
3"STD PIPE, TYP.
COVER PL ½"x7" (TOP AND BOTTOM)
HSS 4½x4½x½" (DIAGONALS)
BUILT-UP INNER SLEEVE, Typ.
TRUSS MODULE – END VIEW
HSS 12x8
3"STD PIPE
6'-0"
8'-0"
TRUSS MODULE – PLAN
18'-0"
3'-0"
6'-0"
6'-0"
3'-0"
HSS 12x8x½"
3"STD PIPE
6'-0"
3½"φ STD PIPE SLEEVE
3"φ STD PIPE
TRUSS MODULE – ELEVATION
18'-0"
3'-0"
6'-0"
6'-0"
3'-0"
HSS 12x8x½"
HSS 4½x4½x½
HSS 3x3x⅜
8'-0"
HSS 4½x4½x½
℄ PIN, TYP.
HSS 6x6x½ SLEEVE
BUILT-UP INNER SLEEVE
HSS 12x8x½"
SECTION A–A
3½"φ STD PIPE SLEEVE
3"φ STD PIPE
¾"φ PIN
SECTION B–B
HSS 6x6x½ SLEEVE
1¼"φ PIN
HSS 4½x4½x½
³⁄₁₆" FILL PL
SECTION C–C
COVER PL ½"x7"
HSS 12x8x½"
2"φ PIN (GR. 50)
½" CHEEK PLATE
BUILT-UP INNER SLEEVE
³⁄₁₆" FILL PL


FOR INFORMATIONAL PURPOSES ONLY
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
HNTB
SEA / ISU / GENESIS
OCTOBER 2011
REV: SEPTEMBER 2012
SHEET NUMBER
CC31
ISOMETRIC VIEW
HILLMAN ROLLER MODIFIED WITH KEEPER PLATES
MAIN GANTRY BEAM
W12x53, TYP.
CRANE RAIL BEAM
W12, TYP.
W10, TYP.
ERECTION TRUSS
HILMAN ROLLER MODIFIED WITH KEEPER PLATES
5'-6"
9"
2'-0"
2'-0"
9"
ROLLING GANTRY SYSTEM – ELEVATION VIEW
W12, TYP.
W10,TYP.
ERECTION TRUSS
6"
3'-0"
7'-0"
3'-0"
6"
VIEW A–A


FOR INFORMATIONAL PURPOSES ONLY
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
HNTB
OCTOBER 2011
SEA / ISU / GENESIS
REV: SEPTEMBER 2012
SHEET NUMBER
CC32
PRECAST PIER CAP
PRECAST PIER COLUMN
ADDC ON NEW SUBSTRUCTURE
PRECAST PIER CAP
PRECAST PIER COLUMN
LTTB ON FALSEWORK
PRECAST PIER CAP
PRECAST PIER COLUMN
CONVENTIONAL ERECTION
PRECAST PIER CAP
PRECAST PIER COLUMN
STRADDLE CARRIER ON LAUNCH BEAM

ABC SAMPLE DESIGN CALCULATIONS

APPENDIX B

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.

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

ABC SAMPLE CALCULATION – 1

Decked Steel Girder Design for ABC

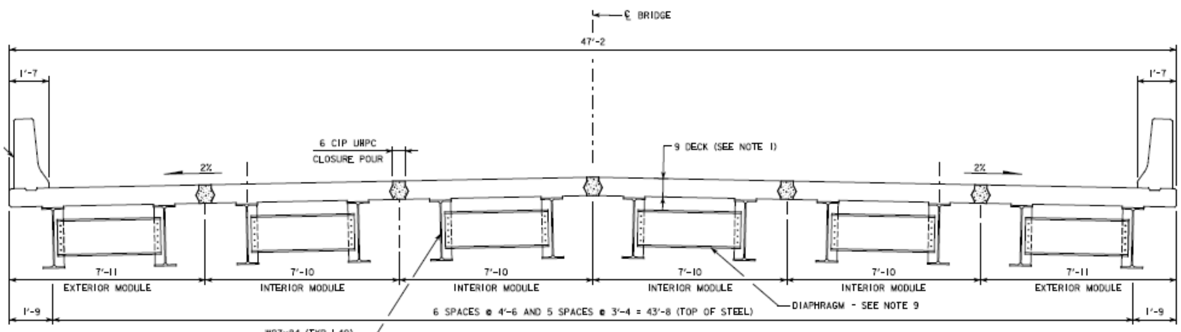
CONCRETE DECKED STEEL GIRDER DESIGN FOR ABC

This document shows the procedure for the design of a steel girder bridge with precast deck element for use in a rapid bridge replacement design in Accelerated Bridge Construction (ABC). 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:



TYPICAL SECTION
(L70 SHOWN; L40, L100 AND L130 SIMILAR)

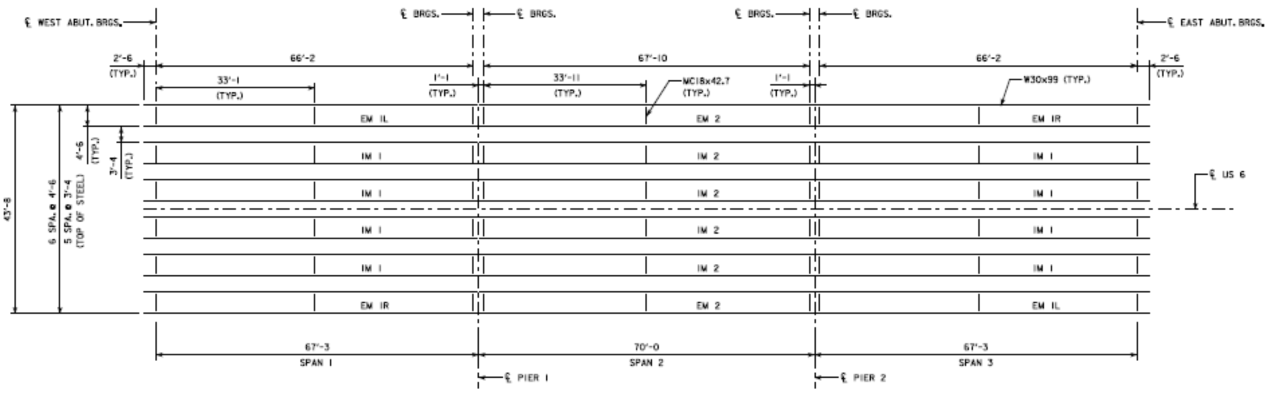
Design member parameters:

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:	$L_{total} := 3 \cdot Length = 210\ ft$
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	$L_{str} := Length - 6 \cdot in = 69.5\ ft$
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 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 quicly locate inputs amongst other data on screen. Units are user inputs.

Height of Structure: $H_{structure} := 25ft$

Example 2: Equations are typed directly into the workspace. Mathcad then reads the operators and executes the calculations.

Panels are 2.5' $N_{panels} := \frac{H_{structure}}{2.5ft}$ $N_{panels} = 10$

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

$$\text{Discount} := \begin{cases} .75 & \text{if } N_{panels} \geq 6 \\ .55 & \text{if } N_{panels} \geq 10 \\ 1 & \text{otherwise} \end{cases} \qquad \text{Discount} = 0.6$$

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%

$$\text{Discount} \leq .55 = 1$$

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 Desing Specifications (5th Edition with 2010 interims)

Design Methodology: Load and Resistance Factor Design (LRFD)

Live Load Requirements: HL-93 S S3.6

Section Constraints:

$W_{mod.max} := 200 \cdot kip$ 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:	$E_c := 33000\cdot\left(\frac{w_c}{1000pcf}\right)^{1.5}\cdot\sqrt{f_c\cdot ksi} = 4286.8\cdot ksi$		
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%

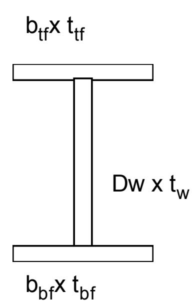
Service II = 1.0DC + 1.0DW + 1.3(LL+IM), where IM = 33%

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 Requeirements Met: S 6.10.2.2

$$\frac{b_{tf}}{2 \cdot t_{tf}} \le 12.0 = 1$$

$$b_{tf} \ge \frac{D_w}{6} = 1$$

$$t_{tf} \ge 1.1 \cdot t_w = 1$$

$$0.1 \le \frac{\frac{t_{bf}^3 \cdot b_{bf}}{12}}{\frac{t_{tf}^3 \cdot b_{tf}}{12}} \le 10 = 1$$

$$\frac{b_{bf}}{2 \cdot t_{bf}} \le 12.0 = 1$$

$$b_{bf} \ge \frac{D_w}{6} = 1$$

$$t_{bf} \ge 1.1 \cdot t_w = 1$$

$$\frac{\frac{t_{bf} \cdot b_{bf}}{12}}{\frac{t_{tf} \cdot b_{tf}}{12}} \ge 0.3 = 1$$

Properties for use when analyzing under beam self weight (steel only):

$A_{tf} := b_{tf} \cdot t_{tf}$ $\quad A_{bf} := b_{bf} \cdot t_{bf}$ $\quad A_w := D_w \cdot t_w$

$A_{steel} := A_{bf} + A_{tf} + A_w$ $\quad A_{steel} = 28.7 \cdot in^2$ $\quad$ Total steel area.

$$y_{steel} := \frac{A_{tf} \cdot \frac{t_{tf}}{2} + A_{bf} \cdot \left(\frac{t_{bf}}{2} + D_w + t_{tf}\right) + A_w \cdot \left(\frac{D_w}{2} + t_{tf}\right)}{A_{steel}} \qquad y_{steel} = 14.8 \cdot in$$

Steel centroid from top.

Calculate Iz: Moment of inertia about Z axis.

$$I_{zsteel} := \frac{t_w \cdot D_w^3}{12} + \frac{b_{tf} \cdot t_{tf}^3}{12} + \frac{b_{bf} \cdot t_{bf}^3}{12} + A_w \cdot \left(\frac{D_w}{2} + t_{tf} - y_{steel}\right)^2 + A_{tf} \cdot \left(y_{steel} - \frac{t_{tf}}{2}\right)^2 + A_{bf} \cdot \left(D_w + \frac{t_{bf}}{2} + t_{tf} - y_{steel}\right)^2$$

Calculate Iy:

$$I_{ysteel} := \frac{D_w \cdot t_w^3 + t_{tf} \cdot b_{tf}^3 + t_{bf} \cdot b_{bf}^3}{12}$$

Moment of inertia about Y axis.

Calculate Ix:

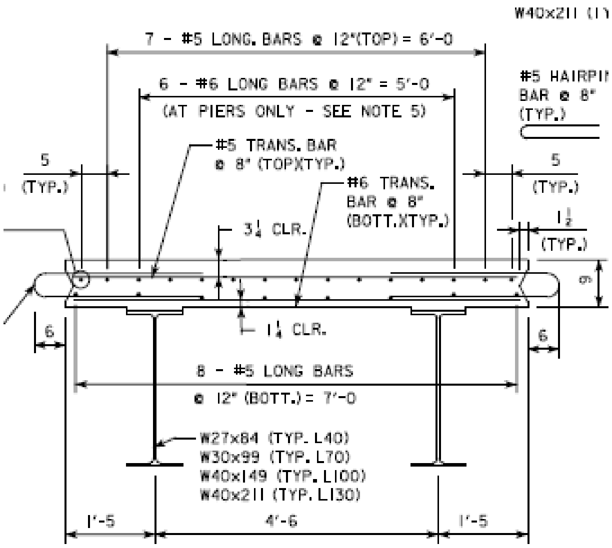
$$I_{xsteel} := \frac{1}{3} \cdot \left(b_{tf} \cdot t_{tf}^3 + b_{bf} \cdot t_{bf}^3 + D_w \cdot t_w^3\right)$$

Moment of inertia about X axis.

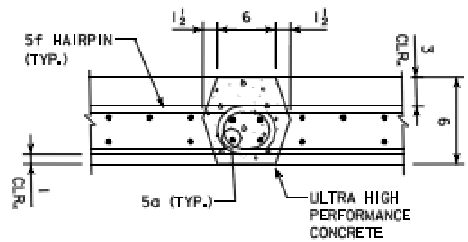
$I_{zsteel} = 3923.795 \cdot in^4$ $\quad I_{ysteel} = 127.762 \cdot in^4$ $\quad I_{xsteel} = 3.4 \cdot in^4$ $\quad A_{steel} = 28.7 \cdot in^2$

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: $t_{slab} := 8in$

$D_t := (t_{slab} + t_{tf} + D_w + t_{bf}) = 37.6 \cdot in$ Total section depth

$b_{eff} := spacing_{int} \quad b_{eff} = 47 \cdot in$ Effective width. S 4.6.2.6.1 LRFD

$b_{tr} := \dfrac{b_{eff}}{n}$ Transformed slab width as steel.

$I_{zslab} := b_{tr} \cdot \dfrac{t_{slab}^{3}}{12}$ Transformed slab moment of inertia about z axis as steel.

$A_{slab} := b_{tr} \cdot t_{slab}$ 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

$$A_{rt} := 0.465 \frac{in^2}{ft} \cdot b_{eff} = 1.8 \cdot in^2$$

$$A_r := A_{rt} + A_{rb} = 4.4 \cdot in^2$$

$$c_{rt} := 2.5in + 0.625in + \left(\frac{5}{16}\right) in = 3.4 \cdot in$$

$$c_r := \frac{(A_{rt} \cdot c_{rt} + A_{rb} \cdot c_{rb})}{A_r} = 4.9 \cdot in$$

Cross Section Over Support

$$A_{rb} := 0.66 \frac{in^2}{ft} \cdot b_{eff} = 2.6 \cdot in^2 \qquad A_{rtadd} := 0.44 \cdot \frac{in^2}{ft} \cdot b_{eff} = 1.7 \cdot in^2$$

$$A_{rneg} := A_r + A_{rtadd} = 6.1 \cdot in^2$$

$$c_{rb} := t_{slab} - 1.75in - \left(\frac{6}{16}\right) in = 5.9 \cdot in$$ ref from top of slab

$$c_{rneg} := \frac{(A_{rt} \cdot c_{rt} + A_{rb} \cdot c_{rb} + A_{rtadd} \cdot c_{rt})}{A_{rneg}} = 4.5 \cdot in$$

Find composite section centroid:

$$A_x := A_{steel} + \frac{A_r \cdot (n-1)}{n} + A_{slab} \qquad y_{slab} := \frac{t_{slab}}{2}$$

$$y_{st} := \frac{A_{tf} \cdot \left(\frac{t_{tf}}{2} + t_{slab}\right) + A_{bf} \cdot \left(\frac{t_{bf}}{2} + D_w + t_{tf} + t_{slab}\right) + A_w \cdot \left(\frac{D_w}{2} + t_{tf} + t_{slab}\right)}{A_{steel}}$$

Centroid of steel from top of slab.

$$y_c := \frac{y_{st} \cdot A_{steel} + \frac{c_r \cdot A_r \cdot (n-1)}{n} + A_{slab} \cdot y_{slab}}{A_x} \qquad y_c = 10.3 \cdot in$$

Centroid of transformed composite section from top of slab.

Calculate Transformed Iz for composite section:

$$I_z := I_{zsteel} + A_{steel} \cdot \left(y_{st} - y_c\right)^2 + I_{zslab} + A_{slab} \cdot \left(y_{slab} - y_c\right)^2 + \frac{A_r \cdot (n-1)}{n} \cdot \left(c_r - y_c\right)^2$$

Transformed moment of inertia about the z axis.

Calculate Transformed Iy for composite section:

$$t_{tr} := \frac{t_{slab}}{n}$$

Transformed slab thickness.

$$I_{yslab} := \frac{t_{tr} \cdot b_{eff}^{\ 3}}{12}$$

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:

$$I_x := \frac{1}{3} \cdot \left(b_{tf} \cdot t_{tf}^{\ 3} + b_{bf} \cdot t_{bf}^{\ 3} + D_w \cdot t_w^{\ 3} + b_{tr} \cdot t_{slab}^{\ 3}\right)$$

Transformed moment of inertia about the x axis.

Results: $A_x = 86.2 \cdot in^2$ $\quad I_y = 10015.7 \cdot in^4$ $\quad I_z = 10959.8 \cdot in^4$ $\quad I_x = 1149.3 \cdot in^4$

Composite Section Properties (Uncracked Section - used for live load negative bending):

Find composite section area and centroid:

$$A_{xneg} := A_{steel} + \frac{A_{rneg} \cdot (n-1)}{n} + A_{slab}$$

$$y_{cneg} := \frac{y_{steel} \cdot A_{steel} + \frac{c_{rneg} \cdot A_{rneg} \cdot (n-1)}{n} + A_{slab} \cdot y_{slab}}{A_{xneg}} \qquad y_{cneg} = 7.6 \cdot in$$

Centroid of transformed composite section from top of slab.

Calculate Transformed Izneg for composite negative moment section:

$$I_{zneg} := I_{zsteel} + A_{steel} \cdot \left(y_{steel} - y_{cneg}\right)^2 + I_{zslab} + A_{slab} \cdot \left(y_{slab} - y_{cneg}\right)^2 + \frac{A_{rneg} \cdot (n-1)}{n} \cdot \left(c_{rneg} - y_{cneg}\right)^2$$

Transformed moment of inertia about the z axis.

$$I_{zneg} = 6457.4 \cdot in^4$$

Composite Section Properties (Cracked Section - used for live load negative bending):

Find cracked section area and centroid:

$$A_{cr} := A_{steel} + A_{rneg} = 34.9 \cdot in^2$$

$$y_{cr} := \frac{\left(A_{steel} \cdot y_{steel} + A_{rneg} \cdot c_{rneg}\right)}{A_{cr}} = 13 \cdot in \qquad y_{crb} := t_{slab} + t_{tf} + D_w + t_{bf} - y_{cr} = 24.6 \cdot in$$

Find cracked section moments of inertia and section moduli:

$$I_{zcr} := I_{zsteel} + A_{steel} \cdot \left(y_{steel} - y_{cr}\right)^2 + A_r \cdot \left(c_r - y_{cr}\right)^2 \qquad I_{zcr} = 4310.8 \cdot in^4$$

$$I_{ycr} := I_{ysteel} \qquad I_{ycr} = 127.8 \cdot in^4$$

$$I_{xcr} := \frac{1}{3} \cdot \left(b_{tf} \cdot t_{tf}^{\ 3} + b_{bf} \cdot t_{tf}^{\ 3} + D_w \cdot t_w^{\ 3}\right) \qquad I_{xcr} = 3.4 \cdot in^4$$

$$d_{topcr} := y_{cr} - c_{rt} \qquad d_{topcr} = 9.6 \cdot in$$

$$d_{botcr} := t_{slab} + t_{tf} + D_w + t_{bf} - y_{cr} \qquad d_{botcr} = 24.6 \cdot in$$

$$S_{topcr} := \frac{I_{zcr}}{d_{topcr}} \qquad S_{topcr} = 450.7 \cdot in^3$$

$$S_{botcr} := \frac{I_{zcr}}{d_{botcr}} \qquad S_{botcr} = 174.9 \cdot in^3$$

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.

$$W_{deck\_int} := w_c \cdot spacing_{int} \cdot t_d \qquad W_{deck\_int} = 514.1 \cdot plf$$

$$W_{deck\_ext} := w_c \cdot spacing_{ext} \cdot t_d \qquad W_{deck\_ext} = 525 \cdot plf$$

$$W_{haunch} := w_c \cdot w_h \cdot t_h \qquad W_{haunch} = 21.9 \cdot plf$$

$$W_{stringer} := w_{s1} \qquad W_{stringer} = 99 \cdot plf$$

Diaphragms:	MC18x42.7	Thickness Conn. Plate	$t_{conn} := \frac{5}{8} in$
Diaphragm Weight	$w_{s2} := 42.7plf$	Width Conn. Plate	$w_{conn} := 5in$
Diaphragm Length	$L_{diaph} := 4ft + 2.5in$	Height Conn. Plate	$h_{conn} := 28.5in$

$$W_{diaphragm} := w_{s2} \cdot \frac{L_{diaph}}{2} \qquad W_{diaphragm} = 89.8 \cdot lbf$$

$$W_{conn} := 2 \cdot w_s \cdot t_{conn} \cdot w_{conn} \cdot h_{conn} \qquad W_{conn} = 50.5 \cdot lbf$$

$$W_{DCpoint} := \left(W_{diaphragm} + W_{conn}\right) \cdot 1.05 \qquad W_{DCpoint} = 147.4 \cdot lbf$$

Equivalent distributed load from DC point loads: $w_{DCpt\_equiv} := \frac{3 \cdot W_{DCpoint}}{L_{str}} = 6.4 \cdot plf$

Interior Uniform Dead Load, Phase 1: $W_{DCuniform1\_int} := W_{deck\_int} + W_{haunch} + W_{stringer} + w_{DCpt\_equiv} = 641.3 \cdot plf$

Exterior Uniform Dead Load, Phase 1: $W_{DCuniform1\_ext} := W_{deck\_ext} + W_{haunch} + W_{stringer} + w_{DCpt\_equiv} = 652.2 \cdot plf$

Moments due to Phase 1 DL: $M_{DC1\_int}(x) := \frac{W_{DCuniform1\_int} \cdot x}{2} \cdot (L_{str} - x)$ $M_{DC1\_ext}(x) := \frac{W_{DCuniform1\_ext} \cdot x}{2} \cdot (L_{str} - x)$

Shear due to Phase 1 DL: $V_{DC1\_int}(x) := W_{DCuniform1\_int} \cdot \left(\frac{L_{str}}{2} - x\right)$ $V_{DC1\_ext}(x) := W_{DCuniform1\_ext} \cdot \left(\frac{L_{str}}{2} - x\right)$

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 $W_{barrier} := \frac{(w_c \cdot A_{barrier})}{2}$ $W_{barrier} = 216.8 \cdot plf$

Interior Dead Load, Phase 2: $W_{DCuniform\_int} := W_{DCuniform1\_int} = 641.3 \cdot plf$

Exterior Dead Load, Phase 2: $W_{DCuniform\_ext} := W_{DCuniform1\_ext} + W_{barrier} = 869 \cdot plf$

Moments due to Phase 2 DL: $M_{DC2\_int}(x) := \frac{W_{DCuniform\_int} \cdot x}{2} \cdot (L_{str} - x)$ $M_{DC2\_ext}(x) := \frac{W_{DCuniform\_ext} \cdot x}{2} \cdot (L_{str} - x)$

Shear due to Phase 2 DL: $V_{DC2\_int}(x) := W_{DCuniform\_int} \cdot \left(\frac{L_{str}}{2} - x\right)$ $V_{DC2\_ext}(x) := W_{DCuniform\_ext} \cdot \left(\frac{L_{str}}{2} - x\right)$

Phase 3: Girders are composite and have been made continuous. Utilities and future wearing surface are applied.

Unit Weight Overlay $w_{ws} := 30psf$

$W_{ws\_int} := w_{ws} \cdot spacing_{int}$ $W_{ws\_int} = 117.5 \cdot plf$

$W_{ws\_ext} := w_{ws} \cdot (spacing_{ext} - 1 \cdot ft - 7in)$ $W_{ws\_ext} = 72.5 \cdot plf$

Unit Weight Utilities $W_u := 15plf$

$W_{DWuniform\_int} := W_{ws\_int} + W_u$ $W_{DWuniform\_int} = 132.5 \cdot plf$

$W_{DWuniform\_ext} := W_{ws\_ext} + W_u$ $W_{DWuniform\_ext} = 87.5 \cdot plf$

Moments due to DW: $M_{DW\_int}(x) := \frac{W_{DWuniform\_int} \cdot x}{2} \cdot (L_{str} - x)$ $M_{DW\_ext}(x) := \frac{W_{DWuniform\_ext} \cdot x}{2} \cdot (L_{str} - x)$

Shears due to DW: $V_{DW\_int}(x) := W_{DWuniform\_int} \cdot \left(\frac{L_{str}}{2} - x\right)$ $V_{DW\_ext}(x) := W_{DWuniform\_ext} \cdot \left(\frac{L_{str}}{2} - x\right)$

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: $W_{int} := \left(L_{str} \cdot W_{DCuniform\_int} + 3 \cdot W_{DCpoint}\right) \cdot 2 \cdot (1 + DLIM) = 117 \cdot kip$

Vertical force at lifting point: $F_{lift\_int} := \dfrac{W_{int}}{4} = 29.3 \cdot kip$

Equivalent distributed load: $w_{int\_IM} := \dfrac{W_{int}}{\left(2 \cdot L_{str}\right)} = 842 \cdot plf$

Min (Neg.) Moment during lifting: $M_{lift\_neg\_max\_int} := -w_{int\_IM} \cdot \dfrac{\left(D_{lift}^{\,2}\right)}{2}$ $\qquad M_{lift\_neg\_max\_int} = -32.2 \cdot kip \cdot ft$

Max (Pos.) Moment during lifting:

$$M_{lift\_pos\_max\_int} := \left| \begin{array}{l} 0 \ \text{ if } \dfrac{w_{int\_IM} \cdot \left(L_{str} - 2 \cdot D_{lift}\right)^2}{8} + M_{lift\_neg\_max\_int} < 0 \\ \dfrac{w_{int\_IM} \cdot \left(L_{str} - 2 \cdot D_{lift}\right)^2}{8} + M_{lift\_neg\_max\_int} \end{array} \right.$$

$M_{lift\_pos\_max\_int} = 252.4 \cdot kip \cdot ft$

Exterior Module:

Total Exterior Module Weight: $W_{ext} := \left(L_{str} \cdot W_{DCuniform\_ext} + 3 \cdot W_{DCpoint} + W_{barrier} \cdot L_{str}\right) \cdot 2 \cdot (1 + DLIM) = 197.3 \cdot kip$

Vertical force at lifting point: $F_{lift\_ext} := \dfrac{W_{ext}}{4} = 49.3 \cdot kip$

Equivalent distributed load: $w_{ext\_IM} := \dfrac{W_{ext}}{2 \cdot L_{str}} = 1419.7 \cdot plf$

Min (Neg.) Moment during lifting: $M_{lift\_neg\_max\_ext} := -w_{ext\_IM} \cdot \dfrac{D_{lift}^{\,2}}{2}$ $\qquad M_{lift\_neg\_max\_ext} = -54.3 \cdot kip \cdot ft$

Max (Pos.) Moment during lifting:

$$M_{lift\_pos\_max\_ext} := \left| \begin{array}{l} 0 \ \text{ if } \dfrac{w_{ext\_IM} \cdot \left(L_{str} - 2 \cdot D_{lift}\right)^2}{8} + M_{lift\_neg\_max\_ext} < 0 \\ \dfrac{w_{ext\_IM} \cdot \left(L_{str} - 2 \cdot D_{lift}\right)^2}{8} + M_{lift\_neg\_max\_ext} \end{array} \right.$$

$M_{lift\_pos\_max\_ext} = 425.5 \cdot kip \cdot ft$

Max Shear during lifting: $V_{lift} := \max\left(w_{ext\_IM} \cdot D_{lift}, F_{lift\_ext} - w_{ext\_IM} \cdot D_{lift}\right) = 36.9 \cdot kip$

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: $I_{zsteel} = 3923.8 \cdot in^4$

Girder Area: $A_{steel} = 28.7 \cdot in^2$

Girder Depth: $d_{gird} = 29.7 \cdot in$

Distance between centroid of deck and centroid of beam: $e_g := \frac{t_d}{2} + t_h + \frac{d_{gird}}{2} = 22.1 \cdot in$

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:

One Lane Loaded: $K_g := n \cdot \left(I_{zsteel} + A_{steel} \cdot e_g^{\,2}\right) = 125670.9 \cdot in^4$ S4.6.2.2.1-1

$$g_{int\_1m} := \left[0.06 + \left(\frac{spacing_{int}}{14ft}\right)^{0.4} \cdot \left(\frac{spacing_{int}}{L_{span}}\right)^{0.3} \cdot \left(\frac{K_g}{L_{span} \cdot t_d^{\,3}}\right)^{0.1}\right] = 0.269$$

Two Lanes Loaded:
$$g_{int\_2m} := \left[0.075 + \left(\frac{spacing_{int}}{9.5ft}\right)^{0.6} \cdot \left(\frac{spacing_{int}}{L_{span}}\right)^{0.2} \cdot \left(\frac{K_g}{L_{span} \cdot t_d^{\,3}}\right)^{0.1}\right] = 0.347$$

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.

$d_e := 2in$

$L_{spa} := 4.5ft$ $\quad r := L_{spa} + d_e - 2ft = 2.7 \cdot ft$

$g_{ext\_1m} := MP_1 \cdot \frac{0.5r}{L_{spa}} = 0.356$

Two Lanes Loaded: $e_{2m} := 0.77 + \frac{d_e}{9.1ft} = 0.7883$

$g_{ext\_2m} := e_{2m} \cdot g_{int\_2m} = 0.273$

Governing Factor: $g_{ext\_m} := \max\left(g_{ext\_1m}, g_{ext\_2m}\right) = 0.356$

Exterior Stringers for Shear:

One Lane Loaded: Use Lever Rule.

$g_{ext\_1v} := g_{ext\_1m} = 0.356$

Two Lanes Loaded: $e_{2v} := 0.6 + \frac{d_e}{10ft} = 0.62$

$g_{ext\_2v} := e_{2v} \cdot g_{int\_2v} = 0.317$

Governing Factor: $g_{ext\_v} := \max\left(g_{ext\_1v}, g_{ext\_2v}\right) = 0.356$

FACTOR TO USE FOR SHEAR: $g_v := \max\left(g_{int\_v}, g_{ext\_v}\right) = 0.517$

FACTOR TO USE FOR MOMENT: $g_m := \max\left(g_{int\_m}, g_{ext\_m}\right) = 0.356$

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 $M_{DC1int} := M_{DC1\_int}\left(\frac{L_{str}}{2}\right) = 387.2 \cdot kip \cdot ft$ $M_{DW1int} := 0 \cdot kip \cdot ft$ $M_{LL1int} := 0 kip \cdot ft$

$V_{DC1int} := V_{DC1\_int}(0) = 22.3 \cdot kip$ $V_{DW1int} := 0 \cdot kip$ $V_{LL1int} := 0 \cdot kip$

Exterior Girder $M_{DC1ext} := M_{DC1\_ext}\left(\frac{L_{str}}{2}\right) = 393.8 \cdot kip \cdot ft$ $M_{DW1ext} := 0 \cdot kip \cdot ft$ $M_{LL1ext} := 0 \cdot kip \cdot ft$

$V_{DC1ext} := V_{DC1\_ext}(0) = 22.7 \cdot kip$ $V_{DW1ext} := 0 \cdot kip$ $V_{LL1ext} := 0 \cdot kip \cdot ft$

Load Cases:

$M_{1\_STR\_I} := \max\left(1.25 \cdot M_{DC1int} + 1.5 \cdot M_{DW1int} + 1.75 \cdot M_{LL1int}, 1.25 \cdot M_{DC1ext} + 1.5 \cdot M_{DW1ext} + 1.75 \cdot M_{LL1ext}\right) = 492.3 \cdot kip \cdot ft$

$V_{1\_STR\_I} := \max\left(1.25 \cdot V_{DC1int} + 1.5 \cdot V_{DW1int} + 1.75 \cdot V_{LL1int}, 1.25 \cdot V_{DC1ext} + 1.5 \cdot V_{DW1ext} + 1.75 \cdot V_{LL1ext}\right) = 28.3 \cdot kip$

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 $M_{DC2int} := M_{DC2\_int}\left(\frac{L_{str}}{2}\right) = 387.2 \cdot kip \cdot ft$ $M_{DW2int} := 0 \cdot kip \cdot ft$ $M_{LL2int} := 0 \cdot kip \cdot ft$

$V_{DC2int} := V_{DC2\_int}(0) = 22.3 \cdot kip$ $V_{DW2int} := 0 \cdot kip$ $V_{LL2int} := 0 \cdot kip$

Exterior Girder $M_{DC2ext} := M_{DC2\_ext}\left(\frac{L_{str}}{2}\right) = 524.7 \cdot kip \cdot ft$ $M_{DW2ext} := 0 \cdot kip \cdot ft$ $M_{LL2ext} := 0 \cdot kip \cdot ft$

$V_{DC2ext} := V_{DC2\_ext}(0) = 30.2 \cdot kip$ $V_{DW2ext} := 0 \cdot kip$ $V_{LL2ext} := 0 \cdot kip$

Load Cases:

$M_{2\_STR\_I} := \max\left(1.25 \cdot M_{DC2int} + 1.5 \cdot M_{DW2int} + 1.75 \cdot M_{LL2int}, 1.25 \cdot M_{DC2ext} + 1.5 \cdot M_{DW2ext} + 1.75 \cdot M_{LL2ext}\right) = 655.8 \cdot kip \cdot ft$

$V_{2\_STR\_I} := \max\left(1.25 \cdot V_{DC2int} + 1.5 \cdot V_{DW2int} + 1.75 \cdot V_{LL2int}, 1.25 \cdot V_{DC2ext} + 1.5 \cdot V_{DW2ext} + 1.75 \cdot V_{LL2ext}\right) = 37.7 \cdot kip$

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 $M_{DC3int} := M_{lift\_pos\_max\_int} = 252.4 \cdot kip \cdot ft$ $M_{DW3int} := 0 \cdot kip \cdot ft$ $M_{LL3int} := 0 \cdot kip \cdot ft$

$M_{DC3int\_neg} := \left|M_{lift\_neg\_max\_int}\right| = 32.2 \cdot kip \cdot ft$ $M_{DW3int\_neg} := 0 \cdot kip \cdot ft$ $M_{LL3int\_neg} := 0 \cdot kip \cdot ft$

$V_{DC3int} := V_{lift} = 36.9 \cdot kip$ $V_{DW3int} := 0 \cdot kip$ $V_{LL3int} := 0 \cdot kip$

Exterior Girder $M_{DC3ext} := M_{lift\_pos\_max\_ext} = 425.5 \cdot kip \cdot ft$ $M_{DW3ext} := 0 \cdot kip \cdot ft$ $M_{LL3ext} := 0 \cdot kip \cdot ft$

$M_{DC3ext\_neg} := \left|M_{lift\_neg\_max\_ext}\right| = 54.3 \cdot kip \cdot ft$ $M_{DW3ext\_neg} := 0 \cdot kip \cdot ft$ $M_{LL3ext\_neg} := 0 \cdot kip \cdot ft$

$V_{DC3ext} := V_{lift} = 36.9 \cdot kip$ $V_{DW3ext} := 0 \cdot kip$ $V_{LL3ext} := 0 \cdot kip$

Load Cases:

$M_{3\_STR\_I} := \max\left(1.5 \cdot M_{DC3int} + 1.5 \cdot M_{DW3int}, 1.5 \cdot M_{DC3ext} + 1.5 \cdot M_{DW3ext}\right) = 638.3 \cdot kip \cdot ft$

$M_{3\_STR\_I\_neg} := \max\left(1.5 \cdot M_{DC3int\_neg} + 1.5 \cdot M_{DW3int\_neg}, 1.5 \cdot M_{DC3ext\_neg} + 1.5 \cdot M_{DW3ext\_neg}\right) = 81.5 \cdot kip \cdot ft$

$$V_{3\_STR\_I} := \max\left(1.5 \cdot V_{DC3int} + 1.5 \cdot V_{DW3int}, 1.5 \cdot V_{DC3ext} + 1.5 \cdot V_{DW3ext}\right) = 55.4 \cdot kip$$

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: $M_{DC4} := 440 \cdot kip \cdot ft$ $\quad M_{DW4} := 43.3 \cdot kip \cdot ft$ $\quad M_{LL4} := 590.3 \cdot kip \cdot ft$

$M_{WS4} := 0 kip \cdot ft$ $\quad M_{W4} := 0 kip \cdot ft$

$M_{DC4neg} := -328.9 \cdot kip \cdot ft$ $\quad M_{DW4neg} := -32.3 \cdot kip \cdot ft$ $\quad M_{LL4neg} := -314.4 kip \cdot ft$

$M_{WS4neg} := 0 \cdot kip \cdot ft$ $\quad M_{WL4neg} := 0 \cdot kip \cdot ft$

$V_u := 145.3 kip$

Load Cases:

$$M_{4\_STR\_I} := 1.25 \cdot M_{DC4} + 1.5 \cdot M_{DW4} + 1.75 \cdot M_{LL4} = 1648 \cdot kip \cdot ft$$

$$M_{4\_STR\_I\_neg} := 1.25 \cdot M_{DC4neg} + 1.5 \cdot M_{DW4neg} + 1.75 \cdot M_{LL4neg} = -1009.8 \cdot kip \cdot ft$$

$$M_{4\_STR\_III} := 1.25 \cdot M_{DC4} + 1.5 \cdot M_{DW4} + 1.4 \cdot M_{WS4} = 614.9 \cdot kip \cdot ft$$

$$M_{4\_STR\_III\_neg} := 1.25 \cdot M_{DC4neg} + 1.5 \cdot M_{DW4neg} + 1.4 \cdot M_{WS4} = -459.6 \cdot kip \cdot ft$$

$$M_{4\_STR\_V} := 1.25 \cdot M_{DC4} + 1.5 \cdot M_{DW4} + 1.35 \cdot M_{LL4} + 0.4 \cdot M_{WS4} + 1.0 \cdot M_{W4} = 1411.9 \cdot kip \cdot ft$$

$$M_{4\_STR\_V\_neg} := 1.25 \cdot M_{DC4neg} + 1.5 \cdot M_{DW4neg} + 1.35 \cdot M_{LL4neg} + 0.4 \cdot M_{WS4neg} + 1.0 \cdot M_{WL4neg} = -884 \cdot kip \cdot ft$$

$$M_{4\_SRV\_I} := 1.0 \cdot M_{DC4} + 1.0 \cdot M_{DW4} + 1.0 \cdot M_{LL4} + 0.3 \cdot M_{WS4} + 1.0 \cdot M_{W4} = 1073.6 \cdot kip \cdot ft$$

$$M_{4\_SRV\_I\_neg} := 1.0 \cdot M_{DC4neg} + 1.0 \cdot M_{DW4neg} + 1.0 \cdot M_{LL4neg} + 0.3 \cdot M_{WS4neg} + 1.0 \cdot M_{WL4neg} = -675.6 \cdot kip \cdot ft$$

$$M_{4\_SRV\_II} := 1.0 \cdot M_{DC4} + 1.0 \cdot M_{DW4} + 1.3 \cdot M_{LL4} = 1250.7 \cdot kip \cdot ft$$

$$M_{4\_SRV\_II\_neg} := 1.0 \cdot M_{DC4neg} + 1.0 \cdot M_{DW4neg} + 1.3 \cdot M_{LL4neg} = -769.9 \cdot kip \cdot ft$$

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:

$P_{rt} := A_{rt} \cdot F_s = 109.3 \cdot kip$ $\quad P_s := 0.85 \cdot f_c \cdot b_{eff} \cdot t_{slab} = 1598 \cdot kip$ $\quad P_w := F_y \cdot D_w \cdot t_w = 736.1 \cdot kip$

$P_{rb} := A_{rb} \cdot F_s = 155.1 \cdot kip$ $\quad P_c := F_y \cdot b_{tf} \cdot t_{tf} = 350.1 \cdot kip$ $\quad P_t := F_y \cdot b_{bf} \cdot t_{bf} = 350.1 \cdot kip$

$$PNA_{pos} := \left|\begin{array}{l} \text{"case 1"} \quad \text{if } \left(P_t + P_w\right) \geq \left(P_c + P_s + P_{rt} + P_{rb}\right) \\ \text{otherwise} \\ \quad \left|\begin{array}{l} \text{"case 2"} \quad \text{if } \left[\left(P_t + P_w + P_c\right) \geq \left(P_s + P_{rt} + P_{rb}\right)\right] \\ \text{otherwise} \\ \quad \left|\begin{array}{l} \text{"case 3"} \quad \text{if } \left[\left(P_t + P_w + P_c\right) \geq \left(\frac{c_{rb}}{t_{slab}} \cdot P_s + P_{rt} + P_{rb}\right)\right] \\ \text{otherwise} \\ \quad \left|\begin{array}{l} \text{"case 4"} \quad \text{if } \left[\left(P_t + P_w + P_c + P_{rb}\right) \geq \left(\frac{c_{rb}}{t_{slab}} \cdot P_s + P_{rt}\right)\right] \\ \text{otherwise} \\ \quad \left|\begin{array}{l} \text{"case 5"} \quad \text{if } \left[\left(P_t + P_w + P_c + P_{rb}\right) \geq \left(\frac{c_{rt}}{t_{slab}} \cdot P_s + P_{rt}\right)\right] \\ \text{otherwise} \\ \quad \left|\begin{array}{l} \text{"case 6"} \quad \text{if } \left(P_t + P_w + P_c + P_{rb} + P_{rt}\right) \geq \left(\frac{c_{rt}}{t_{slab}} \cdot P_s\right) \\ \text{"case 7"} \quad \text{if } \left(P_t + P_w + P_c + P_{rb} + P_{rt}\right) \leq \left(\frac{c_{rt}}{t_{slab}} \cdot P_s\right) \quad \text{otherwise} \end{array}\right. \end{array}\right. \end{array}\right. \end{array}\right. \end{array}\right. \end{array}\right.$$

$PNA_{pos} = \text{"case 4"}$

$$PNA_{neg} := \left|\begin{array}{l} \text{"case 1"} \quad \text{if } \left(P_c + P_w\right) \geq \left(P_t + P_{rt} + P_{rb}\right) \\ \text{"case 2"} \quad \text{if } \left[\left(P_t + P_w + P_c\right) \geq \left(P_{rt} + P_{rb}\right)\right] \quad \text{otherwise} \end{array}\right.$$

$PNA_{neg} = \text{"case 1"}$

Calculate Y, Dp, and Mp: $\quad D := D_w \quad t_s := t_{slab} \quad t_h := 0 \quad C_{rt} := c_{rt} \quad C_{rb} := c_{rb}$

Case I : Plastic Nuetral Axis in the Steel Web

$$Y_1 := \frac{D}{2} \cdot \left(\frac{P_t - P_c - P_s - P_{rt} - P_{rb}}{P_w} + 1\right) \qquad D_{P1} := t_s + t_h + t_{tf} + Y_1$$

$$M_{P1} := \frac{P_w}{2D}\cdot\left[Y_1^{\,2} + (D - Y_1)^2\right] + \left[\begin{array}{l} P_s\cdot\left(Y_1 + \dfrac{t_s}{2} + t_{tf} + t_h\right) + P_{rt}\cdot(t_s - C_{rt} + t_{tf} + Y_1 + t_h) + P_{rb}\cdot(t_s - C_{rb} + t_{tf} + Y_1 + t_h) \ldots \\ + P_c\cdot\left(Y_1 + \dfrac{t_{tf}}{2}\right) + P_t\cdot\left(D - Y_1 + \dfrac{t_{bf}}{2}\right) \end{array}\right]$$

$$Y_{1neg} := \left(\frac{D}{2}\right)\cdot\left[1 + \frac{(P_c - P_t - P_{rt} - P_{rb})}{P_w}\right] \qquad D_{p1neg} := t_s + t_h + t_{tf} + Y_{1neg}$$

$$D_{CP1neg} := \left(\frac{D}{2\cdot P_w}\right)\cdot(P_t + P_w + P_{rb} + P_{rt} - P_c)$$

$$M_{p1neg} := \left[\begin{array}{l} \left(\dfrac{P_w}{2\cdot D}\right)\cdot\left[Y_{1neg}^{\,2} + (D_w - Y_{1neg})^2\right] + P_{rt}\cdot(t_s - C_{rt} + t_{tf} + Y_{1neg} + t_h) + P_{rb}\cdot(t_s - C_{rb} + t_{tf} + Y_{1neg} + t_h) \ldots \\ + P_t\cdot\left(D - Y_{1neg} + \dfrac{t_{bf}}{2}\right) + P_c\cdot\left(Y_{1neg} + \dfrac{t_{tf}}{2}\right) \end{array}\right]$$

<u>Case II:</u> Plastic Nuetral Axis in the Steel Top Flange

$$Y_2 := \frac{t_{tf}}{2}\cdot\left(\frac{P_w + P_t - P_s - P_{rt} - P_{rb}}{P_c} + 1\right) \qquad D_{P2} := t_s + t_h + Y_2$$

$$M_{P2} := \frac{P_c}{2t_{tf}}\cdot\left[Y_2^{\,2} + (t_{tf} - Y_2)^2\right] + \left[\begin{array}{l} P_s\cdot\left(Y_2 + \dfrac{t_s}{2} + t_h\right) + P_{rt}\cdot(t_s - C_{rt} + t_h + Y_2) + P_{rb}\cdot(t_s - C_{rb} + t_h + Y_2) \ldots \\ + P_w\cdot\left(\dfrac{D}{2} + t_{tf} - Y_2\right) + P_t\cdot\left(D - Y_2 + \dfrac{t_{bf}}{2} + t_{tf}\right) \end{array}\right]$$

$$Y_{2neg} := \left(\frac{t_{tf}}{2}\right)\cdot\left[1 + \frac{(P_w + P_c - P_{rt} - P_{rb})}{P_t}\right] \qquad D_{P2neg} := t_s + t_h + Y_{2neg} \qquad D_{CP2neg} := D$$

$$M_{p2neg} := \left(\frac{P_t}{2\cdot t_{tf}}\right)\cdot\left[Y_{2neg}^{\,2} + (t_{tf} - Y_{2neg})^2\right] + \left[\begin{array}{l} P_{rt}\cdot(t_s - C_{rt} + t_h + Y_{2neg}) + P_{rb}\cdot(t_s - C_{rb} + t_h + Y_{2neg}) \ldots \\ + P_w\cdot\left(t_{tf} - Y_{2neg} + \dfrac{D}{2}\right) + P_c\cdot\left(\left|t_s + t_h - Y_{2neg} + \dfrac{t_{tf}}{2}\right|\right) \end{array}\right]$$

<u>Case III:</u> Plastic Nuetral Axis in the Concrete Deck Below the Bottom Reinforcing

$$Y_3 := t_s\cdot\left(\frac{P_c + P_w + P_t - P_{rt} - P_{rb}}{P_s}\right) \qquad D_{P3} := Y_3$$

$$M_{P3} := \frac{P_s}{2t_s}\cdot\left(Y_3^{\,2}\right) + \left[\begin{array}{l} P_{rt}\cdot(Y_3 - C_{rt}) + P_{rb}\cdot(C_{rb} - Y_3) + P_c\cdot\left(\dfrac{t_{tf}}{2} + t_s + t_h - Y_3\right) + P_w\cdot\left(\dfrac{D}{2} + t_{tf} + t_h + t_s - Y_3\right) \ldots \\ + P_t\cdot\left(D + \dfrac{t_{bf}}{2} + t_{tf} + t_s + t_h - Y_3\right) \end{array}\right]$$

<u>Case IV:</u> Plastic Nuetral Axis in the Concrete Deck in the bottom reinforcing layer

$$Y_4 := C_{rb} \qquad D_{P4} := Y_4$$

$$M_{P4} := \frac{P_s}{2t_s}\cdot\left(Y_4^{\,2}\right) + \left[\begin{array}{l} P_{rt}\cdot(Y_4 - C_{rt}) + P_c\cdot\left(\dfrac{t_{tf}}{2} + t_h + t_s - Y_4\right) + P_w\cdot\left(\dfrac{D}{2} + t_{tf} + t_h + t_s - Y_4\right) \ldots \\ + P_t\cdot\left(D + \dfrac{t_{bf}}{2} + t_{tf} + t_h + t_s - Y_4\right) \end{array}\right]$$

Case V: Plastic Nuetral Axis in the Concrete Deck between top and bot reinforcing layers

$$Y_5 := t_s\cdot\left(\frac{P_{rb} + P_c + P_w + P_t - P_{rt}}{P_s}\right) \qquad D_{P5} := Y_5$$

$$M_{P5} := \frac{P_s}{2t_s}\cdot\left(Y_5^{\,2}\right) + \left[\begin{array}{l} P_{rt}\cdot\left(Y_5 - C_{rt}\right) + P_{rb}\cdot\left[\left(t_s - C_{rb}\right) - Y_5\right] + P_c\cdot\left(\frac{t_{tf}}{2} + t_s + t_h - Y_5\right) + P_w\cdot\left(\frac{D}{2} + t_{tf} + t_h + t_s - Y_5\right) \ldots \\ + P_t\cdot\left(D + \frac{t_{bf}}{2} + t_{tf} + t_s + t_h - Y_5\right) \end{array}\right]$$

$$Y_{pos} := \left| \begin{array}{l} Y_1 \text{ if } PNA_{pos} = \text{"case 1"} \\ Y_2 \text{ if } PNA_{pos} = \text{"case 2"} \\ Y_3 \text{ if } PNA_{pos} = \text{"case 3"} \\ Y_4 \text{ if } PNA_{pos} = \text{"case 4"} \\ Y_5 \text{ if } PNA_{pos} = \text{"case 5"} \end{array}\right. \qquad D_{Ppos} := \left| \begin{array}{l} D_{P1} \text{ if } PNA_{pos} = \text{"case 1"} \\ D_{P2} \text{ if } PNA_{pos} = \text{"case 2"} \\ D_{P3} \text{ if } PNA_{pos} = \text{"case 3"} \\ D_{P4} \text{ if } PNA_{pos} = \text{"case 4"} \\ D_{P5} \text{ if } PNA_{pos} = \text{"case 5"} \end{array}\right. \qquad M_{Ppos} := \left| \begin{array}{l} M_{P1} \text{ if } PNA_{pos} = \text{"case 1"} \\ M_{P2} \text{ if } PNA_{pos} = \text{"case 2"} \\ M_{P3} \text{ if } PNA_{pos} = \text{"case 3"} \\ M_{P4} \text{ if } PNA_{pos} = \text{"case 4"} \\ M_{P5} \text{ if } PNA_{pos} = \text{"case 5"} \end{array}\right.$$

$$Y_{pos} = 5.9\cdot in \qquad D_{Ppos} = 5.9\cdot in \qquad M_{Ppos} = 2338.1\cdot kip\cdot ft$$

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).

$$Y_{neg} := \left| \begin{array}{l} Y_{1neg} \text{ if } PNA_{neg} = \text{"case 1"} \\ Y_{2neg} \text{ if } PNA_{neg} = \text{"case 2"} \end{array}\right. \qquad D_{Pneg} := \left| \begin{array}{l} D_{p1neg} \text{ if } PNA_{neg} = \text{"case 1"} \\ D_{P2neg} \text{ if } PNA_{neg} = \text{"case 2"} \end{array}\right. \qquad M_{Pneg} := \left| \begin{array}{l} M_{p1neg} \text{ if } PNA_{neg} = \text{"case 1"} \\ M_{p2neg} \text{ if } PNA_{neg} = \text{"case 2"} \end{array}\right.$$

$$Y_{neg} = 9.1\cdot in \qquad D_{Pneg} = 17.7\cdot in \qquad M_{Pneg} = 19430.1\cdot kip\cdot in$$

Depth of web in compression at the plastic moment [D6.3.2]:

$$A_t := b_{bf}\cdot t_{bf} \qquad A_c := b_{tf}\cdot t_{tf}$$

$$D_{cppos} := \frac{D}{2}\left(\frac{F_y\cdot A_t - F_y\cdot A_c - 0.85\cdot f_c\cdot A_{slab} - F_s\cdot A_r}{F_y\cdot A_w} + 1\right)$$

$$D_{cppos} := \left| \begin{array}{l} (0in) \text{ if } PNA_{pos} \neq \text{"case 1"} \\ (0in) \text{ if } \left(D_{cppos} < 0\right) \\ D_{cppos} \text{ if } PNA_{pos} = \text{"case 1"} \end{array}\right. \qquad D_{cpneg} := \left| \begin{array}{l} D_{CP1neg} \text{ if } PNA_{neg} = \text{"case 1"} \\ D_{CP2neg} \text{ if } PNA_{neg} = \text{"case 2"} \end{array}\right.$$

$$D_{cppos} = 0\cdot in \qquad D_{cpneg} = 19.2\cdot in$$

Positive Flexural Compression Check:

From LRFD Article 6.10.2

Check for compactness:

Web Proportions: Web slenderness Limit:

$$\frac{D_w}{t_w} \leq 150 = 1 \qquad 2\cdot\frac{D_{cppos}}{t_w} \leq 3.76\cdot\sqrt{\frac{E_s}{F_y}} = 1 \qquad \text{S 6.10.6.2.2}$$

Therefore Section is considered compact and shall satisfy the requirements of Article 6.10.7.1.

$$M_n := \left| \begin{array}{l} M_{Ppos} \text{ if } D_{Ppos} \leq 0.1\cdot D_t \\ M_{Ppos}\cdot\left(1.07 - 0.7\cdot\frac{D_{Ppos}}{D_t}\right) \text{ otherwise} \end{array}\right. \qquad M_n = 2246.4\cdot kip\cdot ft$$

Negative Moment Capacity Check (Appendix A6):

Web Slenderness: $D_t = 37.6 \cdot in$ $\quad D_{cneg} := D_t - y_{cr} - t_{bf} = 24 \cdot in$

$$\frac{2 \cdot D_{cneg}}{t_w} < 5.7 \cdot \sqrt{\frac{E_s}{F_y}} = 1$$

S Appendix A6 (for skew less than 20 deg).

Moment ignoring concrete:

$$M_{yt} := F_y \cdot S_{botcr} = 8745.1 \cdot kip \cdot in \qquad M_{yc} := F_s \cdot S_{topcr} = 27039.2 \cdot kip \cdot in$$

$$M_y := \min(M_{yc}, M_{yt}) = 8745.1 \cdot kip \cdot in$$

Web Compactness:

Check for Permanent Deformations (6.10.4.2):

$$D_n := \max(t_{slab} + t_{tf} + D_w - y_c, y_c - t_{slab} - t_{tf}) = 26.7 \cdot in$$

$$Gov := if(y_c - t_{slab} - t_{tf}, y_c - c_{rt}, D_n) = 6.9 \cdot in$$

$$f_n := \left|M_{4\_SRV\_II\_neg}\right| \cdot \frac{Gov}{I_z} = 5.8 \cdot ksi$$

Steel stress on side of Dn

$$\rho := \min\left(1.0, \frac{F_y}{f_n}\right) = 1 \qquad \beta := 2 \cdot D_n \cdot \frac{t_w}{A_{tf}} = 4 \qquad R_h := \frac{\left[12 + \beta \cdot \left(3\rho - \rho^3\right)\right]}{(12 + 2 \cdot \beta)} = 1$$

$$\lambda_{rw} := 5.7 \cdot \sqrt{\frac{E_s}{F_y}}$$

$$\lambda_{PWdcp} := \min\left[\lambda_{rw} \cdot \frac{D_{cpneg}}{D_{cneg}}, \frac{\sqrt{\frac{E_s}{F_y}}}{\left(0.54 \cdot \frac{M_{Pneg}}{R_h \cdot M_y} - 0.09\right)^2}\right] = 19.6$$

$$2 \cdot \frac{D_{cpneg}}{t_w} \le \lambda_{PWdcp} = 0$$

Web Plastification: $R_{pc} := \frac{M_{Pneg}}{M_{yc}} = 0.7$ $\quad R_{pt} := \frac{M_{Pneg}}{M_{yt}} = 2.2$

Flexure Factor: $\phi_f := 1.0$

Tensile Limit: $M_{r\_neg\_t} := \phi_f \cdot R_{pt} \cdot M_{yt} = 1619.2 \cdot kip \cdot ft$

Compressive Limit:

Local Buckling Resistance:

$$\lambda_f := \frac{b_{bf}}{2 \cdot t_{bf}} = 7.8 \qquad \lambda_{rf} := 0.95 \cdot \sqrt{0.76 \cdot \frac{E_s}{F_y}} = 19.9$$

$$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E_s}{F_y}} = 9.2 \qquad F_{yresid} := \max\left(\min\left(0.7 \cdot F_y, R_h \cdot F_y \cdot \frac{S_{topcr}}{S_{botcr}}, F_y\right), 0.5 \cdot F_y\right) = 35.0 \cdot ksi$$

$$M_{ncLB} := \begin{cases} (R_{pc} \cdot M_{yc}) & \text{if } \lambda_f \le \lambda_{pf} \\ \left[R_{pc} \cdot M_{yc} \cdot \left[1 - \left(1 - \frac{F_{yresid} \cdot S_{topcr}}{R_{pc} \cdot M_{yc}}\right)\left(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right]\right] & \text{otherwise} \end{cases}$$

$$M_{ncLB} = 1619.2 \cdot kip \cdot ft$$

Lateral Torsional Buckling Resistance:

$$L_b := \frac{(L_{str})}{2\cdot 3} = 11.6\cdot ft$$

Inflection point assumed to be at 1/6 span

$$r_t := \frac{b_{bf}}{\sqrt{12\cdot\left(1 + \frac{1}{3}\cdot\frac{D_{cneg}\cdot t_w}{b_{bf}\cdot t_{bf}}\right)}} = 2.4\cdot in$$

$$L_p := 1.0\cdot r_t\cdot\sqrt{\frac{E_s}{F_y}} = 57.6\cdot in \qquad h := D + t_{bf} = 29\cdot in \qquad C_b := 1.0$$

$$J_b := \frac{D\cdot t_w^{\,3}}{3} + \frac{b_{bf}\cdot t_{bf}^{\,3}}{3}\cdot\left(1 - 0.63\cdot\frac{t_{bf}}{b_{bf}}\right) + \frac{b_{tf}\cdot t_{tf}^{\,3}}{3}\cdot\left(1 - 0.63\cdot\frac{t_{tf}}{b_{tf}}\right) = 3.3\cdot in^4$$

$$L_r := 1.95\cdot r_t\cdot\frac{E_s}{F_{yresid}}\cdot\sqrt{\frac{J_b}{S_{botcr}\cdot h}}\cdot\sqrt{1 + \sqrt{1 + 6.76\cdot\left(\frac{F_{yresid}}{E_s}\cdot\frac{S_{botcr}\cdot h}{J_b}\right)^2}} = 240\cdot in$$

$$F_{cr} := \frac{C_b\cdot\pi^2\cdot E_s}{\left(\frac{L_b}{r_t}\right)^2}\cdot\sqrt{1 + 0.078\cdot\frac{J_b}{S_{botcr}\cdot h}\cdot\left(\frac{L_b}{r_t}\right)^2} = 91.7\cdot ksi$$

$$M_{ncLTB} := \begin{cases} (R_{pc}\cdot M_{yc}) & \text{if } L_b \le L_p \\ \min\left[C_b\cdot\left[1 - \left(1 - \frac{F_{yresid}\cdot S_{botcr}}{R_{pc}\cdot M_{yc}}\right)\cdot\frac{(L_b - L_p)}{(L_r - L_p)}\right]\cdot R_{pc}\cdot M_{yc}, R_{pc}\cdot M_{yc}\right] & \text{if } L_p < L_b \le L_r \\ \min(F_{cr}\cdot S_{botcr}, R_{pc}\cdot M_{yc}) & \text{if } L_b > L_r \end{cases}$$

$$M_{ncLTB} = 1124.2\cdot kip\cdot ft$$

$$M_{r\_neg\_c} := \phi_f\cdot\min(M_{ncLB}, M_{ncLTB}) = 1124.2\cdot kip\cdot ft$$

Governing negative moment capacity: $M_{r\_neg} := \min(M_{r\_neg\_t}, M_{r\_neg\_c}) = 1124.2\cdot kip\cdot ft$

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 $1.25\cdot M_{DC}$

Max Moment applied, Phase 1: (at midspan)

$$M_{int\_P1} := 1.25\, M_{DC1\_int}\left(\frac{L_{str}}{2}\right) = 484\cdot kip\cdot ft \quad \text{(Interior)}$$

$$M_{ext\_P1} := 1.25\, M_{DC1\_ext}\left(\frac{L_{str}}{2}\right) = 492.3\cdot kip\cdot ft \quad \text{(Exterior)}$$

Maximum Stress, Phase 1:

$$f_{int\_P1} := \frac{M_{int\_P1}\cdot y_{steel}}{I_{zsteel}} = 21.9\cdot ksi \quad \text{(Interior)}$$

$$f_{ext\_P1} := \frac{M_{ext\_P1}\cdot y_{steel}}{I_{zsteel}} = 22.3\cdot ksi \quad \text{(Exterior)}$$

Stress limits: $f_{P1\_max} := \phi_{const}\cdot F_y$

$f_{int\_P1} \le f_{P1\_max} = 1 \qquad f_{ext\_P1} \le f_{P1\_max} = 1$

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 $1.25 \cdot M_{DC}$

Max Moment applied, Phase 2: (at midspan) $M_{2\_STR\_I} = 655.8 \cdot kip \cdot ft$

Capacity for positive flexure: $M_n = 2246.4 \cdot kip \cdot ft$

Check Moment: $M_{2\_STR\_I} \le \phi_{const} \cdot M_n = 1$

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 $1.5 \cdot M_{DC}$ when dynamic construction loads are involved (Section 10).

Loads and stresses on stringer during transport and picking: $M_{3\_STR\_I\_neg} = 81.5 \cdot kip \cdot ft$

Concrete rupture stress $f_r := 0.24 \cdot \sqrt{f_c \cdot ksi} = 0.5 \cdot ksi$

Concrete stress during construction not to exceed:

$$f_{cmax} := \phi_{const} \cdot f_r = 0.5 \cdot ksi$$

$$f_{cconst} := \frac{M_{3\_STR\_I\_neg} \cdot y_c}{I_z \cdot n} = 0.1 \cdot ksi$$

$$f_{cconst} \le f_{cmax} = 1$$

Phase 4: Check flexural capacity under dead load and live load for fully installed continuous composite girders.

Strength I Load Combination $\phi_f := 1.0$

$M_{4\_STR\_I} = 1648 \cdot kip \cdot ft \qquad M_{4\_STR\_I\_neg} = -1009.8 \cdot kip \cdot ft$

$M_{4\_STR\_I} \le \phi_f \cdot M_n = 1 \qquad \left|M_{4\_STR\_I\_neg}\right| \le M_{r\_neg} = 1$

Strength III Load Combination

$M_{4\_STR\_III} = 614.9 \cdot kip \cdot ft \qquad M_{4\_STR\_III\_neg} = -459.6 \cdot kip \cdot ft$

$M_{4\_STR\_III} \le \phi_f \cdot M_n = 1 \qquad \left|M_{4\_STR\_III\_neg}\right| \le M_{r\_neg} = 1$

Strength V Load Combination

$M_{4\_STR\_V} = 1411.9 \cdot kip \cdot ft \qquad M_{4\_STR\_V\_neg} = -884 \cdot kip \cdot ft$

$M_{4\_STR\_V} \le \phi_f \cdot M_n = 1 \qquad \left|M_{4\_STR\_V\_neg}\right| \le M_{r\_neg} = 1$

13. FLEXURAL SERVICE CHECKS

Check service load combinations for the fully continuous beam with live load (Phase 4):

under Service II for stress limits - $M_{4\_SRV\_II} = 1250.7 \cdot kip \cdot ft$

$M_{4\_SRV\_II\_neg} = -769.9 \cdot kip \cdot ft$

under Service I for cracking - $M_{4\_SRV\_I\_neg} = -675.6 \cdot kip \cdot ft$

Ignore positive moment for Service I as there is no tension in the concrete in this case.

Service Load Stress Limits:

Top Flange: $f_{tfmax} := 0.95 \cdot R_h \cdot F_y = 47.5 \cdot ksi$

Bottom Flange: $f_{bfmax} := f_{tfmax} = 47.5 \cdot ksi$

Concrete (Negative bending only): $f_r = 0.5 \cdot ksi$

Service Load Stresses, Positive Moment:

Top Flange: $f_{SRVII\_tf} := M_{4\_SRV\_II} \cdot \dfrac{(y_c - t_{slab})}{I_z} = 3.2 \cdot ksi$

$f_{SRVII\_tf} \le f_{tfmax} = 1$

Bottom Flange: $f_{bfs2} := M_{4\_SRV\_II} \cdot \dfrac{(t_{slab} + t_{tf} + D_w + t_{bf} - y_c)}{I_z} = 37.4 \cdot ksi$

$f_l := 0$ $\quad f_{bfs2} + \dfrac{f_l}{2} \le f_{bfmax} = 1$

Service Load Stresses, Negative Moment:

Top (Concrete):

$f_{con.neg} := \dfrac{M_{4\_SRV\_I\_neg} \cdot y_{cneg}}{n \cdot I_{zneg}} = -1.4 \cdot ksi$ Using Service I Loading

$|f_{con.neg}| \le |f_r| = 0$

Bottom Flange: $f_{bfs2.neg} := \dfrac{M_{4\_SRV\_I\_neg} \cdot (t_{slab} + t_{tf} + D_w + t_{bf} - y_{cneg})}{I_{zneg}} = -37.8 \cdot ksi$

$f_{bfs2.neg} \le f_{bfmax} = 1$

Check LL Deflection:

$\Delta_{DT} := 1.104 \cdot in$ from independent Analysis - includes 100% design truck (w/impact), or 25% design truck (w/impact) + 100% lane load

$DF_\delta := \dfrac{3}{12} = 0.3$ Deflection distribution factor = (no. lanes)/(no. stringers)

$\dfrac{L_{str}}{\Delta_{DT} \cdot DF_\delta} = 3021.7$ Equivalent X, where L/X = Deflection*Distribution Factor

$\dfrac{L_{str}}{\Delta_{DT} \cdot DF_\delta} \ge 800 = 1$

14. SHEAR STRENGTH

Shear Capacity based on AASHTO LRFD 6.10.9

Nominal resistance of unstiffened web:

$F_y = 50.0 \cdot ksi$ $\quad D_w = 28.3 \cdot in$ $\quad t_w = 0.5 \cdot in$ $\quad \phi_v := 1.0$ $\quad k := 5$

$V_p := 0.58 \cdot F_y \cdot D_w \cdot t_w = 426.9 \cdot kip$

$$C_1 := \begin{cases} 1.0 & \text{if } \dfrac{D_w}{t_w} \le 1.12 \cdot \sqrt{\dfrac{E_s \cdot k}{F_y}} \\ \left[\dfrac{1.57}{\left(\dfrac{D_w}{t_w}\right)^2} \cdot \left(\dfrac{E_s \cdot k}{F_y}\right)\right] & \text{if } \dfrac{D_w}{t_w} > 1.40 \cdot \sqrt{\dfrac{E_s \cdot k}{F_y}} \\ \left(\dfrac{1.12}{\dfrac{D_w}{t_w}} \cdot \sqrt{\dfrac{E_s \cdot k}{F_y}}\right) & \text{otherwise} \end{cases}$$

$C_1 = 1$

$V_n := C_1 \cdot V_p = 426.9 \cdot kip$

$V_u \le \phi_v \cdot V_n = 1$

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:

$\Delta F_{TH\_1} := 16 \cdot ksi$ Category B: non-coated weathering steel

$\Delta F_{TH\_2} := 12 \cdot ksi$ Category C': Base metal at toe of transverse stiffener fillet welds

$\Delta F_{TH\_3} := 10 \cdot ksi$ Category C: Base metal at shear connectors

Fatigue Moment Ranges at Detail Locations (from analysis):

$M_{FAT\_B} := 301 \cdot kip \cdot ft$ $\quad M_{FAT\_CP} := 285.7 \cdot kip \cdot ft$ $\quad M_{FAT\_C} := 207.1 kip \cdot ft$

$\gamma_{FATI} := 1.5$ $\quad \gamma_{FATII} := 0.75$

$$n_{fat} := \begin{cases} 2 & \text{if } L_{str} \le 40 \cdot ft \\ 1.0 & \text{otherwise} \end{cases}$$

Constants to use for detail checks:

$ADTT_{SL\_INF\_B} := 860$ $\quad A_{FAT\_B} := 120 \cdot 10^8$

$ADTT_{SL\_INF\_CP} := 660$ $\quad A_{FAT\_CP} := 44 \cdot 10^8$

$ADTT_{SL\_INF\_C} := 1290$ $\quad A_{FAT\_C} := 44 \cdot 10^8$

Category B Check: Stress at Bottom Flange, Fatigue I

$$f_{FATI\_B} := \frac{\gamma_{FATI} \cdot M_{FAT\_B} \cdot \left(t_{slab} + t_{tf} + D_w + t_{bf} - y_c\right)}{I_z} = 13.5 \cdot ksi$$

$f_{FATI\_B} \le \Delta F_{TH\_1} = 1$

$$f_{FATII\_B} := \frac{\gamma_{FATII}}{\gamma_{FATI}} \cdot f_{FATI\_B} = 6.8 \cdot ksi$$

$$ADTT_{SL\_B\_MAX} := \begin{vmatrix} \dfrac{ADTT_{SL\_INF\_B}}{n_{fat}} & \text{if } f_{FATI\_B} \le \Delta F_{TH\_1} \\ \dfrac{A_{FAT\_B} \cdot ksi^3}{365 \cdot 75 \cdot n_{fat} \cdot f_{FATII\_B}^{\ 3}} & \text{otherwise} \end{vmatrix} \qquad ADTT_{SL\_B\_MAX} = 860$$

Category C' Check: Stress at base of transverse stiffener (top of bottom flange)

$$f_{FATI\_CP} := \gamma_{FATI} \cdot M_{FAT\_CP} \cdot \frac{(t_{slab} + t_{tf} + D_w - y_c)}{I_z} = 12.5 \cdot ksi$$

$$f_{FATI\_CP} \le \Delta F_{TH\_2} = 0$$

$$f_{FATII\_CP} := \frac{\gamma_{FATII}}{\gamma_{FATI}} \cdot f_{FATI\_CP} = 6.3 \cdot ksi$$

$$ADTT_{SL\_CP\_MAX} := \begin{vmatrix} \dfrac{ADTT_{SL\_INF\_CP}}{n_{fat}} & \text{if } f_{FATI\_CP} \le \Delta F_{TH\_2} \\ \dfrac{A_{FAT\_CP} \cdot ksi^3}{365 \cdot 75 \cdot n_{fat} \cdot f_{FATII\_CP}^{\ 3}} & \text{otherwise} \end{vmatrix} \qquad ADTT_{SL\_CP\_MAX} = 656$$

Category C Check: Stress at base of shear connectors (top of top flange)

$$f_{FATI\_C} := \gamma_{FATI} \cdot M_{FAT\_C} \cdot \frac{(y_c - t_{slab})}{I_z} = 0.8 \cdot ksi$$

$$f_{FATI\_C} \le \Delta F_{TH\_3} = 1$$

$$f_{FATII\_C} := \frac{\gamma_{FATII}}{\gamma_{FATI}} \cdot f_{FATI\_C} = 0.4 \cdot ksi$$

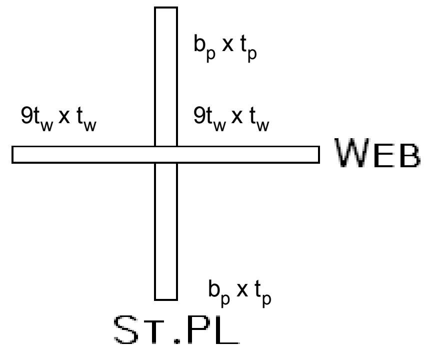
$$ADTT_{SL\_C\_MAX} := \begin{vmatrix} \dfrac{ADTT_{SL\_INF\_C}}{n_{fat}} & \text{if } f_{FATI\_C} \le \Delta F_{TH\_3} \\ \dfrac{A_{FAT\_C} \cdot ksi^3}{365 \cdot 75 \cdot n_{fat} \cdot f_{FATII\_C}^{\ 3}} & \text{otherwise} \end{vmatrix} \qquad ADTT_{SL\_C\_MAX} = 1290$$

FATIGUE CHECK: $ADTT_{SL\_MAX} := \min(ADTT_{SL\_B\_MAX}, ADTT_{SL\_CP\_MAX}, ADTT_{SL\_C\_MAX})$

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:

$t_p := \frac{5}{8}in$ $\quad b_p := 5in$ $\quad \phi_b := 1.0$ $\quad t_{p\_weld} := \left(\frac{5}{16}\right)in$

*Check min weld size

Projecting Width Slenderness Check:

$b_p \le 0.48 t_p \cdot \sqrt{\frac{E_s}{F_y}} = 1$

Stiffener Bearing Resistance:

$A_{pn} := 2 \cdot (b_p - t_{p\_weld}) \cdot t_p$ $\quad A_{pn} = 5.9 \cdot in^2$

$R_{sb\_n} := 1.4 \cdot A_{pn} \cdot F_y$ $\quad R_{sb\_n} = 410.2 \cdot kip$

$R_{sb\_r} := \phi_b \cdot R_{sb\_n}$ $\quad R_{sb\_r} = 410.2 \cdot kip$

$R_{DC} := 26.721kip$ $\quad R_{DW} := 2.62kip$ $\quad R_{LL} := 53.943kip$

$\phi_{DC\_STR\_I} := 1.25$ $\quad \phi_{DW\_STR\_I} := 1.5$ $\quad \phi_{LL\_STR\_I} := 1.75$

$R_u := \phi_{DC\_STR\_I} \cdot R_{DC} + \phi_{DW\_STR\_I} \cdot R_{DW} + \phi_{LL\_STR\_I} \cdot R_{LL}$ $\quad R_u = 131.7 \cdot kip$

$R_u \le R_{sb\_r} = 1$

Weld Check:

$throat := t_{p\_weld} \cdot \frac{\sqrt{2}}{2}$ $\quad throat = 0.2 \cdot in$

$L_{weld} := D_w - 2 \cdot 3in$ $\quad L_{weld} = 22.3 \cdot in$

$A_{eff\_weld} := throat \cdot L_{weld}$ $\quad A_{eff\_weld} = 4.9 \cdot in^2$

$F_{exx} := 70ksi$ $\quad \phi_{e2} := 0.8$

$R_{r\_weld} := 0.6 \cdot \phi_{e2} \cdot F_{exx}$ $\quad R_{r\_weld} = 33.6 \cdot ksi$

$R_{u\_weld} := \frac{R_u}{4 \cdot A_{eff\_weld}}$ $\quad R_{u\_weld} = 6.7 \cdot ksi$

$R_{u\_weld} \le R_{u\_weld} = 1$

Axial Resistance of Bearing Stiffeners: $\quad \phi_c := 0.9$

$A_{eff} := (2 \cdot 9 \cdot t_w + t_p) \cdot t_w + 2 \cdot b_p \cdot t_p$ $\quad A_{eff} = 11.4 \cdot in^2$

$L_{eff} := 0.75 \cdot D_w$ $\quad L_{eff} = 21.2 \cdot in$

$I_{xp} := \frac{2 \cdot 9 \cdot t_w \cdot t_w^3}{12} + \frac{t_p \cdot (2 \cdot b_p + t_w)^3}{12}$ $\quad I_{xp} = 60.7 \cdot in^4$

$I_{yp} := \frac{t_w \cdot (t_p + 2 \cdot 9 \cdot t_w)^3}{12} + \frac{2b_p \cdot t_p^3}{12}$ $\quad I_{yp} = 43.3 \cdot in^4$

$r_p := \sqrt{\frac{\min(I_{xp}, I_{yp})}{A_{eff}}}$ $\quad r_p = 1.9 \cdot in$

$Q := 1$ for bearing stiffeners $\quad K_p := 0.75$

$P_o := Q \cdot F_y \cdot A_{eff} = 572.1 \cdot kip$

$$P_e := \frac{\pi^2 E_s \cdot A_{eff}}{\left(K_p \cdot \frac{L_{eff}}{r_p}\right)^2} = 48919.6 \cdot kip$$

$$P_n := \begin{cases} \left[0.658^{\left(\frac{P_o}{P_e}\right)} \right] \cdot P_o & \text{if } \left(\frac{P_e}{P_o}\right) \geq 0.44 \\ 0.877 \cdot P_e & \text{otherwise} \end{cases}$$ $P_n = 569.3 \cdot kip$

$P_r := \phi_c \cdot P_n$ $P_r = 512.4 \cdot kip$ $R_u \leq P_r = 1$

17. SHEAR CONNECTORS:

Shear Connector design to follow LRFD 6.10.10.

Stud Properties:

$d_s := \frac{7}{8} \cdot in$ Diameter $h_s := 6in$ Height of Stud $\frac{h_s}{d_s} \geq 4 = 1$

$c_s := t_{slab} - h_s$ $c_s \geq 2in = 1$

$s_s := 3.5in$ Spacing $s_s \geq 4d_s = 1$

$n_s := 3$ Studs per row $\frac{\left[b_{tf} - s_s \cdot (n_s - 1) - d_s\right]}{2} \geq 1.0in = 1$

$A_{sc} := \pi \cdot \left(\frac{d_s}{2}\right)^2$ $A_{sc} = 0.6 \cdot in^2$

$F_u := 60ksi$

Fatigue Resistance:

$Z_r := 5.5 \cdot d_s^2 \cdot \frac{kip}{in^2}$ $Z_r = 4.2 \cdot kip$ $Q_{slab} := A_{slab} \cdot (y_c - y_{slab})$ $Q_{slab} = 338.9 \cdot in^3$

$V_f := 47.0kip$

$$V_{fat} := \frac{V_f \cdot Q_{slab}}{I_z} = 1.5 \cdot \frac{kip}{in}$$

$p_s := \frac{n_s \cdot Z_r}{V_{fat}} = 8.7 \cdot in$ $6 \cdot d_s \leq p_s \leq 24in = 1$

Strength Resistance:

$\phi_{sc} := 0.85$

$f_c = 5 \cdot ksi$

$E_c := 33000 \cdot 0.15^{1.5} \cdot \sqrt{f_c ksi} = 4286.8 \cdot ksi$

$Q_n := \min\left(0.5 \cdot A_{sc} \cdot \sqrt{f_c \cdot E_c}, A_{sc} \cdot F_u\right)$ $Q_n = 36.1 \cdot kip$

$Q_r := \phi_{sc} \cdot Q_n$ $Q_r = 30.7 \cdot kip$

$P_{simple} := \min\left(0.85 \cdot f_c \cdot b_{eff} \cdot t_s, F_y \cdot A_{steel}\right)$ $P_{simple} = 1436.2 \cdot kip$

$P_{cont} := P_{simple} + \min\left(0.45 \cdot f_c \cdot b_{eff} \cdot t_s, F_y \cdot A_{steel}\right)$ $P_{cont} = 2282.2 \cdot kip$

$n_{lines} := \frac{P_{cont}}{Q_r \cdot n_s}$ $n_{lines} = 24.8$

Find required stud spacing along the girder (varies as applied shear varies)

$i := 0..23$

$$x := \begin{pmatrix} 0.00 \\ 1.414 \\ 4.947 \\ 8.480 \\ 12.013 \\ 15.546 \\ 19.079 \\ 22.612 \\ 26.145 \\ 29.678 \\ 33.210 \\ 33.917 \\ 34.624 \\ 36.037 \\ 36.743 \\ 40.276 \\ 43.809 \\ 47.342 \\ 50.875 \\ 54.408 \\ 57.941 \\ 61.474 \\ 65.007 \\ 67.833 \end{pmatrix} \cdot ft \quad V_{fi} := \begin{pmatrix} 61.5 \\ 59.2 \\ 56.8 \\ 54.4 \\ 52.0 \\ 49.5 \\ 47.1 \\ 44.7 \\ 42.7 \\ 40.6 \\ 40.6 \\ 40.6 \\ 40.6 \\ 40.6 \\ 40.6 \\ 42.3 \\ 44.2 \\ 46.6 \\ 49.1 \\ 51.5 \\ 53.9 \\ 56.3 \\ 58.7 \\ 61.5 \end{pmatrix} \cdot kip$$

$V_{fati} := \dfrac{V_{fi} \cdot Q_{slab}}{I_z} = $

	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	...

$\cdot \dfrac{kip}{in}$

$P_{max} := \dfrac{n_s \cdot Z_r}{V_{fati}} = $

	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	...

$\cdot in$

$\min(P_{max}) = 6.6 \cdot in$

$\max(P_{max}) = 10.1 \cdot in$

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 $w_c = 150 \cdot pcf$

Deck Thickness for Design $t_{deck} := 8.0in$ $\quad t_{deck} \geq 7in = 1$

Deck Thickness for Loads $t_d = 10.5 \cdot in$

Rebar yield strength $F_s = 60 \cdot ksi$ $\quad$ Strength of concrete $f_c = 5 \cdot ksi$

Concrete clear cover $\quad$ Bottom $\quad$ Top

$c_b := 1.0in$ $\quad c_b \geq 1.0in = 1$ $\quad c_t := 2.5in$ $\quad c_t \geq 2.5in = 1$

Transverse reinforcement

Bottom Reinforcing $\phi_{tb} := \frac{6}{8}in$

Bottom Spacing $s_{tb} := 8in$

$s_{tb} \geq 1.5\phi_{tb} \wedge 1.5in = 1$

$s_{tb} \leq 1.5 \cdot t_{deck} \wedge 18in = 1$

$A_{stb} := \frac{12in}{s_{tb}} \cdot \pi \cdot \left(\frac{\phi_{tb}}{2}\right)^2 = 0.7 \cdot in^2$

Top Reinforcing $\phi_{tt} := \frac{5}{8}in$

Top Spacing $s_{tt} := 8in$

$s_{tt} \geq 1.5\phi_{tt} \wedge 1.5in = 1$

$s_{tt} \leq 1.5 \cdot t_{deck} \wedge 18in = 1$

$A_{stt} := \frac{12in}{s_{tt}} \cdot \pi \cdot \left(\frac{\phi_{tt}}{2}\right)^2 = 0.5 \cdot in^2$

Design depth of Bar $d_{tb} := t_{deck} - \left(c_b + \frac{\phi_{tb}}{2}\right) = 6.6 \cdot in$ $d_{tt} := t_{deck} - \left(c_t + \frac{\phi_{tt}}{2}\right) = 5.2 \cdot in$

Girder Spacing $spacing_{int\_max} := 4ft + 6in$

$spacing_{ext} = 4\ ft$

Equivalent Strip, +M $w_{posM} := \left(26 + 6.6 \cdot \frac{spacing_{int\_max}}{ft}\right) \cdot in$ $w_{posM} = 55.7 \cdot in$

Equivalent Strip, -M $w_{negM} := \left(48 + 3.0 \cdot \frac{spacing_{int\_max}}{ft}\right) \cdot in$ $w_{negM} = 61.5 \cdot in$

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 $w_{deck\_pos} := w_c \cdot t_d \cdot w_{posM}$ $w_{deck\_pos} = 609.2 \cdot plf$

Weight of deck, -M $w_{deck\_neg} := w_c \cdot t_d \cdot w_{negM}$ $w_{deck\_neg} = 672.7 \cdot plf$

Unit weight of barrier $w_b := 433.5plf$

Barrier point load, +M $P_{b\_pos} := w_b \cdot w_{posM}$ $P_{b\_pos} = 2.01 \cdot kip$

Barrier point load, -M $P_{b\_neg} := w_b \cdot w_{negM}$ $P_{b\_neg} = 2.22 \cdot kip$

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 $P_{wheel} := 16kip$

Impact Factor $IM := 1.33$

Multiple presence factors $MP_1 := 1.2$ $MP_2 := 1.0$ $MP_3 := 0.85$

Wheel Loads $P_1 := IM \cdot MP_1 \cdot P_{wheel}$ $P_2 := IM \cdot MP_2 \cdot P_{wheel}$ $P_3 := IM \cdot MP_3 \cdot P_{wheel}$

$P_1 = 25.54 \cdot kip$ $P_2 = 21.3 \cdot kip$ $P_3 = 18.09 \cdot kip$

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

$$M_{pos} := 38.9kip\cdot ft \qquad M_{pos\_dist} := \frac{M_{pos}}{w_{posM}} \qquad M_{pos\_dist} = 8.38\cdot\frac{kip\cdot ft}{ft}$$

$$M_{neg} := -21.0kip\cdot ft \qquad M_{neg\_dist} := \frac{M_{neg}}{w_{negM}} \qquad M_{neg\_dist} = -4.1\cdot\frac{kip\cdot ft}{ft}$$

22. FLEXURAL STRENGTH CAPACITY CHECK:

Consider a 1'-0" strip:

$$\phi_b := 0.9 \qquad b := 12in$$

$$\beta_1 := \begin{cases} 0.85 & \text{if } f_c \le 4ksi \\ 0.85 - 0.05\cdot\left(\frac{f_c}{ksi} - 4\right) & \text{otherwise} \end{cases} \qquad \beta_1 = 0.8$$

Bottom:

$$c_{tb} := \frac{A_{stb}\cdot F_s}{0.85\cdot f_c\cdot\beta_1\cdot b} = 1\cdot in$$

$$a_{tb} := \beta_1\cdot c_{tb} = 0.8\cdot in$$

$$M_{ntb} := \frac{A_{stb}\cdot F_s}{b}\cdot\left(d_{tb} - \frac{a_{tb}}{2}\right) = 20.7\cdot\frac{kip\cdot ft}{ft}$$

$$M_{rtb} := \phi_b\cdot M_{ntb} = 18.6\cdot\frac{kip\cdot ft}{ft}$$

$$M_{rtb} \ge \left|M_{pos\_dist}\right| = 1$$

Top:

$$c_{tt} := \frac{A_{stt}\cdot F_s}{0.85\cdot f_c\cdot\beta_1\cdot b} = 0.7\cdot in$$

$$a_{tt} := \beta_1\cdot c_{tt} = 0.5\cdot in$$

$$M_{ntt} := \frac{A_{stt}\cdot F_s}{ft}\cdot\left(d_{tt} - \frac{a_{tt}}{2}\right) = 11.3\cdot\frac{kip\cdot ft}{ft}$$

$$M_{rtt} := \phi_b\cdot M_{ntt} = 10.2\cdot\frac{kip\cdot ft}{ft}$$

$$M_{rtt} \ge \left|M_{neg\_dist}\right| = 1$$

23. LONGITUDINAL DECK REINFORCEMENT DESIGN:

Longitudinal reinforcement

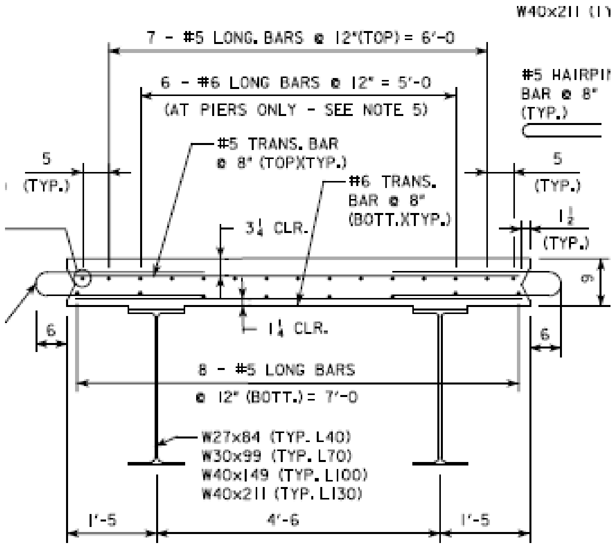
$$\phi_{lb} := \frac{5}{8}in \qquad s_{lb} := 12in \qquad \phi_{lt} := \frac{5}{8}in \qquad s_{lt} := 12in$$

$$A_{slb} := \frac{12in}{s_{lb}}\cdot\pi\cdot\left(\frac{\phi_{lb}}{2}\right)^2 = 0.3\cdot in^2 \qquad A_{slt} := \frac{12in}{s_{lt}}\cdot\pi\left(\frac{\phi_{lt}}{2}\right)^2 = 0.3\cdot in^2$$

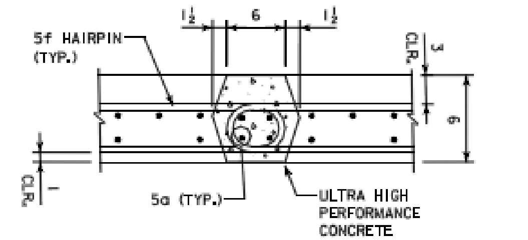
Distribution Reinforcement (AASHTO 9.7.3.2)

$$A_{\%dist} := \frac{\min\left(\frac{220}{\sqrt{\frac{spacing_{int\_max}}{ft}}}, 67\right)}{100} = 67\cdot\%$$

$$A_{dist} := A_{\%dist}\cdot\left(A_{stb}\right) = 0.4\cdot in^2 \qquad A_{slb} + A_{slt} \ge A_{dist} = 1$$



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 $f_r := 0.37 \cdot \sqrt{f_c \cdot ksi} = 0.8 \cdot ksi$ $\quad E_c = 4286.8 \cdot ksi$

$E_s = 29000 \cdot ksi$

Section Modulus $S_{nc} := \dfrac{b \cdot t_{deck}^{\ 2}}{6} = 128 \cdot in^3$

$A_{deck} := t_{deck} \cdot b = 96 \cdot in^2$

$$y_{bar\_tb} := \frac{A_{deck} \cdot \dfrac{t_{deck}}{2} + (n-1) \cdot A_{stb} \cdot d_{tb}}{A_{deck} + (n-1) \cdot A_{stb}} = 4.1 \cdot in$$

$$y_{bar\_tt} := \frac{A_{deck} \cdot \dfrac{t_{deck}}{2} + (n-1) \cdot A_{stt} \cdot d_{tt}}{A_{deck} + (n-1) \cdot A_{stt}} = 4 \cdot in$$

$$I_{tb} := \frac{b \cdot t_{deck}^{\ 3}}{12} + A_{deck} \cdot \left(\frac{t_{deck}}{2} - y_{bar\_tb}\right)^2 + (n-1) \cdot A_{stb} \cdot \left(d_{tb} - y_{bar\_tb}\right)^2 = 538.3 \cdot in^4$$

$$I_{tt} := \frac{b \cdot t_{deck}^{\ 3}}{12} + A_{deck} \cdot \left(\frac{t_{deck}}{2} - y_{bar\_tt}\right)^2 + (n-1) \cdot A_{stt} \cdot \left(d_{tt} - y_{bar\_tt}\right)^2 = 515.8 \cdot in^4$$

$S_{c\_tb} := \dfrac{I_{tb}}{t_{deck} - y_{bar\_tb}} = 138.2 \cdot in^3$ $\quad S_{c\_tt} := \dfrac{I_{tt}}{t_{deck} - y_{bar\_tt}} = 130 \cdot in^3$

Unfactored Dead Load $M_{dnc\_pos\_t} := 1.25 \dfrac{kip \cdot ft}{ft}$ $\quad M_{dnc\_neg\_t} := -0.542 \dfrac{kip \cdot ft}{ft}$

Cracking Moment

$$M_{cr\_tb} := \max\left[\frac{S_{c\_tb} \cdot f_r}{ft} - \left|M_{dnc\_pos\_t}\right| \cdot \left(\frac{S_{c\_tb}}{S_{nc}} - 1\right), \frac{S_{c\_tb} \cdot f_r}{ft}\right] = 9.5 \cdot \frac{kip \cdot ft}{ft}$$

S 5.7.3.3.2

$$M_{cr\_tt} := \max\left[\frac{S_{c\_tt} \cdot f_r}{ft} - \left|M_{dnc\_neg\_t}\right| \cdot \left(\frac{S_{c\_tt}}{S_{nc}} - 1\right), \frac{S_{c\_tt} \cdot f_r}{ft}\right] = 9 \cdot \frac{kip \cdot ft}{ft}$$

Minimum Factored Flexural Resistance

$$M_{r\_min\_tb} := \min\left(1.2 \cdot M_{cr\_tb}, 1.33 \cdot \left|M_{pos\_dist}\right|\right) = 11.1 \cdot \frac{kip \cdot ft}{ft} \qquad M_{rtb} \geq M_{r\_min\_tb} = 1$$

$$M_{r\_min\_tt} := \min\left(1.2 \cdot M_{cr\_tt}, 1.33 \cdot \left|M_{neg\_dist}\right|\right) = 5.4 \cdot \frac{kip \cdot ft}{ft} \qquad M_{rtt} \geq M_{r\_min\_tt} = 1$$

CHECK CRACK CONTROL (AASHTO LRFD 5.7.3.4):

$$\gamma_{eb} := 1.0 \qquad \gamma_{et} := 0.75$$

$$M_{SL\_pos} := 29.64 kip \cdot ft \qquad M_{SL\_neg} := 29.64 kip \cdot ft$$

$$M_{SL\_pos\_dist} := \frac{M_{SL\_pos}}{w_{posM}} = 6.4 \cdot \frac{kip \cdot ft}{ft} \qquad M_{SL\_neg\_dist} := \frac{M_{SL\_neg}}{w_{negM}} = 5.8 \cdot \frac{kip \cdot ft}{ft}$$

$$f_{ssb} := \frac{M_{SL\_pos\_dist} \cdot b \cdot n}{\frac{I_{tb}}{d_{tb} - y_{bar\_tb}}} = 2.5 \cdot ksi \qquad f_{sst} := \frac{M_{SL\_neg\_dist} \cdot b \cdot n}{\frac{I_{tt}}{d_{tt} - y_{bar\_tt}}} = 1.1 \cdot ksi$$

$$d_{cb} := c_b + \frac{\phi_{tb}}{2} = 1.4 \cdot in \qquad d_{ct} := c_t + \frac{\phi_{tt}}{2} = 2.8 \cdot in$$

$$\beta_{sb} := 1 + \frac{d_{cb}}{0.7 \cdot \left(t_{deck} - d_{cb}\right)} = 1.3 \qquad \beta_{st} := 1 + \frac{d_{ct}}{0.7 \cdot \left(t_{deck} - d_{ct}\right)} = 1.8$$

$$s_b := \frac{700 \cdot \gamma_{eb} \cdot kip}{\beta_{sb} \cdot f_{ssb} \cdot in} - 2 \cdot d_{cb} = 212.2 \cdot in \qquad s_t := \frac{700 \cdot \gamma_{et} \cdot kip}{\beta_{st} \cdot f_{sst} \cdot in} - 2 \cdot d_{ct} = 266.5 \cdot in$$

$$s_{tb} \leq s_b = 1 \qquad s_{tt} \leq s_t = 1$$

SHRINKAGE AND TEMPERATURE REINFORCING (AASHTO LRFD 5.10.8):

$$A_{st} := \left| \begin{array}{l} \frac{1.30 \cdot b \cdot t_{deck}}{2 \cdot \left(b + t_{deck}\right) \cdot F_s} \cdot \frac{kip}{in} \ \text{ if } 0.11 in^2 \leq \frac{1.30 \cdot b \cdot t_{deck}}{2 \cdot \left(b + t_{deck}\right) \cdot F_s} \cdot \frac{kip}{in} \leq 0.60 in^2 \\ 0.11 in^2 \ \text{ if } \frac{1.30 \cdot b \cdot t_{deck}}{2 \cdot \left(b + t_{deck}\right) \cdot F_s} \cdot \frac{kip}{in} < 0.11 in^2 \\ 0.60 in^2 \ \text{ if } \frac{1.30 \cdot b \cdot t_{deck}}{2 \cdot \left(b + t_{deck}\right) \cdot F_s} \cdot \frac{kip}{in} > 0.60 in^2 \end{array} \right. = 0.1 \cdot in^2$$

$$A_{stb} \geq A_{st} = 1 \qquad A_{stt} \geq A_{st} = 1$$

$$A_{slb} \geq A_{st} = 1 \qquad A_{slt} \geq A_{st} = 1$$

SHEAR RESISTANCE (AASHTO LRFD 5.8.3.3):

$$\phi := 0.9 \qquad \beta := 2 \qquad \theta := 45 deg \qquad b = 1\ ft$$

$$d_{v\_tb} := \max\left(0.72 \cdot t_{deck}, d_{tb} - \frac{a_{tb}}{2}, 0.9 \cdot d_{tb}\right) = 6.2 \cdot in$$

$$d_{v\_tt} := \max\left(0.72 \cdot t_{deck}, d_{tt} - \frac{a_{tt}}{2}, 0.9 \cdot d_{tt}\right) = 5.8 \cdot in$$

$$d_v := \min\left(d_{v\_tb}, d_{v\_tt}\right) = 5.8 \cdot in$$

$$V_c := 0.0316 \cdot \beta \cdot \sqrt{f_c \cdot ksi} \cdot b \cdot d_v = 9.8 \cdot kip$$

$V_s := 0kip$ Shear capacity of reinforcing steel

$V_{ps} := 0kip$ Shear capacity of prestressing steel

$$V_{ns} := \min\left(V_c + V_s + V_{ps}, 0.25 \cdot f_c \cdot b \cdot d_v + V_{ps}\right) = 9.8 \cdot kip$$

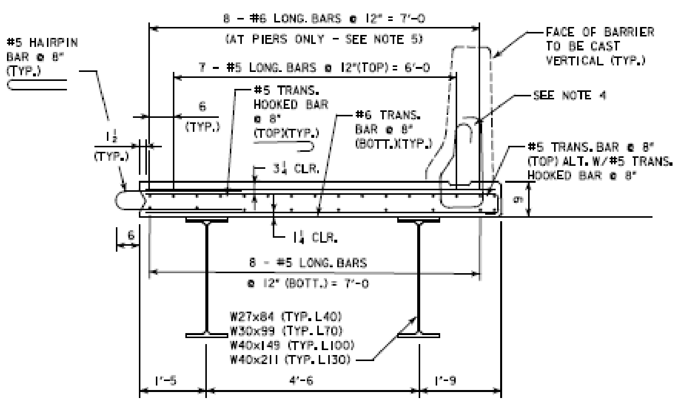
$V_r := \phi \cdot V_{ns} = 8.8 \cdot kip$ Total factored resistance

$V_{us} := 8.38kip$ Total factored load $V_r \ge V_{us} = 1$

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	$W_{par} := w_c \cdot A_p = 426 \cdot plf$		
Width at base	$w_{base} := 1ft + 5in$	Average width of wall	$w_{wall} := \frac{13in + 9.5in}{2} = 11.3 \cdot in$
Height of top portion of parapet	$h_1 := 2ft$	Width at top of parapet	$width_1 := 9.5 \cdot in = 9.5 \cdot in$
Height of middle portion of parapet	$h_2 := 7in$	Width at middle transition of parapet	$width_2 := 12 \cdot in = 12 \cdot in$

Height of lower portion of parapet $h_3 := 3in$ Width at base of parapet $width_3 := 1ft + 5 \cdot in = 17 \cdot in$

$b_1 := width_1$ $b_2 := width_2 - width_1$ $b_3 := width_3 - width_2$

Parapet Center of Gravity

$$CG_p := \frac{(h_1 + h_2 + h_3) \cdot \frac{b_1^2}{2} + \frac{1}{2} \cdot h_1 \cdot b_2 \cdot \left(b_1 + \frac{b_2}{3}\right) \ldots + (h_2 + h_3) \cdot (b_2 + b_3) \cdot \left(b_1 + \frac{b_2 + b_3}{2}\right) - \frac{1}{2} \cdot h_2 \cdot b_3 \cdot \left(b_1 + b_2 + \frac{2b_3}{3}\right)}{(h_1 + h_2 + h_3) \cdot b_1 + \frac{1}{2} \cdot h_1 \cdot b_2 + (h_2 + h_3) \cdot (b_2 + b_3) - \frac{1}{2} \cdot h_2 \cdot b_3} = 6.3 \cdot in$$

Parapet Reinforcement

	Vertically Aligned Bars in Wall	Horizontal Bars
Rebar spacing:	$s_{pa} := 12in$	$n_{pl} := 5$
Rebar Diameter:	$\phi_{pa} := \frac{5}{8} in$	$\phi_{pl} := \frac{5}{8} in$
Rebar Area:	$A_{st\_p} := \pi \cdot \left(\frac{\phi_{pa}}{2}\right)^2 \cdot \frac{b}{s_{pa}} = 0.3 \cdot in^2$	$A_{sl\_p} := \pi \cdot \left(\frac{\phi_{pl}}{2}\right)^2 = 0.3 \cdot in^2$
Cover:	$c_{st} := 3in$	$c_{sl} := 2in + \phi_{pa} = 2.6 \cdot in$
Effective Depth:	$d_{st} := w_{base} - c_{st} - \frac{\phi_{pa}}{2} = 13.7 \cdot in$	$d_{sl} := w_{wall} - c_{sl} - \frac{\phi_{pl}}{2} = 8.3 \cdot in$

Parapet Moment Resistance About Horizontal Axis: $\phi_{ext} := 1.0$

Depth of Equivalent Stress Block: $a_h := \frac{A_{st\_p} \cdot F_s}{0.85 \cdot f_c \cdot b} = 0.4 \cdot in$

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 $w_1 := \frac{width_1 + width_2}{2} = 10.7 \cdot in$

Cover $c_{st1} := 2in$

Depth $d_{h1} := w_1 - c_{st1} - \frac{\phi_{pa}}{2} = 8.4 \cdot in$

Factored Moment Resistance $\phi M_{nh1} := \frac{\phi_{ext} \cdot A_{st\_p} \cdot F_s \cdot \left(d_{h1} - \frac{a_h}{2}\right)}{b} = 12.7 \cdot \frac{kip \cdot ft}{ft}$

Moment Capacity of Middle Segment of Barrier (about longitudinal axis):

Average width of section $w_2 := \frac{width_2 + width_3}{2} = 14.5 \cdot in$

Cover $c_{st2} := 3in$

Depth $d_{h2} := w_2 - c_{st2} - \frac{\phi_{pa}}{2} = 11.2 \cdot in$

Factored Moment Resistance $\phi M_{nh2} := \frac{\phi_{ext} \cdot A_{st\_p} \cdot F_s \cdot \left(d_{h2} - \frac{a_h}{2}\right)}{b} = 16.9 \cdot \frac{kip \cdot ft}{ft}$

Parapet Base Moment Resistance (about longitudinal axis):

development in tension

$$c_{st3} := 3in \qquad cover_{base\_vert} := c_{st3} + \frac{\phi_{pa}}{2} = 3.3 \cdot in$$

$$m_{inc\_ta} := \begin{vmatrix} 1.5 & \text{if } c_{st3} < 3 \cdot \phi_{pa} \vee s_{pa} - \phi_{pa} < 6 \cdot \phi_{pa} \\ 1.2 & \text{otherwise} \end{vmatrix} = 1.2$$

$$m_{dec\_ta} := \begin{vmatrix} 0.8 & \text{if } s_{pa} \geq 6in \\ 1.0 & \text{otherwise} \end{vmatrix} = 0.8$$

$$l_{db\_ta} := \begin{vmatrix} \max\left(\dfrac{1.25in \cdot A_{st\_p} \cdot \dfrac{F_s}{kip}}{\sqrt{\dfrac{f_c}{ksi}}}, 0.4 \cdot \phi_{pa} \cdot \dfrac{F_s}{ksi}\right) & \text{if } \phi_{pa} \leq \dfrac{11}{8}in \\ \dfrac{2.70in \cdot \dfrac{F_s}{ksi}}{\sqrt{\dfrac{f_c}{ksi}}} & \text{if } \phi_{pa} = \dfrac{14}{8}in \\ \dfrac{3.50in \cdot \dfrac{F_s}{ksi}}{\sqrt{\dfrac{f_c}{ksi}}} & \text{if } \phi_{pa} = \dfrac{18}{8}in \end{vmatrix}$$

$$l_{dt\_ta} := l_{db\_ta} \cdot m_{inc\_ta} \cdot m_{dec\_ta} = 14.4 \cdot in$$

hooked bar developed in tension

$$l_{hb\_ta} := \frac{38 \cdot \phi_{pa}}{\sqrt{\dfrac{f_c}{ksi}}} = 10.6 \cdot in \qquad m_{inc} = 1.2$$

$$l_{dh\_ta} := \max(6in, 8 \cdot \phi_{pa}, m_{inc} \cdot l_{hb\_ta}) = 12.7 \cdot in$$

lap splice in tension

$$l_{lst\_ta} := \max(12in, 1.3 \cdot l_{dt\_ta}) = 18.7 \cdot in$$

$$benefit := l_{dt\_ta} - l_{dh\_ta} = 1.7 \cdot in$$

$$l_{dev\_a} := \left(7 + \frac{13}{16}\right)in$$

$$F_{dev} := \frac{benefit + l_{dev\_a}}{l_{dt\_ta}} = 0.7$$

$$F_d := 0.75$$

Distance from NA to Compressive Face

$$c_{t\_b} := \frac{F_d \cdot A_{st\_p} \cdot F_s}{0.85 \cdot f_c \cdot \beta_1 \cdot b} = 0.3 \cdot in$$ S 5.7.3.1.2-4

Depth of Equivalent Stress Block

$$a_t := \beta_1 \cdot c_{t\_b} = 0.3 \cdot in$$ S 5.7.3.2.3

Nominal Moment Resistance

$$M_{nt} := F_d \cdot A_{st\_p} \cdot F_s \cdot \left(d_{st} - \frac{a_t}{2}\right) = 15.6 \cdot kip \cdot ft$$ S 5.7.3.2.2-1

Factored Moment Resistance

$$M_{cb} := \phi_{ext} \cdot \frac{M_{nt}}{ft} = 15.6 \cdot \frac{kip \cdot ft}{ft}$$ S 5.7.3.2

Average Moment Capacity of Barrier (about longitudinal axis):

Factored Moment Resistance about Horizontal Axis

$$M_c := \frac{\phi M_{nh1} \cdot h_1 + \phi M_{nh2} \cdot h_2 + M_{cb} \cdot h_3}{h_1 + h_2 + h_3} = 13.8 \cdot \frac{kip \cdot ft}{ft}$$

Parapet Moment Resistance (about vertical axis):

Height of Transverse Reinforcement in Parapet

$y_1 := 5in$

$y_2 := 11.5in$

$y_3 := 18in$

$y_4 := 24.5in$

$y_5 := 31in$

Width of Parapet at Transverse Reinforcement

$$x_1 := width_3 - \frac{(y_1 - h_3) \cdot b_3}{h_2} = 15.6 \cdot in$$

$$x_2 := b_1 + b_2 - \frac{(y_2 - h_3 - h_2) \cdot b_2}{h_1} = 11.8 \cdot in$$

$$x_3 := b_1 + b_2 - \frac{(y_3 - h_3 - h_2) \cdot b_2}{h_1} = 11.2 \cdot in$$

$$x_4 := b_1 + b_2 - \frac{(y_4 - h_3 - h_2) \cdot b_2}{h_1} = 10.5 \cdot in$$

$$x_5 := b_1 + b_2 - \frac{(y_5 - h_3 - h_2) \cdot b_2}{h_1} = 9.8 \cdot in$$

Depth of Equivalent Stress Block

$$a := \frac{n_{pl} \cdot A_{sl\_p} \cdot F_s}{0.85 \cdot f_c \cdot H_{par}} = 0.6 \cdot in$$

Concrete Cover in Parapet

$cover_r := 2in$

$$cover_{rear} := cover_r + \phi_{pa} + \frac{\phi_{pl}}{2} = 2.9 \cdot in$$

$$cover_{base} := c_{st3} + \phi_{pa} + \frac{\phi_{pl}}{2} = 3.9 \cdot in$$

$cover_f := 2in$

$$cover_{front} := 2in + \phi_{pa} + \frac{\phi_{pl}}{2}$$

$$cover_t := \frac{x_5}{2} = 4.9 \cdot in$$

$cover_{top} := cover_t = 4.9 \cdot in$

Design depth

$d_{1i} := x_1 - cover_{base} = 11.6 \cdot in$ $\quad d_{1o} := x_1 - cover_{rear} = 12.6 \cdot in$

$d_{2i} := x_2 - cover_{front} = 8.9 \cdot in$ $\quad d_{2o} := x_2 - cover_{rear} = 8.9 \cdot in$

$d_{3i} := x_3 - cover_{front} = 8.2 \cdot in$ $\quad d_{3o} := x_3 - cover_{rear} = 8.2 \cdot in$

$d_{4i} := x_4 - cover_{front} = 7.6 \cdot in$ $\quad d_{4o} := x_4 - cover_{rear} = 7.6 \cdot in$

$d_{5i} := x_5 - cover_{top} = 4.9 \cdot in$ $\quad d_{5o} := x_5 - cover_{top} = 4.9 \cdot in$

Nominal Moment Resistance - Tension on Inside Face

$$\phi Mn_{1i} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{1i} - \frac{a}{2}\right) = 208.3 \cdot kip \cdot in$$

$$\phi Mn_{2i} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{2i} - \frac{a}{2}\right) = 158.1 \cdot kip \cdot in$$

$$\phi Mn_{3i} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{3i} - \frac{a}{2}\right) = 145.6 \cdot kip \cdot in$$

$$\phi Mn_{4i} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{4i} - \frac{a}{2}\right) = 133.2 \cdot kip \cdot in$$

$$\phi Mn_{5i} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{5i} - \frac{a}{2}\right) = 84.5 \cdot kip \cdot in$$

$$M_{wi} := \phi Mn_{1i} + \phi Mn_{2i} + \phi Mn_{3i} + \phi Mn_{4i} + \phi Mn_{5i} = 60.8 \cdot kip \cdot ft$$

Nominal Moment Resistance - Tension on Outside Face

$$\phi Mn_{1o} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{1o} - \frac{a}{2}\right) = 18.9 \cdot kip \cdot ft$$

$$\phi Mn_{2o} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{2o} - \frac{a}{2}\right) = 13.2 \cdot kip \cdot ft$$

$$\phi Mn_{3o} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{3o} - \frac{a}{2}\right) = 12.1 \cdot kip \cdot ft$$

$$\phi Mn_{4o} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{4o} - \frac{a}{2} \right) = 11.1 \cdot kip \cdot ft$$

$$\phi Mn_{5o} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{5o} - \frac{a}{2} \right) = 7 \cdot kip \cdot ft$$

$$M_{wo} := \phi Mn_{1o} + \phi Mn_{2o} + \phi Mn_{3o} + \phi Mn_{4o} + \phi Mn_{5o} = 62.3 \cdot kip \cdot ft$$

Vertical Nominal Moment Resistance of Parapet

$$M_w := \frac{2 \cdot M_{wi} + M_{wo}}{3} = 61.3 \cdot kip \cdot ft$$

Parapet Design Factors:

Crash Level

$$CL := \text{"TL-4"}$$

Transverse Design Force

$$F_t := \begin{cases} 13.5kip & \text{if } CL = \text{"TL-1"} \\ 27.0kip & \text{if } CL = \text{"TL-2"} \\ 54.0kip & \text{if } CL = \text{"TL-3"} \\ 54.0kip & \text{if } CL = \text{"TL-4"} \\ 124.0kip & \text{if } CL = \text{"TL-5"} \\ 175.0kip & \text{otherwise} \end{cases} = 54 \cdot kip \qquad L_t := \begin{cases} 4.0ft & \text{if } CL = \text{"TL-1"} \\ 4.0ft & \text{if } CL = \text{"TL-2"} \\ 4.0ft & \text{if } CL = \text{"TL-3"} \\ 3.5ft & \text{if } CL = \text{"TL-4"} \\ 8.0ft & \text{if } CL = \text{"TL-5"} \\ 8.0ft & \text{otherwise} \end{cases} = 3.5 \cdot ft$$

Longitudinal Design Force

$$F_l := \begin{cases} 4.5kip & \text{if } CL = \text{"TL-1"} \\ 9.0kip & \text{if } CL = \text{"TL-2"} \\ 18.0kip & \text{if } CL = \text{"TL-3"} \\ 18.0kip & \text{if } CL = \text{"TL-4"} \\ 41.0kip & \text{if } CL = \text{"TL-5"} \\ 58.0kip & \text{otherwise} \end{cases} = 18 \cdot kip \qquad L_l := \begin{cases} 4.0ft & \text{if } CL = \text{"TL-1"} \\ 4.0ft & \text{if } CL = \text{"TL-2"} \\ 4.0ft & \text{if } CL = \text{"TL-3"} \\ 3.5ft & \text{if } CL = \text{"TL-4"} \\ 8.0ft & \text{if } CL = \text{"TL-5"} \\ 8.0ft & \text{otherwise} \end{cases} = 3.5 \cdot ft$$

Vertical Design Force (Down)

$$F_v := \begin{cases} 4.5kip & \text{if } CL = \text{"TL-1"} \\ 4.5kip & \text{if } CL = \text{"TL-2"} \\ 4.5kip & \text{if } CL = \text{"TL-3"} \\ 18.0kip & \text{if } CL = \text{"TL-4"} \\ 80.0kip & \text{if } CL = \text{"TL-5"} \\ 80.0kip & \text{otherwise} \end{cases} = 18 \cdot kip \qquad L_v := \begin{cases} 18.0ft & \text{if } CL = \text{"TL-1"} \\ 18.0ft & \text{if } CL = \text{"TL-2"} \\ 18.0ft & \text{if } CL = \text{"TL-3"} \\ 18.0ft & \text{if } CL = \text{"TL-4"} \\ 40.0ft & \text{if } CL = \text{"TL-5"} \\ 40.0ft & \text{otherwise} \end{cases} = 18 \cdot ft$$

Critical Length of Yield Line Failure Pattern:

$$M_b := 0kip \cdot ft$$

$$L_c := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2} \right)^2 + \frac{8 \cdot H_{par} \cdot (M_b + M_w)}{M_c}} = 11.9 \cdot ft$$

S A13.3.1-2

$$R_w := \frac{2}{2 \cdot L_c - L_t} \cdot \left(8 \cdot M_b + 8 \cdot M_w + \frac{M_c \cdot L_c^2}{H_{par}} \right) = 116.2 \cdot kip$$

S A13.3.1-1

$$T := \frac{R_w \cdot b}{L_c + 2 \cdot H_{par}} = 6.6 \cdot kip$$

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

S A13.4.1
S Table 3.4.1-1

$\phi_{ext} = 1 \qquad \gamma_{DC} := 1.0 \qquad \gamma_{DW} := 1.0 \qquad \gamma_{LL} := 0.5$

$l_{lip} := 2in \qquad w_{base} = 17 \cdot in$

$A_{deck\_1A} := t_{deck} \cdot (l_{lip} + w_{base}) = 152 \cdot in^2 \qquad A_p = 2.8 \cdot ft^2$

$W_{deck\_1A} := w_c \cdot A_{deck\_1A} = 0.2 \cdot klf \qquad W_{par} = 0.4 \cdot klf$

$$M_{DCdeck\_1A} := \gamma_{DC} \cdot W_{deck\_1A} \cdot \frac{l_{lip} + w_{base}}{2} = 0.1 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{DCpar\_1A} := \gamma_{DC} \cdot W_{par} \cdot (l_{lip} + CG_p) = 0.3 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{total\_1A} := M_{cb} + M_{DCdeck\_1A} + M_{DCpar\_1A} = 16 \cdot \frac{kip \cdot ft}{ft}$$

$\phi_{tt\_add} := \frac{5}{8} in \qquad s_{tt\_add} := 8in$

$$A_{stt\_p} := \frac{12in}{s_{tt}} \cdot \pi \cdot \left(\frac{\phi_{tt}}{2}\right)^2 + \frac{12in}{s_{tt\_add}} \cdot \pi \cdot \left(\frac{\phi_{tt\_add}}{2}\right)^2 = 0.9 \cdot in^2$$

$$d_{tt\_add} := t_{deck} - \left(c_t + \frac{\phi_{tt\_add}}{2}\right) = 5.2 \cdot in$$

$$c_{tt\_p} := \frac{A_{stt\_p} \cdot F_s}{0.85 \cdot f_c \cdot \beta_1 \cdot b} = 1.4 \cdot in \qquad a_{tt\_p} := \beta_1 \cdot c_{tt\_p} = 1.1 \cdot in$$

$$M_{ntt\_p} := \frac{A_{stt\_p} \cdot F_s}{ft} \cdot \left(d_{tt\_add} - \frac{a_{tt\_p}}{2}\right) = 21.4 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{rtt\_p} := \phi_b \cdot M_{ntt\_p} = 19.2 \cdot \frac{kip \cdot ft}{ft} \qquad M_{rtt\_p} \geq M_{total\_1A} = 1$$

$A_{sT} := A_{stt} + A_{stb} = 1.1 \cdot in^2$

$\phi P_n := \phi_{ext} \cdot A_{sT} \cdot F_s = 67.4 \cdot kip \qquad \phi P_n \geq T = 1$

$$M_{u\_1A} := M_{rtt\_p} \cdot \left(1 - \frac{T}{\phi P_n}\right) = 17.4 \cdot \frac{kip \cdot ft}{ft} \qquad M_{u\_1A} \geq M_{total\_1A} = 1$$

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.

$A_{deck\_1B} := t_{deck} \cdot L_o = 168 \cdot in^2 \qquad A_p = 2.8 \cdot ft^2$

$W_{deck\_1B} := w_c \cdot A_{deck\_1B} = 0.2 \cdot klf$ $\qquad W_{par} = 0.4 \cdot klf$

$M_{DCdeck\_1B} := \gamma_{DC} \cdot W_{deck\_1B} \cdot \frac{L_o}{2} = 0.2 \cdot \frac{kip \cdot ft}{ft}$

$M_{DCpar\_1B} := \gamma_{DC} \cdot W_{par} \cdot \left(L_o - l_{lip} - CG_p\right) = 0.5 \cdot \frac{kip \cdot ft}{ft}$

$L_{spread\_B} := L_o - l_{lip} - width_3 = 2 \cdot in$ $\qquad spread := 30deg$

$w_{spread\_B} := L_{spread\_B} \cdot \tan(spread) = 1.2 \cdot in$

$M_{cb\_1B} := \frac{M_{cb} \cdot L_c}{L_c + 2 \cdot w_{spread\_B}} = 15.3 \cdot \frac{kip \cdot ft}{ft}$

$M_{total\_1B} := M_{cb\_1B} + M_{DCdeck\_1B} + M_{DCpar\_1B} = 15.9 \cdot \frac{kip \cdot ft}{ft}$

$M_{rtt\_p} = 19.2 \cdot \frac{kip \cdot ft}{ft}$ $\qquad M_{rtt\_p} \geq M_{total\_1B} = 1$

$\phi P_n = 67.4 \cdot kip$

$P_u := \frac{T \cdot \left(L_c + 2 \cdot H_{par}\right)}{L_c + 2 \cdot H_{par} + 2 \cdot w_{spread\_B}} = 6.5 \cdot kip$ $\qquad \phi P_n \geq P_u = 1$

$M_{u\_1B} := M_{rtt\_p} \cdot \left(1 - \frac{P_u}{\phi P_n}\right) = 17.4 \cdot \frac{kip \cdot ft}{ft}$ $\qquad M_{u\_1B} \geq M_{total\_1B} = 1$

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.

$M_{par\_G1} := M_{DCpar\_1B} = 0.5 \cdot \frac{kip \cdot ft}{ft}$

$M_{par\_G2} := -0.137 \frac{kip \cdot ft}{ft}$ $\qquad$ (From model output)

$M_1 := M_{cb} = 15.6 \cdot \frac{kip \cdot ft}{ft}$

$M_2 := M_1 \cdot \frac{M_{par\_G2}}{M_{par\_G1}} = -4.7 \cdot \frac{kip \cdot ft}{ft}$

$b_f := 10.5in$

$M_{c\_M2M1} := M_1 + \frac{\frac{1}{4} \cdot b_f \cdot \left(-M_1 + M_2\right)}{spacing_{int\_max}} = 14.6 \cdot \frac{kip \cdot ft}{ft}$

$L_{spread\_C} := L_o - l_{lip} - w_{base} + \frac{b_f}{4} = 4.6 \cdot in$

$w_{spread\_C} := L_{spread\_C} \cdot \tan(spread) = 2.7 \cdot in$

$M_{cb\_1C} := \frac{M_{c\_M2M1} \cdot L_c}{L_c + 2 \cdot w_{spread\_C}} = 14.1 \cdot \frac{kip \cdot ft}{ft}$

$$M_{total\_1C} := M_{cb\_1C} + M_{DCdeck\_1B} + M_{DCpar\_1B} = 14.7 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{rtt\_p} = 19.2 \cdot \frac{kip \cdot ft}{ft} \qquad M_{rtt\_p} \geq M_{total\_1C} = 1$$

$$\phi P_n = 67.4 \cdot kip$$

$$P_{uC} := \frac{T \cdot (L_c + 2 \cdot H_{par})}{L_c + 2 \cdot H_{par} + 2 \cdot w_{spread\_C}} = 6.4 \cdot kip \qquad \phi P_n \geq P_{uC} = 1$$

$$M_{u\_1C} := M_{rtt\_p} \cdot \left(1 - \frac{P_u}{\phi P_n}\right) = 17.4 \cdot \frac{kip \cdot ft}{ft} \qquad M_{u\_1B} \geq M_{total\_1B} = 1$$

Compute Overhang Reinforcement Cut-off Length Requirement

Maximum crash load moment at theoretical cut-ff point:

$$M_{c\_max} := M_{rtt} = 10.2 \cdot \frac{kip \cdot ft}{ft}$$

$$L_{Mc\_max} := \frac{M_2 - M_{rtt}}{M_2 - M_1} \cdot spacing_{int\_max} = 3.3 \cdot ft$$

$$L_{spread\_D} := L_o - l_{lip} - w_{base} + L_{Mc\_max} = 41.6 \cdot in$$

$$w_{spread\_D} := L_{spread\_D} \cdot \tan(spread) = 24 \cdot in$$

$$M_{cb\_max} := \frac{M_{c\_max} \cdot L_c}{L_c + 2 \cdot w_{spread\_D}} = 7.6 \cdot \frac{kip \cdot ft}{ft}$$

$$extension := \max(d_{tt\_add}, 12 \cdot \phi_{tt\_add}, 0.0625 \cdot spacing_{int\_max}) = 7.5 \cdot in$$

$$cutt\_off := L_{Mc\_max} + extension = 47.1 \cdot in$$

$$A_{tt\_add} := \pi \cdot \left(\frac{\phi_{tt\_add}}{2}\right)^2 = 0.3 \cdot in^2$$

$$m_{thick\_tt\_add} := \begin{vmatrix} 1.4 \ \text{if} \ t_{deck} - c_t \geq 12in \\ 1.0 \ \text{otherwise} \end{vmatrix} = 1$$

$$m_{epoxy\_tt\_add} := \begin{vmatrix} 1.5 \ \text{if} \ c_t < 3 \cdot \phi_{tt\_add} \vee \frac{s_{tt\_add}}{2} - \phi_{tt\_add} < 6 \cdot \phi_{tt\_add} \\ 1.2 \ \text{otherwise} \end{vmatrix} = 1.5$$

$$m_{inc\_tt\_add} := \min(m_{thick\_tt\_add} \cdot m_{epoxy\_tt\_add}, 1.7) = 1.5$$

$$m_{dec\_tt\_add} := \begin{vmatrix} 0.8 \ \text{if} \ \frac{s_{tt\_add}}{2} \geq 6in \\ 1.0 \ \text{otherwise} \end{vmatrix} = 1$$

$$l_{db\_tt\_add} := \begin{cases} \max\left(\dfrac{1.25in \cdot A_{tt\_add} \cdot \dfrac{F_s}{kip}}{\sqrt{\dfrac{f_c}{ksi}}}, 0.4 \cdot \phi_{tt\_add} \cdot \dfrac{F_s}{ksi}\right) & \text{if } \phi_{tt\_add} \le \dfrac{11}{8} in \\ \dfrac{2.70in \cdot \dfrac{F_s}{ksi}}{\sqrt{\dfrac{f_c}{ksi}}} & \text{if } \phi_{tt\_add} = \dfrac{14}{8} in \\ \dfrac{3.50in \cdot \dfrac{F_s}{ksi}}{\sqrt{\dfrac{f_c}{ksi}}} & \text{if } \phi_{tt\_add} = \dfrac{18}{8} in \end{cases} \qquad l_{db\_tt\_add} = 15 \cdot in$$

$l_{dt\_tt\_add} := l_{db\_tt\_add} \cdot m_{inc\_tt\_add} \cdot m_{dec\_tt\_add} = 22.5 \cdot in$

$Cuttoff_{point} := L_{Mc\_max} + l_{dt\_tt\_add} - spacing_{int\_max} = 8.1 \cdot in$ extension past second interior girder

Check for Cracking in Overhang under Service Limit State:

Does not govern - no live load on overhang.

25. COMPRESSION SPLICE:

See sheet S7 for drawing.

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: $M_{LLPier} := 541.8 \cdot kip \cdot ft$

Factored LL moment: $M_{UPier} := 1.75 \cdot M_{LLPier} = 948.1 \cdot kip \cdot ft$

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: $I_z = 10959.8 \cdot in^4$

Distance to center of bottom flange from composite section centroid: $y_{bf} := \dfrac{t_{bf}}{2} + D_w + t_{tf} + t_{slab} - y_c = 27 \cdot in$

Stress in bottom flange: $f_{bf} := M_{UPier} \cdot \dfrac{y_{bf}}{I_z} = 28 \cdot ksi$

Calculate Bottom Flange Force:

Design Stress: $F_{bf} := \max\left(\dfrac{f_{bf} + F_y}{2}, 0.75 \cdot F_y\right) = 39 \cdot ksi$

Effective Flange Area: $A_{ef} := b_{bf} \cdot t_{bf} = 7 \cdot in^2$

Force in Flange: $C_{nf} := F_{bf} \cdot A_{ef} = 273.2 \cdot kip$

Calculate Bottom Flange Stress, Ignoring Concrete:

Moment of inertia: $I_{zsteel} = 3923.8 \cdot in^4$

Distance to center of bottom flange: $y_{bfsteel} := \dfrac{t_{bf}}{2} + D_w + t_{tf} - y_{steel} = 14.5 \cdot in$

Stress in bottom flange: $f_{bfsteel} := M_{UPier} \cdot \frac{y_{bfsteel}}{I_{zsteel}} = 42 \cdot ksi$

Bottom Flange Force for design:

Design Stress: $F_{cf} := \max\left(\frac{f_{bfsteel} + F_y}{2}, 0.75 \cdot F_y\right) = 46 \cdot ksi$

Design Force: $C_n := \max\left(F_{bf}, F_{cf}\right) \cdot A_{ef} = 322.1 \cdot kip$

Compression Splice Plate Dimensions:

Bottom Splice Plate:	$b_{bsp} := b_{bf} = 10.4 \cdot in$	$t_{bsp} := 0.75in$	$A_{bsp} := b_{bsp} \cdot t_{bsp} = 7.8 \cdot in^2$
Built-Up Angle Splice Plate Horizontal Leg:	$b_{asph} := 4.25in$	$t_{asph} := 0.75in$	$A_{asph} := 2 \cdot b_{asph} \cdot t_{asph} = 6.4 \cdot in^2$
Built-Up Angle Splice Plate Vertical Leg:	$b_{aspv} := 7.75in$	$t_{aspv} := 0.75in$	$A_{aspv} := 2 \cdot b_{aspv} \cdot t_{aspv} = 11.6 \cdot in^2$

Total Area: $A_{csp} := A_{bsp} + A_{asph} + A_{aspv} = 25.8 \cdot in^2$

Average Stress: $f_{cs} := \frac{C_n}{A_{csp}} = 12.5 \cdot ksi$

Proportion Load into each plate based on area:

$$C_{bsp} := \frac{C_n \cdot A_{bsp}}{A_{csp}} = 97.7 \cdot kip \qquad C_{asph} := \frac{C_n \cdot A_{asph}}{A_{csp}} = 79.5 \cdot kip \qquad C_{aspv} := \frac{C_n \cdot A_{aspv}}{A_{csp}} = 144.9 \cdot kip$$

Check Plates Compression Capacity:

Bottom Splice Plate: $k_{cps} := 0.75$ for bolted connection

$l_{cps} := 9in$

$$r_{bsp} := \sqrt{\frac{\min\left(\frac{b_{bsp} \cdot t_{bsp}^3}{12}, \frac{t_{bsp} \cdot b_{bsp}^3}{12}\right)}{A_{bsp}}} = 0.2 \cdot in$$

$$P_{ebsp} := \frac{\pi^2 \cdot E_s \cdot A_{bsp}}{\left(\frac{k_{cps} \cdot l_{cps}}{r_{bsp}}\right)^2} = 2307.9 \cdot kip$$

$$Q_{bsp} := \begin{cases} 1.0 & \text{if } \frac{b_{bsp}}{t_{bsp}} \le 0.45 \cdot \sqrt{\frac{E_s}{F_y}} \\ \left[1.34 - 0.76 \cdot \left(\frac{b_{bsp}}{t_{bsp}}\right) \cdot \sqrt{\frac{F_y}{E_s}}\right] & \text{if } 0.45 \cdot \sqrt{\frac{E_s}{F_y}} \le \frac{b_{bsp}}{t_{bsp}} \le 0.91 \cdot \sqrt{\frac{E_s}{F_y}} \\ \frac{0.53 \cdot E_s}{F_y \cdot \left(\frac{b_{bsp}}{t_{bsp}}\right)^2} & \text{otherwise} \end{cases} = 0.9$$

$$P_{obsp} := Q_{bsp} \cdot F_y \cdot A_{bsp} = 352.8 \cdot kip$$

$$P_{nbsp} := \begin{array}{|l} \left[\left[0.658^{\left(\frac{P_{obsp}}{P_{ebsp}}\right)}\right]\cdot P_{obsp}\right] \text{ if } \frac{P_{ebsp}}{P_{obsp}} \ge 0.44 \\ \left(0.877\cdot P_{ebsp}\right) \text{ otherwise} \end{array} = 330.9\cdot kip$$

$$P_{nbsp\_allow} := 0.9\cdot P_{nbsp} = 297.8\cdot kip \qquad Check := \begin{array}{|l} \text{"NG" if } C_{bsp} \ge P_{nbsp\_allow} \\ \text{"OK" if } P_{nbsp\_allow} \ge C_{bsp} \end{array} = \text{"OK"}$$

Horizontal Angle Leg: $k_{cps} = 0.75$ for bolted connection

$$l_{cps} = 9\cdot in$$

$$r_{asph} := \sqrt{\frac{\min\left(\frac{b_{asph}\cdot t_{asph}^3}{12}, \frac{t_{asph}\cdot b_{asph}^3}{12}\right)}{A_{asph}}} = 0.153\cdot in$$

$$P_{easph} := \frac{\pi^2\cdot E_s\cdot A_{asph}}{\left(\frac{k_{cps}\cdot l_{cps}}{r_{asph}}\right)^2} = 938.6\cdot kip$$

$$Q_{asph} := \begin{array}{|l} 1.0 \text{ if } \frac{b_{asph}}{t_{asph}} \le 0.45\cdot\sqrt{\frac{E_s}{F_y}} \\ \left[1.34 - 0.76\cdot\left(\frac{b_{asph}}{t_{asph}}\right)\cdot\sqrt{\frac{F_y}{E_s}}\right] \text{ if } 0.45\cdot\sqrt{\frac{E_s}{F_y}} \le \frac{b_{asph}}{t_{asph}} \le 0.91\cdot\sqrt{\frac{E_s}{F_y}} \\ \frac{0.53\cdot E_s}{F_y\cdot\left(\frac{b_{asph}}{t_{asph}}\right)^2} \text{ otherwise} \end{array} = 1$$

$$P_{oasph} := Q_{asph}\cdot F_y\cdot A_{asph} = 318.7\cdot kip$$

$$P_{nasph} := \begin{array}{|l} \left[\left[0.658^{\left(\frac{P_{oasph}}{P_{easph}}\right)}\right]\cdot P_{oasph}\right] \text{ if } \frac{P_{easph}}{P_{oasph}} \ge 0.44 \\ \left(0.877\cdot P_{easph}\right) \text{ otherwise} \end{array} = 276.5\cdot kip$$

$$P_{nasph\_allow} := 0.9\cdot P_{nasph} = 248.9\cdot kip \qquad Check2 := \begin{array}{|l} \text{"NG" if } C_{asph} \ge P_{nasph\_allow} \\ \text{"OK" if } P_{nasph\_allow} \ge C_{asph} \end{array} = \text{"OK'}$$

Vertical Angle Leg: $k_{cps} = 0.75$ for bolted connection

$$l_{cps} = 9\cdot in$$

$$r_{aspv} := \sqrt{\frac{\min\left(\frac{b_{aspv}\cdot t_{aspv}^3}{12}, \frac{t_{aspv}\cdot b_{aspv}^3}{12}\right)}{A_{aspv}}} = 0.153\cdot in$$

$$P_{easpv} := \frac{\pi^2\cdot E_s\cdot A_{aspv}}{\left(\frac{k_{cps}\cdot l_{cps}}{r_{aspv}}\right)^2} = 1711.6\cdot kip$$

$$Q_{aspv} := \begin{cases} 1.0 & \text{if } \dfrac{b_{aspv}}{t_{aspv}} \le 0.45\cdot\sqrt{\dfrac{E_s}{F_y}} \\ \left[1.34 - 0.76\cdot\left(\dfrac{b_{aspv}}{t_{aspv}}\right)\cdot\sqrt{\dfrac{F_y}{E_s}}\right] & \text{if } 0.45\cdot\sqrt{\dfrac{E_s}{F_y}} \le \dfrac{b_{aspv}}{t_{aspv}} \le 0.91\cdot\sqrt{\dfrac{E_s}{F_y}} \\ \dfrac{0.53\cdot E_s}{F_y\cdot\left(\dfrac{b_{aspv}}{t_{aspv}}\right)^2} & \text{otherwise} \end{cases} = 1$$

$$P_{oaspv} := Q_{aspv}\cdot F_y\cdot A_{aspv} = 581.2\cdot kip$$

$$P_{naspv} := \begin{cases} \left[\left[0.658^{\left(\frac{P_{oaspv}}{P_{easpv}}\right)}\right]\cdot P_{oaspv}\right] & \text{if } \dfrac{P_{easpv}}{P_{oaspv}} \ge 0.44 \\ (0.877\cdot P_{easpv}) & \text{otherwise} \end{cases} = 504.2\cdot kip$$

$$P_{naspv\_allow} := 0.9\cdot P_{naspv} = 453.8\cdot kip \qquad Check3 := \begin{cases} \text{"NG"} & \text{if } C_{aspv} \ge P_{naspv\_allow} \\ \text{"OK"} & \text{if } P_{naspv\_allow} \ge C_{aspv} \end{cases} = \text{"OK'}$$

Additional Checks: Design Bolted Connections of the splice plates to the girders, checking for shear, bearing, and slip critical connections.

26. CLOSURE POUR DESIGN:

See sheet S2 for drawing of closure pour.

Check the closure pour according to the negative bending capacity of the section.

Use the minimum reinforcing properties for design, to be conservative.

$$A_{steel} = 28.7\cdot in^2 \qquad A_{rt} = 1.8\cdot in^2 \qquad A_{rb} = 2.6\cdot in^2$$

$$cg_{steel} := t_{slab} + y_{steel} = 22.8\cdot in \qquad cg_{rt} := 3in + 1.5\cdot\frac{5}{8}in = 3.9\cdot in \qquad cg_{rb} := t_{slab} - \left(1in + 1.5\cdot\frac{5}{8}in\right) = 6.1\cdot in$$

Overall CG: $A_{neg} := A_{steel} + A_{rt} + A_{rb} = 33.1\cdot in^2$ $\qquad cg_{neg} := \dfrac{A_{steel}\cdot cg_{steel} + A_{rt}\cdot cg_{rt} + A_{rb}\cdot cg_{rb}}{A_{neg}} = 20.5\cdot in$

Moment of Inertia: $I_{zstl} := 3990in^4$

$$I_{neg} := I_{zstl} + A_{steel}\cdot(cg_{steel} - cg_{neg})^2 + A_{rt}\cdot(cg_{rt} - cg_{neg})^2 + A_{rb}\cdot(cg_{rb} - cg_{neg})^2 = 5183.7\cdot in^4$$

Section Moduli:

$$S_{top\_neg} := \frac{I_{neg}}{cg_{neg} - cg_{rt}} = 313.4\cdot in^3 \qquad r_{neg} := \sqrt{\frac{I_{neg}}{A_{neg}}} = 12.5\cdot in$$

$$S_{bot\_neg} := \frac{I_{neg}}{(t_{slab} + t_{tf} + D_w + t_{bf} - cg_{neg})} = 301.9\cdot in^3$$

Concrete Properties: $f_c = 5\cdot ksi$ $\qquad E_c = 4286.8\cdot ksi$

Steel Properties: $F_y = 50\cdot ksi$ $\qquad E_s = 29000\cdot ksi$ $\qquad L_{bneg} := 13.42ft$

$$F_{yr} := 0.7 \cdot F_y = 35 \cdot ksi$$

Negative Flexural Capacity:

Slenderness ratio for compressive flange: $\lambda_{fneg} := \dfrac{b_{bf}}{2 \cdot t_{bf}} = 7.8$

Limiting ratio for compactness: $\lambda_{pfneg} := 0.38 \cdot \sqrt{\dfrac{E_s}{F_y}} = 9.2$

Limiting ratio for noncompact $\lambda_{rfneg} := 0.56 \cdot \sqrt{\dfrac{E_s}{F_{yr}}} = 16.1$

Hybrid Factor: $R_h = 1$

$$D_{cneg2} := \frac{D_w}{2} = 14.2 \cdot in \qquad a_{wc} := \frac{2 \cdot D_{cneg2} \cdot t_w}{b_{bf} \cdot t_{bf}} = 2.1$$

$$R_b := \left| \begin{array}{l} 1.0 \;\; \text{if } 2 \cdot \dfrac{D_{cneg2}}{t_w} \le 5.7 \cdot \sqrt{\dfrac{E_s}{F_y}} \\ \min\left[1.0, 1 - \dfrac{a_{wc}}{1200 + 300 \cdot a_{wc}} \cdot \left(2 \cdot \dfrac{D_{cneg2}}{t_w} - 5.7 \cdot \sqrt{\dfrac{E_s}{F_y}}\right)\right] \;\; \text{otherwise} \end{array} \right.$$

$$R_b = 1$$

Flange compression resistance:

$$F_{nc1} := \left| \begin{array}{l} R_b \cdot R_h \cdot F_y \;\; \text{if } \lambda_{fneg} \le \lambda_{pfneg} \\ \left[\left[1 - \left(1 - \dfrac{F_{yr}}{R_h \cdot F_y}\right) \cdot \dfrac{(\lambda_{fneg} - \lambda_{pfneg})}{(\lambda_{rfneg} - \lambda_{pfneg})}\right] \cdot R_b \cdot R_h \cdot F_y\right] \;\; \text{otherwise} \end{array} \right.$$

$$F_{nc1} = 50 \cdot ksi$$

Lateral Torsional Buckling Resistance:

$$r_{tneg} := \frac{b_{bf}}{\sqrt{12 \cdot \left(1 + \dfrac{D_{cneg2} \cdot t_w}{3 \cdot b_{bf} \cdot t_{bf}}\right)}} = 2.6 \cdot in$$

$$L_{pneg} := 1.0 \cdot r_{tneg} \cdot \sqrt{\frac{E_s}{F_y}} = 62.5 \cdot in$$

$$L_{rneg} := \pi \cdot r_{tneg} \cdot \sqrt{\frac{E_s}{F_{yr}}} = 234.7 \cdot in$$

$$C_b = 1$$

$$F_{nc2} := \left| \begin{array}{l} R_b \cdot R_h \cdot F_y \;\; \text{if } L_{bneg} \le L_{pneg} \\ \min\left[C_b \cdot \left[1 - \left(1 - \dfrac{F_{yr}}{R_h \cdot F_y}\right) \cdot \dfrac{(L_{bneg} - L_{pneg})}{(L_{rneg} - L_{pneg})}\right] \cdot R_b \cdot R_h \cdot F_y, R_b \cdot R_h \cdot F_y\right] \end{array} \right.$$

$$F_{nc2} = 41.4 \cdot ksi$$

Compressive Resistance: $F_{nc} := \min(F_{nc1}, F_{nc2}) = 41.4 \cdot ksi$

Tensile Flexural Resistance: $F_{nt} := R_h \cdot F_y = 50 \cdot ksi$ For Strength

$F_{nt\_Serv} := 0.95 \cdot R_h \cdot F_y = 47.5 \cdot ksi$ For Service

Ultimate Moment Resistance: $M_{n\_neg} := \min\left(F_{nt} \cdot S_{top\_neg}, F_{nc} \cdot S_{bot\_neg}\right) = 1042 \cdot kip \cdot ft$

$M_{UPier} = 948.1 \cdot kip \cdot ft$ from external FE analysis

$Check4 := M_{n\_neg} \geq M_{UPier} = 1$

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.

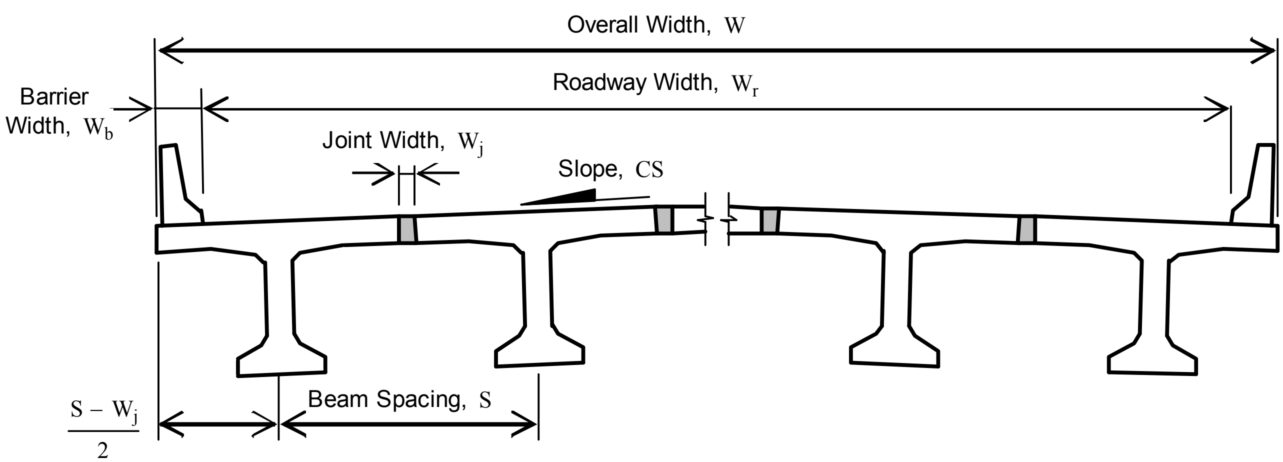
ABC SAMPLE CALCULATION – 2

Decked Precast Prestressed Concrete girder Design for ABC

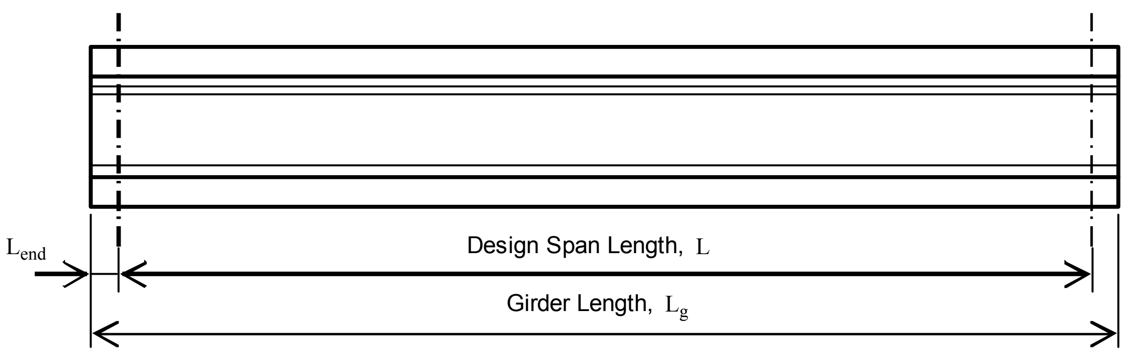
DECKED PRECAST PRESTRESSED CONCRETE GIRDER DESIGN FOR ABC

Unit Definition: $kcf \equiv kip \cdot ft^{-3}$

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:

$L := 70 \cdot ft$ $\qquad L_{end} := 2 \cdot ft$ $\qquad skew := 0 \cdot deg$

$W := 47.167 \cdot ft$ $\qquad W_b := 1.5 \cdot ft$

$S_{max} := 8 \cdot ft$ $\qquad W_j := 0.5 \cdot ft$

$N_g := ceil\left(\frac{W + W_j}{S_{max}}\right) = 6$ Minimum number of girders in cross-section

$S := \frac{W + W_j}{N_g} = 7.945 \cdot ft$ Girder spacing

ORDER OF CALCULATIONS

1. Introduction
2. Design Philosophy
3. Design Criteria
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

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. Sacrifical wearing thickness, use of stainles 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 the 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 transporation 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:

$W_{pc.max} := 200 \cdot kip$ 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

$S_{max} = 8 \cdot ft$ Upper limit on girder spacing and, therefore, girder flange width (defined on first page)

Prestress Limits:

$F_{hd.single} := 4 \cdot kip$ Maximum hold-down force for a single strand

$F_{hd.group} := 48 \cdot kip$ 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.

3. DESIGN CRITERIA (cont'd)

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.

$f_{t.all.ser} := 0 \cdot ksi$ 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.

$f_{c.rel}(f) := 0.80 \cdot f$ 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 considerd 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

$f_{c.erec}(f) := 0.90 \cdot f$ Assumed attained concrete strength during lifting and transportation

$FS_c := 1.5$ Factor of safety against cracking during lifting transportation

$f_{t.erec}(f) := \dfrac{-0.24 \cdot \sqrt{f \cdot ksi}}{FS_c}$ 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.

$h := 42 \cdot in$	Beam section depth
$t_{flange} := 9 \cdot in$	Flange thickness at tip
$t_{sac} := 1 \cdot in$	Total sacrificial depth for grinding and wear

$b_1 := 26 \cdot in$ $\quad b_2 := 26 \cdot in$ $\quad d_1 := 7 \cdot in$

$b_2 = 26 \cdot in$ $\quad b_3 := 6 \cdot in$ $\quad d_2 := 3 \cdot in$

$b_3 = 6 \cdot in$ $\quad b_4 := 6 \cdot in$

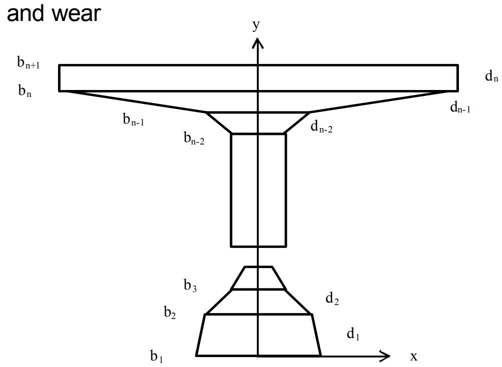
$b_4 = 6 \cdot in$ $\quad b_5 := 10 \cdot in$ $\quad d_4 := 2 \cdot in$

$b_5 = 10 \cdot in$ $\quad b_6 := 49 \cdot in$ $\quad d_5 := 3 \cdot in$

$b_6 = 49 \cdot in$ $\quad b_7 := S - W_j$ $\quad d_6 := 0 \cdot in$

$b_7 = 89.334 \cdot in$ $\quad b_8 := S - W_j$ $\quad d_7 := t_{flange} - t_{sac}$

$d_3 := h - t_{sac} - \sum d$ $\quad d_3 = 18 \cdot in$

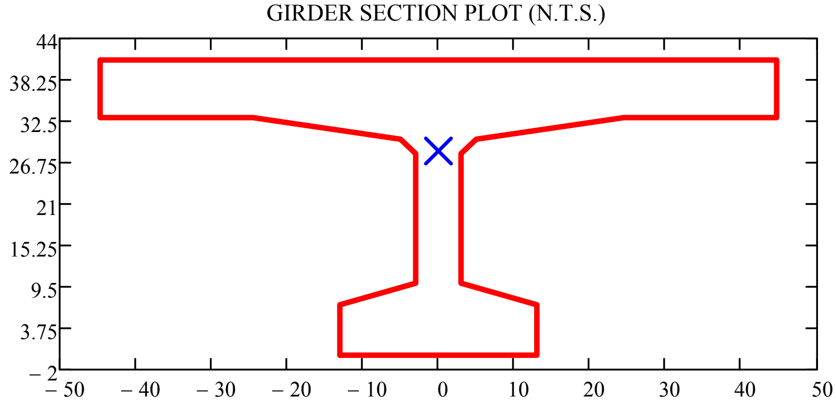


TYPICAL GIRDER SECTION COMPRISED OF n TRAPEZOIDAL REGIONS

Gross Section Properties

$b_f = 89.334 \cdot in$		Precast girder flange width
$A_g = 1157.172 \cdot in^2$		Cross-sectional area (does not include sacrifical thickness)
$I_{xg} = 203462 \cdot in^4$		Moment of inertia (does not include sacrificial thickness)
$y_{tg} = 12.649 \cdot in$	$y_{bg} = -28.351 \cdot in$	Top and bottom fiber distances from neutal axis (positive up)
$S_{tg} = 16085.5 \cdot in^3$	$S_{bg} = -7176.5 \cdot in^3$	Top and bottom section moduli
$I_{yg} = 493395 \cdot in^4$		Weak-axis moment of inertia

GIRDER SECTION PLOT (N.T.S.)



5. MATERIAL PROPERTIES

Concrete:

$f_c := 8 \cdot ksi$ Minimum 28-day compressive strength of concrete

$f_{ci} := f_{c.rel}(f_c) = 6.4 \cdot ksi$ Minimum strength of concrete at release

$\gamma_c := .150 \cdot kcf$ Unit weight of concrete

$K_1 := 1.0$ Correction factor for standard aggregate (5.4.2.4)

$E_{ci} := 33000 \cdot K_1 \cdot \left(\frac{\gamma_c}{kcf}\right)^{1.5} \cdot \sqrt{f_{ci} \cdot ksi} = 4850 \cdot ksi$ Modulus of elasticity at release (5.4.2.4-1)

$E_c := 33000 \cdot K_1 \cdot \left(\frac{\gamma_c}{kcf}\right)^{1.5} \cdot \sqrt{f_c \cdot ksi} = 5422 \cdot ksi$ Modulus of elasticity (5.4.2.4-1)

$f_{r.cm} := 0.37 \cdot \sqrt{f_c \cdot ksi} = 1.047 \cdot ksi$ Modulus of rupture for cracking moment (5.4.2.6)

$f_{r.cd} := 0.24 \cdot \sqrt{f_c \cdot ksi} = 0.679 \cdot ksi$ Modulus of rupture for camber and deflection (5.4.2.6)

$H := 70$ Relative humidity (5.4.2.3)

Prestressing Steel:

$f_{pu} := 270 \cdot ksi$ Ultimate tensile strength

$f_{py} := 0.9 \cdot f_{pu} = 243 \cdot ksi$ Yield strength, low-relaxation strand (Table 5.4.4.1-1)

$f_{pbt.max} := 0.75 \cdot f_{pu} = 202.5 \cdot ksi$ Maximum stress in steel immediately prior to transfer

$f_{pe.max} := 0.80 \cdot f_{py} = 194.4 \cdot ksi$ Maximum stress in steel after all losses

$E_p := 28500 \cdot ksi$ Modulus of elasticity (5.4.4.2)

$d_{ps} := 0.5 \cdot in$ Strand diameter

$A_p := 0.153 \cdot in^2$ 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:

$f_y := 60 \cdot ksi$ Specified minimum yield strength

$E_s := 29000 \cdot ksi$ 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.

$BeamLoc := 1$	Location of beam within the cross-section (0 - Interior, 1 - Exterior)

Load at Release:

$\gamma_{c.DL} := .155 \cdot kcf$	Concrete density used for weight calculations
$A_{g.DL} := A_g + t_{sac} \cdot (S - W_j) = 1246.506 \cdot in^2$	Area used for weight calculations, including sacrificial thickness
$w_g := A_{g.DL} \cdot \gamma_{c.DL} = 1.342 \cdot klf$	Uniform load due to self-weight, including sacrificial thickness
$L_g := L + 2 \cdot L_{end} = 74 \cdot ft$	Span length at release
$M_{gr}(x) := \frac{w_g \cdot x}{2} \cdot (L_g - x)$	Moment due to beam self-weight (supported at ends)
$V_{gr}(x) := w_g \cdot \left(\frac{L_g}{2} - x\right)$	Shear due to beam self-weight (supported at ends)

Load at Erection:

$M_g(x) := \frac{w_g \cdot x}{2} \cdot (L - x)$	Moment due to beam self-weight
$V_g(x) := w_g \cdot \left(\frac{L}{2} - x\right)$	Shear due to beam self-weight
$w_{bar} := 0.430 \cdot klf$	Uniform load due to barrier weight, exterior beams only
$w_{bar} := if(BeamLoc = 1, w_{bar}, 0) = 0.43 \cdot klf$	Redfine to 0 if interior beam (BeamLoc = 0)
$M_{bar}(x) := \frac{w_{bar} \cdot x}{2} \cdot (L - x)$	Moment due to beam self-weight
$V_{bar}(x) := w_{bar} \cdot \left(\frac{L}{2} - x\right)$	Shear due to beam self-weight

6. **PERMANENT LOADS (cont'd)**

Load at Service:

$p_{fws} := 25 \cdot psf$ — Assumed weight of future wearing surface

$w_{fws} := p_{fws} \cdot S = 0.199 \cdot klf$ — Uniform load due to future wearing surface

$M_{fws}(x) := \dfrac{w_{fws} \cdot x}{2} \cdot (L - x)$ — Moment due to future wearing surface

$V_{fws}(x) := w_{fws} \cdot \left(\dfrac{L}{2} - x\right)$ — Shear due to future wearing surface

$w_j := W_j \cdot d_7 \cdot \gamma_{c.DL} = 0.052 \cdot klf$ — Uniform load due to weight of longitudinal closure joint

$M_j(x) := \dfrac{w_j \cdot x}{2} \cdot (L - x)$ — Moment due to longitudinal closure joint

$V_j(x) := w_j \cdot \left(\dfrac{L}{2} - x\right)$ — Shear due to longitudinal closure joint

7. PRECAST LIFTING WEIGHT

Precast Superstructure

$W_g := (w_g + w_{bar}) \cdot L_g = 131.1 \cdot kip$ — Precast girder, including barrier if necessary

Substructure Precast with Superstructure

$L_{corb} := 1 \cdot ft$ — Length of approach slab corbel

$B_{corb} := b_f$ $\quad b_f = 89.334 \cdot in$ — Width of corbel cast with girder

$D_{corb} := 1.5 \cdot ft$ — Average depth of corbel

$V_{corb} := L_{corb} \cdot B_{corb} \cdot D_{corb} = 11.17 \cdot ft^3$ — Volume of corbel

$L_{ia} := 2.167 \cdot ft$ — Length of integral abutment

$L_{gia} := 1.167 \cdot ft$ — Length of girder embedded in integral abutment

$B_{ia} := S - W_j = 7.444 \cdot ft$ — Width of integral abutment cast with girder

$D_{ia} := h + 4 \cdot in = 46 \cdot in$ — Depth of integral abutment

$V_{ia} := V_{corb} + \left[L_{ia} \cdot B_{ia} \cdot D_{ia} - \left(A_g - t_{flange} \cdot b_f\right) \cdot L_{gia}\right] = 70.14 \cdot ft^3$ — Volume of integral abutment cast with girder

$W_{ia} := V_{ia} \cdot \gamma_c = 11 \cdot kip$ — Weight of integral abutment cast with girder

$L_{sa} := 2.167 \cdot ft$ — Length of semi-integral abutment

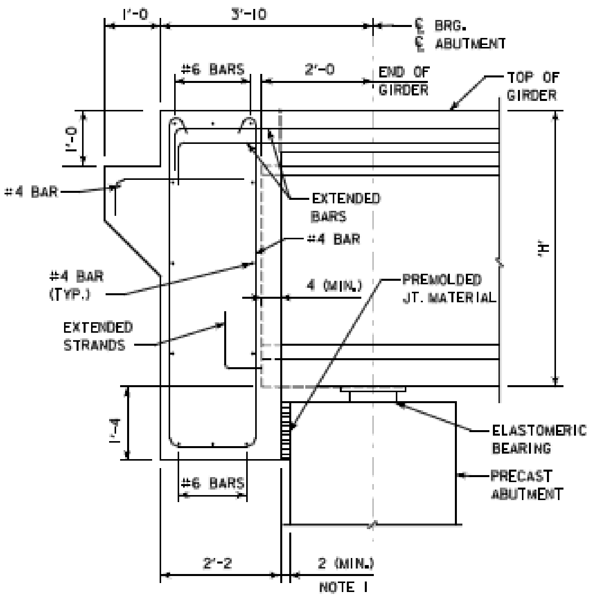
$L_{gsa} := 4 \cdot in$ — Length of girder embedded in semi-integral abutment

$B_{sa} := S - W_j = 7.444 \cdot ft$ — Width of semi-integral abutment cast with girder

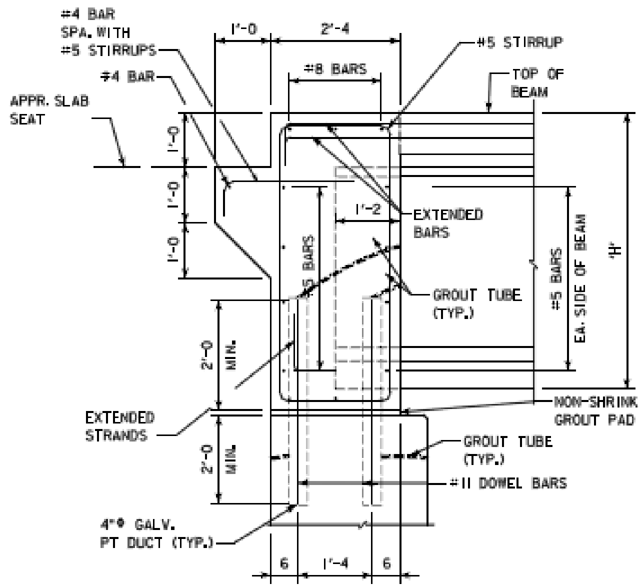
$D_{sa} := h + 16 \cdot in = 58 \cdot in$ — Depth of semi-integral abutment

$V_{sa} := V_{corb} + \left[L_{sa} \cdot B_{sa} \cdot D_{sa} - \left(A_g - t_{flange} \cdot b_f\right) \cdot L_{gsa}\right] = 88.32 \cdot ft^3$ — Volume of semi-integral abutment cast with girder

$W_{sa} := V_{sa} \cdot \gamma_c = 13 \cdot kip$ — 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,

$I_{bb} = 59851 \cdot in^4$ Moment of inertia of section below the top flange

$A_{bb} = 442.5 \cdot in^2$ Area of beam section below the top flange

$e_g := h - \left(t_{sac} + \frac{t_s}{2}\right) + y_{bb} = 22.617 \cdot in$ Distance between c.g.'s of beam and flange

$K_g := 1.0 \cdot \left(I_{bb} + A_{bb} \cdot e_g^2\right) = 286209 \cdot in^4$ 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.

$$\text{CheckMint} := \text{if}\left[(S \le 16 \cdot ft) \cdot (S \ge 3.5 \cdot ft) \cdot \left(t_s \ge 4.5 \cdot in\right) \cdot \left(t_s \le 12.0 \cdot in\right) \cdot (L \ge 20 \cdot ft) \cdot (L \le 240 \cdot ft), \text{"OK"}, \text{"No Good"}\right]$$

$$\text{CheckMint} := \text{if}\left[(\text{CheckMint} = \text{"OK"}) \cdot \left(N_g \ge 4\right) \cdot \left(K_g \ge 10000 \cdot in^4\right) \cdot \left(K_g \le 7000000 \cdot in^4\right), \text{"OK"}, \text{"No Good"}\right]$$

$$\text{CheckMint} = \text{"OK"}$$

$g_{mint1} := 0.06 + \left(\frac{S}{14 \cdot ft}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{L \cdot t_s^3}\right)^{0.1} = 0.458$ Single loaded lane

$g_{mint2} := 0.075 + \left(\frac{S}{9.5 \cdot ft}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{L \cdot t_s^3}\right)^{0.1} = 0.633$ Two or more loaded lanes

$g_{mint} := \max\left(g_{mint1}, g_{mint2}\right) = 0.633$ Distribution factor for moment at interior beams

8. LIVE LOAD (cont'd)

From Table 4.6.2.2.2d-1 for moment in exterior girders,

$$d_e := \frac{S}{2} - W_b = 29.667 \cdot in$$

$$CheckMext := if\left[\left(d_e \geq -1 \cdot ft\right) \cdot \left(d_e \leq 5.5 \cdot ft\right) \cdot \left(N_g \geq 4\right), "OK", "No\ Good"\right] = "OK"$$

For a single loaded lane, use the Lever Rule.

$$g_{mext1} := \frac{\left(S + 0.5 \cdot b_f - W_b - 5 \cdot ft\right)}{S} = 0.65$$ Single loaded lane

$$e_m := 0.77 + \frac{d_e}{9.1 \cdot ft} = 1.042$$

$$g_{mext2} := e_m \cdot g_{mint} = 0.659$$ 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.

$$CheckVint := if\left[(S \leq 16 \cdot ft) \cdot (S \geq 3.5 \cdot ft) \cdot \left(t_s \geq 4.5 \cdot in\right) \cdot \left(t_s \leq 12.0in\right) \cdot (L \geq 20 \cdot ft) \cdot (L \leq 240 \cdot ft), "OK", "No\ Good"\right]$$

$$CheckVint := if\left[(CheckMint = "OK") \cdot \left(N_g \geq 4\right), "OK", "No\ Good"\right]$$

$$CheckVint = "OK"$$

$$g_{vint1} := 0.36 + \left(\frac{S}{25 \cdot ft}\right) = 0.678$$ Single loaded lane

$$g_{vint2} := 0.2 + \left(\frac{S}{12 \cdot ft}\right) - \left(\frac{S}{35 \cdot ft}\right)^{2.0} = 0.811$$ Two or more loaded lanes

$$g_{vint} := max\left(g_{vint1}, g_{vint2}\right) = 0.811$$ Distribution factor for shear at interior beams

8. LIVE LOAD (cont'd)

From Table 4.6.2.2.3b-1 for shear in exterior girders,

For a single loaded lane, use the Lever Rule.

$$\text{CheckVext} := \text{if}\left[\left(d_e \geq -1\cdot\text{ft}\right)\cdot\left(d_e \leq 5.5\cdot\text{ft}\right)\cdot\left(N_g \geq 4\right), \text{"OK"}, \text{"No Good"}\right] = \text{"OK"}$$

$$g_1 := \frac{\left(S + 0.5\cdot b_f - W_b - 5\cdot\text{ft}\right)}{S} = 0.65$$

Single loaded lane (same as for moment)

$$e_v := 0.6 + \frac{d_e}{10\cdot\text{ft}} = 0.847$$

$$g_2 := e_v\cdot g_{vint} = 0.687$$

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,

$$\theta := \text{skew} = 0\cdot\text{deg}$$

$$\text{CheckSkew} := \text{if}\left[(\theta \leq 60\cdot\text{deg})\cdot(3.5\cdot\text{ft} \leq S \leq 16\cdot\text{ft})\cdot(20\cdot\text{ft} \leq L \leq 240\cdot\text{ft})\cdot\left(N_g \geq 4\right), \text{"OK"}, \text{"No Good"}\right] = \text{"OK"}$$

$$c_{skew} := 1.0 + 0.20\cdot\left(\frac{L\cdot t_s^{\,3}}{K_g}\right)^{0.3}\cdot\tan(\theta) = 1.00$$

Correction factor for skew

8. LIVE LOAD (cont'd)

Design Live Load Moment at Midspan:

$w_{lane} := 0.64 \cdot klf$ Design lane load

$P_{truck} := 32 \cdot kip$ Design truck axle load

$IM := 33\%$ Dynamic load allowance (truck only)

$M_{lane}(x) := \dfrac{w_{lane} \cdot x}{2} \cdot (L - x)$ Design lane load moment

$\delta(x) := \dfrac{x \cdot L - x^2}{L}$ Influence coefficient for truck moment calculation

$M_{truck}(x) := P_{truck} \cdot \delta(x) \cdot \max\left[\dfrac{9 \cdot x \cdot (L - x) - 14 \cdot ft \cdot (3 \cdot x + L)}{4 \cdot x \cdot (L - x)}, \dfrac{9 \cdot (L - x) - 84 \cdot ft}{4 \cdot (L - x)}\right]$ Design truck moment

$M_{HL93}(x) := M_{lane}(x) + (1 + IM) \cdot M_{truck}(x)$ HL93 design live load moment per lane

$M_{ll.i}(x) := M_{HL93}(x) \cdot g_{mint}$ Design live load moment at interior beam

$M_{ll.e}(x) := M_{HL93}(x) \cdot g_{mext}$ Design live load moment at exterior beam

$M_{ll}(x) := \text{if}\left(BeamLoc = 1, M_{ll.e}(x), M_{ll.i}(x)\right)$ Design live load moment

Design Live Load Shear:

$V_{lane}(x) := w_{lane} \cdot \left(\dfrac{L}{2} - x\right)$ Design lane load shear

$V_{truck}(x) := P_{truck} \cdot \left(\dfrac{9 \cdot L - 9 \cdot x - 84 \cdot ft}{4 \cdot L}\right)$ Design truck shear

$V_{HL93}(x) := V_{lane}(x) + (1 + IM) \cdot V_{truck}(x)$ HL93 design live load shear

$V_{ll.i}(x) := V_{HL93}(x) \cdot g_{vint}$ Design live load shear at interior beam

$V_{ll.e}(x) := V_{HL93}(x) \cdot g_{vext}$ Design live load shear at exterior beam

$V_{ll}(x) := \text{if}\left(BeamLoc = 1, V_{ll.e}(x), V_{ll.i}(x)\right)$ 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.

$y_{p.est} := 5 \cdot in$ Assumed distance from bottom of beam to centroid of prestress at midspan

$y_{cgp.est} := y_{bg} + y_{p.est} = -23.35 \cdot in$ 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.

$X := \frac{L}{2}$ Distance from support

$M_{dl.ser} := M_g(X) + M_{fws}(X) + M_j(X) + M_{bar}(X) = 1238 \cdot kip \cdot ft$ Total dead load moment

$f_{b.serIII} := \frac{M_{dl.ser} + 0.8 \cdot M_{ll}(X)}{S_{bg}} = -3.567 \cdot ksi$ Total bottom fiber service stress

$f_{pj} := f_{pbt.max} = 202.5 \cdot ksi$ Prestress jacking force

$f_{pe.est} := f_{pj} \cdot \left(1 - \Delta f_{p.est}\right) = 151.9 \cdot ksi$ Estimate of effective prestress force

$A_{ps.est} := A_g \cdot \frac{\left(\frac{-f_{b.serIII} + f_{t.all.ser}}{f_{pe.est}}\right)}{1 + \frac{A_g \cdot y_{cgp.est}}{S_{bg}}} = 5.703 \cdot in^2$ Estimated minimum area of prestressing steel

$N_{ps.est} := ceil\left(\frac{A_{ps.est}}{A_p}\right) = 38$ 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

9. **PRESTRESS PROPERTIES (cont'd)**

$$N_h := \begin{cases} 2 & \text{if } N_{ps} \le 12 \\ 4 & \text{if } 12 < N_{ps} \le 24 \\ 6 & \text{if } 24 < N_{ps} \le 30 \\ 6 + (N_{ps} - 30) & \text{if } N_{ps} > 30 \end{cases}$$

$N_h = 14$

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)

$$N_h := \min\left(N_h + N_{h.add}, 16, 2 \cdot \text{floor}\left(\frac{N_{ps}}{4}\right)\right)$$

$N_h = 16$

16 strands or half of total strands maximum harped in web

$$y_h := 1 \cdot in + (2 \cdot in) \cdot \left(1 + \frac{0.5 \cdot N_h - 1}{2}\right)$$

$y_h = 10 \cdot in$

Centroid of harped strands from bottom, equally spaced

$$y_{hb} := 5 \cdot in$$

Centroid of harped strands from bottom, bundled

$$N_s := N_{ps} - N_h$$

$N_s = 22$

Number of straight strands in flange

$$y_s := 1 \cdot in + \begin{cases} 2 \cdot in & \text{if } N_s \le 10 \\ \dfrac{(4 \cdot in) \cdot N_s - 20 \cdot in}{N_s} & \text{if } 10 < N_s \le 20 \\ \dfrac{(6 \cdot in) \cdot N_s - 60 \cdot in}{N_s} & \text{if } 20 < N_s \le 24 \\ 3.5 \cdot in & \text{otherwise} \end{cases}$$

$y_s = 4.273 \cdot in$

Centroid of straight strands from bottom

$$y_p := \frac{N_s \cdot y_s + N_h \cdot y_{hb}}{N_s + N_h} = 4.579 \cdot in$$

Centroid of prestress from bottom at midspan

$$y_{cgp} := y_{bg} + y_p = -23.77 \cdot in$$

Eccentricity of prestress from neutral axis

$$A_{ps.req} := A_g \cdot \frac{\left(\dfrac{-f_{b.serIII} + f_{t.all.ser}}{f_{pe.est}}\right)}{1 + \dfrac{A_g \cdot y_{cgp}}{S_{bg}}} = 5.623 \cdot in^2$$

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

$$\text{CheckNps} := \text{if}\left[(N_{ps} \le N_{ps.max}) \cdot (N_{ps.req} \le N_{ps}), \text{"OK"}, \text{"No Good"}\right] = \text{"OK"}$$

$$A_{ps.h} := N_h \cdot A_p = 2.448 \cdot in^2$$

Area of prestress in web (harped)

$$A_{ps.s} := N_s \cdot A_p = 3.366 \cdot in^2$$

Area of prestress in flange (straight)

$A_{ps} := A_{ps.h} + A_{ps.s} = 5.814 \cdot in^2$ Total area of prestress

9. PRESTRESS PROPERTIES (cont'd)

Compute transformed section properties based on prestress layout.

Transformed Section Properties

Initial Transformed Section (release):

$A_{ti} = 1185.5 \cdot in^2$

$I_{xti} = 219101 \cdot in^4$

$y_{tti} = 13.217 \cdot in$ $S_{tti} = 16577 \cdot in^3$

$y_{cgpi} = -23.204 \cdot in$ $S_{cgpi} = -9442 \cdot in^3$

$y_{bti} = -27.783 \cdot in$ $S_{bti} = -7886 \cdot in^3$

Final Transformed Section (service):

$A_{tf} = 1181.9 \cdot in^2$

$I_{xtf} = 217153 \cdot in^4$

$y_{ttf} = 13.146 \cdot in$ $S_{ttf} = 16518 \cdot in^3$

$y_{cgpf} = -23.275 \cdot in$ $S_{cgpf} = -9330 \cdot in^3$

$y_{btf} = -27.854 \cdot in$ $S_{btf} = -7796 \cdot in^3$

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.

$P_j := f_{pj} \cdot A_{ps} = 1177.3 \cdot kip$ Jacking force in prestress, prior to losses

$$f_{cgpi} := P_j \cdot \left(\frac{1}{A_{ti}} + \frac{y_{cgpi}}{S_{cgpi}} \right) + \frac{M_{gr}\left(\frac{L_g}{2}\right)}{S_{cgpi}} = 2.719 \cdot ksi$$

Stress in concrete at the level of prestress after instantaneous losses

$\Delta f_{pES} := n_{pi} \cdot f_{cgpi} = 15.978 \cdot ksi$ Prestress loss due to elastic shortening (5.9.5.2.3a-1)

$f_{pi} := f_{pj} - \Delta f_{pES} = 186.522 \cdot ksi$ Initial prestress after instantaneous losses

$P_i := f_{pi} \cdot A_{ps} = 1084.4 \cdot kip$ Initial prestress force

Determine deflection of harped strands required to satisfy allowable stresses at the end of the beam at release.

$f_{c.all.rel} := 0.6 \cdot f_{ci} = 3.84 \cdot ksi$ Allowable compression before losses (5.9.4.1.1)

$f_{t.all.rel} := \max\left(-0.0948 \cdot \sqrt{f_{ci} \cdot ksi}, -0.2 \cdot ksi\right) = -0.200 \cdot ksi$ Allowable tension before losses (Table 5.9.4.1.2-1)

$L_t := 60 \cdot d_{ps} = 2.5 \cdot ft$ Transfer length (AASHTO 5.11.4.1)

$$y_{cgp.t} := \left(\frac{f_{t.all.rel} - \frac{M_{gr}(L_t)}{S_{tti}}}{P_i} - \frac{1}{A_{ti}} \right) \cdot S_{tti} = -18.367 \cdot in$$

Prestress eccentricity required for tension

$$y_{cgp.b} := \left(\frac{f_{c.all.rel} - \frac{M_{gr}(L_t)}{S_{bti}}}{P_i} - \frac{1}{A_{ti}} \right) \cdot S_{bti} = -22.6 \cdot in$$

Prestress eccentricity required for compression

9. PRESTRESS PROPERTIES (cont'd)

$$y_{cgp.req} := \max(y_{cgp.t}, y_{cgp.b}) = -18.367 \cdot in$$

Required prestress eccentricity at end of beam

$$y_{h.brg.req} := \frac{(y_{cgp.req} - y_{bti}) \cdot (N_s + N_h) - y_s \cdot N_s}{N_h} = 16.488 \cdot in$$

Minimum distance to harped prestress centroid from bottom of beam at centerline of bearing

$$y_{top.min} := 18 \cdot in$$

Minimum distance between uppermost strand and top of beam

$$\alpha_{hd} := 0.4$$

Hold-down point, fraction of the design span length

$$slope_{max} := if\left(d_{ps} = 0.6 \cdot in, \frac{1}{12}, \frac{1}{8} \right) = 0.125$$

Maximum slope of an individual strand to limit hold-down force to 4 kips/strand

$$y_{h.brg} := h - y_{top.min} - \left(\frac{0.5 \cdot N_h - 1}{2} \right) \cdot (2 \cdot in) = 17 \cdot in$$

Set centroid of harped strands as high as possible to minimize release and handling stresses

$$y_{h.brg} := \min(y_{h.brg}, y_{hb} + slope_{max} \cdot \alpha_{hd} \cdot L) = 17 \cdot in$$

Verify that slope requirement is satisfied at uppermost strand

$$CheckEndPrestress := if(y_{h.brg} \geq y_{h.brg.req}, \text{"OK"}, \text{"Verify release stresses."}) = \text{"OK"}$$

$$y_{p.brg} := \frac{N_s \cdot y_s + N_h \cdot y_{h.brg}}{N_s + N_h} = 9.632 \cdot in$$

Centroid of prestress from bottom at bearing

$$slope_{cgp} := \frac{y_{p.brg} - y_p}{\alpha_{hd} \cdot L} = 0.015$$

Slope of prestress centroid within the harping length

$$y_{px}(x) := \begin{cases} y_p + slope_{cgp} \cdot (L_{end} + \alpha_{hd} \cdot L - x) & \text{if } x \leq L_{end} + \alpha_{hd} \cdot L \\ y_p & \text{otherwise} \end{cases}$$

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-dependendent 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)

$VS := \frac{A_g}{Peri} = 4.023 \cdot in$ Volume-to-surface ratio of the precast section

$k_s := \max\left(1.45 - 0.13 \cdot \frac{VS}{in}, 1.0\right) = 1.00$ Factor for volume-to-surface ratio (5.4.2.3.2-2)

$k_{hc} := 1.56 - 0.008 \cdot H = 1.00$ Humidity factor for creep (5.4.2.3.2-3)

$k_{hs} := 2.00 - 0.014 \cdot H = 1.02$ Humidity factor for shrinkage (5.4.2.3.3-2)

$k_f := \frac{5}{1 + \frac{f_{ci}}{ksi}} = 0.676$ Factor for effect of concrete strength (5.4.2.3.2-4)

10. PRESTRESS LOSSES (cont'd)

$$k_{td}(t) := \frac{t}{61 - 4\cdot\frac{f_{ci}}{ksi} + t}$$
Time development factor (5.4.2.3.2-5)

$$\psi(t, t_{init}) := 1.9\cdot k_s\cdot k_{hc}\cdot k_f\cdot k_{td}(t)\cdot(t_{init})^{-0.118}$$
Creep coefficient (5.4.2.3.2-1)

$$\varepsilon_{sh}(t) := k_s\cdot k_{hs}\cdot k_f\cdot k_{td}(t)\cdot\left(0.48\cdot 10^{-3}\right)$$
Concrete shrinkage strain (5.4.2.3.3-1)

Time from Transfer to Erection:

$$e_{pg} := -(y_p + y_{bg}) = 23.772\cdot in$$
Eccentricity of prestress force with respect to the neutral axis of the gross non-composite beam, positive below the beam neutral axis

$$f_{cgp} := P_i\cdot\left(\frac{1}{A_g} + \frac{e_{pg}^{\,2}}{I_{xg}}\right) + \frac{M_g\left(\frac{L}{2}\right)}{I_{xg}}\cdot(y_p + y_{bg}) = 2.797\cdot ksi$$
Stress in the concrete at the center prestress immediately after transfer

$$f_{pt} := \max(f_{pi}, 0.55\cdot f_{py}) = 186.522\cdot ksi$$
Stress in strands immediately after transfer (5.9.5.4.2c-1)

$$\psi_{bid} := \psi(t_d, t_i) = 0.589$$
Creep coefficient at erection due to loading at transfer

$$\psi_{bif} := \psi(t_f, t_i) = 1.282$$
Creep coefficient at final due to loading at transfer

$$\varepsilon_{bid} := \varepsilon_{sh}(t_d - t_i) = 1.490 \times 10^{-4}$$
Concrete shrinkage between transfer and erection

$$K_{id} := \frac{1}{1 + n_{pi}\cdot\frac{A_{ps}}{A_g}\cdot\left(1 + \frac{A_g\cdot e_{pg}^{\,2}}{I_{xg}}\right)\cdot(1 + 0.7\cdot\psi_{bif})} = 0.809$$
Age-adjusted transformed section coefficient (5.9.5.4.2a-2)

$$\Delta f_{pSR} := \varepsilon_{bid}\cdot E_p\cdot K_{id} = 3.435\cdot ksi$$
Loss due to beam shrinkage (5.9.5.4.2a-1)

$$\Delta f_{pCR} := n_{pi}\cdot f_{cgp}\cdot\psi_{bid}\cdot K_{id} = 7.831\cdot ksi$$
Loss due to creep (5.9.5.4.2b-1)

$$\Delta f_{pR1} := \left[\frac{f_{pt}}{K_L}\cdot\frac{\log(24\cdot t_d)}{\log(24\cdot t_i)}\cdot\left(\frac{f_{pt}}{f_{py}} - 0.55\right)\right]\cdot\left[1 - \frac{3\cdot(\Delta f_{pSR} + \Delta f_{pCR})}{f_{pt}}\right]\cdot K_{id} = 1.237\cdot ksi$$
Loss due to relaxation (C5.9.5.4.2c-1

$$\Delta f_{pid} := \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1} = 12.502\cdot ksi$$

10. PRESTRESS LOSSES (cont'd)

Time from Erection to Final:

$e_{pc} := e_{pg} = 23.772 \cdot in$ — Eccentricity of prestress force does not change

$A_c := A_g \qquad I_c := I_{xg}$ — Section properties remain unchanged

$$\Delta f_{cd} := \frac{M_{fws}\left(\frac{L}{2}\right) + M_j\left(\frac{L}{2}\right)}{S_{cgpf}} + \frac{\Delta f_{pid}}{n_p} = 2.182 \cdot ksi$$

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

$$K_{df} := \frac{1}{1 + n_{pi} \cdot \frac{A_{ps}}{A_c} \cdot \left(1 + \frac{A_c \cdot e_{pc}^{\,2}}{I_c}\right) \cdot \left(1 + 0.7 \cdot \psi_{bif}\right)} = 0.809$$

Age-adjusted transformed section coefficient remains unchanged

$\Delta f_{pSD} := \varepsilon_{bdf} \cdot E_p \cdot K_{df} = 4.179 \cdot ksi$ — Loss due to beam shrinkage

$\Delta f_{pCD} := n_{pi} \cdot f_{cgp} \cdot (\psi_{bif} - \psi_{bid}) \cdot K_{df} + n_p \cdot \Delta f_{cd} \cdot \psi_{bdf} \cdot K_{df} = 17.168 \cdot ksi$ — Loss due to creep

$\Delta f_{pR2} := \Delta f_{pR1} = 1.237 \cdot ksi$ — Loss due to relaxation

$\Delta f_{pSS} := 0$ — Loss due to deck shrinkage

$\Delta f_{pdf} := \Delta f_{pSD} + \Delta f_{pCD} + \Delta f_{pR2} + \Delta f_{pSS} = 22.584 \cdot ksi$

Prestress Loss Summary

$\Delta f_{pES} = 15.978 \cdot ksi \qquad \frac{\Delta f_{pES}}{f_{pj}} = 7.9 \cdot \%$

$\Delta f_{pLT} := \Delta f_{pid} + \Delta f_{pdf} = 35.087 \cdot ksi \qquad \frac{\Delta f_{pLT}}{f_{pj}} = 17.3 \cdot \%$

$\Delta f_{pTotal} := \Delta f_{pES} + \Delta f_{pLT} = 51.065 \cdot ksi \qquad \frac{\Delta f_{pTotal}}{f_{pj}} = 25.2 \cdot \% \qquad \Delta f_{p.est} = 25 \cdot \%$

$f_{pe} := f_{pj} - \Delta f_{pTotal} = 151.4 \cdot ksi$ — 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.

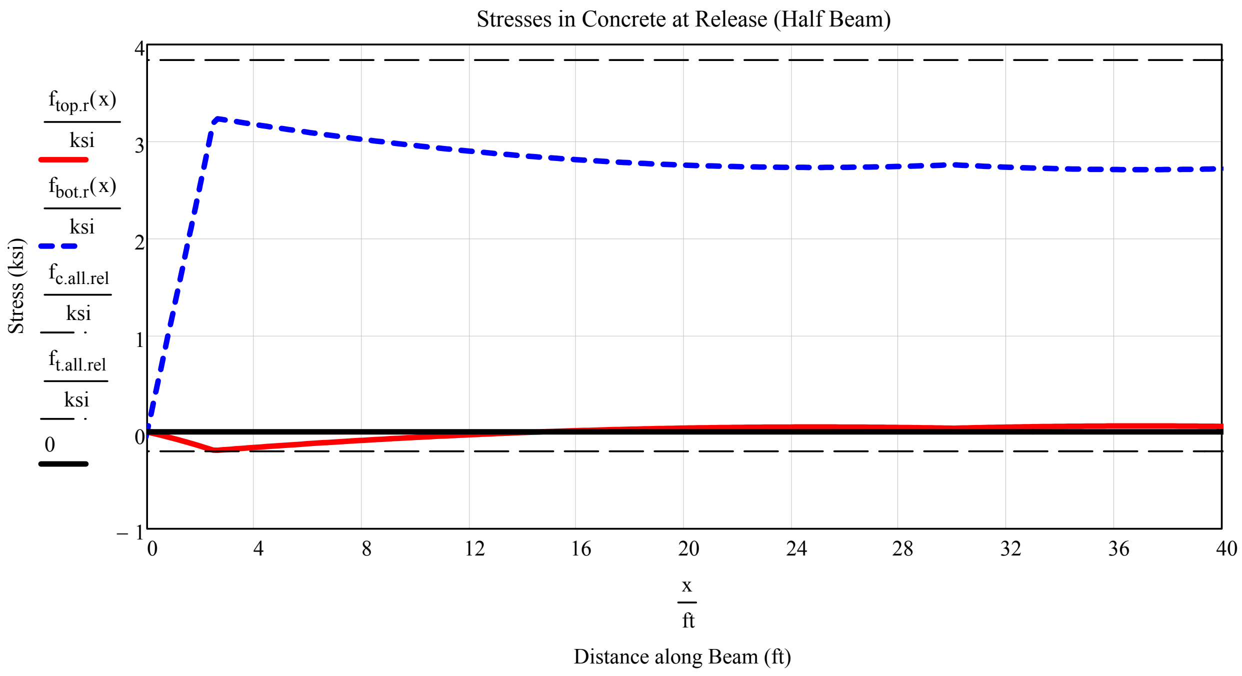
Define locations for which stresses are to be calculated:

$$x_r := L_g \cdot \left(0 \quad \min\left(\frac{L_t}{L_g}, \frac{L_{end}}{L_g}\right) \quad \max\left(\frac{L_t}{L_g}, \frac{L_{end}}{L_g}\right) \quad 0.1 \quad 0.2 \quad 0.3 \quad \alpha_{hd} \quad 0.5\right)^T \qquad ir := 1 .. \mathrm{last}(x_r)$$

Functions for computing beam stresses:

$$f_{top.r}(x) := \min\left(\frac{x}{L_t}, 1\right) \cdot P_i \cdot \left(\frac{1}{A_{ti}} + \frac{y_{bti} + y_{px}(x)}{S_{tti}}\right) + \frac{M_{gr}(x)}{S_{tti}}$$ Top fiber stress at release

$$f_{bot.r}(x) := \min\left(\frac{x}{L_t}, 1\right) \cdot P_i \cdot \left(\frac{1}{A_{ti}} + \frac{y_{bti} + y_{px}(x)}{S_{bti}}\right) + \frac{M_{gr}(x)}{S_{bti}}$$ Bottom fiber stress at release



11. CONCRETE STRESSES (cont'd)

Compare beam stresses to allowable stresses.

$f_{t.all.rel} = -0.2 \cdot ksi$ Allowable tension at release

$f_{c.all.rel} = 3.84 \cdot ksi$ Allowable compression at release

$TopRel_{ir} := f_{top.r}\left(x_{r_{ir}}\right)$ $TopRel^{T} = (0.000\ \ -0.148\ \ -0.192\ \ -0.097\ \ 0.002\ \ 0.047\ \ 0.040\ \ 0.062) \cdot ksi$

$CheckTopRel := if\left[\left(max(TopRel) \le f_{c.all.rel}\right) \cdot \left(min(TopRel) \ge f_{t.all.rel}\right), "OK", "No Good"\right] = "OK"$

$BotRel_{ir} := f_{bot.r}\left(x_{r_{ir}}\right)$ $BotRel^{T} = (0.000\ \ 2.582\ \ 3.241\ \ 3.042\ \ 2.834\ \ 2.738\ \ 2.754\ \ 2.708) \cdot ksi$

$CheckBotRel := if\left[\left(max(BotRel) \le f_{c.all.rel}\right) \cdot \left(min(BotRel) \ge f_{t.all.rel}\right), "OK", "No Good"\right] = "OK"$

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 occuring between transfer and erection (i.e., the Δf_{pid}).

$a := h = 3.5 \cdot ft$ Maximum distance to lift point from bearing line

$a' := a + L_{end} = 5.5 \cdot ft$ Distance to lift point from end of beam

$P_{dia} := max\left(W_{ia}, W_{sa}\right) = 13.2 \cdot kip$ Approximate abutment weight

$P_m := P_j \cdot \left[1 - \frac{\left(\Delta f_{pES} + \Delta f_{pid}\right)}{f_{pj}}\right] = 1011.7 \cdot kip$ Effective prestress during lifting and shipping

Define locations for which stresses are to be calculated:

$$x_e := L_g \cdot \left(0\ \ min\left(\frac{L_t}{L_g}, \frac{L_{end}}{L_g}\right)\ \ max\left(\frac{L_t}{L_g}, \frac{L_{end}}{L_g}\right)\ \ \frac{a'}{L_g}\ \ \alpha_{hd}\ \ 0.5\right)^{T} \qquad ie := 1 .. last\left(x_e\right)$$

Compute moment in the girder during lifting with supports at the lift points.

$$M_{lift}(x) := \begin{cases} -\left[\frac{\left(w_g + w_{bar}\right) \cdot x^2}{2} + P_{dia} \cdot x\right] & \text{if } x \le a' \\ M_{gr}(x) - \left[M_{gr}(a') + \frac{\left(w_g + w_{bar}\right) \cdot (a')^2}{2} + P_{dia} \cdot a'\right] & \text{otherwise} \end{cases}$$

11. CONCRETE STRESSES (cont'd)

Functions for computing beam stresses:

$$f_{top.lift}(x) := \min\left(\frac{x}{L_t}, 1\right) \cdot P_m \cdot \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{ttf}}\right) + \frac{M_{lift}(x)}{S_{ttf}}$$

Top fiber stress during lifting

$$f_{top.DIM.inc}(x) := \min\left(\frac{x}{L_t}, 1\right) \cdot P_m \cdot \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{ttf}}\right) + \frac{M_{lift}(x)}{S_{ttf}} \cdot (1 + DIM)$$

Top fiber stress during lifting, impact increasing dead load

$$f_{top.DIM.dec}(x) := \min\left(\frac{x}{L_t}, 1\right) \cdot P_m \cdot \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{ttf}}\right) + \frac{M_{lift}(x)}{S_{ttf}} \cdot (1 - DIM)$$

Top fiber stress during lifting, impact decreasing dead load

$TopLift1_{ie} := f_{top.lift}\left(x_{e_{ie}}\right)$ $\quad TopLift1^T = (0.000 \quad -0.230 \quad -0.294 \quad -0.371 \quad -0.181 \quad -0.158) \cdot ksi$

$TopLift2_{ie} := f_{top.DIM.inc}\left(x_{e_{ie}}\right)$ $\quad TopLift2^T = (0.000 \quad -0.236 \quad -0.302 \quad -0.393 \quad -0.065 \quad -0.035) \cdot ksi$

$TopLift3_{ie} := f_{top.DIM.dec}\left(x_{e_{ie}}\right)$ $\quad TopLift3^T = (0.000 \quad -0.223 \quad -0.285 \quad -0.349 \quad -0.296 \quad -0.282) \cdot ksi$

$$f_{bot.lift}(x) := \min\left(\frac{x}{L_t}, 1\right) \cdot P_m \cdot \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{btf}}\right) + \frac{M_{lift}(x)}{S_{btf}}$$

Bottom fiber stress during lifting

$$f_{bot.DIM.inc}(x) := \min\left(\frac{x}{L_t}, 1\right) \cdot P_m \cdot \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{btf}}\right) + \frac{M_{lift}(x)}{S_{btf}} \cdot (1 + DIM)$$

Bottom fiber stress during lifting, impact increasing dead load

$$f_{bot.DIM.dec}(x) := \min\left(\frac{x}{L_t}, 1\right) \cdot P_m \cdot \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{btf}}\right) + \frac{M_{lift}(x)}{S_{btf}} \cdot (1 - DIM)$$

Bottom fiber stress during lifting, impact decreasing dead load

$BotLift1_{ie} := f_{bot.lift}\left(x_{e_{ie}}\right)$ $\quad BotLift1^T = (0.000 \quad 2.623 \quad 3.292 \quad 3.456 \quad 3.052 \quad 3.005) \cdot ksi$

$BotLift2_{ie} := f_{bot.DIM.inc}\left(x_{e_{ie}}\right)$ $\quad BotLift2^T = (0.000 \quad 2.637 \quad 3.310 \quad 3.502 \quad 2.808 \quad 2.744) \cdot ksi$

$BotLift3_{ie} := f_{bot.DIM.dec}\left(x_{e_{ie}}\right)$ $\quad BotLift3^T = (0.000 \quad 2.609 \quad 3.274 \quad 3.410 \quad 3.297 \quad 3.267) \cdot ksi$

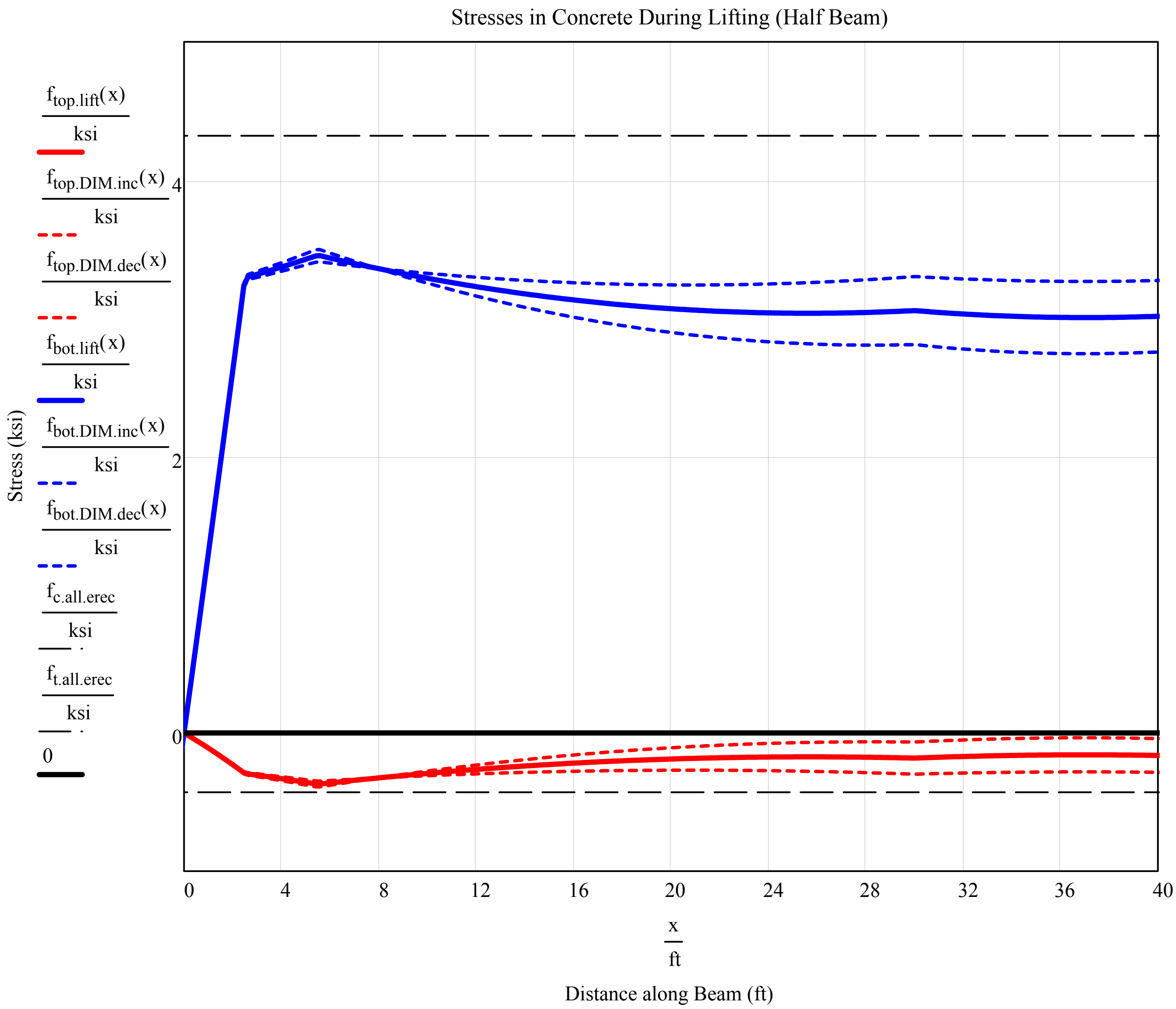
Allowable stresses during handling:

$f_{cm} := f_{c.erec}\left(f_c\right) = 7.2 \cdot ksi$ — Assumed concrete strength when handling operations begin

$f_{c.all.erec} := 0.6 \cdot f_{cm} = 4.32 \cdot ksi$ — Allowable compression during lifting and shipping

$f_{t.all.erec} := f_{t.erec}\left(f_{cm}\right) = -0.429 \cdot ksi$ — Allowable tension during lifting and shipping

11. CONCRETE STRESSES (cont'd)



Compare beam stresses to allowable stresses.

$$TopLiftMax_{ie} := \max\left(TopLift1_{ie}, TopLift2_{ie}, TopLift3_{ie}\right) \qquad TopLiftMax^{T} = (0 \quad -0.223 \quad -0.285 \quad -0.349 \quad -0.065 \quad -0.035\,)\cdot ks$$

$$TopLiftMin_{ie} := \min\left(TopLift1_{ie}, TopLift2_{ie}, TopLift3_{ie}\right) \qquad TopLiftMin^{T} = (0 \quad -0.236 \quad -0.302 \quad -0.393 \quad -0.296 \quad -0.282\,)\cdot ks$$

$$CheckTopLift := if\left[\left(\max(TopLiftMax) \le f_{c.all.erec}\right)\cdot\left(\min(TopLiftMin) \ge f_{t.all.erec}\right), \text{"OK"}, \text{"No Good"}\right] = \text{"OK"}$$

$$BotLiftMax_{ie} := \max\left(BotLift1_{ie}, BotLift2_{ie}, BotLift3_{ie}\right) \qquad BotLiftMax^{T} = (0 \quad 2.637 \quad 3.31 \quad 3.502 \quad 3.297 \quad 3.267\,)\cdot ksi$$

$$BotLiftMin_{ie} := \min\left(BotLift1_{ie}, BotLift2_{ie}, BotLift3_{ie}\right) \qquad BotLiftMin^{T} = (0 \quad 2.609 \quad 3.274 \quad 3.41 \quad 2.808 \quad 2.744\,)\cdot ksi$$

$$CheckBotLift := if\left[\left(\max(BotLiftMax) \le f_{c.all.erec}\right)\cdot\left(\min(BotLiftMin) \ge f_{t.all.erec}\right), \text{"OK"}, \text{"No Good"}\right] = \text{"OK"}$$

11. CONCRETE STRESSES (cont'd)

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.

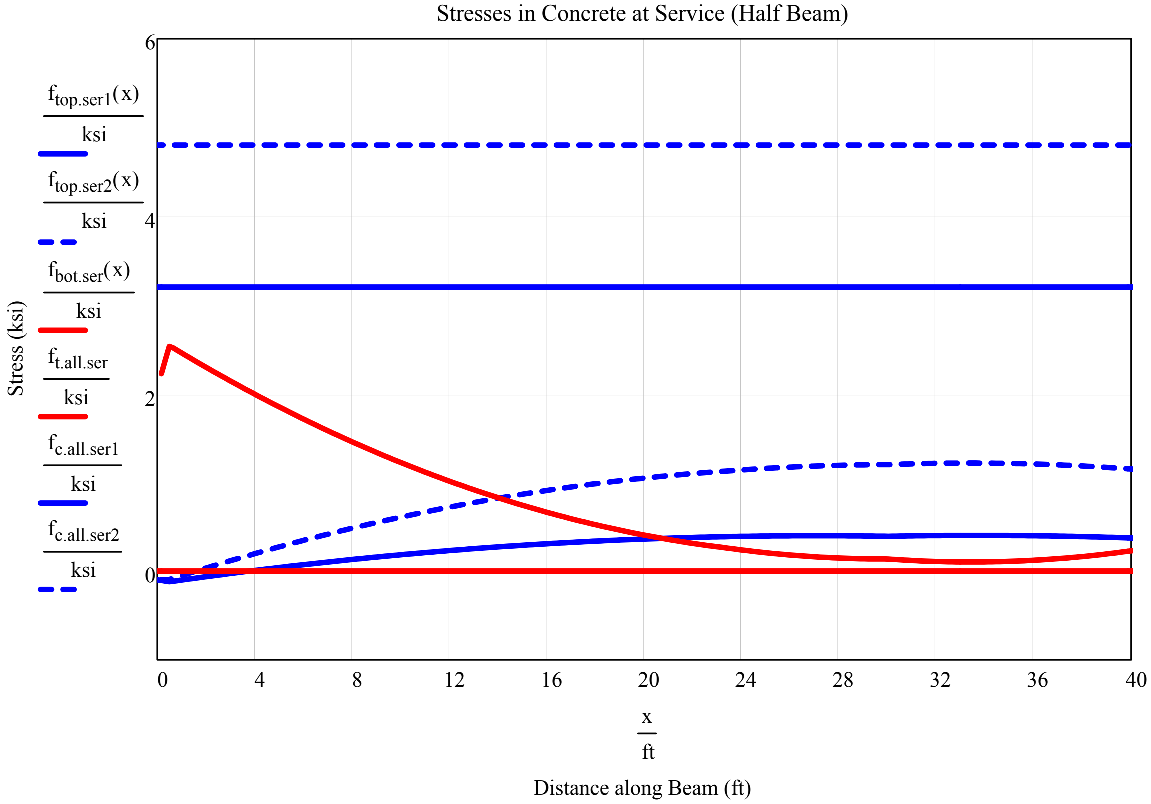
$f_{c.all.ser1} := 0.4 \cdot f_c = 3.2 \cdot ksi$	Allowable compression due to effective prestress and dead load (Table 5.9.4.2.1-1)
$f_{c.all.ser2} := 0.6 \cdot f_c = 4.8 \cdot ksi$	Allowable compression due to effective prestress, permanent load, and transient loads, as well as stresses during shipping and handling (Table 5.9.4.2.1-1)
$f_{t.all.ser} = 0 \cdot ksi$	Allowable tension (computed previously)
$P_e := f_{pe} \cdot A_{ps} = 880.4 \cdot kip$	Effective prestress after all losses

Compute stresses at midspan and compare to allowable values.

$$f_{top.ser1}(x) := \min\left(\frac{L_{end} + x}{L_t}, 1\right) \cdot P_e \cdot \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{ttf}}\right) + \frac{M_g(x + L_{end})}{S_{tti}} + \frac{M_{bar}(x) + M_{fws}(x) + M_j(x)}{S_{ttf}}$$

$$f_{top.ser2}(x) := \min\left(\frac{L_{end} + x}{L_t}, 1\right) \cdot P_e \cdot \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{ttf}}\right) + \frac{M_g(x + L_{end})}{S_{tti}} + \frac{M_{bar}(x) + M_{fws}(x) + M_j(x) + M_{ll}(x)}{S_{ttf}}$$

$$f_{bot.ser}(x) := \min\left(\frac{L_{end} + x}{L_t}, 1\right) \cdot P_e \cdot \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{btf}}\right) + \frac{M_g(x + L_{end})}{S_{bti}} + \frac{M_{bar}(x) + M_{fws}(x) + M_j(x) + 0.8 \cdot M_{ll}(x)}{S_{btf}}$$



11. CONCRETE STRESSES (cont'd)

Compare beam stresses to allowable stresses.

$$x_s := L\cdot\begin{pmatrix}\frac{L_t}{L} & 0.1 & 0.15 & 0.2 & 0.25 & 0.3 & 0.35 & \alpha_{hd} & 0.45 & 0.5\end{pmatrix}^T \qquad is := 1\,..\,\mathrm{last}(x_s)$$

$$TopSer1_{is} := f_{top.ser1}\left(x_{s_{is}}\right) \quad TopSer1^T = (-0.046\ \ 0.101\ \ 0.195\ \ 0.272\ \ 0.330\ \ 0.370\ \ 0.393\ \ 0.397\ \ 0.398\ \ 0.400)\cdot ksi$$

$$TopSer2_{is} := f_{top.ser2}\left(x_{s_{is}}\right) \quad TopSer2^T = (0.075\ \ 0.415\ \ 0.636\ \ 0.820\ \ 0.966\ \ 1.074\ \ 1.148\ \ 1.191\ \ 1.211\ \ 1.212)\cdot ksi$$

$$CheckCompSerI := \mathrm{if}\left[\left(\max(TopSer1) \le f_{c.all.ser1}\right)\cdot\left(\max(TopSer2) \le f_{c.all.ser2}\right), \text{"OK"}, \text{"No Good"}\right] = \text{"OK"}$$

$$BotSer_{is} := f_{bot.ser}\left(x_{s_{is}}\right) \quad BotSer^T = (2.218\ \ 1.581\ \ 1.168\ \ 0.825\ \ 0.554\ \ 0.355\ \ 0.221\ \ 0.146\ \ 0.112\ \ 0.109)\cdot ksi$$

$$CheckTenSerIII := \mathrm{if}\left(\min(BotSer) \ge f_{t.all.ser}, \text{"OK"}, \text{"No Good"}\right) = \text{"OK"}$$

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

$M_{StrI}(x) := 1.25\cdot M_{DC}(x) + 1.5\cdot M_{DW}(x) + 1.75\cdot M_{LL}(x)$ — Factored design moment

For minimum reinforcement check, per 5.7.3.3.2

$f_{cpe} := P_e\cdot\left(\frac{1}{A_g} + \frac{y_{cgp}}{S_{bg}}\right) = 3.677\cdot ksi$ — Concrete compression at extreme fiber due to effective prestress

$M_{cr} := -\left(f_{r.cm} + f_{cpe}\right)\cdot S_{bg} = 2825\cdot kip\cdot ft$ — Cracking moment (5.7.3.3.2-1)

$M_u(x) := \max\left(M_{StrI}(x), \min\left(1.33\cdot M_{StrI}(x), 1.2\cdot M_{cr}\right)\right)$ — Design moment

12. FLEXURAL STRENGTH (cont'd)

Compute factored flexural resistance.

$$\beta_1 := \max\left[0.65, 0.85 - 0.05\cdot\left(\frac{f_c}{ksi} - 4\right)\right] = 0.65$$

Stress block factor (5.7.2.2)

$$k := 2\cdot\left(1.04 - \frac{f_{py}}{f_{pu}}\right) = 0.28$$

Tendon type factor (5.7.3.1.1-2)

$$d_p(x) := h - y_{px}(x + L_{end}) \qquad d_p(X) = 37.421\cdot in$$

Distance from compression fiber to prestress centroid

$$h_f := d_7 = 8\cdot in$$

Structural flange thickness

$$b_{taper} := \frac{b_6 - b_5}{2} = 19.5\cdot in$$

Average width of taper at bottom of flange

$$h_{taper} := d_5 = 3\cdot in$$

Depth of taper at bottom of flange

$$a(x) := \frac{A_{ps}\cdot f_{pu}}{0.85\cdot f_c\cdot b_f + \frac{k}{\beta_1}\cdot A_{ps}\cdot\left(\frac{f_{pu}}{d_p(x)}\right)} \qquad a(X) = 2.509\cdot in$$

Depth of equivalent stress block for rectangular section

$$c(x) := \frac{a(x)}{\beta_1} \qquad c(X) = 3.861\cdot in$$

Neutral axis location

$$CheckTC := if\left[\frac{c(X)}{d_p(X)} \le \left(\frac{.003}{.003 + .005}\right), \text{"OK"}, \text{"NG"}\right] = \text{"OK"}$$

Tension-controlled section check (midspan)

$$\varphi_f := \min\left[1.0, \max\left[0.75, 0.583 + 0.25\cdot\left(\frac{d_p(X)}{c(X)} - 1\right)\right]\right] = 1.00$$

Resistance factor for prestressed concrete (5.5.4.2)

$$f_{ps} := f_{pu}\cdot\left(1 - k\cdot\frac{c(X)}{d_p(X)}\right) = 262.2\cdot ksi$$

Average stress in the prestressing steel (5.7.3.1.1-1)

$$L_d := \frac{1.6}{ksi}\cdot\left(f_{ps} - \frac{2}{3}\cdot f_{pe}\right)\cdot d_{ps} = 10.75\cdot ft$$

Bonded strand devlepment length (5.11.4.2-1)

$$f_{px}(x) := \begin{cases} \frac{f_{pe}\cdot(x + L_{end})}{L_t} & \text{if } x \le L_t - L_{end} \\ f_{pe} + \frac{(x + L_{end}) - L_t}{L_d - L_t}\cdot(f_{ps} - f_{pe}) & \text{if } L_t - L_{end} < x \le L_d - L_{end} \\ f_{ps} & \text{otherwise} \end{cases}$$

Stress in prestressing steel along the length for bonded strand (5.11.4.2)

$$M_r(x) := \varphi_f\cdot\left[A_{ps}\cdot f_{px}(x)\cdot\left(d_p(x) - \frac{a(x)}{2}\right)\right]$$

Flexure resistance along the length

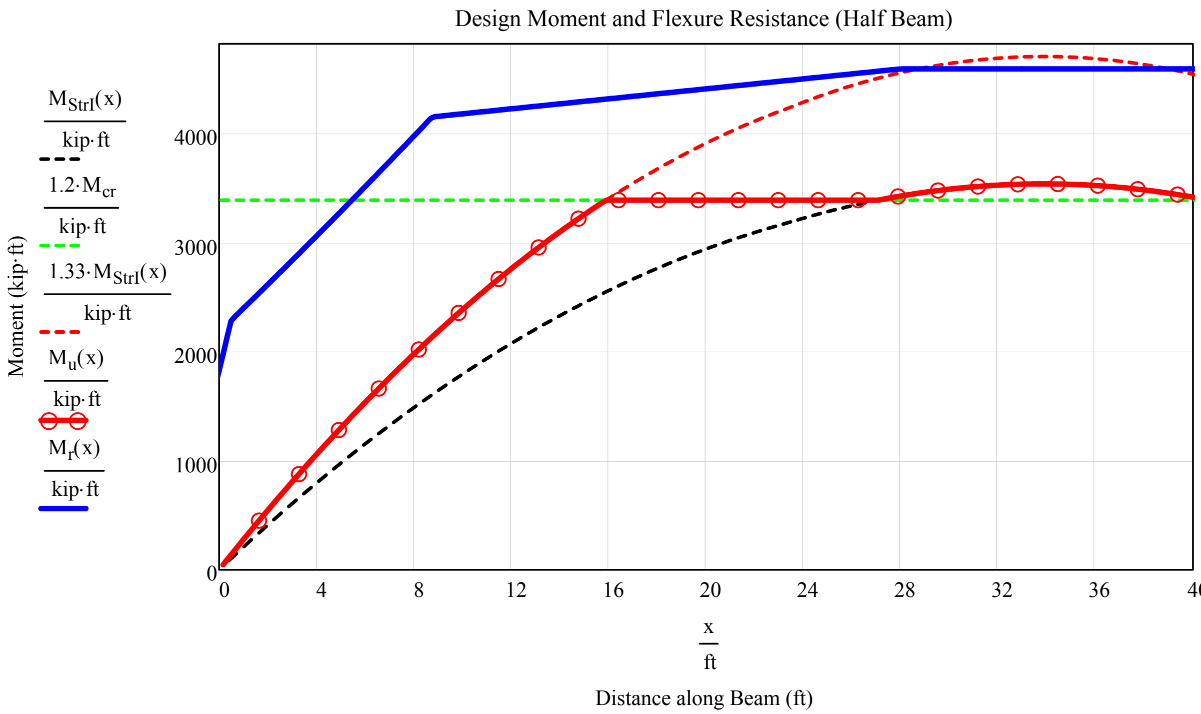
12. FLEXURAL STRENGTH (cont'd)

$$x_{mom} := L \cdot \left(0.01 \quad \frac{L_t - L_{end}}{L} \quad \frac{L_d - L_{end}}{L} \quad \alpha_{hd} \quad 0.5 \right)^T \qquad imom := 1 .. last(x_{mom})$$

$$M_{rx_{imom}} := M_r\left(x_{mom_{imom}}\right) \qquad M_{ux_{imom}} := M_u\left(x_{mom_{imom}}\right)$$

$$DC_{mom} := \frac{M_{ux}}{M_{rx}} \qquad max(DC_{mom}) = 0.769$$ Demand-Capacity ratio for moment

$$CheckMom := if\left(max(DC_{mom}) \le 1.0, \text{"OK"}, \text{"No Good"}\right) = \text{"OK"}$$ 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

$V_u(x) := 1.25 \cdot V_{DC}(x) + 1.5 \cdot V_{DW}(x) + 1.75 \cdot V_{LL}(x)$ — Factored design shear

$\varphi_v := 0.90$ — Resistance factor for shear in normal weight concrete (AASHTO LRFD 5.5.4.2)

$d_{end} := h - y_{px}(L_{end}) = 32.368 \cdot in$ — Depth to steel centroid at bearing

$d_v := \min(0.9 \cdot d_{end}, 0.72 \cdot h) = 29.132 \cdot in$ — Effective shear depth lower limit at end

$$V_p(x) := \begin{cases} P_e \cdot slope_{cgp} \cdot \dfrac{x + L_{end}}{L_t} & \text{if } x \le L_t - L_{end} \\ P_e \cdot slope_{cgp} & \text{if } L_t - L_{end} < x \le \alpha_{hd} \cdot L \\ 0 & \text{otherwise} \end{cases}$$

Vertical component of effective prestress force

$b_v := b_3 = 6 \cdot in$ — Web thickness

$$v_u(x) := \frac{\left|V_u(x) - \varphi_v \cdot V_p(x)\right|}{\varphi_v \cdot b_v \cdot d_v}$$

Shear stress on concrete (5.8.2.9-1)

$M_{ushr}(x) := \max\left(M_{StrI}(x), \left|V_u(x) - V_p(x)\right| \cdot d_v\right)$ — Factored moment for shear

$f_{po} := 0.7 \cdot f_{pu} = 189 \cdot ksi$ — Stress in prestressing steel due to locked-in strain after casting concrete

$$\varepsilon_s(x) := \max\left(-0.4 \cdot 10^{-3}, \frac{\dfrac{\left|M_u(x)\right|}{d_v} + \left|V_u(x) - V_p(x)\right| - A_{ps} \cdot f_{po}}{E_p \cdot A_{ps}}\right)$$

Steel strain at the centroid of the prestressing steel

$$\beta(x) := \frac{4.8}{1 + 750 \cdot \varepsilon_s(x)}$$

Shear resistance parameter

$\theta(x) := (29 + 3500 \cdot \varepsilon_s(x)) \cdot deg$ — Principal compressive stress angle

$$V_c(x) := 0.0316 \cdot ksi \cdot \beta(x) \cdot \sqrt{\frac{f_c}{ksi}} \cdot b_v \cdot d_v$$

Concrete contribution to total shear resistance

13. SHEAR STRENGTH (cont'd)

$\alpha := 90 \cdot deg$ Angle of inclination of transverse reinforcement

$A_v := (1.02 \ \ 0.62 \ \ 0.62 \ \ 0.62 \ \ 0.31)^T \cdot in^2$ $s_v := (3 \ \ 6 \ \ 6 \ \ 12 \ \ 12)^T \cdot in$ Transverse reinforcement area and spacing provided

$x_v := (0 \ \ 0.25 \cdot h \ \ 1.5 \cdot h \ \ 0.3 \cdot L \ \ 0.5 \cdot L \ \ 0.6 \cdot L)^T$ $x_v^T = (0 \ \ 0.875 \ \ 5.25 \ \ 21 \ \ 35 \ \ 42) \cdot ft$

$$A_{vs}(x) := \left| \begin{array}{l} \text{for } i \in 1 .. \text{last}(A_v) \\ \quad out \leftarrow \dfrac{A_{v_i}}{s_{v_i}} \text{ if } x_{v_i} \le x \le x_{v_{i+1}} \\ out \end{array} \right.$$

$V_s(x) := A_{vs}(x) \cdot f_y \cdot d_v \cdot (\cot(\theta(x)) + \cot(\alpha)) \cdot \sin(\alpha)$ Steel contribution to total shear resistance

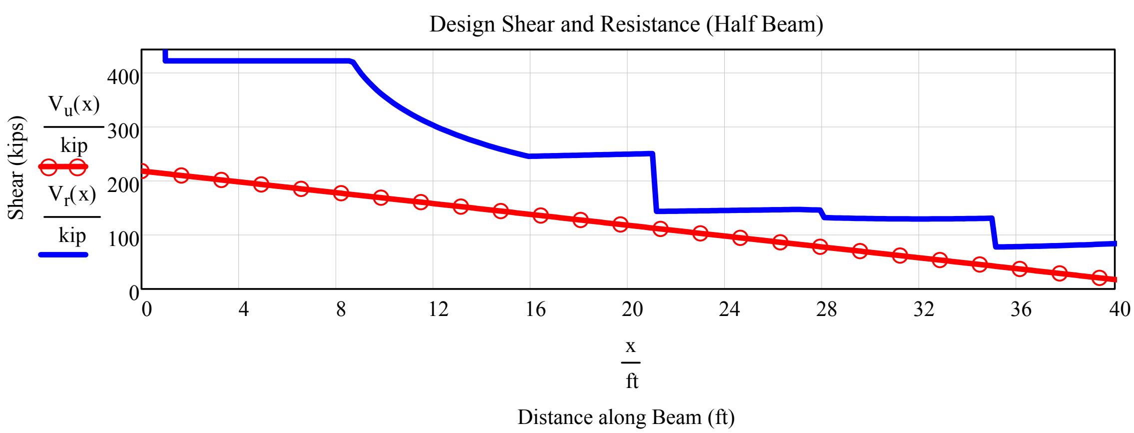
$V_r(x) := \varphi_v \cdot (V_c(x) + V_s(x) + V_p(x))$ Factored shear resistance

$$x_{shr} := \left| \begin{array}{l} \text{for } i \in 1 .. 100 \\ \quad out_i \leftarrow i \cdot \dfrac{0.5 \cdot L}{100} \\ out \end{array} \right.$$ $ishr := 1 .. \text{last}(x_{shr})$

$V_{ux_{ishr}} := V_u(x_{shr_{ishr}})$ $V_{rx_{ishr}} := V_r(x_{shr_{ishr}})$

$DC_{shr} := \dfrac{V_{ux}}{V_{rx}}$ $\max(DC_{shr}) = 0.787$

$CheckShear := \text{if}(\max(DC_{shr}) \le 1.0, \text{"OK"}, \text{"No Good"}) = \text{"OK"}$ Shear resistance check



13. SHEAR STRENGTH (cont'd)

Longitudinal Reinforcement

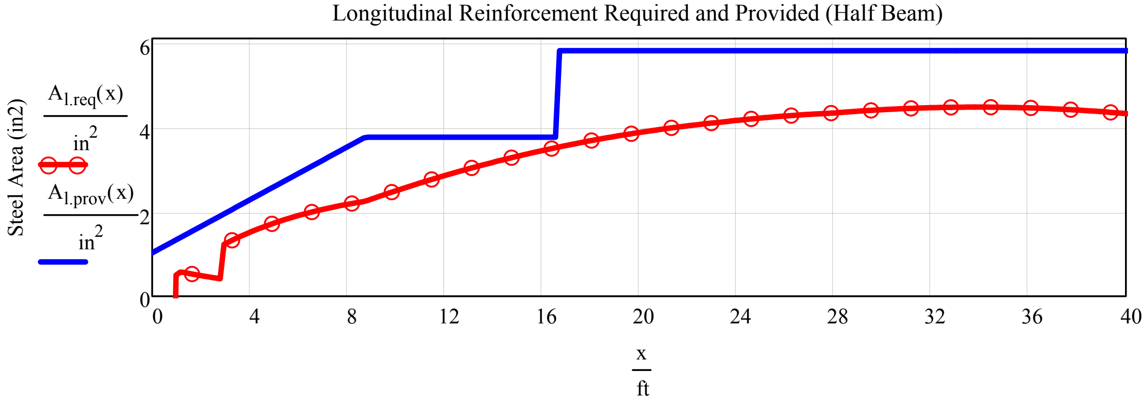
$$A_{l.req}(x) := \begin{array}{|l} a1 \leftarrow \dfrac{M_{StrI}(x)}{\varphi_f \cdot f_{px}(x) \cdot \left(d_p(x) - \dfrac{a(x)}{2}\right)} \\ a2 \leftarrow \dfrac{\left(\dfrac{V_u(x)}{\varphi_v} - 0.5 \cdot V_s(x) - V_p(x)\right) \cdot \cot(\theta(x))}{f_{px}(x)} \\ a3 \leftarrow \dfrac{\dfrac{M_{ushr}(x)}{d_v \cdot \varphi_f} + \left(\left|\dfrac{V_u(x)}{\varphi_v} - V_p(x)\right| - 0.5 \cdot V_s(x)\right) \cdot \cot(\theta(x))}{f_{px}(x)} \\ \min(a1, a2) \quad \text{if } x \le d_v + 5 \cdot in \\ \min(a1, a3) \quad \text{otherwise} \end{array}$$

Longitudinal reinforcement required for shear (5.8.3.5)

$A_{s.add} := 0.40 \cdot in^2$ $\qquad L_{d.add} := 18.67 \cdot ft$ $\qquad$ Additional longitudinal steel and developed length from end of beam

$$A_{l.prov}(x) := \text{if}\left(x < L_{d.add} - L_{end}, A_{s.add}, 0\right) + \begin{array}{|l} A_p \cdot N_s \cdot \dfrac{x + L_{end}}{L_d} \quad \text{if } x \le L_d - L_{end} \\ A_p \cdot N_s \quad \text{if } L_d - L_{end} < x \le \dfrac{y_{h.brg} - 0.5 \cdot h}{slope_{cgp}} + \left(\dfrac{0.5 \cdot N_h - 1}{2}\right) \cdot (2 \cdot in) \cdot \cot\left(slope_{cgp}\right) \\ A_p \cdot \left(N_h + N_s\right) \quad \text{otherwise} \end{array}$$

Longitudinal Reinforcement Required and Provided (Half Beam)



Distance along Beam (ft)

$A_{l.req_{ishr}} := A_{l.req}\left(x_{shr_{ishr}}\right)$ $\qquad A_{l.prov_{ishr}} := A_{l.prov}\left(x_{shr_{ishr}}\right)$

$DC_{long} := \dfrac{A_{l.req}}{A_{l.prov}}$ $\qquad \max\left(DC_{long}\right) = 0.93$

$CheckLong := \text{if}\left(\max\left(DC_{long}\right) \le 1.0, \text{"OK"}, \text{"No Good"}\right) = \text{"OK"}$ $\qquad$ 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.

$$A_s := \frac{A_{v_1} \cdot x_{v_2}}{s_{v_1}} = 3.57 \cdot in^2$$

$f_s := 20 \cdot ksi$ Limiting stress in steel for crack control (5.10.10.1)

$P_r := f_s \cdot A_s = 71.4 \cdot kip$ Splitting resistance provided (5.10.10.1-1)

$P_{r.min} := 0.04 \cdot P_j = 47.1 \cdot kip$ Minimum splitting resistance required

$\text{CheckSplit} := \text{if}\left(P_r \geq P_{r.min}, \text{"OK"}, \text{"No Good"}\right) = \text{"OK"}$ Splitting resistance check

15. CAMBER AND DEFLECTIONS

$$\Delta_{ps} := \frac{-P_i}{E_{ci} \cdot I_{xg}} \cdot \left[\frac{y_{cgp} \cdot L_g^2}{8} - \frac{\left(y_{bg} + y_{p.brg}\right) \cdot \left(\alpha_{hd} \cdot L + L_{end}\right)^2}{6}\right] = 2.131 \cdot in$$ Deflection due to prestress at release

$$\Delta_{gr} := \frac{-5}{384} \cdot \frac{w_g \cdot L_g^4}{E_{ci} \cdot I_{xg}} = -0.917 \cdot in$$ Deflection due to self-weight at release

$$\Delta_{bar} := \frac{-5}{384} \cdot \frac{w_{bar} \cdot L_g^4}{E_c \cdot I_{xg}} = -0.263 \cdot in$$ Deflection due to barrier weight

$$\Delta_j := \frac{-5}{384} \cdot \frac{w_j \cdot L^4}{E_c \cdot I_{xg}} \cdot \text{if}(\text{BeamLoc} = 0, 1, 0.5) = -0.013 \cdot in \qquad 2$$ Deflection due to longitudinal joint

$$\Delta_{fws} := \frac{-5}{384} \cdot \frac{w_{fws} \cdot L^4}{E_c \cdot I_{xg}} \cdot \text{if}\left(\text{BeamLoc} = 0, 1, \frac{S - W_b}{S}\right) = -0.079 \cdot in$$ Deflection due to future wearing surface

$t_{bar} := 20$ Age at which barrier is assumed to be cast

$T := \left(t_i \;\; 7 \;\; 14 \;\; 21 \;\; 28 \;\; 60 \;\; 120 \;\; 240 \;\; \infty\right)^T$ Concrete ages at which camber is computed

15. CAMBER AND DEFLECTIONS (cont'd)

$$\Delta_{cr1}(t) := \psi(t - t_i, t_i)(\Delta_{gr} + \Delta_{ps})$$

$$\Delta_{cr2}(t) := \left(\psi(t - t_i, t_i) - \psi(t_{bar} - t_i, t_i)\right)\cdot(\Delta_{gr} + \Delta_{ps}) + \psi(t - t_{bar}, t_{bar})\cdot\Delta_{bar}$$

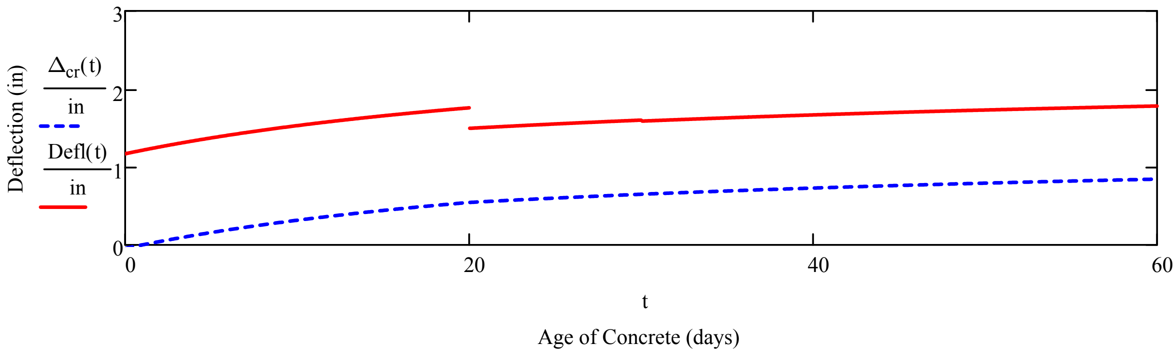
$$\Delta_{cr3}(t) := \left(\psi(t - t_i, t_i) - \psi(t_d - t_i, t_i)\right)\cdot(\Delta_{gr} + \Delta_{ps}) + \left(\psi(t - t_{bar}, t_{bar}) - \psi(t_d - t_{bar}, t_{bar})\right)\cdot\Delta_{bar} \ldots$$
$$+\psi(t - t_d, t_d)\cdot(\Delta_j)$$

$$\Delta_{cr}(t) := \begin{cases} \Delta_{cr1}(t) & \text{if } t \le t_{bar} \\ \Delta_{cr1}(t_{bar}) + \Delta_{cr2}(t) & \text{if } t_{bar} < t \le t_d \\ \Delta_{cr1}(t_{bar}) + \Delta_{cr2}(t_d) + \Delta_{cr3}(t) & \text{if } t > t_d \end{cases}$$

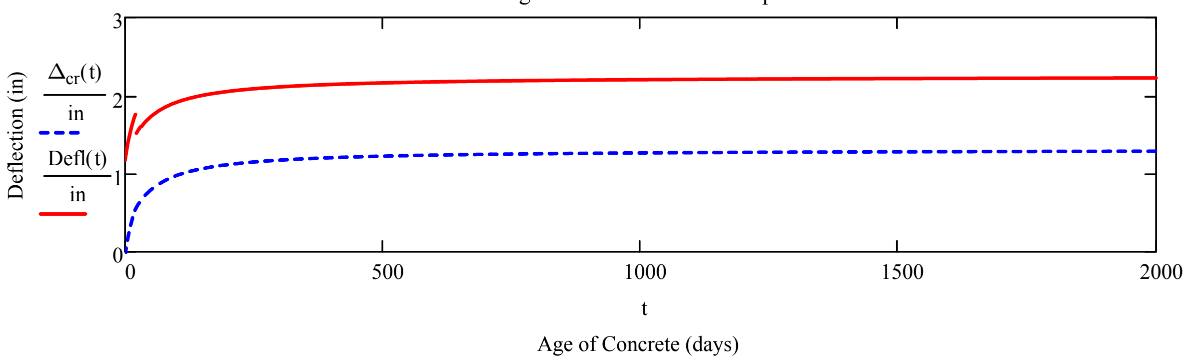
$$\mathrm{Defl}(t) := \begin{cases} (\Delta_{gr} + \Delta_{ps}) + \Delta_{cr1}(t) & \text{if } t \le t_{bar} \\ (\Delta_{gr} + \Delta_{ps}) + \Delta_{cr1}(t_{bar}) + \Delta_{bar} + \Delta_{cr2}(t) & \text{if } t_{bar} < t \le t_d \\ (\Delta_{gr} + \Delta_{ps}) + \Delta_{cr1}(t_{bar}) + \Delta_{bar} + \Delta_{cr2}(t_d) + \Delta_j + \Delta_{cr3}(t) & \text{if } t > t_d \end{cases}$$

$$C := \begin{array}{|l} \text{for } j \in 1..\mathrm{last}(T) \\ \quad out_j \leftarrow \mathrm{Defl}(T_j) \\ out \end{array} \qquad C^T = (1.213 \quad 1.439 \quad 1.632 \quad 1.506 \quad 1.581 \quad 1.78 \quad 1.955 \quad 2.081 \quad 2.247)\cdot in$$

60-Day Deflection at Midspan



Long-term Deflection at Midspan



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.

Live Load Truck and Truck Train Moment Calculations

$\min(M_{truck}) = -889 \cdot kip \cdot ft$ — Maximum negative moment due to a single truck

$\min(M_{train}) = -1650 \cdot kip \cdot ft$ — Maximum negative moment due to two trucks in a single lane

$M_{neg.lane} := \dfrac{-w_{lane} \cdot L^2}{2} = -1568 \cdot kip \cdot ft$ — Negative moment due to lane load on adjacent spans

$M_{neg.truck} := M_{neg.lane} + (1 + IM) \cdot \min(M_{truck}) = -2750 \cdot kip \cdot ft$ — Live load negative moment for single truck

$M_{neg.train} := 0.9 \cdot \left[M_{neg.lane} + (1 + IM) \cdot \min(M_{train})\right] = -3387 \cdot kip \cdot ft$ — Live load negative moment for two trucks in a single lane

$M_{HL93.neg} := \min(M_{neg.truck}, M_{neg.train}) = -3387 \cdot kip \cdot ft$ — Design negative live load moment, per design lane

$M_{ll.neg.i} := M_{HL93.neg} \cdot g_{mint} = -2144 \cdot kip \cdot ft$ — Design negative live load moment at interior beam

$M_{ll.neg.e} := M_{HL93.neg} \cdot g_{mext} = -2233 \cdot kip \cdot ft$ — Design negative live load moment at exterior beam

$M_{LL.neg} := \text{if}(BeamLoc = 1, M_{ll.neg.e}, M_{ll.neg.i}) = -2233 \cdot kip \cdot ft$ — 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.

$M_{DW.neg} := \dfrac{-w_{fws} \cdot L^2}{2} = -487 \cdot kip \cdot ft$ — Superimposed dead load resisted by continuity section

$M_{u.neg.StrI} := 1.5 \cdot M_{DW.neg} + 1.75 \cdot M_{LL.neg} = -4638 \cdot kip \cdot ft$ — Strength Limit State

$M_{u.neg.StrI} := 1.0 \cdot M_{DW.neg} + 1.0 \cdot M_{LL.neg} = -2720 \cdot kip \cdot ft$ — Service Limit State

16. NEGATIVE MOMENT FLEXURAL STRENGTH (cont'd)

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)
$b_c := b_1 = 26 \cdot in$	Width of compression block at bottom flange
$d_{nms} := h - t_{sac} - 0.5 \cdot (t_{flange} - t_{sac}) = 37 \cdot in$	Distance to centroid of negative moment steel, taken at mid-depth of top flange
$R_u := \dfrac{\left\|M_{u.neg.StrI}\right\|}{\varphi_f \cdot b_c \cdot d_{nms}^{\,2}} = 1.019 \cdot ksi$	Factored load, in terms of stress in concrete at depth of steel, for computing steel requirement
$m := \dfrac{f_y}{0.85 \cdot f_c} = 8.824$	Steel-to-concrete strength ratio
$\rho_{req} := \dfrac{1}{m} \cdot \left(1 - \sqrt{1 - \dfrac{2 \cdot m \cdot R_u}{f_y}}\right) = 0.0185$	Required negative moment steel ratio
$A_{nms.req} := \rho_{req} \cdot b_c \cdot d_{nms} = 17.787 \cdot in^2$	Required negative moment steel in top flange
$A_{s.long.t} := 2.0 \cdot in^2 \qquad A_{s.long.b} := 2.0 \cdot in^2$	Full-length longitudinal reinforcement to be made continuous across joint
$A_{bar} := 0.44 \cdot in^2$	Additional negative moment reinforcing bar area
$A_{nms.t} := \dfrac{2}{3} \cdot A_{nms.req} - A_{s.long.t} = 9.858 \cdot in^2$	Additional reinforcement area required in the top mat (2/3 of total)
$n_{bar.t} := ceil\left(\dfrac{A_{nms.t}}{A_{bar}}\right) = 23$	Additional bars required in the top mat
$A_{nms.b} := \dfrac{1}{3} \cdot A_{nms.req} - A_{s.long.b} = 3.929 \cdot in^2$	Additional reinforcement area required in the bottom mat
$n_{bar.b} := ceil\left(\dfrac{A_{nms.b}}{A_{bar}}\right) = 9$	Additional bars required in the top mat
$s_{bar.top} := \dfrac{S - W_j - 6 \cdot in}{n_{bar.t} - 1} = 3.788 \cdot in$	Spacing of bars in top mat
$A_{s.nms} := (n_{bar.t} + n_{bar.b}) \cdot A_{bar} + A_{s.long.t} + A_{s.long.b} = 18.08 \cdot in^2$	Total reinforcing steel provided over pier
$a := \dfrac{A_{s.nms} \cdot f_y}{0.85 \cdot f_c \cdot b_c} = 6.136 \cdot in$	Depth of compression block
$M_{r.neg} := \varphi_f \cdot A_{s.nms} \cdot f_y \cdot \left(d_{nms} - \dfrac{a}{2}\right) = 2761 \cdot kip \cdot ft$	Factored flexural resistance at interior pier
$DC_{neg.mom} := \dfrac{\left\|M_{u.neg.StrI}\right\|}{M_{r.neg}} = 0.985$	
$CheckNegMom := if(DC_{neg.mom} \le 1.0, "OK", "No Good") = "OK"$	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)

Nomenclature

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 = heigth·of·Foam·for·Blockout

LFoam = Length·fo·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

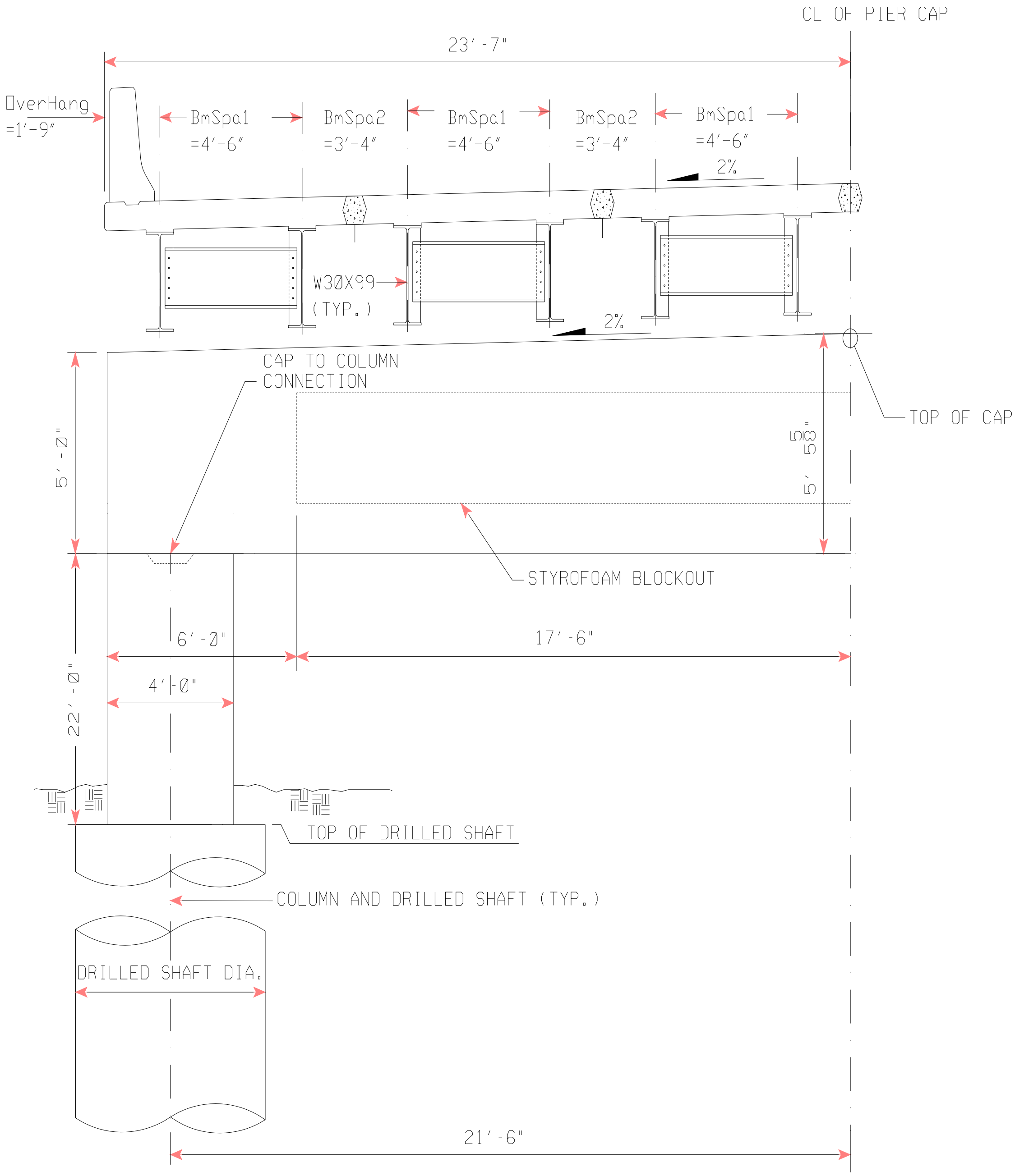
$TorsionDC_{Ext}$ = DeadLoad·Torsion·in·a·Cap·due·to·difference·in·Forward·and·Backward·span·length·per·Exterior·Beam

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:

$FNofBm := 12$ $FSpan := 70\cdot ft$ $FDeckW := \frac{283}{6}\cdot ft$ $FBmAg := 29.1\cdot in^2$ $FBmFlange := 10.5\cdot in$

$FHaunch := 0\cdot in$ $FBmD := 29.7\cdot in$ $FBmIg := 3990\cdot in^4$ $y_{Ft} := 14.85\cdot in$

BACKWARD SPAN PARAMETER INPUT:

$BNofBm := 12$ $BSpan := 70\cdot ft$ $BDeckW := \frac{283}{6}\cdot ft$ $BBmAg := 29.1\cdot in^2$ $BBmFlange := 10.5\cdot in$

$BHaunch := 0\cdot in$ $BBmD := 29.7\cdot in$ $BBmIg := 3990\cdot in^4$ $y_{Bt} := 14.85\cdot in$

COMMON BRIDGE PARAMETER INPUT: Intermediate Bent between Forward and Backward span Parameters

$SlabTh := 9\cdot in$ $Overlay := 25\cdot psf$ $\theta := 0\cdot deg$ $DeckOH := 1.75\cdot ft$ $BrgTh := 3.5\cdot in$

$RailWt := 0.43\cdot klf$ $RailW := 19\cdot in$ $RailH := 34.0\cdot in$ $tBrgSeat := 0\cdot in$ $bBrgSeat := 0\cdot ft$

$DeckW := \frac{283}{6}\cdot ft$ $NofLane := 3$ $m := 0.85$ $w_c := 0.150\cdot kcf$ $f'_c := 5\cdot ksi$ (Cap)

$wCap := 4.5\cdot ft$ $hCap := 5\cdot ft$ $CapL := 47\cdot ft$ $NofDs := 2$ $DsDia := 6\cdot ft$

$wCol := 4\cdot ft$ $bCol := 4\cdot ft$ $NofCol := 2$ $HCol := 22.00\cdot ft$ $f'_{cs} := 4\cdot ksi$ (Slab)

$\gamma_c := 0.150\cdot kcf$ $e_{brg} := 13\cdot in$ $NofBm := 12$ $Sta := 0.25\cdot\frac{ft}{incr}$ $DiapWt := 0.2\cdot kip$

$wEarWall := 0\cdot ft$ $hEarWall := 0\cdot ft$ $tEarWall := 0\cdot in$ $IM := 0.33$ $BmMat := Steel$

$LFoam := 35\cdot ft$ $wFoam := 14\cdot in$ $hFoam := 31\cdot in$ $h_{bS} := 15\cdot in$ (Bottom Solid Depth of Section)

$E_s := 29000\cdot ksi$ $\gamma_{st} := 490\cdot pcf$ (steel)

Modulus of elasticity of Concrete:

$E(f_c) := 33000\cdot(w_c)^{1.5}\cdot\sqrt{f_c\cdot ksi}$ (AASHTO LRFD EQ 5.4.2.4-1 for $K_1 = 1$)

$E_{slab} := E(f'_{cs})$ $E_{slab} = 3834.254\cdot ksi$

$E_{cap} := E(f'_c)$ $E_{cap} = 4286.826\cdot ksi$

Modulus of Beam or Girder: Input Beam Material, BmMat = Steel or Concrete

$E_{beam} := if(BmMat = Steel, E_s, E(f'_c))$ $E_{beam} = 29000\cdot ksi$

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 taker 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

$FBmSpa1 := 4.5\cdot ft$ $\qquad$ $FBmSpa2 := \frac{10}{3}\cdot ft$

$FIntBmTriW := \frac{FBmSpa1}{2} + \frac{FBmSpa2}{2}$ $\qquad$ $FIntBmTriW = 3.917\cdot ft$

$FExtBmTriW := \frac{FBmSpa1}{2} + DeckOH$ $\qquad$ $FExtBmTriW = 4\cdot ft$

$RoadW := 0.25\cdot(FDeckW + 3\cdot DeckW) - 2\cdot RailW$ $\qquad$ $RoadW = 44\cdot ft$

$SlabDC_{Int} := \gamma_c\cdot FIntBmTriW\cdot SlabTh\cdot\left(\frac{FSpan}{2}\right)$ $\qquad$ $SlabDC_{Int} = 15.422\cdot\frac{kip}{beam}$

$SlabDC_{Ext} := \gamma_c\cdot FExtBmTriW\cdot SlabTh\cdot\left(\frac{FSpan}{2}\right)$ $\qquad$ $SlabDC_{Ext} = 15.75\cdot\frac{kip}{beam}$

$BeamDC := \gamma_{st}\cdot FBmAg\cdot\left(\frac{FSpan}{2}\right)$ $\qquad$ $BeamDC = 3.466\cdot\frac{kip}{beam}$

$HaunchDC := \gamma_c\cdot FHaunch\cdot FBmFlange\cdot\left(\frac{FSpan}{2}\right)$ $\qquad$ $HaunchDC = 0\cdot\frac{kip}{beam}$

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 < 4^o
6. Bridge cross-section is consistent with one of the x-section shown in AASHTO LRFD TABLE 4.6.2.2.1-1

$RailDC := \frac{2\cdot RailWt}{FNofBm}\cdot\left(\frac{FSpan}{2}\right)$ $\qquad$ $RailDC = 2.508\cdot\frac{kip}{beam}$

$OverlayDW := \frac{RoadW\cdot Overlay}{FNofBm}\cdot\left(\frac{FSpan}{2}\right)$ $\qquad$ $OverlayDW = 3.208\cdot\frac{kip}{beam}$

Forward Span Superstructure DC & DW per Interior and Exterior Beam:

$FSuperDC_{Int} := RailDC + BeamDC + SlabDC_{Int} + HaunchDC + DiapWt$ $\qquad FSuperDC_{Int} = 21.596 \cdot \frac{kip}{beam}$

$FSuperDC_{Ext} := RailDC + BeamDC + SlabDC_{Ext} + HaunchDC + 0.5 \cdot DiapWt$ $\qquad FSuperDC_{Ext} = 21.824 \cdot \frac{kip}{beam}$

$FSuperDW := OverlayDW$ $\qquad FSuperDW = 3.208 \cdot \frac{kip}{beam}$

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

$BBmSpa1 := 4.5 \cdot ft$ $\qquad BBmSpa2 := \frac{10}{3} \cdot ft$

$BIntBmTriW := \frac{BBmSpa1}{2} + \frac{BBmSpa2}{2}$ $\qquad BIntBmTriW = 3.917 \cdot ft$

$BExtBmTriW := \frac{BBmSpa1}{2} + DeckOH$ $\qquad BExtBmTriW = 4 \cdot ft$

$RoadW := 0.25 \cdot (BDeckW + 3 \cdot DeckW) - 2 \cdot RailW$ $\qquad RoadW = 44 \cdot ft$

$SlabDC_{Int} := \gamma_c \cdot BIntBmTriW \cdot SlabTh \cdot \left(\frac{BSpan}{2}\right)$ $\qquad SlabDC_{Int} = 15.422 \cdot \frac{kip}{beam}$

$SlabDC_{Ext} := \gamma_c \cdot BExtBmTriW \cdot SlabTh \cdot \left(\frac{BSpan}{2}\right)$ $\qquad SlabDC_{Ext} = 15.75 \cdot \frac{kip}{beam}$

$BeamDC := \gamma_{st} \cdot BBmAg \cdot \left(\frac{BSpan}{2}\right)$ $\qquad BeamDC = 3.466 \cdot \frac{kip}{beam}$

$HaunchDC := \gamma_c \cdot BHaunch \cdot BBmFlange \cdot \left(\frac{BSpan}{2}\right)$ $\qquad HaunchDC = 0 \cdot \frac{kip}{beam}$

$RailDC := \frac{2 \cdot RailWt}{BNofBm} \cdot \left(\frac{BSpan}{2}\right)$ $\qquad RailDC = 2.508 \cdot \frac{kip}{beam}$

$OverlayDW := \frac{RoadW \cdot Overlay}{BNofBm} \cdot \left(\frac{BSpan}{2}\right)$ $\qquad OverlayDW = 3.208 \cdot \frac{kip}{beam}$

Total Backward Span Superstructure DC & DW per Interior and Exterior Beam:

$BSuperDC_{Int} := RailDC + BeamDC + SlabDC_{Int} + HaunchDC + DiapWt$ $\qquad BSuperDC_{Int} = 21.596 \cdot \frac{kip}{beam}$

$BSuperDC_{Ext} := RailDC + BeamDC + SlabDC_{Ext} + HaunchDC + 0.5 \cdot DiapWt$ $\qquad BSuperDC_{Ext} = 21.824 \cdot \frac{kip}{beam}$

$BSuperDW := OverlayDW$ $\qquad BSuperDW = 3.208 \cdot \frac{kip}{beam}$

Total Superstructure DC & DW per Beam on Bent Cap:

$SuperDC_{Int} := FSuperDC_{Int} + BSuperDC_{Int}$ $\qquad SuperDC_{Int} = 43.192 \cdot \frac{kip}{beam}$

$SuperDC_{Ext} := FSuperDC_{Ext} + BSuperDC_{Ext}$ $\qquad SuperDC_{Ext} = 43.648 \cdot \frac{kip}{beam}$

$SuperDW := FSuperDW + BSuperDW$ $\qquad SuperDW = 6.417 \cdot \frac{kip}{beam}$

$TorsionDC_{Int} := \left(\max\left(FSuperDC_{Int}, BSuperDC_{Int}\right) - \min\left(FSuperDC_{Int}, BSuperDC_{Int}\right)\right) \cdot e_{brg}$ $\qquad TorsionDC_{Int} = 0 \cdot \frac{kft}{beam}$

$TorsionDC_{Ext} := \left(\max\left(FSuperDC_{Ext}, BSuperDC_{Ext}\right) - \min\left(FSuperDC_{Ext}, BSuperDC_{Ext}\right)\right) \cdot e_{t}$ $\qquad TorsionDC_{Ext} = 0 \cdot \frac{kft}{beam}$

$TorsionDW := (\max(FSuperDW, BSuperDW) - \min(FSuperDW, BSuperDW)) \cdot e_{brg}$ $\qquad TorsionDW = 0 \cdot \frac{kft}{beam}$

CAP, EAR WALL & BEARING SEAT WEIGHT:

The Bent cap has two sections along the length. One is a solid rectangular section 6ft from the 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.

$CapDC1 := wCap \cdot hCap \cdot \gamma_c$ $\quad Applicable \cdot for \cdot (0 \cdot ft \le CapL \le 6 \cdot ft), (41 \cdot ft \le CapL \le 47 \cdot ft)$ $\qquad CapDC1 = 3.375 \cdot \frac{kip}{ft}$

$CapDC2 := (wCap \cdot hCap - 2 \cdot wFoam \cdot hFoam) \cdot \gamma_c$ $\quad Applicable \cdot for \cdot (6 \cdot ft \le CapL \le 41 \cdot ft)$ $\qquad CapDC2 = 2.471 \cdot \frac{kip}{ft}$

$EarWallDC := (wEarWall \cdot hEarWall \cdot tEarWall) \cdot \gamma_c$ $\qquad EarWallDC = 0 \cdot kip$

$BrgSeatDC := tBrgSeat \cdot bBrgSeat \cdot (wCap) \cdot \gamma_c$ $\qquad BrgSeatDC = 0 \cdot \frac{kip}{beam}$

Distribution Factor

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: $P_1 := 8\cdot kip$ $P_2 := 32\cdot kip$ $P_3 := 32\cdot kip$ (AASHTO·LRFD·3.6.1.2.2) $TruckT := P_1 + P_2 + P_3$

Design Lane Load: $w_{lane} := 0.64\cdot klf$ (AASHTO·LRFD·3.6.1.2.4)

$LongSpan := \max(FSpan, BSpan)$ $ShortSpan := \min(FSpan, BSpan)$

$L_{long} := LongSpan$ $L_{short} := ShortSpan$

Lane Load Reaction

$$Lane := w_{lane}\cdot\left(\frac{L_{long} + L_{short}}{2}\right) \qquad Lane = 44.8\cdot\frac{kip}{lane}$$

Truck Load Reaction

$$Truck := P_2 + P_3\cdot\frac{(L_{long} - 14ft)}{L_{long}} + P_1\cdot\max\left[\frac{(L_{long} - 28ft)}{L_{long}}, \frac{(L_{short} - 14ft)}{L_{short}}\right] \qquad Truck = 64\cdot\frac{kip}{lane}$$

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)

$$LLRxn := Lane + Truck\cdot(1 + IM) \qquad LLRxn = 129.92\cdot\frac{kip}{lane}$$

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:

$$P := (0.5\cdot P_3)\cdot(1 + IM) \qquad P = 21.28\cdot kip$$

The Design Lane Load Width Transversely in a Lane

$wlaneTransW := 10\cdot ft$ AASHTO LRFD Article 3.6.1.2.1

The uniform load portion of the Live Load, kip/station for Cap Loading Program:

$$W := \frac{(LLRxn - 2\cdot P)\cdot Sta}{wlaneTransW} \qquad W = 2.184\cdot\frac{kip}{incr}$$

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:

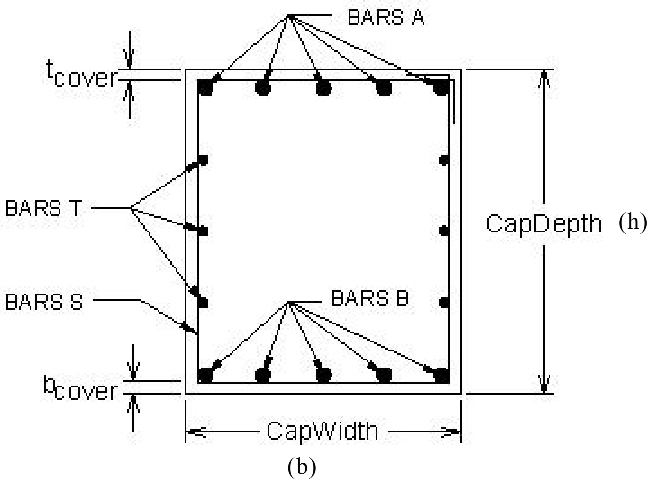
Torsional load about center line of bent cap occurs due to horizontal loads acting on the superstructure perpendicular to the bent length or along the bridge length. Braking force, Centrifugal force, WS on superstructure, and WL cause torsion on bent.

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:



$f'_c := 5.0 \cdot ksi$ $\quad f_y := 60 \cdot ksi$ $\quad E_s := 29000 \cdot ksi$ $\quad \phi_m := 0.9$ $\quad \phi_v := 0.9$ $\quad \phi_n := 1$

$\gamma_c := 0.150 \cdot kcf$ $\quad b_{cover} := 2 \cdot in$ $\quad t_{cover} := 2 \cdot in$ $\quad h := 5 \cdot ft$ $\quad b := 4.5 \cdot ft$ $\quad E_c := E_{cap}$

$n := \text{round}\left(\frac{E_s}{E_c}, 0\right)$ (AASHTO LRFD 5.7.1) $\quad n = 7$

$EI_{cap1} := E_c \cdot \frac{(b \cdot h^3)}{12}$ $\quad$ Applicable·for·$0 \le CapL \le 6, 41 \le CapL \le 47$ $\quad EI_{cap1} = 2.894 \times 10^7 \cdot kip \cdot ft^2$

$$y_{cg2} := \frac{wCap \cdot hCap \cdot \frac{hCap}{2} - 2 \cdot (wFoam \cdot hFoam) \cdot \left(\frac{hFoam}{2} + h_{bS}\right)}{wCap \cdot hCap - 2 \cdot (wFoam \cdot hFoam)}$$

(y_{cg} of from Bottom of Cap Section) $\quad y_{cg2} = 29.817 \cdot in$

$$I_{cap2} := \frac{wCap \cdot hCap^3}{12} + wCap \cdot hCap \cdot \left(\frac{hCap}{2} - y_{cg2}\right)^2 \ldots$$
$$+ -2 \cdot \left[\frac{wFoam \cdot hFoam^3}{12} + wFoam \cdot hFoam \cdot \left(\frac{hFoam}{2} + h_{bS} - y_{cg2}\right)^2\right]$$

$I_{cap2} = 902191.259 \cdot in^4$

$EI_{cap2} := E_c \cdot I_{cap2}$ $\quad$ Applicable·for·$6 \le CapL \le 41$ $\quad EI_{cap2} = 2.686 \times 10^7 \cdot kip \cdot ft^2$

OUTPUT of BENT CAP LOADING PROGRAM: The maximum load effects from different applicable limit states:

DEAD LOAD $\quad M_{dlPos} := 3309.6 \cdot kft$ $\quad M_{dlNeg} := 30.1 \cdot kft$

SERVICE I $\quad M_{sPos} := 5377.1 \cdot kft$ $\quad M_{sNeg} := 45.1 \cdot kft$

STRENGTH I $M_{uPos} := 7830.6 \cdot kft$ $M_{uNeg} := 64.6 \cdot kft$

FLEXURE DESIGN:

MINIMUM FLEXURAL REINFORCEMENT *AASHTO LRFD 5.7.3.3.2*

Factored Flexural Resistance, Mr, must be greater than or equal to the lesser of 1.2Mcr or 1.33 Mu. Applicable to both positive and negative moment.

Modulus of rupture

$f_r := 0.37\sqrt{f'_c \cdot ksi}$ (AASHTO LRFD EQ 5.4.2.6) $f_r = 0.827 \cdot ksi$

$S := \dfrac{I_{cap2}}{y_{cg2}}$ (Bottom Section Modulus for Positive Moment) $S = 30257.581 \cdot in^3$

Cracking moment

$M_{cr} := S \cdot f_r$ (AASHTO LRFD EQ 5.7.3.3.2-1) $M_{cr} = 2086.122 \cdot kip \cdot ft$

$M_{cr1} := 1.2 \cdot M_{cr}$ $M_{cr1} = 2503.346 \cdot kip \cdot ft$

$M_{cr2} := 1.33 \cdot \max(M_{uPos}, M_{uNeg})$ $M_{cr2} = 10414.698 \cdot kip \cdot ft$

$M_{cr\_min} := \min(M_{cr1}, M_{cr2})$ Therefore Mr must be <u>greater</u> than $M_{cr\_min} = 2503.346 \cdot kip \cdot ft$

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

$A_{bp} := (1.56\ 1.56\ 1.56\ 0\ 0\ 0\ 0\ 0\ 0\ 0\ 0\ 0) \cdot in^2$

Input center to center vertical distance between each rebar row starting from bottom of cap

$clp := (3.5\ 4\ 4\ 0\ 0\ 0\ 0\ 0\ 0\ 0\ 0\ 0) \cdot in$

dc Calc for Pos Moment

$ns_{Pos} = 3$ (No. of Bottom or Positive Steel Layers)

Distance from centroid of positive rebar to extreme bottom tension fiber (d_{cPos}):

$d_{cPos} := (Ayp_{0,0}) \cdot in$ $d_{cPos} = 7.5 \cdot in$

Effective depth from centroid of bottom rebar to extreme compression fiber (d_{Pos}):

$d_{Pos} := h - d_{cPos}$ $d_{Pos} = 52.5 \cdot in$

Compression Block depth under ultimate load *AASHTO LRFD 5.7.2.2*

$$\beta_1 := \min\left[0.85, \max\left[0.65, 0.85 - \frac{0.05}{ksi}\left(f'_c - 4\cdot ksi\right)\right]\right] \qquad \beta_1 = 0.8$$

The Amount of Bottom or Positive Steel A_s Required,

$$A_{sReq} := \left(\frac{0.85\cdot f'_c\cdot b\cdot d_{Pos}}{f_y}\right)\cdot\left(1 - \sqrt{1 - \frac{2\cdot M_{uPos}}{0.85\cdot\phi_m\cdot f'_c\cdot b\cdot d_{Pos}{}^2}}\right) \qquad A_{sReq} = 36.454\cdot in^2$$

The Amount of Positive A_s Provided,

$$NofBars_{Pos} := \sum N_p \qquad NofBars_{Pos} = 27$$

$$A_{sPos} := \left(Ayp_{0,1}\right)\cdot in^2 \qquad A_{sPos} = 42.12\cdot in^2$$

$$h_{tS} := h - hFoam - h_{bS} \quad \text{(Top solid depth)} \qquad h_{tS} = 14\cdot in$$

Compression depth under ultimate load

$$c_{Pos} := \frac{A_{sPos}\cdot f_y}{0.85\cdot f'_c\cdot\beta_1\cdot b} \quad \text{(AASHTO LRFD EQ 5.7.3.1.1-4)} \qquad c_{Pos} = 13.765\cdot in$$

$$a_{Pos} := \beta_1\cdot c_{Pos} \quad \left(a_{Pos} < h_{tS}, OK\right) \quad \text{(AASHTO LRFD 5.7.3.2.2)} \qquad a_{Pos} = 11.012\cdot in$$

Nominal flexural resistance:

$$M_{nPos} := A_{sPos}\cdot f_y\cdot\left(d_{Pos} - \frac{a_{Pos}}{2}\right) \quad \text{(AASHTO LRFD EQ 5.7.3.2.2-1)} \qquad M_{nPos} = 9896.961\cdot kip\cdot ft$$

Tension controlled resistance factor for flexure

$$\phi_{mPos} := \min\left[0.65 + 0.15\cdot\left(\frac{d_{Pos}}{c_{Pos}} - 1\right), 0.9\right] \quad \text{(AASHTO LRFD EQ 5.5.4.2.1-2)} \qquad \phi_{mPos} = 0.9$$

or simply use, $\phi_m = 0.9$ (AASHTO LRFD 5.5.4.2)

$$M_{rPos} := \phi_{mPos}\cdot M_{nPos} \quad \text{(AASHTO LRFD EQ 5.7.3.2.1-1)} \qquad M_{rPos} = 8907.265\cdot kip\cdot ft$$

$$M_{uPos} = 7830.6\cdot kip\cdot ft$$

$$MinReinChkPos := if\left[\left(M_{rPos} \ge M_{cr\_min}\right), \text{"OK"}, \text{"NG"}\right] \qquad MinReinChkPos = \text{"OK"}$$

$\text{UltimateMomChkPos} := \text{if}\left[\left(M_{rPos} \geq M_{uPos}\right), \text{"OK"}, \text{"NG"}\right]$ $\text{UltimateMomChkPos} = \text{"OK"}$

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

$A_{bn} := (0.6\ \ 1.27\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0)\cdot in^2$

Input center to center vertical distance between each rebar row starting from top of cap

$cln := (3.5\ \ 4\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0)\cdot in$

dc Calc for Neg. Moment

$ns_{Neg} = 2$ (No. of Negative or Top Steel Layers)

Distance from centroid of negative rebar to top extreme tension fiber (d_{cNeg}):

$d_{cNeg} := \left(Ayn_{0,0}\right)\cdot in$ $d_{cNeg} = 6.217\cdot in$

Effective depth from centroid of top rebar to extreme compression fiber (d_{Neg}):

$d_{Neg} := h - d_{cNeg}$ $d_{Neg} = 53.783\cdot in$

The Amount of Negative A_s Required,

$$A_{sReq} := \left(\frac{0.85\cdot f'_c\cdot b\cdot d_{Neg}}{f_y}\right)\cdot\left(1 - \sqrt{1 - \frac{2\cdot M_{uNeg}}{0.85\cdot\phi_m\cdot f'_c\cdot b\cdot {d_{Neg}}^2}}\right)$$ $A_{sReq} = 0.267\cdot in^2$

The Amount of Negative A_s Provided,

$NofBars_{Neg} := \sum N_n$ $NofBars_{Neg} = 12$

$A_{sNeg} := \left(Ayn_{0,1}\right)\cdot in^2$ $A_{sNeg} = 11.22\cdot in^2$

Compression depth under ultimate load

$$c_{Neg} := \frac{A_{sNeg}\cdot f_y}{0.85\cdot f'_c\cdot\beta_1\cdot b}$$ $c_{Neg} = 3.667\cdot in$

$a_{Neg} := \beta_1\cdot c_{Neg}$ $a_{Neg} = 2.933\cdot in$

Thus, nominal flexural resistance:

$M_{nNeg} := A_{sNeg} \cdot f_y \cdot \left(d_{Neg} - \frac{a_{Neg}}{2} \right)$ $M_{nNeg} = 2934.97 \cdot kip \cdot ft$

Factored flexural resistance

$M_{rNeg} := \phi_m \cdot M_{nNeg}$ $M_{rNeg} = 2641.473 \cdot kip \cdot ft$

$M_{uNeg} = 64.6 \cdot kip \cdot ft$

$MinReinChkNeg := if\left[\left(M_{rNeg} \geq M_{cr\_min}\right), "OK", "NG"\right]$ $MinReinChkNeg = "OK"$

$UltimateMomChkNeg := if\left[\left(M_{rNeg} \geq M_{uNeg}\right), "OK", "NG"\right]$ $UltimateMomChkNeg = "OK"$

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)

$\gamma_e := if(exposure\_cond = 1, 1, 0.75)$ (Exposure condition factor) $\gamma_e = 1$

$\left(side_{cTop} \quad side_{cBot} \right) := (5.625 \quad 4.75) \cdot in$ (Input side cover for Top and Bottom Rebars)

Positive Moment (Bottom Bars B) To find S_{max}: S is spacing of first layer of rebar closest to tension face

$n := round\left(\frac{E_s}{E_c}, 0\right)$ (modular ratio) $n = 7$

$\rho_{Pos} := \frac{A_{sPos}}{b \cdot d_{Pos}}$ $\rho_{Pos} = 0.0149$

$k_{Pos} := \sqrt{\left(\rho_{Pos} \cdot n + 1\right)^2 - 1} - \rho_{Pos} \cdot n$ (Applicable for Solid Rectangular Section) $k_{Pos} = 0.364$

$kd_P := k_{Pos} \cdot d_{Pos}$ Location of NA from Top of Cap for Pos Moment $kd_P = 19.098 \cdot in$

$StressBlock_{Pos} := if\left(kd_P \geq h_{tS}, "T-Section", "Rec-Section"\right)$ $StressBlock_{Pos} = "T-Section"$

$Comression \cdot Force = Tension \cdot Force$ OR $Moment \cdot of \cdot Comression \cdot Area = Moment \cdot of \cdot Tension \cdot Area \cdot about \cdot NA$

$$b \cdot \left(kd_{Pos}\right)^2 - 2 \cdot wFoam \cdot \left(kd_{Pos} - hStop\right)^2 = 2 \cdot n \cdot A_{sPos} \cdot \left(d_{Pos} - kd_{Pos}\right)$$

$$(b - 2 \cdot wFoam) \cdot \left(kd_{Pos}\right)^2 + \left(2 \cdot n \cdot A_{sPos} + 4 \cdot wFoam \cdot hStop\right) \cdot \left(kd_{Pos}\right) - 2 \cdot \left(wFoam \cdot hStop^2 + n \cdot A_{sPos} \cdot d_{Pos}\right) = 0$$

$$kd_{Pos} := \frac{-\left(2\cdot n\cdot A_{sPos} + 4\cdot wFoam\cdot h_{tS}\right) + \sqrt{\left(2\cdot n\cdot A_{sPos} + 4\cdot wFoam\cdot h_{tS}\right)^2 \ldots + 4\cdot(b - 2\cdot wFoam)\cdot 2\cdot\left(wFoam\cdot h_{tS}^{\,2} + n\cdot A_{sPos}\cdot d_{Pos}\right)}}{2\cdot(b - 2\cdot wFoam)}$$

$kd_{Pos} = 19.405\cdot in$ Location of NA from Top of Cap

Location of Resultant Compression force from NA for Positive Moment:

$$x_{Pos} := \frac{b\cdot\frac{\left(kd_{Pos}\right)^2}{3} - \frac{2}{3}\cdot wFoam\cdot\left(kd_{Pos} - h_{tS}\right)^2\cdot\left(1 - \frac{h_{tS}}{kd_{Pos}}\right)}{\frac{1}{2}\cdot b\cdot kd_{Pos} - wFoam\cdot\left(kd_{Pos} - h_{tS}\right)\cdot\left(1 - \frac{h_{tS}}{kd_{Pos}}\right)} \qquad x_{Pos} = 13.328\cdot in$$

$$jd_{Pos} := d_{Pos} - kd_{Pos} + x_{Pos} \qquad jd_{Pos} = 46.423\cdot in$$

Tensile Stress at Service Limit State

$$f_{ssPos} := \frac{M_{sPos}}{A_{sPos}\cdot jd_{Pos}} \qquad f_{ssPos} = 33\cdot ksi$$

$d_{c1Pos} := clp_{0,0}$ (Distance of bottom first row rebar closest to tension face) $d_{c1Pos} = 3.5\cdot in$

$$\beta_{sPos} := 1 + \frac{d_{c1Pos}}{0.7\cdot\left(h - d_{c1Pos}\right)} \qquad \beta_{sPos} = 1.088$$

$$s_{maxPos} := \frac{700\,\frac{kip}{in}\cdot\gamma_e}{\beta_{sPos}\cdot f_{ssPos}} - 2\cdot d_{c1Pos}$$ AASHTO LRFD EQ (5.7.3.4-1) $s_{maxPos} = 12.488\cdot in$

$$s_{ActualPos} := \frac{b - 2\cdot side_{cBot}}{N_{p_{0,0}} - 1}$$ (Equal horizontal spacing of bottom first rebar row closest to tension face) $s_{ActualPos} = 5.563\cdot in$

Actual Max Spacing Provided in Bottom first row closest to Tension Face, $s_{aPosProvided} := 7\cdot in$

$$s_{ActualPos} := \max\left(s_{aPosProvided}, s_{ActualPos}\right) \qquad s_{ActualPos} = 7\cdot in$$

SpacingCheckPos := if$\left[\left(s_{maxPos} \geq s_{ActualPos}\right), \text{"OK"}, \text{"NG"}\right]$ SpacingCheckPos = "OK"

Negative Moment (Top Bars A)

$$\rho_{Neg} := \frac{A_{sNeg}}{b \cdot d_{Neg}}$$ $\rho_{Neg} = 3.863 \times 10^{-3}$

$$k_{Neg} := \sqrt{(\rho_{Neg} \cdot n + 1)^2 - 1} - \rho_{Neg} \cdot n$$ (Applicable for Solid Rectangular Section) $k_{Neg} = 0.207$

$kd_N := k_{Neg} \cdot d_{Neg}$ Location of NA from Bottom of Cap for Neg Moment $kd_N = 11.138 \cdot in$

$StressBlock_{Neg} := if(kd_N \geq h_{bS}, \text{"T-Section"}, \text{"Rec-Section"})$ $StressBlock_{Neg} = \text{"Rec-Section"}$

$$j_{Neg} := 1 - \frac{k_{Neg}}{3}$$ $j_{Neg} = 0.931$

$$f_{ssNeg} := \frac{M_{sNeg}}{A_{sNeg} \cdot j_{Neg} \cdot d_{Neg}}$$ $f_{ssNeg} = 0.963 \cdot ksi$

$d_{c1Neg} := cln_{0,0}$ (Distance of top first row rebar closest to tension face) $d_{c1Neg} = 3.5 \cdot in$

$$\beta_{sNeg} := 1 + \frac{d_{c1Neg}}{0.7 \cdot (h - d_{c1Neg})}$$ $\beta_{sNeg} = 1.088$

$$s_{maxNeg} := \frac{700 \frac{kip}{in} \cdot \gamma_e}{\beta_{sNeg} \cdot f_{ssNeg}} - 2 \cdot d_{c1Neg}$$ $s_{maxNeg} = 660.561 \cdot in$

$$s_{ActualNeg} := \frac{b - 2 \cdot side_{cTop}}{N_{n_{0,0}} - 1}$$ (Equal horizontal spacing of top first rebar row closest to tension face) $s_{ActualNeg} = 8.55 \cdot in$

Actual Max Spacing Provided in Top first row closest to Tension Face, $s_{aNegProvided} := 11.125 \cdot in$

$s_{ActualNeg} := max(s_{aNegProvided}, s_{ActualNeg})$ $s_{ActualNeg} = 11.125 \cdot in$

$SpacingCheckNeg := if[(s_{maxNeg} \geq s_{ActualNeg}), \text{"OK"}, \text{"NG"}]$ $SpacingCheckNeg = \text{"OK"}$

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, $A_{skBar} := 0.79 \cdot in^2$

$d_{cTop} := \sum cln$ $\qquad d_{cTop} = 7.5 \cdot in$

$d_{cBot} := \sum clp$ $\qquad d_{cBot} = 11.5 \cdot in$

Effective Depth from centroid of ExtremeTension Steel to Extreme compression Fiber (d_l):

$d_l := \max\left(h - clp_{0,0}, h - cln_{0,0}\right)$ $\qquad d_l = 56.5 \cdot in$

Effective Depth from centroid of Tension Steel to Extreme compression Fiber (d_e):

$d_e := \max\left(d_{Pos}, d_{Neg}\right)$ $\qquad d_e = 53.783 \cdot in$

$A_s := \min\left(A_{sNeg}, A_{sPos}\right)$ min. of negative and positive reinforcement $\qquad A_s = 11.22 \cdot in^2$

$d_{skin} := h - \left(d_{cTop} + d_{cBot}\right)$ $\qquad d_{skin} = 41 \cdot in$

Skin Reinforcement Requirement: AASHTO LRFD EQ 5.7.3.4-2

$$A_{skReq} := \text{if}\left[d_l > 3ft, \min\left[0.012 \cdot \frac{in}{ft} \cdot \left(d_l - 30 \cdot in\right) \cdot d_{skin}, \frac{A_s + A_{ps}}{4}\right], 0in^2\right] \qquad A_{skReq} = 1.087 \cdot in^2$$

$$NoA_{skbar1} := R\left(\frac{A_{skReq}}{A_{skBar}}\right) \qquad NoA_{skbar1} = 2$$ per Side

Maximum Spacing of Skin Reinforcement:

$$S_{skMax} := \min\left(\frac{d_e}{6}, 12 \cdot in\right) \qquad S_{skMax} = 8.964 \cdot in$$ AASHTO LRFD 5.7.3.4

$$NoA_{skbar2} := \text{if}\left(d_l > 3ft, R\left(\frac{d_{skin}}{S_{skMax}} - 1\right), 1\right) \qquad NoA_{skbar2} = 4$$ per Side

$$NofSideBars_{req} := \max\left(NoA_{skbar1}, NoA_{skbar2}\right) \qquad NofSideBars_{req} = 4$$

$$S_{skRequired} := \frac{d_{skin}}{1 + NofSideBars_{req}} \qquad S_{skRequired} = 8.2 \cdot in$$

$NofSideBars := 5$ (No. of Side Bars Provided)

$S_{skProvided} := \dfrac{d_{skin}}{1 + NofSideBars}$ $S_{skProvided} = 6.833 \cdot in$

$S_{skChk} := if\left(S_{skProvided} < S_{skMax}, "OK", "N.G."\right)$ $S_{skChk} = "OK"$

Therefore Use: $NofSideBars = 5$ and Size $SkBarNo = 8$

3. BENT CAP SHEAR AND TORSION DESIGN

SHEAR DESIGN OF CAP:

Effective Shear Depth, $d_v = \max\left(\left(\begin{matrix} d_e - \dfrac{a}{2} \\ 0.9 \cdot d_e \\ 0.72 \cdot h \end{matrix}\right)\right)$ (*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,	$A_s := A_{sPos}$	$A_s = 42.12 \cdot in^2$
Nominal Flexure,	$M_n := M_{nPos}$	$M_n = 9896.961 \cdot kft$
Stress block Depth,	$a := a_{Pos}$	$a = 11.012 \cdot in$
Effective Depth,	$d_e := d_{Pos}$	$d_e = 52.5 \cdot in$
Effective web Width at critical Location,	$b_v := b$	$b_v = 4.5 \cdot ft$
Input initial θ,	$\theta := 35 \cdot deg$	$cot\theta := \cot(\theta)$
Shear Resistance Factor,	$\phi_v := 0.9$	
Cap Depth & Width,	$h = 60 \cdot in$	$b = 54 \cdot in$

Moment Arm, $\left(d_e - \frac{a}{2}\right) = 46.994 \cdot in$ $\quad 0.9 \cdot d_e = 47.25 \cdot in$ $\quad 0.72 \cdot h = 43.2 \cdot in$

Effective Shear Depth at Critical Location, $d_v := \max\left(\begin{pmatrix} d_e - \frac{a}{2} \\ 0.9 \cdot d_e \\ 0.72 \cdot h \end{pmatrix}\right)$ (*AASHTO LRFD 5.8.2.9*) $\quad d_v = 47.25 \cdot in$

$h_h := h - t_{cover} - b_{cover}$ (Height of shear reinforcement) $\quad h_h = 56 \cdot in$

$b_h := b - 2 \cdot b_{cover}$ (Width of shear reinforcement) $\quad b_h = 50 \cdot in$

$p_h := 2\left(h_h + b_h\right)$ (Perimeter of shear reinforcement) $\quad p_h = 212 \cdot in$

$A_{oh} := \left(h_h\right) \cdot \left(b_h\right)$ (Area enclosed by the shear reinforcement) $\quad A_{oh} = 2800 \cdot in^2$

$A_o := 0.85 \cdot A_{oh}$ (AASHTO LRFD C5.8.2.1) $\quad A_o = 2380 \cdot in^2$

$A_c := 0.5 \cdot b \cdot h$ (AASHTO·LRFD·FIGURE·5.8.3.4.2 − 1) $\quad A_c = 1620 \cdot in^2$

Yield strength & Modulus of Elasticity of Steel Reinforcement:

$\left(f_y \;\; E_s\right) := (60 \;\; 29000) \cdot ksi$ (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)

$\left(M_u \;\; T_u\right) := (1314.8 \;\; 964.6) \cdot kft$ $\quad \left(V_u \;\; N_u\right) := (665.4 \;\; 0) \cdot kip$

$M'_u := \max\left(M_u, \left|V_u - V_p\right| \cdot d_v\right)$ AASHTO LRFD B5.2 $\quad M'_u = 2620.013 \cdot kip \cdot ft$

$V'_u := \sqrt{{V_u}^2 + \left(\frac{0.9 \cdot p_h \cdot T_u}{2 \cdot A_o}\right)^2}$ (Equivalent shear) AASHTO LRFD EQ (5.8.2.1-6) for solid section $\quad V'_u = 811.194 \cdot kip$

Assuming·atleast·minimum·transverse·reinforcement·is·provided (Always provide min. transverse reinf.)

$$\varepsilon_x = \frac{\left(\frac{M'_u}{d_v}\right) + 0.5 \cdot N_u + 0.5 \cdot \left(V'_u - V_p\right) \cdot \cot\theta - A_{ps} \cdot f_{po}}{2 \cdot \left(E_s \cdot A_s + E_p \cdot A_{ps}\right)}$$

(Strain from Appendix B5) AASHTO LRFD EQ (B5.2-1)

$v_u := \dfrac{(V_u - \phi_v \cdot V_p)}{\phi_v \cdot b_v \cdot d_v}$ (Shear Stress) AASHTO LRFD EQ (5.8.2.9-1) $v_u = 0.29 \cdot ksi$

$r := \max\left(0.075, \dfrac{v_u}{f'_c}\right)$ (Shear stress ratio) $r = 0.075$

Determining Beta & Theta

After Interpolating the value of $(\Theta \quad B)$

$\Theta = 30.773 \cdot deg$ $B = 2.572$

Nominal Shear Resistance by Concrete,

$V_c := 0.0316 \cdot B \cdot \sqrt{f'_c \cdot ksi} \cdot b_v \cdot d_v$ AASHTO LRFD EQ (5.8.3.3-3) $V_c = 463.7 \cdot kip$

$V_u = 665.4 \cdot kip$ $0.5 \cdot \phi_v \cdot (V_c + V_p) = 208.673 \cdot kip$

REGION REQUIRING TRANSVERSE REINFORCEMENT: AASHTO LRFD 5.8.2.4

$V_u > 0.5 \cdot \phi_v \cdot (V_c + V_p)$ AASHTO LRFD EQ (5.8.2.4-1)

check := if$[V_u > 0.5 \cdot \phi_v \cdot (V_c + V_p)$, "Provide Shear Reinf", "No reinf."$]$ check = "Provide Shear Reinf"

$V_n = \min\left(\begin{pmatrix} V_c + V_s + V_p \\ 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p \end{pmatrix}\right)$ (Nominal Shear Resistance) AASHTO·LRFD·EQ·(5.8.3.3 − 1, 2)

$V_s = \dfrac{A_v \cdot f_y \cdot d_v \cdot (\cot\theta + \cot\alpha) \cdot \sin\alpha}{S}$ (Shear Resistance of Steel) AASHTO·LRFD·EQ·(5.8.3.3 − 4)

$V_s = \dfrac{A_v \cdot f_y \cdot d_v \cdot \cot\theta}{S}$ (Shear·Resistance·of·Steel·when, $\alpha = 90 \cdot deg$) AASHTO LRFD EQ (C5.8.3.3-1)

$S_v := 6 \cdot in$ (Input Stirrup Spacing) $V_p = 0 \cdot kip$ $(V_u \quad V_c) = (665.4 \quad 463.718) \cdot kip$

$f_y = 60 \cdot ksi$ $d_v = 47.25 \cdot in$ $\Theta = 30.773 \cdot deg$

$A_{v\_req} := \left(\dfrac{V_u}{\phi_v} - V_c - V_p\right) \cdot \left(\dfrac{S_v}{f_y \cdot d_v \cdot \cot\Theta}\right)$ (Derive from AASHTO LRFD EQ 5.8.3.3-1, C5.8.3.3-1 and ϕVn >= Vu) $A_{v\_req} = 0.3474 \cdot in^2$

Torsional Steel:

$A_t := \dfrac{T_u}{2\cdot\phi_v\cdot A_o\cdot f_y\cdot \cot\Theta}\cdot S_v$ (Derive from AASHTO LRFD EQ 5.8.3.6.2-1 and $\phi Tn >= Tu$) $A_t = 0.161\cdot in^2$

$A_{vt\_req} := A_{v\_req} + 2\cdot A_t$ (Shear + Torsion) $A_{vt\_req} = 0.669\cdot in^2$

$A_{vt} := 4\cdot\left(0.44\cdot in^2\right)$ (Use 2 #6 double leg Stirrup at S_v c/c,) *Provided*, $A_{vt} = 1.76\cdot in^2$

$A_{vt\_check} := if\left(A_{vt} > A_{vt\_req}, \text{"OK"}, \text{"NG"}\right)$ $A_{vt\_check} = \text{"OK"}$

Maximum Spacing Check: AASHTO·LRFD·Article·5.8.2.7

$V_u = 665.4\cdot kip$ $0.125\cdot f'_c\cdot b_v\cdot d_v = 1594.69\cdot kip$

$S_{vmax} := if\left(V_u < 0.125\cdot f'_c\cdot b_v\cdot d_v, \min\left(0.8\cdot d_v, 24\cdot in\right), \min\left(0.4\cdot d_v, 12\cdot in\right)\right)$ $S_{vmax} = 24\cdot in$

$S_{vmax\_check} := if\left(S_v < S_{vmax}, \text{"OK"}, \text{"use lower spacing"}\right)$ $S_{vmax\_check} = \text{"OK"}$

$A_v := A_{vt} - A_t$ (Shear Reinf. without Torsion Reinf.) $A_v = 1.599\cdot in^2$

$V_s := \dfrac{A_v\cdot f_y\cdot d_v\cdot \cot\Theta}{S_v}$ $V_s = 1268.855\cdot kip$

Minimum Transverse Reinforcement Check: AASHTO·LRFD·Article·5.8.2.5 $b_v = 54\cdot in$

$A_{vmin} := 0.0316\cdot\sqrt{f'_c\cdot ksi}\cdot\dfrac{b_v\cdot S_v}{f_y}$ AASHTO·LRFD·EQ·(5.8.2.5 − 1) $A_{vmin} = 0.382\cdot in^2$

$A_{vmin\_check} := if\left(A_{vt} > A_{vmin}, \text{"OK"}, \text{"NG"}\right)$ $A_{vmin\_check} = \text{"OK"}$

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*

$0.25\cdot f_c\cdot b_v\cdot d_v + V_p = 3189.375\cdot kip$ $V_c + V_s + V_p = 1732.573\cdot kip$

$V_n := \min\left(\begin{pmatrix} V_c + V_s + V_p \\ 0.25\cdot f'_c\cdot b_v\cdot d_v + V_p \end{pmatrix}\right)$ AASHTO·LRFD·EQ·(5.8.3.3 − 1, 2) $V_n = 1732.573\cdot kip$

$\phi_v\cdot V_n = 1559.316\cdot kip$ $V_u = 665.4\cdot kip$

$\phi V_{n\_check} := if\left(\phi_v\cdot V_n > V_u, \text{"OK"}, \text{"NG"}\right)$ $\phi V_{n\_check} = \text{"OK"}$

Torsional Resistance,

$$T_n := \frac{2 \cdot A_o \cdot (0.5 \cdot A_{vt}) \cdot f_y \cdot \cot\Theta}{S_v}$$ AASHTO·LRFD·EQ·(5.8.3.6.2 − 1) $\qquad \phi_v \cdot T_n = 5275.8 \cdot kip \cdot ft$

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

$$A_{ps} \cdot f_{ps} + A_s \cdot f_y \ge \left(\frac{M'_u}{\phi_m \cdot d_v}\right) + \frac{0.5 \cdot N_u}{\phi_n} + \cot\Theta \cdot \sqrt{\left(\frac{V_u}{\phi_v} - V_p - 0.5 \cdot V'_s\right)^2 + \left(\frac{0.45 \cdot p_h \cdot T_u}{2 \cdot \phi_v \cdot A_o}\right)^2}$$

$(\phi_m\ \phi_v\ \phi_n) := (0.9\ \ 0.9\ \ 1)$ $\qquad A_s \cdot f_y + A_{ps} \cdot f_{ps} = 2527.2 \cdot kip$

$M'_u = 2620.013 \cdot kip \cdot ft$ $\qquad V_u = 665.4 \cdot kip$ $\qquad N_u = 0 \cdot kip$ $\qquad V_s = 1268.855 \cdot kip$

$T_u = 964.6 \cdot kip \cdot ft$ $\qquad p_h = 212 \cdot in$ $\qquad V_p = 0 \cdot kip$ $\qquad A_s = 42.12 \cdot in^2$

$$V'_s := \min\left(\frac{V_u}{\phi_v}, V_s\right)$$ AASHTO·LRFD·5.8.3.5 $\qquad V'_s = 739.333 \cdot kip$

$$F := \left(\frac{M'_u}{\phi_m \cdot d_v}\right) + \frac{0.5 \cdot N_u}{\phi_n} + \cot\Theta \cdot \sqrt{\left(\frac{V_u}{\phi_v} - V_p - 0.5 \cdot V'_s\right)^2 + \left(\frac{0.45 \cdot T_u \cdot p_h}{2 \cdot \phi_v \cdot A_o}\right)^2}$$ $\qquad F = 1496.141 \cdot kip$

$F_{check} := \text{if}\left(A_{ps} \cdot f_{ps} + A_s \cdot f_y \ge F, \text{"OK"}, \text{"NG"}\right)$ AASHTO·LRFD·EQ(5.8.3.6.3 − 1) $\qquad F_{check} = \text{"OK"}$

4. COLUMN/DRILLED SHAFT LOADING AND DESIGN

Superstructure to substructure force: AASHTO·LRFD·SECTION·3·LOADS·and·LOAD·COMBINATIONS

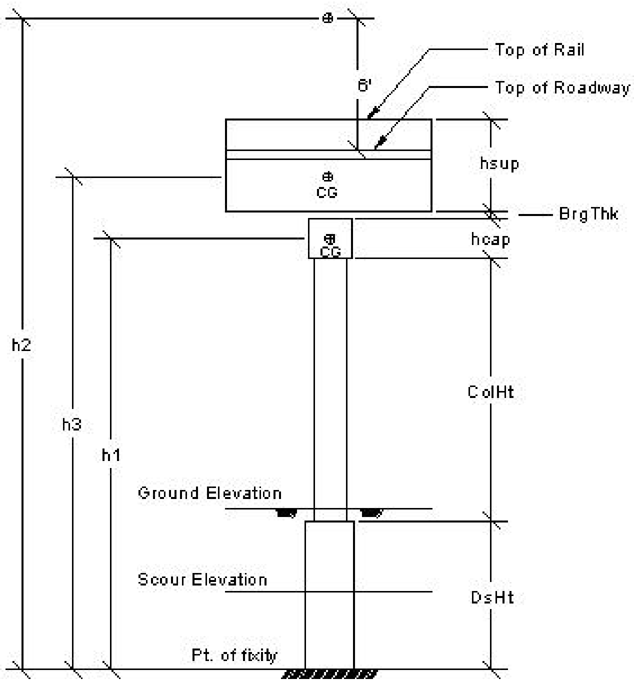
Subscript*: *X* = *Parallel to the Bent cap Length and* *Z* = *Perpendicular to the bent Cap Length

$t_h := 2 \cdot in$ (Haunch Thickness)

Beam Depth, $BmH := FBmD$

$ColH := HCol + 0 \cdot ft$ (Column height + 0 ft Column Capital)

$$TribuLength := \frac{FSpan + BSpan}{2}$$



Scour Depth:

$h_{scour} := 0 \cdot ft$

Scour to Fixity Depth:

$h_{scf} := \min(3 \cdot DsDia, 10 \cdot ft)$

Total Drilled Shaft height:

$DsH := h_{scour} + h_{scf}$

$DsH = 10 \cdot ft$

$h_o := BrgTh + BmH + t_h + SlabTh$ (Top of cap to top of slab height) $h_o = 3.683 \cdot ft$

$h_6 := h_o + 6ft$ (Top of cap to top of slab height + 6 ft) $h_6 = 9.683 \cdot ft$

$hsup := BmH + t_h + SlabTh + RailH$ (Height of Superstructure) $hsup = 6.225 \cdot ft$

$h1 := DsH + ColH + \frac{hCap}{2}$ (Height of Cap cg from Fixity of Dshaft) $h1 = 34.5 \cdot ft$

$h2 := DsH + ColH + hCap + h_6$ $h2 = 46.683 \cdot ft$

$h3 := DsH + ColH + hCap + BrgTh + \frac{hsup}{2}$ $h3 = 40.404 \cdot ft$

Tributary area for Superstructure,

$A_{super} := (hsup) \cdot (TribuLength)$ $A_{super} = 435.75 \cdot ft^2$

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):

$$FTruck := P_3 + P_2 \cdot \left[\frac{(FSpan - 14 \cdot ft)}{FSpan}\right] + P_1 \cdot \frac{(FSpan - 28ft)}{FSpan} \qquad FTruck = 62.4 \cdot kip$$

$$FLane := w_{lane} \cdot \left(\frac{FSpan}{2}\right) \qquad FLane = 22.4 \cdot \frac{kip}{lane}$$

Forward Span Live Load Reactions with Impact (FLLRxn):

$$FLLRxn := FLane + FTruck \cdot (1 + IM) \qquad FLLRxn = 105.392 \cdot \frac{kip}{lane}$$

Maximum Backward Span Design Truck (BTruck) & Lane Load Reaction (BLane):

$$BTruck := P_3 + P_2 \cdot \left[\frac{(BSpan - 14 \cdot ft)}{BSpan}\right] + P_1 \cdot \frac{(BSpan - 28ft)}{BSpan} \qquad BTruck = 62.4 \cdot kip$$

$$BLane := w_{lane} \cdot \left(\frac{BSpan}{2}\right) \qquad BLane = 22.4 \cdot \frac{kip}{lane}$$

Backward Span Live Load Reactions with Impact (BLLRxn):

$$BLLRxn := BLane + BTruck \cdot (1 + IM) \qquad BLLRxn = 105.392 \cdot \frac{kip}{lane}$$

Live Load Reactions per Beam with Impact (BmLLRxn) using Distribution Factors:

$$BmLLRxn := (LLRxn) \cdot max\left(DFS_{Fmax}, DFS_{Bmax},\right) (Max \cdot reaction \cdot when \cdot mid \cdot axle \cdot on \cdot support) \qquad BmLLRxn = 72.556 \cdot \frac{kip}{beam}$$

$$FBmLLRxn := (FLLRxn) \cdot DFS_{Fmax} \qquad (Only \cdot Forward \cdot Span \cdot is \cdot Loaded) \qquad FBmLLRxn = 58.858 \cdot \frac{kip}{beam}$$

$$BBmLLRxn := (BLLRxn) \cdot DFS_{Bmax} \qquad (Only \cdot Backward \cdot Span \cdot is \cdot Loaded) \qquad BBmLLRxn = 58.858 \cdot \frac{kip}{beam}$$

Torsion due to the eccentricity from CL of Bearing to CL of Bent when only Longer Span is loaded with HL-93 Loading

$$TorsionLL := max(FBmLLRxn, BBmLLRxn) \cdot e_{brg} \qquad TorsionLL = 63.763 \cdot \frac{kip \cdot ft}{beam}$$

***Live Load Reactions per Beam without Impact ($BmLLRxn_n$) using Distribution Factors*:**

$$BmLLRxn_n := (Lane + Truck) \cdot max\left(DFS_{Fmax}, DFS_{Bmax}\right) \qquad BmLLRxn_n = 60.761 \cdot \frac{kip}{beam}$$

$$FBmLLRxn_n := (FLane + FTruck) \cdot \left(DFS_{Fmax}\right) \qquad FBmLLRxn_n = 47.358 \cdot \frac{kip}{beam}$$

$$BBmLLRxn_n := (BLane + BTruck) \cdot \left(DFS_{Bmax}\right) \qquad BBmLLRxn_n = 47.358 \cdot \frac{kip}{beam}$$

Torsion due to the eccentricity of CL of Bearing and CL of Bent without Impact

$TorsionLL_n := \max(FBmLLRxn_n, BBmLLRxn_n) \cdot e_{brg}$ $\qquad TorsionLL_n = 51.305 \cdot \frac{kft}{beam}$

CENTRIFUGAL FORCE: CF (*AASHTO LRFD 3.6.3*) Skew Angle of Bridge, $\theta := 0 \cdot deg$

Design Speed $v := 45 \cdot mph$

Degree of Curve, $\phi_c := 0.00001 \cdot deg$ (Input 4° curve or 0.00001° for 0° curve) $\qquad (f \;\; g) := \left(\frac{4}{3} \;\; 32.2 \cdot \frac{ft}{sec^2}\right)$

Radius of Curvature, $R_c := \frac{(360 \cdot deg) \cdot 100 \cdot ft}{2 \cdot \pi \cdot \phi_c}$ $\qquad R_c = 572957795.131 \cdot ft \left(R_c = \infty \cdot ft\right)$

Centri. Force Factor, $C := f \cdot \frac{v^2}{R_c \cdot g}$ (AASHTO·LRFD·EQ·3.6.3 − 1) $\qquad C = 0$

$P_{cf} := C \cdot TruckT \cdot (NofLane) \cdot (m)$ $\qquad P_{cf} = 0 \cdot kip$

Centrifugal force **parallel** to bent (X-direction)

$CF_X := \left(\frac{P_{cf} \cdot \cos(\theta)}{NofBm}\right)$ $\qquad CF_X = 0 \cdot \frac{kip}{beam}$

Centrifugal force **normal** to bent (Z-direction)

$CF_Z := \left(\frac{P_{cf} \cdot \sin(\theta)}{NofBm}\right)$ $\qquad CF_Z = 0 \cdot \frac{kip}{beam}$

Moments at cg of the Bent Cap due to Centrifugal Force

$M_{CF\_X} := CF_Z \cdot \left(h_6 + \frac{hCap}{2}\right)$ $\qquad M_{CF\_X} = 0 \cdot \frac{kft}{beam}$

$M_{CF\_Z} := CF_X \cdot \left(h_6 + \frac{hCap}{2}\right)$ $\qquad M_{CF\_Z} = 0 \cdot \frac{kft}{beam}$

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.

$P_{br1} := 5\% \cdot (Lane + TruckT) \cdot (NofLane) \cdot (m)$ (Truck + Lane) $\qquad P_{br1} = 14.892 \cdot kip$

$P_{br2} := 5\% \cdot (Lane + 50 \cdot kip) \cdot (NofLane) \cdot (m)$ (Tandem + Lane) $\qquad P_{br2} = 12.087 \cdot kip$

$P_{br3} := 25\% \cdot (TruckT) \cdot (NofLane) \cdot (m)$ (DesignTruck) $\qquad P_{br3} = \blacksquare \cdot kip$

$P_{br} := \max(P_{br1}, P_{br2}, P_{br3})$ $\qquad P_{br} = 45.9 \cdot kip$

Braking force **parallel** to bent (X-direction)

$BR_X := \frac{P_{br} \cdot \sin(\theta)}{NofBm}$ $\qquad BR_X = 0 \cdot \frac{kip}{beam}$

Braking force **normal** to bent (Z-direction)

$$BR_Z := \frac{P_{br} \cdot \cos(\theta)}{NofBm} \qquad BR_Z = 3.825 \cdot \frac{kip}{beam}$$

Moments at cg of the Bent Cap due to Braking Force

$$M_{BR\_X} := BR_Z \cdot \left(h_6 + \frac{hCap}{2}\right) \qquad M_{BR\_X} = 46.601 \cdot \frac{kft}{beam}$$

$$M_{BR\_Z} := BR_X \cdot \left(h_6 + \frac{hCap}{2}\right) \qquad M_{BR\_Z} = 0 \cdot \frac{kft}{beam}$$

WATER LOADS: WA (*AASHTO LRFD 3.7*)

Note : To be applied only on bridge components below design high water surface.

Substructure:

$V := 0 \frac{ft}{sec}$ (Design Stream Velocity) Specific Weight, $\gamma_{water} := 62.4 \cdot pcf$

Longitudinal Stream Pressure: *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
debries 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, $C_{D\_ds} := 0.7$

$$p_T = C_D \cdot \frac{V^2}{2 \cdot g} \cdot \gamma_{water}$$ (Longitudinal stream pressure) AASHTO LRFD EQ (C3.7.3.1-1)

$$p_{T\_col} := C_{D\_col} \cdot \frac{V^2}{2 \cdot g} \cdot \gamma_{water} \qquad p_{T\_col} = 0 \cdot ksf$$

$$p_{T\_ds} := C_{D\_ds} \cdot \frac{V^2}{2 \cdot g} \cdot \gamma_{water} \qquad p_{T\_ds} = 0 \cdot ksf$$

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 flowr and longitudina axis of the pie	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, $p_L := C_L \cdot \frac{V^2}{2 \cdot g} \cdot \gamma_{water}$ $p_L = 0 \cdot ksf$

Bent Cap: Longitudinal stream pressure $C_L := 1.4$

$p_{Tcap} := C_L \cdot \frac{V^2}{2 \cdot g} \cdot \gamma_{water}$ $p_{Tcap} = 0 \cdot ksf$

WA on Columns

Water force on column **parallel** to bent (X-direction)

$WA_{col\_X} := wCol \cdot p_{T\_col}$ $WA_{col\_X} = 0 \cdot \frac{kip}{ft}$

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)

$WA_{col\_Z} := bCol \cdot p_L$ $WA_{col\_Z} = 0 \cdot \frac{kip}{ft}$

WA on Drilled Shafts

Water force on drilled shaft **parallel** to bent (X-direction)

$WA_{dshaft\_X} := DsDia \cdot p_{T\_ds}$ $WA_{dshaft\_X} = 0 \cdot \frac{kip}{ft}$

Water force on drilled shaft **normal** to bent (Z-direction)

$WA_{dshaft\_Z} := DsDia \cdot p_L$ $WA_{dshaft\_Z} = 0 \cdot \frac{kip}{ft}$

WA on Bent Cap (input as a punctual load)

Water force on bent cap **parallel** to bent (X-direction)

$WA_{cap\_X} := wCap \cdot hCap \cdot (p_{Tcap})$ (If design HW is below cap then input zero) $WA_{cap\_X} = 0 \cdot kip$

Water force on bent cap **normal** to bent (Z-direction)

$WA_{cap\_Z} := hCap \cdot p_L$ (If design HW is below cap then input zero) $WA_{cap\_Z} = 0 \cdot \frac{kip}{ft}$

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.

$p_{tsup} := 0.05\text{ksf}$ Normal to superstructure (conservative suggested value 0.050 ksf)

$p_{lsup} := 0.012\text{ksf}$ Along Superstructure (conservative suggested value 0.019 ksf)

$WS_{chk} := \text{if}\left(p_{tsup} \cdot hsup \geq 0.3 \cdot klf, \text{"OK"}, \text{"N.G."}\right)$ $WS_{chk} = \text{"OK"}$

$$Wsup_{Long} := \frac{p_{lsup} \cdot hsup \cdot TribuLength}{NofBm} \qquad Wsup_{Long} = 0.436 \cdot \frac{kip}{beam}$$

$$Wsup_{Trans} := \frac{p_{tsup} \cdot hsup \cdot TribuLength}{NofBm} \qquad Wsup_{Trans} = 1.816 \cdot \frac{kip}{beam}$$

Wind force on superstructure **parallel** to bent (X-direction)

$$WS_{super\_X} := Wsup_{Long} \cdot \sin(\theta) + Wsup_{Trans} \cdot \cos(\theta) \qquad WS_{super\_X} = 1.816 \cdot \frac{kip}{beam}$$

Wind force on superstructure **normal** to bent (Z-direction)

$$WS_{super\_Z} := Wsup_{Long} \cdot \cos(\theta) + Wsup_{Trans} \cdot \sin(\theta) \qquad WS_{super\_Z} = 0.436 \cdot \frac{kip}{beam}$$

Moments at cg of the Bent Cap due to Wind load on superstructure

$$M_{super\_X} := WS_{super\_Z} \cdot \left(\frac{hCap}{2} + BrgTh + \frac{hsup}{2}\right) \qquad M_{super\_X} = 2.573 \cdot \frac{kft}{beam}$$

$$M_{super\_Z} := WS_{super\_X} \cdot \left(\frac{hCap}{2} + BrgTh + \frac{hsup}{2}\right) \qquad M_{super\_Z} = 10.72 \cdot \frac{kft}{beam}$$

WIND ON SUBSTRUCTURE: WS (*AASHTO LRFD 3.8.1.2.3*)

Base Wind pressure, $p_{sub} := 0.04 \cdot ksf$ will be applied on exposed substructure both transverse & longitudinal direction

Wind on Columns

Wind force on columns **parallel** to bent (X-direction)

$WS_{col\_X} := \left[p_{sub} \cdot (bCol \cdot \cos(\theta) + wCol \cdot \sin(\theta))\right]$ $\qquad WS_{col\_X} = 0.16 \cdot \frac{kip}{ft}$

Apply WS loads at all columns even with zero degree attack angle.

Wind force on columns **normal** to bent (Z-direction)

$WS_{col\_Z} := \left[p_{sub} \cdot (bCol \cdot \sin(\theta) + wCol \cdot \cos(\theta))\right]$ $\qquad WS_{col\_Z} = 0.16 \cdot \frac{kip}{ft}$

Wind on Bent Cap & Ear Wall

$WS_{ew\_X} := p_{sub} \cdot hEarWall \cdot (wEarWall \cdot \sin(\theta) + wCap \cdot \cos(\theta))$ $\qquad WS_{ew\_X} = 0 \cdot kip$

$WS_{ew\_Z} := p_{sub} \cdot hEarWall \cdot (wEarWall \cdot \cos(\theta) + wCap \cdot \sin(\theta))$ $\qquad WS_{ew\_Z} = 0 \cdot kip$

Wind force on bent cap **parallel** to bent (X-direction)

$WS_{cap\_X} := \left[p_{sub} \cdot hCap \cdot (CapL \cdot \sin(\theta) + wCap \cdot \cos(\theta))\right] + WS_{ew\_X}$ (punctual load) $\qquad WS_{cap\_X} = 0.9 \cdot kip$

Wind force on bent cap **normal** to bent (Z-direction)

$WS_{cap\_Z} := \frac{\left[p_{sub} \cdot hCap \cdot (CapL \cdot \cos(\theta) + wCap \cdot \sin(\theta))\right] + WS_{ew\_Z}}{CapL}$ $\qquad WS_{cap\_Z} = 0.2 \cdot \frac{kip}{ft}$

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) $\qquad p_{WLt} := 0.1 \frac{kip}{ft}$

(suggested value 0.038 kip/ft) $\qquad p_{WLl} := 0.04 \frac{kip}{ft}$

$WL_{Par} := \frac{p_{WLl} \cdot TribuLength}{NofBm}$ $\qquad WL_{Par} = 0.233 \cdot \frac{kip}{beam}$

$WL_{Nor} := \frac{p_{WLt} \cdot TribuLength}{NofBm}$ $\qquad WL_{Nor} = 0.583 \cdot \frac{kip}{beam}$

Wind force on live load **parallel** to bent (X-direction)

$WL_X := WL_{Nor} \cdot \cos(\theta) + WL_{Par} \cdot \sin(\theta)$ $\qquad WL_X = 0.583 \cdot \frac{kip}{beam}$

Wind force on live load **normal** to bent (Z-direction)

$WL_Z := WL_{Nor} \cdot \sin(\theta) + WL_{Par} \cdot \cos(\theta)$ $\qquad WL_Z = 0.233 \cdot \frac{kip}{beam}$

Moments at cg of the Bent Cap due to Wind load on Live Load

$M_{WL\_X} := WL_Z \cdot \left(h_6 + \frac{hCap}{2}\right)$ $\qquad M_{WL\_X} = 2.843 \cdot \frac{kft}{beam}$

$M_{WL\_Z} := WL_X \cdot \left(h_6 + \frac{hCap}{2}\right)$ $\qquad M_{WL\_Z} = 7.107 \cdot \frac{kft}{beam}$

Vertical Wind Pressure: *(AASHTO LRFD 3.8.2)*

$DeckWidth := FDeckW$ Bridge deck width including parapet and sidewalk

$P_{uplift} := -(0.02ksf) \cdot DeckWidth \cdot TribuLength$ (Acts upword Y-direction) $\qquad P_{uplift} = -66.033 \cdot kip$

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

$STRENGTH\_I = 1.25 \cdot DC + 1.5 \cdot DW + 1.75 \cdot (LL + BR + CF) + 1.0 \cdot WA$

$STRENGTH\_IA = 0.9 \cdot DC + 0.65 \cdot DW + 1.75 \cdot (LL + BR + CF) + 1.0 \cdot WA$

$STRENGTH\_III = 1.25 \cdot DC + 1.5 \cdot DW + 1.4 \cdot WS + 1.0 \cdot WA + 1.4 \cdot P_{uplift}$

$STRENGTH\_IIIA = 0.9 \cdot DC + 0.65 \cdot DW + 1.4 \cdot WS + 1.0 \cdot WA + 1.4 \cdot P_{uplift}$

$STRENGTH\_V = 1.25 \cdot DC + 1.5 \cdot DW + 1.35 \cdot (LL + BR + CF) + 0.4 \cdot WS + 1.0 \cdot WA + 1.0 \cdot WL$

$STRENGTH\_VA = 0.9 \cdot DC + 0.65 \cdot DW + 1.35 \cdot (LL + BR + CF) + 0.4 \cdot WS + 1.0 \cdot WA + 1.0 \cdot WL$

$SERVICE\_I = 1.0 \cdot DC + 1.0 \cdot DW + 1.0 \cdot \left(LL_{no\_Impact} + BR + CF\right) + 0.3 \cdot WS + 1.0 \cdot WA + 1.0 \cdot WL$

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}):

$F_{FDC} := (FNofBm - 2) \cdot FSuperDC_{Int} + 2 \cdot FSuperDC_{Ext}$ $F_{FDC} = 259.607 \cdot kip$

$F_{FDW} := (FNofBm) \cdot FSuperDW$ $F_{FDW} = 38.5 \cdot kip$

Backward Span Superstructure DC (F_{BDC}) & DW (F_{BDW}):

$F_{BDC} := (BNofBm - 2) \cdot BSuperDC_{Int} + 2 \cdot BSuperDC_{Ext}$ $F_{BDC} = 259.607 \cdot kip$

$F_{BDW} := (BNofBm) \cdot BSuperDW$ $F_{BDW} = 38.5 \cdot kip$

Total Cap Dead Load Weight (TCapDC):

$CapDC := CapDC1 \cdot (CapL - LFoam) + CapDC2 \cdot LFoam$ $CapDC = 126.979 \cdot kip$

$TCapDC := CapDC + (NofBm) \cdot (BrgSeatDC) + EarWallDC$ $TCapDC = 126.979 \cdot kip$

Total DL on columns including Cap weight (F_{DC}):

$F_{DL} := (F_{FDC} + F_{FDW}) + (F_{BDC} + F_{BDW}) + TCapDC$ $F_{DL} = 723.194 \cdot kip$

Column & Drilled Shaft Self Weight:

DSahft Length, $DsHt := 0 \cdot ft$ if Rounded Col, $ColDia := 0 \cdot ft$

$ColDC := \text{if}\left[ColDia > 0ft, \frac{\pi}{4} \cdot (ColDia)^2 \cdot (HCol) \cdot \gamma_c, wCol \cdot bCol \cdot HCol \cdot \gamma_c\right]$ Column Wt, $ColDC = 52.8 \cdot kip$

$DsDC := \frac{\pi}{4} \cdot (DsDia)^2 \cdot (DsHt) \cdot \gamma_c$ Dr Shaft Wt, $DsDC = 0 \cdot kip$

Total Dead Load on Drilled Shaft (DL_on_DShaft):

$DL\_on\_DShaft := F_{DL} + (NofCol) \cdot (ColDC) + (NofDs) \cdot (DsDC)$ $DL\_on\_DShaft = 828.794 \cdot kip$

Live Load on Drilled Shaft:

$m = 0.85$ (Multiple Presence Factors for 3 Lanes) (AASHTO·LRFD·Table·3.6.1.1.2 – 1

$R_{LL} := (Lane + Truck) \cdot (NofLane) \cdot (m)$ (Total LIVE LOAD without Impact) $R_{LL} = 277.44 \cdot kip$

Total Load, DL+LL per Drilled Shaft of Intermediate Bent:

$Load\_on\_DShaft := \frac{DL\_on\_DShaft + R_{LL}}{NofDs}$ $Load\_on\_DShaft = 276.6 \cdot ton$

5. PRECAST COMPONENT DESIGN

Precast Cap Construction and Handling:

$w_1 := b \cdot h \cdot \gamma_c$ applicable for $0 \cdot ft \le L_{cap} \le 6 \cdot ft$ $\quad w_1 = 3.375 \cdot klf$ (Cap selfweight)

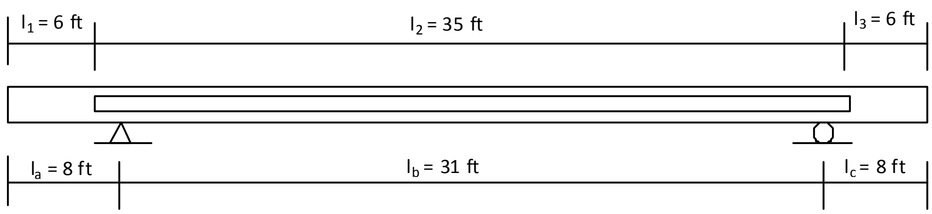
$w_2 := (b \cdot h - 2 \cdot wFoam \cdot hFoam) \cdot \gamma_c$ applicable for $6 \cdot ft \le L_{cap} \le 41 \cdot ft$ $\quad w_2 = 2.471 \cdot klf$ (Cap selfweight)

$w_3 := b \cdot h \cdot \gamma_c$ applicable for $41 \cdot ft \le L_{cap} \le 47 \cdot ft$ $\quad w_3 = 3.375 \cdot klf$ (Cap selfweight)

$l_1 := 6 \cdot ft \qquad l_2 := 35 \cdot ft \qquad l_3 := 6 \cdot ft$

$L_{cap} := l_1 + l_2 + l_3$ (Total Cap Length) $\qquad L_{cap} = 47 \cdot ft$

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.



$l_a := 8 \cdot ft \qquad l_b := 31 \cdot ft \qquad l_c := 8ft$

Construction factor:

$\lambda_{cons} := 1.25 \qquad \lambda_{cons} = 1.25$

Maximum Positive Moment (M_{maxP}) & Negative Moment (M_{maxN}):

$R_{xn} := 0.5 \cdot (w_1 \cdot l_1 + w_2 \cdot l_2 + w_3 \cdot l_3) \qquad R_{xn} = 63.49 \cdot kip$

$$M_{maxP} := R_{xn} \cdot \frac{l_b}{2} - w_1 \cdot l_1 \cdot \left(\frac{l_1}{2} + l_a - l_1 + \frac{l_b}{2}\right) - \frac{w_2}{2} \cdot \left(l_a - l_1 + \frac{l_b}{2}\right)^2 \qquad M_{maxP} = 190.617 \cdot kft$$

$$M_{maxN} := w_1 \cdot l_1 \cdot \left(\frac{l_1}{2} + l_a - l_1\right) + \frac{w_2}{2} \cdot (l_a - l_1)^2 \qquad M_{maxN} = 106.192 \cdot kft$$

Factored Maximum Positive Moment (M_{uP}) & Negative Moment (M_{uN}):

$M_{uP} := \lambda_{cons} \cdot M_{maxP}$ (Positive Moment at the middle of the cap) $\qquad M_{uP} = 238.271 \cdot kft$

$M_{uN} := \lambda_{cons} \cdot M_{maxN}$ (Negative Moment at the support point) $\qquad M_{uN} = 132.74 \cdot kft$

Maximum Positive Stress (f_{tP}) & Negative Stress (f_{tN}):

$$f_{tP} := \frac{M_{uP} \cdot (h - y_{cg2})}{I_{cap2}}$$ $f_{tP} = 95.657 \cdot psi$

$$f_{tN} := \frac{M_{uN} \cdot y_{cg2}}{I_{cap2}}$$ $f_{tN} = 52.644 \cdot psi$

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

$f'_c := 5 \cdot ksi$ (Compressive Strength of Concrete) Unit weight factor, $\lambda := 1$

$f_r := 5 \cdot \lambda \cdot \sqrt{f'_c \cdot psi}$ (PCI EQ 5.3.3.2) $f_r = 353.553 \cdot psi$

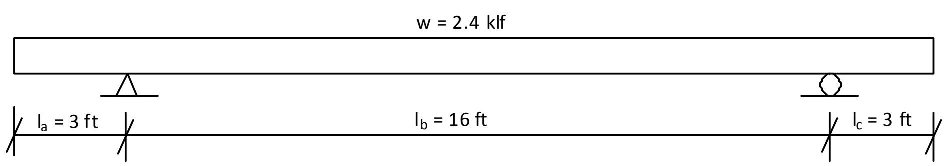
$f_{r\_check} := \text{if}\left[(f_r > f_{tP}) \cdot (f_r > f_{tN}), \text{"OK"}, \text{"N.G."}\right]$ $f_{r\_check} = \text{"OK"}$

Precast Column Construction and Handling:

$wCol = 4 \cdot ft$ (Column width) Column breadth, $bCol = 4 \cdot ft$

$w_{col} := wCol \cdot bCol \cdot \gamma_c$ (Column self weight) $w_{col} = 2.4 \cdot klf$

Due to the location of girder bolts on column, pickup 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.



$l_a := 3 \cdot ft$ $l_b := 16 \cdot ft$ $l_c := 3 \cdot ft$

Maximum Positive Moment (M_{maxP}) & Negative Moment (M_{maxN}):

$$M_{maxP} := \frac{w_{col} \cdot HCol}{2} \cdot \left(\frac{HCol}{4} - l_a\right)$$ $M_{maxP} = 66 \cdot kft$

$$M_{maxN} := \frac{w_{col} \cdot l_a^2}{2}$$ $M_{maxN} = 10.8 \cdot kft$

Factored Maximum Positive Moment (M_{uP}) & Negative Moment (M_{uN}):

$M_{uP} := \lambda_{cons} \cdot M_{maxP}$ $M_{uP} = 82.5 \cdot kft$

$M_{uN} := \lambda_{cons} \cdot M_{maxN}$ $M_{uN} = 13.5 \cdot kft$

$S_{col} := \frac{wCol \cdot bCol^2}{6}$ (Column Section Modulus) $S_{col} = 18432 \cdot in^3$

Maximum Positive Stress (f_{tP}) & Negative Stress (f_{tN}):

$f_{tP} := \frac{M_{uP}}{S_{col}}$ $f_{tP} = 53.711 \cdot psi$

$f_{tN} := \frac{M_{uN}}{S_{col}}$ $f_{tN} = 8.789 \cdot psi$

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

$f'_c := 5 \cdot ksi$ (Compressive Strength of Concrete) Unit weight factor, $\lambda := 1$

$f_r := 5 \cdot \lambda \cdot \sqrt{f'_c \cdot psi}$ (PCI EQ 5.3.3.2) $f_r = 353.553 \cdot psi$

$f_{r\_check} := \text{if}\left[\left(f_r > f_{tP}\right) \cdot \left(f_r > f_{tN}\right), \text{"OK"}, \text{"N.G."}\right]$ $f_{r\_check} = \text{"OK"}$

DEVELOPMENT LENGTH: AASHTO LRFD 5.11

$A_b := 1.56 \cdot in^2$ (Area of Bar) $d_b := 1.41 \cdot in$ (Diameter of Bar) $f'_c := 5 \cdot ksi$

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

$l_{db} := \max\left[1.25 \cdot \left(\frac{A_b}{in}\right) \cdot \frac{f_y}{\sqrt{f'_c \cdot ksi}}, 0.4 \cdot d_b \cdot \frac{f_y}{ksi}, 12 \cdot in\right]$ (AASHTO LRFD 5.11.2.1.1) $l_{db} = 52.324 \cdot in$

$l_d := \left(\lambda_{mod}\right) \cdot l_{db}$ $l_d = 4.36 \cdot ft$

Basic Compression Development: AASHTO LRFD 5.11.2.2

$l_{db} := \max\left(\frac{0.63 \cdot d_b \cdot f_y}{\sqrt{f'_c \cdot ksi}}, 0.3 \cdot d_b \cdot \frac{f_y}{ksi}, 8 \cdot in\right)$ AASHTO·LRFD·EQ·(5.11.2.2.1 − 1,2) $l_{db} = 25.38 \cdot in$

$l_d := \left(\lambda_{mod}\right) \cdot l_{db}$ $l_d = 2.115 \cdot ft$

ABC SAMPLE CALCULATION – 3b

Precast Pier Design for ABC (70’ Conventional Pier)

PRECAST PIER DESIGN FOR ABC (70' SPAN CONVENTIONAL PIER)

Nomenclature

$kip := 1000 \cdot lb$ $\quad klf := \frac{kip}{ft}$ $\quad ksi := \frac{kip}{in^2}$ $\quad psi := \frac{lb}{in^2}$ $\quad kcf := \frac{kip}{ft^3}$ $\quad ksf := \frac{kip}{ft^2}$ $\quad plf := \frac{lb}{ft}$

$pcf := \frac{lb}{ft^3}$ $\quad incr := 1$ $\quad kft := kip \cdot ft$ $\quad psf := \frac{lb}{ft^2}$ $\quad wingwall := 1$ $\quad lane := 1$ $\quad beam := 1$

SlabTh = Thickness of Slab, in
BmWt = Weight of Beam per unit length, klf
BmSpa = Spacing of beams, ft
Haunch = Haunch thickness, in
wcap = Width of Abutment/Bent Cap, ft
hcap = Depth of Abutment/Bent Cap, ft
Railwt =Weight of rail per unit length, klf
Ohang = Length of overhang from centreline of the edge beam, ft
BmH = height of beam, in
BmFlange = Top flange Width of the Beam, in
NofCol = Number of Columns per bent
DsH = length of Drilled shaft from pt. of fixity to col base, ft
DsDia= Shaft diameter, ft
ColH = ht of column, ft
V= Stream flo velocity, ft/sec
Ncomp =Normal wind load component, kip/ft
Pcomp= Parallel wind load component, kip/ft
BrWidth = Overall Bridge width, ft
CapL = Length of Bent cap, ft
h'= superstructure depth below surface of water, ft
LatLoad = Wind pressure normal to superstructure, ksf
LongLoad= wind pressure parallel to superstructure, ksf

$Steel := 1 \quad Concrete := 2$

Nomenclature

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

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_{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

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

$SlabDC_{Int}$ = 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

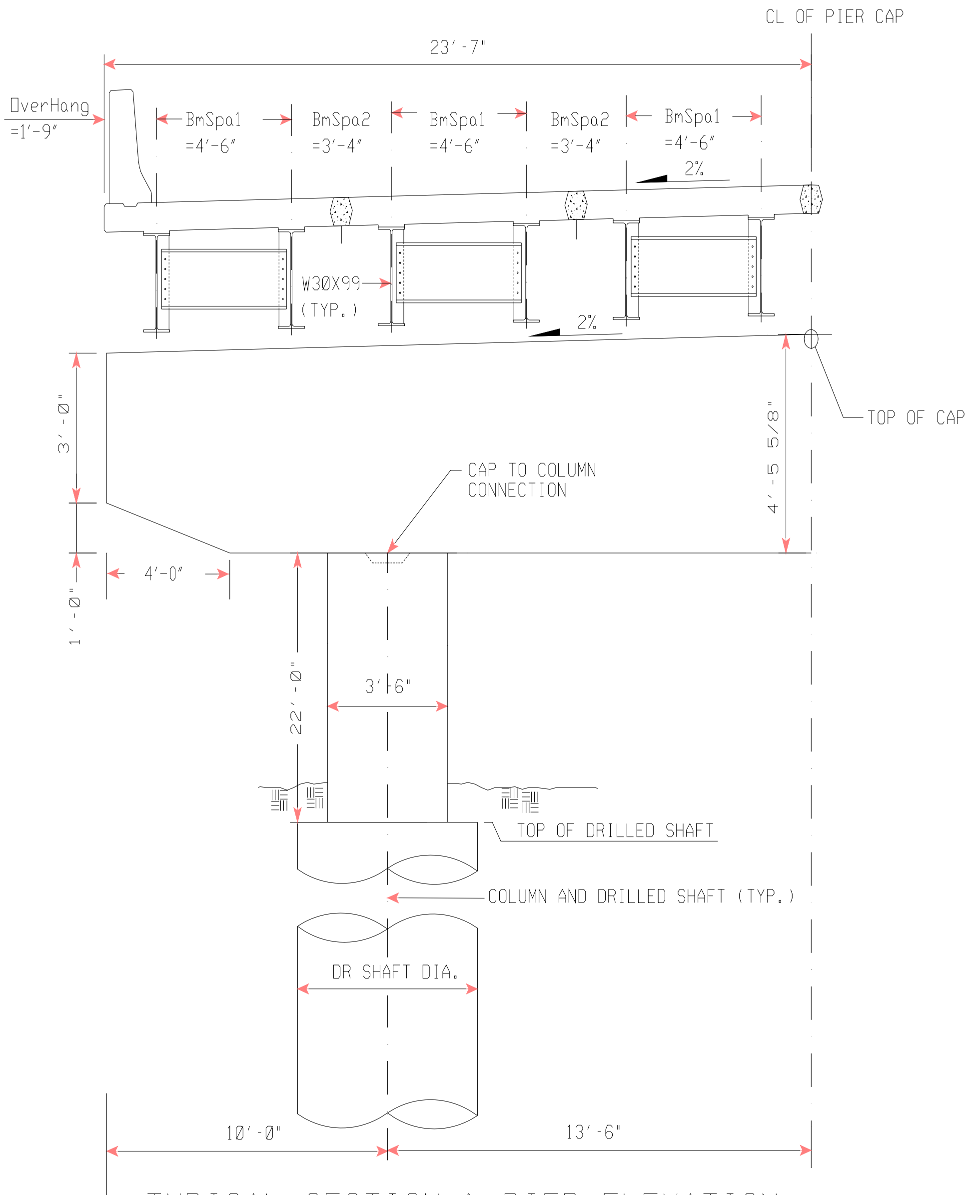
BSuperDW = Half·of·Backward·Span·Overlay·Dead·Load·Component·per·Beam

$TorsionDC_{Int}$ = 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 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:

$FNofBm := 12$ $FSpan := 70 \cdot ft$ $FDeckW := \frac{283}{6} \cdot ft$ $FBmAg := 29.1 \cdot in^2$ $FBmFlange := 10.5 \cdot in$

$FHaunch := 0 \cdot in$ $FBmD := 29.7 \cdot in$ $FBmIg := 3990 \cdot in^4$ $y_{Ft} := 14.85 \cdot in$

BACKWARD SPAN PARAMETER INPUT:

$BNofBm := 12$ $BSpan := 70 \cdot ft$ $BDeckW := \frac{283}{6} \cdot ft$ $BBmAg := 29.1 \cdot in^2$ $BBmFlange := 10.5 \cdot in$

$BHaunch := 0 \cdot in$ $BBmD := 29.7 \cdot in$ $BBmIg := 3990 \cdot in^4$ $y_{Bt} := 14.85 \cdot in$

COMMON BRIDGE PARAMETER INPUT: Bent in Question Parameters

$SlabTh := 9 \cdot in$ $Overlay := 25 \cdot psf$ $\theta := 0 \cdot deg$ $DeckOH := 1.75 \cdot ft$ $BrgTh := 3.5 \cdot in$

$RailWt := 0.43 \cdot klf$ $RailW := 19 \cdot in$ $RailH := 34.0 \cdot in$ $tBrgSeat := 0 \cdot in$ $bBrgSeat := 0 \cdot ft$

$DeckW := \frac{283}{6} \cdot ft$ $NofLane := 3$ $m := 0.85$ $w_c := 0.150 \cdot kcf$ $f'_c := 5 \cdot ksi$ (Cap)

$wCap := 4.0 \cdot ft$ $hCap := 4.0 \cdot ft$ $CapL := 47 \cdot ft$ $NofDs := 2$ $DsDia := 5 \cdot ft$

$wCol := 3.5 \cdot ft$ $bCol := 3.5 \cdot ft$ $NofCol := 2$ $HCol := 22.00 \cdot ft$ $f'_{cs} := 4 \cdot ksi$ (Slab)

$\gamma_c := 0.150 \cdot kcf$ $e_{brg} := 13 \cdot in$ $NofBm := 12$ $Sta := 0.25 \cdot \frac{ft}{incr}$ $DiapWt := 0.2 \cdot kip$

$wEarWall := 0 \cdot ft$ $hEarWall := 0 \cdot ft$ $tEarWall := 0 \cdot in$ $IM := 0.33$ $BmMat := Steel$

$E_s := 29000 \cdot ksi$ $\gamma_{st} := 490 \cdot pcf$ (steel)

Modulus of elasticity of Concrete:

$E(f_c) := 33000 \cdot (w_c)^{1.5} \cdot \sqrt{f_c \cdot ksi}$ (AASHTO LRFD EQ 5.4.2.4-1 for $K_1 = 1$)

$E_{slab} := E(f'_{cs})$ $E_{slab} = 3834.254 \cdot ksi$

$E_{cap} := E(f'_c)$ $E_{cap} = 4286.826 \cdot ksi$

Modulus of Beam or Girder: Input Beam Material, BmMat = Steel or Concrete

$E_{beam} := if(BmMat = Steel, E_s, E(f'_c))$ $E_{beam} = 29000 \cdot ksi$

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 taker 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

$FBmSpa1 := 4.5 \cdot ft$ $\qquad FBmSpa2 := \frac{10}{3} \cdot ft$

$FIntBmTriW := \frac{FBmSpa1}{2} + \frac{FBmSpa2}{2}$ $\qquad FIntBmTriW = 3.917 \cdot ft$

$FExtBmTriW := \frac{FBmSpa1}{2} + DeckOH$ $\qquad FExtBmTriW = 4 \cdot ft$

$RoadW := 0.25 \cdot (FDeckW + 3 \cdot DeckW) - 2 \cdot RailW$ $\qquad RoadW = 44 \cdot ft$

$SlabDC_{Int} := \gamma_c \cdot FIntBmTriW \cdot SlabTh \cdot \left(\frac{FSpan}{2}\right)$ $\qquad SlabDC_{Int} = 15.422 \cdot \frac{kip}{beam}$

$SlabDC_{Ext} := \gamma_c \cdot FExtBmTriW \cdot SlabTh \cdot \left(\frac{FSpan}{2}\right)$ $\qquad SlabDC_{Ext} = 15.75 \cdot \frac{kip}{beam}$

$BeamDC := \gamma_{st} \cdot FBmAg \cdot \left(\frac{FSpan}{2}\right)$ $\qquad BeamDC = 3.466 \cdot \frac{kip}{beam}$

$HaunchDC := \gamma_c \cdot FHaunch \cdot FBmFlange \cdot \left(\frac{FSpan}{2}\right)$ $\qquad HaunchDC = 0 \cdot \frac{kip}{beam}$

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 < 4°
6. Bridge cross-section is consistent with one of the x-section shown in AASHTO LRFD TABLE 4.6.2.2.1-1

$RailDC := \frac{2 \cdot RailWt}{FNofBm} \cdot \left(\frac{FSpan}{2}\right)$ $\qquad RailDC = 2.508 \cdot \frac{kip}{beam}$

$$OverlayDW := \frac{RoadW \cdot Overlay}{FNofBm} \cdot \left(\frac{FSpan}{2}\right) \qquad OverlayDW = 3.208 \cdot \frac{kip}{beam}$$

Forward Span Superstructure DC & DW per Interior and Exterior Beam:

$$FSuperDC_{Int} := RailDC + BeamDC + SlabDC_{Int} + HaunchDC + DiapWt \qquad FSuperDC_{Int} = 21.596 \cdot \frac{kip}{beam}$$

$$FSuperDC_{Ext} := RailDC + BeamDC + SlabDC_{Ext} + HaunchDC + 0.5 \cdot DiapWt \qquad FSuperDC_{Ext} = 21.824 \cdot \frac{kip}{beam}$$

$$FSuperDW := OverlayDW \qquad FSuperDW = 3.208 \cdot \frac{kip}{beam}$$

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

$$BBmSpa1 := 4.5 \cdot ft \qquad BBmSpa2 := \frac{10}{3} \cdot ft$$

$$BIntBmTriW := \frac{BBmSpa1}{2} + \frac{BBmSpa2}{2} \qquad BIntBmTriW = 3.917 \cdot ft$$

$$BExtBmTriW := \frac{BBmSpa1}{2} + DeckOH \qquad BExtBmTriW = 4 \cdot ft$$

$$RoadW := 0.25 \cdot (BDeckW + 3 \cdot DeckW) - 2 \cdot RailW \qquad RoadW = 44 \cdot ft$$

$$SlabDC_{Int} := \gamma_c \cdot BIntBmTriW \cdot SlabTh \cdot \left(\frac{BSpan}{2}\right) \qquad SlabDC_{Int} = 15.422 \cdot \frac{kip}{beam}$$

$$SlabDC_{Ext} := \gamma_c \cdot BExtBmTriW \cdot SlabTh \cdot \left(\frac{BSpan}{2}\right) \qquad SlabDC_{Ext} = 15.75 \cdot \frac{kip}{beam}$$

$$BeamDC := \gamma_{st} \cdot BBmAg \cdot \left(\frac{BSpan}{2}\right) \qquad BeamDC = 3.466 \cdot \frac{kip}{beam}$$

$$HaunchDC := \gamma_c \cdot BHaunch \cdot BBmFlange \cdot \left(\frac{BSpan}{2}\right) \qquad HaunchDC = 0 \cdot \frac{kip}{beam}$$

$$RailDC := \frac{2 \cdot RailWt}{BNofBm} \cdot \left(\frac{BSpan}{2}\right) \qquad RailDC = 2.508 \cdot \frac{kip}{beam}$$

$$OverlayDW := \frac{RoadW \cdot Overlay}{BNofBm} \cdot \left(\frac{BSpan}{2}\right) \qquad OverlayDW = 3.208 \cdot \frac{kip}{beam}$$

Total Backward Span Superstructure DC & DW per Interior and Exterior Beam:

$$BSuperDC_{Int} := RailDC + BeamDC + SlabDC_{Int} + HaunchDC + DiapWt \qquad BSuperDC_{Int} = 21.596 \cdot \frac{kip}{beam}$$

$$BSuperDC_{Ext} := RailDC + BeamDC + SlabDC_{Ext} + HaunchDC + 0.5 \cdot DiapWt \qquad BSuperDC_{Ext} = 21.824 \cdot \frac{kip}{beam}$$

$BSuperDW := OverlayDW$ $\qquad BSuperDW = 3.208 \cdot \frac{kip}{beam}$

Total Superstructure DC & DW Reactions per Beam on Bent Cap:

$SuperDC_{Int} := FSuperDC_{Int} + BSuperDC_{Int}$ $\qquad SuperDC_{Int} = 43.192 \cdot \frac{kip}{beam}$

$SuperDC_{Ext} := FSuperDC_{Ext} + BSuperDC_{Ext}$ $\qquad SuperDC_{Ext} = 43.648 \cdot \frac{kip}{beam}$

$SuperDW := FSuperDW + BSuperDW$ $\qquad SuperDW = 6.417 \cdot \frac{kip}{beam}$

$TorsionDC_{Int} := \left(\max\left(FSuperDC_{Int}, BSuperDC_{Int}\right) - \min\left(FSuperDC_{Int}, BSuperDC_{Int}\right)\right) \cdot e_{brg}$ $\qquad TorsionDC_{Int} = 0 \cdot \frac{kft}{beam}$

$TorsionDC_{Ext} := \left(\max\left(FSuperDC_{Ext}, BSuperDC_{Ext}\right) - \min\left(FSuperDC_{Ext}, BSuperDC_{Ext}\right)\right) \cdot e_{b}$ $\qquad TorsionDC_{Ext} = 0 \cdot \frac{kft}{beam}$

$TorsionDW := (\max(FSuperDW, BSuperDW) - \min(FSuperDW, BSuperDW)) \cdot e_{brg}$ $\qquad TorsionDW = 0 \cdot \frac{kft}{beam}$

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.

$CapDC := wCap \cdot hCap \cdot \gamma_c$ $\qquad CapDC = 2.4 \cdot klf$

$CapDC_{sta} := \left(wCap \cdot hCap \cdot \gamma_c\right) \cdot (Sta)$ $\qquad CapDC_{sta} = 0.6 \cdot \frac{kip}{incr}$

$EarWallDC := (wEarWall \cdot hEarWall \cdot tEarWall) \cdot \gamma_c$ $\qquad EarWallDC = 0 \cdot kip$

$BrgSeatDC := tBrgSeat \cdot bBrgSeat \cdot (wCap) \cdot \gamma_c$ $\qquad BrgSeatDC = 0 \cdot \frac{kip}{beam}$

$EI_{cap} := E_{cap} \cdot \left(\frac{wCap \cdot hCap^3}{12}\right)$ $\qquad EI_{cap} = 1.317 \times 10^7 \cdot kip \cdot ft^2$

Distribution Factor

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: $P_1 := 8 \cdot kip$ $P_2 := 32 \cdot kip$ $P_3 := 32 \cdot kip$ (AASHTO·LRFD·3.6.1.2.2) $TruckT := P_1 + P_2 + P_3$

Design Lane Load: $w_{lane} := 0.64 \cdot klf$ (AASHTO·LRFD·3.6.1.2.4)

Longer Span Length, $L_{long} := \max(FSpan, BSpan)$ Shorter Span Length, $L_{short} := \min(FSpan, BSpan)$

Lane Load Reaction:

$$Lane := w_{lane} \cdot \left(\frac{L_{long} + L_{short}}{2} \right) \qquad Lane = 44.8 \cdot \frac{kip}{lane}$$

Truck Load Reaction:

$$Truck := P_2 + P_3 \cdot \frac{(L_{long} - 14ft)}{L_{long}} + P_1 \cdot \max\left[\frac{(L_{long} - 28ft)}{L_{long}}, \frac{(L_{short} - 14ft)}{L_{short}} \right] \qquad Truck = 64 \cdot \frac{kip}{lane}$$

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)

$$LLRxn := Lane + Truck \cdot (1 + IM) \qquad LLRxn = 129.92 \cdot \frac{kip}{lane}$$

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 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:

$$P := (0.5 \cdot P_3) \cdot (1 + IM) \qquad P = 21.28 \cdot kip$$

The Design Lane Load Width Transversely in a Lane

$wlaneTransW := 10 \cdot ft$ AASHTO LRFD Article 3.6.1.2.1

The uniform load portion of the Live Load, kip/station for Cap Loading Program:

$$W := \frac{(LLRxn - 2 \cdot P) \cdot Sta}{wlaneTransW} \qquad W = 2.184 \cdot \frac{kip}{incr}$$

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:

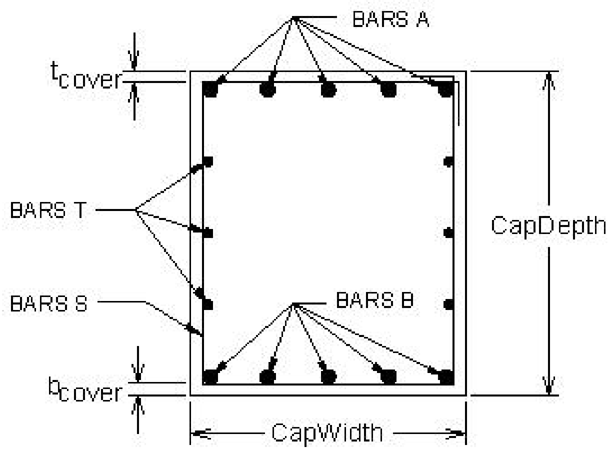
Torsional load about center line of bent cap occurs due to horizontal loads acting on the superstructure perpendicular to the bent length or along the bridge length. Braking force, Centrifugal force, WS on superstructure, and WL cause torsion on bent.

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)

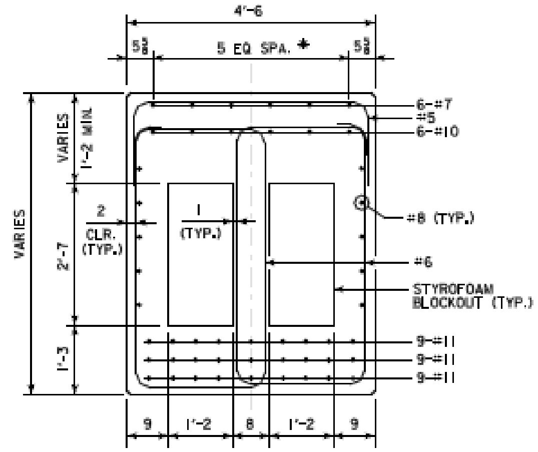
$f'_c := 5.0 \cdot ksi$ $f_y := 60 \cdot ksi$ $E_s := 29000 \cdot ksi$ $\phi_m := 0.9$ $\phi_v := 0.9$ $\phi_n := 1$

$\gamma_c := 0.150 \cdot kcf$ $b_{cover} := 2.5 \cdot in$ $t_{cover} := 2.5 \cdot in$ $h := 4.0 \cdot ft$ $b := 4.0 \cdot ft$ $E_c := E_{cap}$

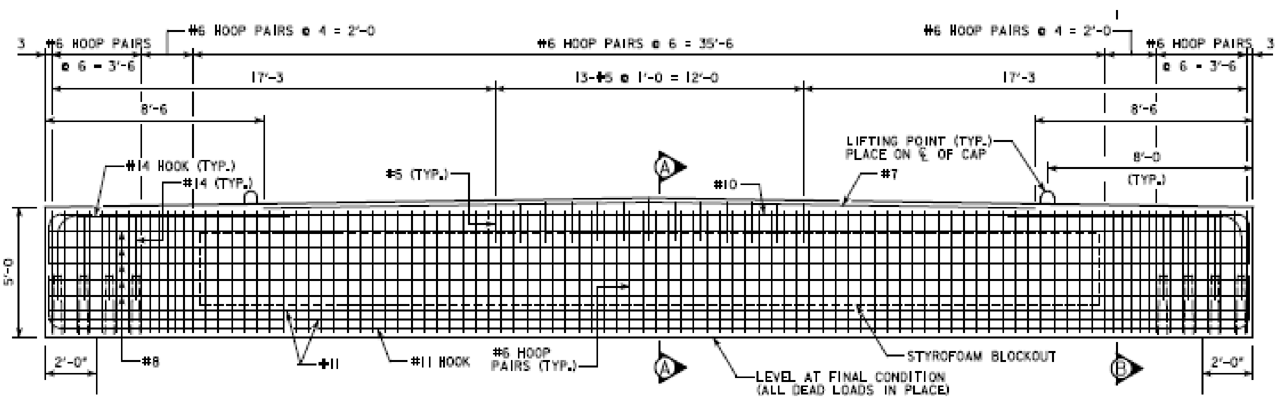
OUTPUT of BENT CAP LOADING PROGRAM: The maximum load effects from different applicable limit states:

DEAD LOAD	$M_{dlPos} := 627.2 \cdot kft$	$M_{dlNeg} := 783.4 \cdot kft$
SERVICE I	$M_{sPos} := 1462.5 \cdot kft$	$M_{sNeg} := 1297.7 \cdot kft$
STRENGTH I	$M_{uPos} := 1900.5 \cdot kft$	$M_{uNeg} := 2262.8 \cdot kft$

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, Mr, must be greater than or equal to the lesser of 1.2Mcr or 1.33 Mu. Applicable to both positive and negative moment.

Modulus of rupture

$f_r := 0.37\sqrt{f'_c \cdot ksi}$ (AASHTO LRFD EQ 5.4.2.6) $f_r = 0.827 \cdot ksi$

$S := \dfrac{b \cdot h^2}{6}$ (Section Modulus) $S = 18432 \cdot in^3$

Cracking moment

$M_{cr} := S \cdot f_r$ (AASHTO LRFD EQ 5.7.3.3.2-1) $M_{cr} = 1270.802 \cdot kip \cdot ft$

$M_{cr1} := 1.2 \cdot M_{cr}$ $M_{cr1} = 1524.963 \cdot kip \cdot ft$

$M_{cr2} := 1.33 \cdot \max(M_{uPos}, M_{uNeg})$ $M_{cr2} = 3009.524 \cdot kip \cdot ft$

$M_{cr\_min} := \min(M_{cr1}, M_{cr2})$ Therefore Mr must be <u>greater</u> than $M_{cr\_min} = 1524.963 \cdot kip \cdot ft$

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

$A_{bp} := (1.56 \ \ 1.56 \ \ 0 \ \ 0 \ \ 0 \ \ 0 \ \ 0 \ \ 0 \ \ 0 \ \ 0 \ \ 0 \ \ 0) \cdot in^2$

Input center to center vertical distance between each rebar row starting from bottom of cap

$clp := (3.5 \ \ 4 \ \ 0 \ \ 0 \ \ 0 \ \ 0 \ \ 0 \ \ 0 \ \ 0 \ \ 0 \ \ 0 \ \ 0) \cdot in$

dc Calc for Pos Moment

$ns_{Pos} = 2$ (No. of Bottom or Positive Steel Layers)

Distance from centroid of positive rebar to extreme bottom tension fiber (d_{cPos}):

$d_{cPos} := \left(Ayp_{0,0}\right) \cdot in$ $d_{cPos} = 5.5 \cdot in$

Effective depth from centroid of bottom rebar to extreme compression fiber (d_{Pos}):

$d_{Pos} := h - d_{cPos}$ $d_{Pos} = 42.5 \cdot in$

Compression Block depth under ultimate load *AASHTO LRFD 5.7.2.2*

$\beta_1 := \min\left[0.85, \max\left[0.65, 0.85 - \frac{0.05}{ksi}\left(f'_c - 4 \cdot ksi\right)\right]\right]$ $\beta_1 = 0.8$

The Amount of Bottom or Positive Steel A_s Required, $b = 48 \cdot in$

$A_{sReq} := \left(\frac{0.85 \cdot f'_c \cdot b \cdot d_{Pos}}{f_y}\right) \cdot \left(1 - \sqrt{1 - \frac{2 \cdot M_{uPos}}{0.85 \cdot \phi_m \cdot f'_c \cdot b \cdot d_{Pos}^{\ 2}}}\right)$ $A_{sReq} = 10.305 \cdot in^2$

The Amount of Positive A_s Provided,

$NofBars_{Pos} := \sum N_p$ $\qquad NofBars_{Pos} = 10$

$A_{sPos} := \left(Ayp_{0,1}\right)\cdot in^2$ $\qquad A_{sPos} = 15.6\cdot in^2$

Compression depth under ultimate load

$c_{Pos} := \dfrac{A_{sPos}\cdot f_y}{0.85\cdot f'_c\cdot \beta_1\cdot b}$ (AASHTO LRFD EQ 5.7.3.1.1-4) $\qquad c_{Pos} = 5.735\cdot in$

$a_{Pos} := \beta_1\cdot c_{Pos}$ (AASHTO LRFD 5.7.3.2.2) $\qquad a_{Pos} = 4.588\cdot in$

Nominal flexural resistance:

$M_{nPos} := A_{sPos}\cdot f_y\cdot\left(d_{Pos} - \dfrac{a_{Pos}}{2}\right)$ (AASHTO LRFD EQ 5.7.3.2.2-1) $\qquad M_{nPos} = 3136.059\cdot kip\cdot ft$

Tension controlled resistance factor for flexure

$\phi_{mPos} := \min\left[0.65 + 0.15\cdot\left(\dfrac{d_{Pos}}{c_{Pos}} - 1\right), 0.9\right]$ (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)

$M_{rPos} := \phi_{mPos}\cdot M_{nPos}$ (AASHTO LRFD EQ 5.7.3.2.1-1) $\qquad M_{rPos} = 2822.453\cdot kip\cdot ft$

$MinReinChkPos := if\left[\left(M_{rPos} \ge M_{cr\_min}\right), \text{"OK"}, \text{"NG"}\right]$ $\qquad MinReinChkPos = \text{"OK"}$

$UltimateMomChkPos := if\left[\left(M_{rPos} \ge M_{uPos}\right), \text{"OK"}, \text{"NG"}\right]$ $\qquad UltimateMomChkPos = \text{"OK"}$

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)

$N_n := (8\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0)$

Input area of rebar corresponding to above rows from top of cap, not applicable for mixed rebar in a single row

$A_{bn} := (1.56\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0)\cdot in^2$

Input center to center vertical distance between each rebar row starting from top of cap

$cln := (3.5\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0)\cdot in$

dc Calc for Neg. Moment

$ns_{Neg} = 1$ (No. of Negative or Top Steel Layers)

Distance from centroid of negative rebar to top extreme tension fiber (d_{cNeg}):

$d_{cNeg} := \left(Ayn_{0,0}\right)\cdot in$ $\qquad d_{cNeg} = 3.5\cdot in$

Effective depth from centroid of top rebar to extreme compression fiber (d_{Neg}):

$d_{Neg} := h - d_{cNeg}$ $\qquad d_{Neg} = 44.5\cdot in$

The Amount of Negative A_s Required,

$$A_{sReq} := \left(\frac{0.85\cdot f'_c\cdot b\cdot d_{Neg}}{f_y}\right)\cdot\left(1 - \sqrt{1 - \frac{2\cdot M_{uNeg}}{0.85\cdot \phi_m\cdot f'_c\cdot b\cdot d_{Neg}{}^2}}\right) \qquad A_{sReq} = 11.757\cdot in^2$$

The Amount of Negative A_s Provided,

$NofBars_{Neg} := \sum N_n$ $\qquad NofBars_{Neg} = 8$

$A_{sNeg} := \left(Ayn_{0,1}\right)\cdot in^2$ $\qquad A_{sNeg} = 12.48\cdot in^2$

Compression depth under ultimate load

$$c_{Neg} := \frac{A_{sNeg}\cdot f_y}{0.85\cdot f'_c\cdot \beta_1\cdot b} \qquad c_{Neg} = 4.588\cdot in$$

$a_{Neg} := \beta_1\cdot c_{Neg}$ $\qquad a_{Neg} = 3.671\cdot in$

Thus, nominal flexural resistance:

$$M_{nNeg} := A_{sNeg}\cdot f_y\cdot\left(d_{Neg} - \frac{a_{Neg}}{2}\right) \qquad M_{nNeg} = 2662.278\cdot kip\cdot ft$$

$M_{rNeg} := \phi_m\cdot M_{nNeg}$ (Factored flexural resistance) $\qquad M_{rNeg} = 2396.05\cdot kip\cdot ft$

$MinReinChkNeg := if\left[\left(M_{rNeg} \ge M_{cr\_min}\right), \text{"OK"}, \text{"NG"}\right]$ $\qquad MinReinChkNeg = \text{"OK"}$

$UltimateMomChkNeg := if\left[\left(M_{rNeg} \ge M_{uNeg}\right), \text{"OK"}, \text{"NG"}\right]$ $\qquad UltimateMomChkNeg = \text{"OK"}$

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) $\qquad \gamma_e = 1$

$\left(side_{cTop}\ \ side_{cBot}\right) := (4.75\ \ 4.75)\cdot in$ (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) (AASHTO LRFD 5.7.1) $n = 7$

$\rho_{Pos} := \frac{A_{sPos}}{b \cdot d_{Pos}}$ $\rho_{Pos} = 0.0076$

$k_{Pos} := \sqrt{\left(\rho_{Pos} \cdot n + 1\right)^2 - 1} - \rho_{Pos} \cdot n$ (Applicable for Solid Rectangular Section) $k_{Pos} = 0.278$

$j_{Pos} := 1 - \frac{k_{Pos}}{3}$ $j_{Pos} = 0.907$

$f_{ssPos} := \frac{M_{sPos}}{A_{sPos} \cdot j_{Pos} \cdot d_{Pos}}$ (Tensile Stress at Service Limit State) $f_{ssPos} = 29.174 \cdot ksi$

$d_{c1Pos} := clp_{0,0}$ (Distance of bottom first row rebar closest to tension face) $d_{c1Pos} = 3.5 \cdot in$

$\beta_{sPos} := 1 + \frac{d_{c1Pos}}{0.7 \cdot \left(h - d_{c1Pos}\right)}$ $\beta_{sPos} = 1.112$

$s_{maxPos} := \frac{700 \frac{kip}{in} \cdot \gamma_e}{\beta_{sPos} \cdot f_{ssPos}} - 2 \cdot d_{c1Pos}$ AASHTO LRFD EQ (5.7.3.4-1) $s_{maxPos} = 14.57 \cdot in$

$s_{ActualPos} := \frac{b - 2 \cdot side_{cBot}}{N_{p_{0,0}} - 1}$ (Equal horizontal spacing of Bottom first Rebar row closest to Tension Face) $s_{ActualPos} = 9.625 \cdot in$

Actual Max Spacing in Bottom first Layer, $s_{aPosProvided} := 7 \cdot in$

$s_{ActualPos} := \max\left(s_{aPosProvided}, s_{ActualPos}\right)$ $s_{ActualPos} = 9.625 \cdot in$

$SpacingCheckPos := \text{if}\left[\left(s_{maxPos} \geq s_{ActualPos}\right), \text{"OK"}, \text{"NG"}\right]$ $SpacingCheckPos = \text{"OK"}$

Negative Moment (Top Bars A)

$\rho_{Neg} := \frac{A_{sNeg}}{b \cdot d_{Neg}}$ $\rho_{Neg} = 0.006$

$k_{Neg} := \sqrt{\left(\rho_{Neg} \cdot n + 1\right)^2 - 1} - \rho_{Neg} \cdot n$ (Applicable for Solid Rectangular Section) $k_{Neg} = 0.248$

$j_{Neg} := 1 - \frac{k_{Neg}}{3}$ $j_{Neg} = 0.917$

$$f_{ssNeg} := \frac{M_{sNeg}}{A_{sNeg} \cdot j_{Neg} \cdot d_{Neg}}$$ $f_{ssNeg} = 30.567 \cdot ksi$

$d_{c1Neg} := cln_{0,0}$ (Distance of Top first layer rebar closest to tension face) $d_{c1Neg} = 3.5 \cdot in$

$$\beta_{sNeg} := 1 + \frac{d_{c1Neg}}{0.7 \cdot (h - d_{c1Neg})}$$ $\beta_{sNeg} = 1.112$

$$s_{maxNeg} := \frac{700 \frac{kip}{in} \cdot \gamma_e}{\beta_{sNeg} \cdot f_{ssNeg}} - 2 \cdot d_{c1Neg}$$ $s_{maxNeg} = 13.587 \cdot in$

$$s_{ActualNeg} := \frac{b - 2 \cdot side_{cTop}}{N_{n_{0,0}} - 1}$$ (Equal horizontal spacing of top first Rebar row closest to Tension Face) $s_{ActualNeg} = 5.5 \cdot in$

Actual Max Spacing Provided in Top first row closest to Tension Face, $s_{aNegProvided} := 11.125 \cdot in$

$s_{ActualNeg} := \max(s_{aNegProvided}, s_{ActualNeg})$ $s_{ActualNeg} = 11.125 \cdot in$

$SpacingCheckNeg := if[(s_{maxNeg} \ge s_{ActualNeg}), "OK", "NG"]$ 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, $A_{skBar} := 0.31 \cdot in^2$

$d_{cTop} := \sum cln$ $d_{cTop} = 3.5 \cdot in$

$d_{cBot} := \sum clp$ $d_{cBot} = 7.5 \cdot in$

Effective Depth from centroid of Extreme Tension Steel to Extreme compression Fiber (d_l):

$d_l := \max(h - clp_{0,0}, h - cln_{0,0})$ $d_l = 44.5 \cdot in$

Effective Depth from centroid of Tension Steel to Extreme compression Fiber (d_e):

$d_e := \max(d_{Pos}, d_{Neg})$ $\qquad$ $d_e = 44.5 \cdot in$

$A_s := \min(A_{sNeg}, A_{sPos})$ min. of negative and positive reinforcement $\qquad$ $A_s = 12.48 \cdot in^2$

$d_{skin} := h - (d_{cTop} + d_{cBot})$ $\qquad$ $d_{skin} = 37 \cdot in$

Skin Reinforcement Requirement: AASHTO LRFD EQ 5.7.3.4-2

$$A_{skReq} := \text{if}\left[d_l > 3ft, \min\left[0.012 \cdot \frac{in}{ft} \cdot (d_l - 30 \cdot in) \cdot d_{skin}, \frac{A_s + A_{ps}}{4}\right], 0in^2\right] \qquad A_{skReq} = 0.537 \cdot in^2$$

$$NoA_{skbar1} := R\left(\frac{A_{skReq}}{A_{skBar}}\right) \qquad NoA_{skbar1} = 2$$ per Side

Maximum Spacing of Skin Reinforcement:

$$S_{skMax} := \min\left(\frac{d_e}{6}, 12 \cdot in\right)$$ AASHTO LRFD 5.7.3.4 $\qquad$ $S_{skMax} = 7.417 \cdot in$

$$NoA_{skbar2} := \text{if}\left(d_l > 3ft, R\left(\frac{d_{skin}}{S_{skMax}} - 1\right), 1\right) \qquad NoA_{skbar2} = 4$$ per Side

$NofSideBars_{req} := \max(NoA_{skbar1}, NoA_{skbar2})$ $\qquad$ $NofSideBars_{req} = 4$

$$S_{skRequired} := \frac{d_{skin}}{1 + NofSideBars_{req}} \qquad S_{skRequired} = 7.4 \cdot in$$

$NofSideBars := 4$ (No. of Side Bars Provided)

$$S_{skProvided} := \frac{d_{skin}}{1 + NofSideBars} \qquad S_{skProvided} = 7.4 \cdot in$$

$S_{skChk} := \text{if}(S_{skProvided} < S_{skMax}, \text{"OK"}, \text{"N.G."})$ $\qquad$ $S_{skChk} = \text{"OK"}$

Therefore Use: $NofSideBars = 4$ and Size $SkBarNo = 5$

3. BENT CAP SHEAR AND TORSION DESIGN

SHEAR DESIGN OF CAP:

Effective Shear Depth, $d_v = \max\left(\begin{pmatrix} d_e - \frac{a}{2} \\ 0.9 \cdot d_e \\ 0.72 \cdot h \end{pmatrix}\right)$ (*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,	$A_s := A_{sNeg}$	$A_s = 12.48 \cdot in^2$
Nominal Flexure,	$M_n := M_{nNeg}$	$M_n = 2662.278 \cdot kft$
Stress block Depth,	$a := a_{Neg}$	$a = 3.671 \cdot in$
Effective Depth,	$d_e := d_{Neg}$	$d_e = 44.5 \cdot in$
Effective web Width at critical Location,	$b_v := b$	$b_v = 4 \cdot ft$
Input initial θ,	$\theta := 35 \cdot deg$	$\cot\theta := \cot(\theta)$
Shear Resistance Factor,	$\phi_v := 0.9$	

Cap Depth & Width, $h = 48 \cdot in$ $b = 48 \cdot in$

Moment Arm, $\left(d_e - \frac{a}{2}\right) = 42.665 \cdot in$ $0.9 \cdot d_e = 40.05 \cdot in$ $0.72 \cdot h = 34.56 \cdot in$

Effective Shear Depth at Critical Location, $d_v := \max\left(\left(\begin{array}{c} d_e - \frac{a}{2} \\ 0.9 \cdot d_e \\ 0.72 \cdot h \end{array}\right)\right)$ (*AASHTO LRFD 5.8.2.9*) $d_v = 42.665 \cdot in$

$h_h := h - t_{cover} - b_{cover}$ (Height of shear reinforcement) $h_h = 43 \cdot in$

$b_h := b - 2 \cdot b_{cover}$ (Width of shear reinforcement) $b_h = 43 \cdot in$

$p_h := 2(h_h + b_h)$ (Perimeter of shear reinforcement) $p_h = 172 \cdot in$

$A_{oh} := (h_h) \cdot (b_h)$ (Area enclosed by the shear reinforcement) $A_{oh} = 1849 \cdot in^2$

$A_o := 0.85 \cdot A_{oh}$ (AASHTO LRFD C5.8.2.1) $A_o = 1571.65 \cdot in^2$

$A_c := 0.5 \cdot b \cdot h$ (AASHTO·LRFD·FIGURE·5.8.3.4.2 − 1) $A_c = 1152 \cdot in^2$

Yield strength & Modulus of Elasticity of Steel Reinforcement:

$(f_y \;\; E_s) := (60 \;\; 29000) \cdot ksi$ (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)

$(M_u \;\; T_u) := (1398.6 \;\; 570.2) \cdot kft$ $(V_u \;\; N_u) := (463.4 \;\; 0) \cdot kip$

$M'_u := \max(M_u, |V_u - V_p| \cdot d_v)$ AASHTO LRFD B5.2 $M'_u = 1647.569 \cdot kip \cdot ft$

$V'_u := \sqrt{V_u^2 + \left(\frac{0.9 \cdot p_h \cdot T_u}{2 \cdot A_o}\right)^2}$ (Equivalent shear) AASHTO LRFD EQ (5.8.2.1-6) for solid section $V'_u = 572.966 \cdot kip$

Assuming·atleast·minimum·transverse·reinforcement·is·provided (Always provide min. transverse reinf.)

$$\varepsilon_x = \frac{\left(\frac{M'_u}{d_v}\right) + 0.5 \cdot N_u + 0.5 \cdot (V'_u - V_p) \cdot \cot\theta - A_{ps} \cdot f_{po}}{2 \cdot (E_s \cdot A_s + E_p \cdot A_{ps})}$$

(Strain from Appendix B5) AASHTO LRFD EQ (B5.2-1)

$v_u := \dfrac{(V_u - \phi_v \cdot V_p)}{\phi_v \cdot b_v \cdot d_v}$ (Shear Stress) AASHTO LRFD EQ (5.8.2.9-1) $v_u = 0.251 \cdot ksi$

$r := \max\left(0.075, \dfrac{v_u}{f'_c}\right)$ (Shear stress ratio) $r = 0.075$

Determining Beta & Theta

After Interpolating the value of $(\Theta \ \ B)$

$\Theta = 36.4 \cdot deg$ $B = 2.23$

Nominal Shear Resistance by Concrete,

$V_c := 0.0316 \cdot B \cdot \sqrt{f'_c \cdot ksi} \cdot b_v \cdot d_v$ AASHTO LRFD EQ (5.8.3.3-3) $V_c = 322.7 \cdot kip$

$V_u = 463.4 \cdot kip$ $0.5 \cdot \phi_v \cdot (V_c + V_p) = 145.211 \cdot kip$

REGION REQUIRING TRANSVERSE REINFORCEMENT: AASHTO LRFD 5.8.2.4

$V_u > 0.5 \cdot \phi_v \cdot (V_c + V_p)$ AASHTO LRFD EQ (5.8.2.4-1)

check := if$[V_u > 0.5 \cdot \phi_v \cdot (V_c + V_p)$, "Provide Shear Reinf" , "No reinf."$]$ check = "Provide Shear Reinf"

$V_n = \min\left(\begin{pmatrix} V_c + V_s + V_p \\ 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p \end{pmatrix}\right)$ (Nominal Shear Resistance) AASHTO·LRFD·EQ·(5.8.3.3 − 1, 2)

$V_s = \dfrac{A_v \cdot f_y \cdot d_v \cdot (\cot\theta + \cot\alpha) \cdot \sin\alpha}{S}$ (Shear Resistance of Steel) AASHTO·LRFD·EQ·(5.8.3.3 − 4)

$V_s = \dfrac{A_v \cdot f_y \cdot d_v \cdot \cot\theta}{S}$ (Shear·Resistance·of·Steel·when, $\alpha = 90 \cdot deg$) AASHTO LRFD EQ (C5.8.3.3-1)

$S_v := 9 \cdot in$ (Input Stirrup Spacing) $V_p = 0 \cdot kip$ $(V_u \ \ V_c) = (463.4 \ \ 322.691) \cdot kip$

$f_y = 60 \cdot ksi$ $d_v = 42.665 \cdot in$ $\Theta = 36.4 \cdot deg$

$A_{v\_req} := \left(\dfrac{V_u}{\phi_v} - V_c - V_p\right) \cdot \left(\dfrac{S_v}{f_y \cdot d_v \cdot \cot\Theta}\right)$ (Derive from AASHTO LRFD EQ 5.8.3.3-1, C5.8.3.3-1 and ϕVn >= Vu) $A_{v\_req} = 0.4982 \cdot in^2$

Torsional Steel:

$A_t := \dfrac{T_u}{2 \cdot \phi_v \cdot A_o \cdot f_y \cdot \cot\Theta} \cdot S_v$ (Derive from AASHTO LRFD EQ 5.8.3.6.2-1 and ϕTn >= Tu) $A_t = 0.267 \cdot in^2$

$A_{vt\_req} := A_{v\_req} + 2 \cdot A_t$ (Shear + Torsion) $A_{vt\_req} = 1.033 \cdot in^2$

$A_{vt} := 4 \cdot \left(0.44 \cdot in^2\right)$ (Use 2 #6 double leg Stirrup at S_v c/c,) *Provided*, $A_{vt} = 1.76 \cdot in^2$

$A_{vt\_check} := \text{if}\left(A_{vt} > A_{vt\_req}, \text{"OK"}, \text{"NG"}\right)$ $A_{vt\_check} = \text{"OK"}$

Maximum Spacing Check: $\text{AASHTO} \cdot \text{LRFD} \cdot \text{Article} \cdot 5.8.2.7$

$V_u = 463.4 \cdot kip$ $0.125 \cdot f'_c \cdot b_v \cdot d_v = 1279.94 \cdot kip$

$S_{vmax} := \text{if}\left(V_u < 0.125 \cdot f'_c \cdot b_v \cdot d_v, \min\left(0.8 \cdot d_v, 24 \cdot in\right), \min\left(0.4 \cdot d_v, 12 \cdot in\right)\right)$ $S_{vmax} = 24 \cdot in$

$S_{vmax\_check} := \text{if}\left(S_v < S_{vmax}, \text{"OK"}, \text{"use lower spacing"}\right)$ $S_{vmax\_check} = \text{"OK"}$

$A_v := A_{vt} - A_t$ (Shear Reinf. without Torsion Reinf.) $A_v = 1.493 \cdot in^2$

$V_s := \dfrac{A_v \cdot f_y \cdot d_v \cdot \cot\Theta}{S_v}$ $V_s = 575.804 \cdot kip$

Minimum Transverse Reinforce Check: $\text{AASHTO} \cdot \text{LRFD} \cdot \text{Article} \cdot 5.8.2.5$ $b_v = 48 \cdot in$

$A_{vmin} := 0.0316 \cdot \sqrt{f'_c \cdot ksi} \cdot \dfrac{b_v \cdot S_v}{f_y}$ $\text{AASHTO} \cdot \text{LRFD} \cdot \text{EQ} \cdot (5.8.2.5 - 1)$ $A_{vmin} = 0.509 \cdot in^2$

$A_{vmin\_check} := \text{if}\left(A_{vt} > A_{vmin}, \text{"OK"}, \text{"NG"}\right)$ $A_{vmin\_check} = \text{"OK"}$

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*

$0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p = 2559.882 \cdot kip$ $V_c + V_s + V_p = 898.495 \cdot kip$

$V_n := \min\left(\begin{pmatrix} V_c + V_s + V_p \\ 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p \end{pmatrix}\right)$ $\text{AASHTO} \cdot \text{LRFD} \cdot \text{EQ} \cdot (5.8.3.3 - 1, 2)$ $V_n = 898.495 \cdot kip$

$\phi_v \cdot V_n = 808.645 \cdot kip$ $V_u = 463.4 \cdot kip$

$\phi V_{n\_check} := \text{if}\left(\phi_v \cdot V_n > V_u, \text{"OK"}, \text{"NG"}\right)$ $\phi V_{n\_check} = \text{"OK"}$

Torsional Resistance,

$$T_n := \frac{2 \cdot A_o \cdot (0.5 \cdot A_{vt}) \cdot f_y \cdot \cot\Theta}{S_v}$$ AASHTO·LRFD·EQ·(5.8.3.6.2 − 1) $\phi_v \cdot T_n = 1875.9 \cdot kip \cdot ft$

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

$$A_{ps} \cdot f_{ps} + A_s \cdot f_y \geq \left(\frac{M'_u}{\phi_m \cdot d_v}\right) + \frac{0.5 \cdot N_u}{\phi_n} + \cot\Theta \cdot \sqrt{\left(\frac{V_u}{\phi_v} - V_p - 0.5 \cdot V'_s\right)^2 + \left(\frac{0.45 \cdot p_h \cdot T_u}{2 \cdot \phi_v \cdot A_o}\right)^2}$$

$(\phi_m \ \phi_v \ \phi_n) := (0.9 \ \ 0.9 \ \ 1)$ $A_s \cdot f_y + A_{ps} \cdot f_{ps} = 748.8 \cdot kip$

$M'_u = 1647.569 \cdot kip \cdot ft$ $V_u = 463.4 \cdot kip$ $N_u = 0 \cdot kip$ $V_s = 575.804 \cdot kip$

$T_u = 570.2 \cdot kip \cdot ft$ $p_h = 172 \cdot in$ $V_p = 0 \cdot kip$ $A_s = 12.48 \cdot in^2$

$$V'_s := \min\left(\frac{V_u}{\phi_v}, V_s\right)$$ AASHTO·LRFD·5.8.3.5 $V'_s = 514.889 \cdot kip$

$$F := \left(\frac{M'_u}{\phi_m \cdot d_v}\right) + \frac{0.5 \cdot N_u}{\phi_n} + \cot\Theta \cdot \sqrt{\left(\frac{V_u}{\phi_v} - V_p - 0.5 \cdot V'_s\right)^2 + \left(\frac{0.45 \cdot T_u \cdot p_h}{2 \cdot \phi_v \cdot A_o}\right)^2}$$ $F = 946.64 \cdot kip$

$F_{check} := \text{if}\left(A_{ps} \cdot f_{ps} + A_s \cdot f_y \geq F, \text{"OK"}, \text{"N.G."}\right)$ AASHTO·LRFD·EQ(5.8.3.6.3 − 1) $F_{check} = \text{"N.G."}$

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

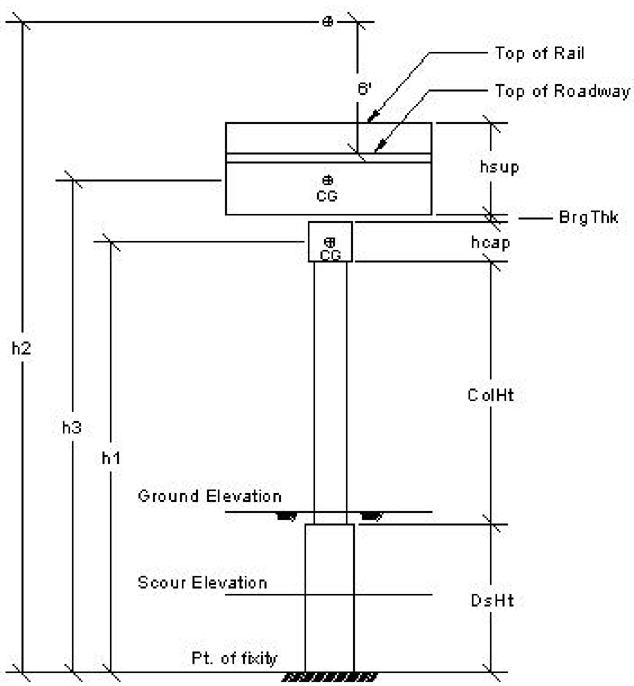
Subscript*: *X* = *Parallel to the Bent cap Length and* *Z* = *Perpendicular to the bent Cap Length

$t_h := 2.5 \cdot in$ (Haunch Thickness)

Beam Depth, $BmH := FBmD$

$ColH := HCol + 0 \cdot ft$ (Column height + 0 ft Column Capital)

$TribuLength := \dfrac{FSpan + BSpan}{2}$



Scour Depth:

$h_{scour} := 0 \cdot ft$

Scour to Fixity Depth:

$h_{scf} := \min(3 \cdot DsDia, 10 \cdot ft)$

Total Drilled Shaft height:

$DsH := h_{scour} + h_{scf}$

$DsH = 10 \cdot ft$

$h_o := BrgTh + BmH + t_h + SlabTh$ (Top of cap to top of slab height) $h_o = 3.725 \cdot ft$

$h_6 := h_o + 6ft$ (Top of cap to top of slab height + 6 ft) $h_6 = 9.725 \cdot ft$

$hsup := BmH + t_h + SlabTh + RailH$ (Height of Superstructure) $hsup = 6.267 \cdot ft$

$h1 := DsH + ColH + \frac{hCap}{2}$ (Height of Cap cg from Fixity of Dshaft) $h1 = 34 \cdot ft$

$h2 := DsH + ColH + hCap + h_6$ $h2 = 45.725 \cdot ft$

$h3 := DsH + ColH + hCap + BrgTh + \frac{hsup}{2}$ $h3 = 39.425 \cdot ft$

Tributary area for Superstructure,

$A_{super} := (hsup) \cdot (TribuLength)$ $A_{super} = 438.667 \cdot ft^2$

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):

$FTruck := P_3 + P_2 \cdot \left[\frac{(FSpan - 14 \cdot ft)}{FSpan}\right] + P_1 \cdot \frac{(FSpan - 28ft)}{FSpan}$ $FTruck = 62.4 \cdot kip$

$FLane := w_{lane} \cdot \left(\frac{FSpan}{2}\right)$ $FLane = 22.4 \cdot \frac{kip}{lane}$

Forward Span Live Load Reactions with Impact (FLLRxn):

$FLLRxn := FLane + FTruck \cdot (1 + IM)$ $FLLRxn = 105.392 \cdot \frac{kip}{lane}$

Maximum Backward Span Design Truck (BTruck) & Lane Load Reaction (BLane):

$BTruck := P_3 + P_2 \cdot \left[\frac{(BSpan - 14 \cdot ft)}{BSpan}\right] + P_1 \cdot \frac{(BSpan - 28ft)}{BSpan}$ $BTruck = 62.4 \cdot kip$

$BLane := w_{lane} \cdot \left(\frac{BSpan}{2}\right)$ $BLane = 22.4 \cdot \frac{kip}{lane}$

Backward Span Live Load Reactions with Impact (BLLRxn):

$BLLRxn := BLane + BTruck \cdot (1 + IM)$ $BLLRxn = 105.392 \cdot \frac{kip}{lane}$

Live Load Reactions per Beam with Impact (BmLLRxn) using Distribution Factors:

$BmLLRxn := (LLRxn) \cdot \max\left(DFS_{Fmax}, DFS_{Bmax}\right)$ (Max·reaction·when·mid·axle·on·support) $BmLLRxn = 72.556 \cdot \frac{kip}{beam}$

$FBmLLRxn := (FLLRxn) \cdot DFS_{Fmax}$ (Only·Forward·Span·is·Loaded) $FBmLLRxn = 58.858 \cdot \frac{kip}{beam}$

$BBmLLRxn := (BLLRxn) \cdot DFS_{Bmax}$ (Only·Backward·Span·is·Loaded) $BBmLLRxn = 58.858 \cdot \frac{kip}{beam}$

Torsion due to the eccentricity from CL of Bearing to CL of Bent when only Longer Span is loaded with HL-93 Loading

$TorsionLL := max(FBmLLRxn, BBmLLRxn) \cdot e_{brg}$ $\quad$ $TorsionLL = 63.763 \cdot \frac{kip \cdot ft}{beam}$

Live Load Reactions per Beam without Impact ($BmLLRxn_n$) using Distribution Factors:

$BmLLRxn_n := (Lane + Truck) \cdot max\left(DFS_{Fmax}, DFS_{Bmax}\right)$ $\quad$ $BmLLRxn_n = 60.761 \cdot \frac{kip}{beam}$

$FBmLLRxn_n := (FLane + FTruck) \cdot \left(DFS_{Fmax}\right)$ $\quad$ $FBmLLRxn_n = 47.358 \cdot \frac{kip}{beam}$

$BBmLLRxn_n := (BLane + BTruck) \cdot \left(DFS_{Bmax}\right)$ $\quad$ $BBmLLRxn_n = 47.358 \cdot \frac{kip}{beam}$

Torsion due to the eccentricity of CL of Bearing and CL of Bent without Impact

$TorsionLL_n := max\left(FBmLLRxn_n, BBmLLRxn_n\right) \cdot e_{brg}$ $\quad$ $TorsionLL_n = 51.305 \cdot \frac{kft}{beam}$

CENTRIFUGAL FORCE: CF (*AASHTO LRFD 3.6.3*) Skew Angle of Bridge, $\theta := 0 \cdot deg$

Design Speed $v := 45 \cdot mph$

Degree of Curve, $\phi_c := 0.00001 \cdot deg$ (Input 4º curve or 0.00001º for 0º curve) $\quad$ $(f \;\; g) := \left(\frac{4}{3} \;\; 32.2 \cdot \frac{ft}{sec^2}\right)$

Radius of Curvature, $R_c := \frac{(360 \cdot deg) \cdot 100 \cdot ft}{2 \cdot \pi \cdot \phi_c}$ $\quad$ $R_c = 572957795.131 \cdot ft \left(R_c = \infty\right)$

Centri. Force Factor, $C := f \cdot \frac{v^2}{R_c \cdot g}$ $\quad$ $(AASHTO \cdot LRFD \cdot EQ \cdot 3.6.3 - 1)$ $\quad$ $C = 0$

$P_{cf} := C \cdot TruckT \cdot (NofLane) \cdot (m)$ $\quad$ $P_{cf} = 0 \cdot kip$

Centrifugal force **parallel** to bent (X-direction)

$CF_X := \left(\frac{P_{cf} \cdot \cos(\theta)}{NofBm}\right)$ $\quad$ $CF_X = 0 \cdot \frac{kip}{beam}$

Centrifugal force **normal** to bent (Z-direction)

$CF_Z := \left(\frac{P_{cf} \cdot \sin(\theta)}{NofBm}\right)$ $\quad$ $CF_Z = 0 \cdot \frac{kip}{beam}$

Moments at cg of the Bent Cap due to Centrifugal Force

$M_{CF\_X} := CF_Z \cdot \left(h_6 + \frac{hCap}{2}\right)$ $\quad$ $M_{CF\_X} = 0 \cdot \frac{kft}{beam}$

$M_{CF\_Z} := CF_X \cdot \left(h_6 + \frac{hCap}{2}\right)$ $\quad$ $M_{CF\_Z} = 0 \cdot \frac{kft}{beam}$

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.

$P_{br1} := 5\% \cdot (Lane + TruckT) \cdot (NofLane) \cdot (m)$ (Truck + Lane) $\quad$ $P_{br1} = 14.892 \cdot kip$

$P_{br2} := 5\%\cdot(\text{Lane} + 50\cdot\text{kip})\cdot(\text{NofLane})\cdot(m)$ (Tandem + Lane) $P_{br2} = 12.087\cdot\text{kip}$

$P_{br3} := 25\%\cdot(\text{TruckT})\cdot(\text{NofLane})\cdot(m)$ (DesignTruck) $P_{br3} = 45.9\cdot\text{kip}$

$P_{br} := \max(P_{br1}, P_{br2}, P_{br3})$ $P_{br} = 45.9\cdot\text{kip}$

Braking force **parallel** to bent (X-direction)

$$BR_X := \frac{P_{br}\cdot\sin(\theta)}{\text{NofBm}} \qquad BR_X = 0\cdot\frac{\text{kip}}{\text{beam}}$$

Braking force **normal** to bent
(Z-direction)

$$BR_Z := \frac{P_{br}\cdot\cos(\theta)}{\text{NofBm}} \qquad BR_Z = 3.825\cdot\frac{\text{kip}}{\text{beam}}$$

Moments at cg of the Bent Cap due to Braking Force

$$M_{BR\_X} := BR_Z\cdot\left(h_6 + \frac{hCap}{2}\right) \qquad M_{BR\_X} = 44.848\cdot\frac{\text{kft}}{\text{beam}}$$

$$M_{BR\_Z} := BR_X\cdot\left(h_6 + \frac{hCap}{2}\right) \qquad M_{BR\_Z} = 0\cdot\frac{\text{kft}}{\text{beam}}$$

WATER LOADS: WA (*AASHTO LRFD 3.7*)

Note : To be applied only on bridge components below design high water surface.

Substructure:

$V := 0\frac{\text{ft}}{\text{sec}}$ (Design Stream Velocity) Specific Weight, $\gamma_{water} := 62.4\cdot\text{pcf}$

Longitudinal Stream Pressure: *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
debries 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, $C_{D\_ds} := 0.7$

$$p_T = C_D\cdot\frac{V^2}{2\cdot g}\cdot\gamma_{water}$$ (Longitudinal stream pressure) AASHTO LRFD EQ (C3.7.3.1-1)

$$p_{T\_col} := C_{D\_col}\cdot\frac{V^2}{2\cdot g}\cdot\gamma_{water} \qquad p_{T\_col} = 0\cdot\text{ksf}$$

$$p_{T\_ds} := C_{D\_ds} \cdot \frac{V^2}{2 \cdot g} \cdot \gamma_{water} \qquad p_{T\_ds} = 0 \cdot ksf$$

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 flowr and longitudina axis of the pie	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, $p_L := C_L \cdot \frac{V^2}{2 \cdot g} \cdot \gamma_{water}$ $\qquad p_L = 0 \cdot ksf$

Bent Cap: Longitudinal stream pressure $\qquad C_L := 1.4$

$$p_{Tcap} := C_L \cdot \frac{V^2}{2 \cdot g} \cdot \gamma_{water} \qquad p_{Tcap} = 0 \cdot ksf$$

WA on Columns

Water force on column **parallel** to bent (X-direction)

$$WA_{col\_X} := wCol \cdot p_{T\_col} \qquad WA_{col\_X} = 0 \cdot \frac{kip}{ft}$$

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)

$$WA_{col\_Z} := bCol \cdot p_L \qquad WA_{col\_Z} = 0 \cdot \frac{kip}{ft}$$

WA on Drilled Shafts

Water force on drilled shaft **parallel** to bent (X-direction)

$$WA_{dshaft\_X} := DsDia \cdot p_{T\_ds} \qquad WA_{dshaft\_X} = 0 \cdot \frac{kip}{ft}$$

Water force on drilled shaft **normal** to bent (Z-direction)

$$WA_{dshaft\_Z} := DsDia \cdot p_L \qquad WA_{dshaft\_Z} = 0 \cdot \frac{kip}{ft}$$

WA on Bent Cap (input as a punctual load)

Water force on bent cap **parallel** to bent (X-direction)

$WA_{cap\_X} := wCap \cdot hCap \cdot (p_{Tcap})$ (If design HW is below cap then input zero) $\qquad WA_{cap\_X} = 0 \cdot kip$

Water force on bent cap **normal** to bent (Z-direction)

$WA_{cap\_Z} := hCap \cdot p_L$ (If design HW is below cap then input zero) $WA_{cap\_Z} = 0 \cdot \frac{kip}{ft}$

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)

$WS_{chk} := if\left(p_{tsup} \cdot hsup \geq 0.3 \cdot klf, "OK", "N.G."\right)$ $WS_{chk} = "OK"$

$Wsup_{Long} := \frac{p_{lsup} \cdot hsup \cdot TribuLength}{NofBm}$ $Wsup_{Long} = 0.439 \cdot \frac{kip}{beam}$

$Wsup_{Trans} := \frac{p_{tsup} \cdot hsup \cdot TribuLength}{NofBm}$ $Wsup_{Trans} = 1.828 \cdot \frac{kip}{beam}$

Wind force on superstructure **parallel** to bent (X-direction)

$WS_{super\_X} := Wsup_{Long} \cdot \sin(\theta) + Wsup_{Trans} \cdot \cos(\theta)$ $WS_{super\_X} = 1.828 \cdot \frac{kip}{beam}$

Wind force on superstructure **normal** to bent (Z-direction)

$WS_{super\_Z} := Wsup_{Long} \cdot \cos(\theta) + Wsup_{Trans} \cdot \sin(\theta)$ $WS_{super\_Z} = 0.439 \cdot \frac{kip}{beam}$

Moments at cg of the Bent Cap due to Wind load on superstructure

$M_{super\_X} := WS_{super\_Z} \cdot \left(\frac{hCap}{2} + BrgTh + \frac{hsup}{2}\right)$ $M_{super\_X} = 2.38 \cdot \frac{kft}{beam}$

$M_{super\_Z} := WS_{super\_X} \cdot \left(\frac{hCap}{2} + BrgTh + \frac{hsup}{2} \right)$ $\qquad$ $M_{super\_Z} = 9.916 \cdot \frac{kft}{beam}$

WIND ON SUBSTRUCTURE: WS (*AASHTO LRFD 3.8.1.2.3*)

Base Wind pressure, $p_{sub} := 0.04 \cdot ksf$ will be applied on exposed substructure both transverse & longitudinal direction

Wind on Columns

Wind force on columns **parallel** to bent (X-direction)

$WS_{col\_X} := \left[p_{sub} \cdot (bCol \cdot \cos(\theta) + wCol \cdot \sin(\theta)) \right]$ $\qquad$ $WS_{col\_X} = 0.14 \cdot \frac{kip}{ft}$

Apply WS loads at all columns even with zero degree attack angle.

Wind force on columns **normal** to bent (Z-direction)

$WS_{col\_Z} := \left[p_{sub} \cdot (bCol \cdot \sin(\theta) + wCol \cdot \cos(\theta)) \right]$ $\qquad$ $WS_{col\_Z} = 0.14 \cdot \frac{kip}{ft}$

Wind on Bent Cap & Ear Wall

$WS_{ew\_X} := p_{sub} \cdot hEarWall \cdot (wEarWall \cdot \sin(\theta) + wCap \cdot \cos(\theta))$ $\qquad$ $WS_{ew\_X} = 0 \cdot kip$

$WS_{ew\_Z} := p_{sub} \cdot hEarWall \cdot (wEarWall \cdot \cos(\theta) + wCap \cdot \sin(\theta))$ $\qquad$ $WS_{ew\_Z} = 0 \cdot kip$

Wind force on bent cap **parallel** to bent (X-direction)

$WS_{cap\_X} := \left[p_{sub} \cdot hCap \cdot (CapL \cdot \sin(\theta) + wCap \cdot \cos(\theta)) \right] + WS_{ew\_X}$ (punctual load) $\qquad$ $WS_{cap\_X} = 0.64 \cdot kip$

Wind force on bent cap **normal** to bent (Z-direction)

$WS_{cap\_Z} := \frac{\left[p_{sub} \cdot hCap \cdot (CapL \cdot \cos(\theta) + wCap \cdot \sin(\theta)) \right] + WS_{ew\_Z}}{CapL}$ $\qquad$ $WS_{cap\_Z} = 0.16 \cdot \frac{kip}{ft}$

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) $\qquad$ $p_{WLt} := 0.1 \frac{kip}{ft}$

(suggested value 0.038 kip/ft) $\qquad$ $p_{WLl} := 0.04 \frac{kip}{ft}$

$$WL_{Par} := \frac{p_{WLl} \cdot TribuLength}{NofBm} \qquad WL_{Par} = 0.233 \cdot \frac{kip}{beam}$$

$$WL_{Nor} := \frac{p_{WLt} \cdot TribuLength}{NofBm} \qquad WL_{Nor} = 0.583 \cdot \frac{kip}{beam}$$

Wind force on live load **parallel** to bent (X-direction)

$$WL_X := WL_{Nor} \cdot \cos(\theta) + WL_{Par} \cdot \sin(\theta) \qquad WL_X = 0.583 \cdot \frac{kip}{beam}$$

Wind force on live load **normal** to bent (Z-direction)

$$WL_Z := WL_{Nor} \cdot \sin(\theta) + WL_{Par} \cdot \cos(\theta) \qquad WL_Z = 0.233 \cdot \frac{kip}{beam}$$

Moments at cg of the Bent Cap due to Wind load on Live Load

$$M_{WL\_X} := WL_Z \cdot \left(h_6 + \frac{hCap}{2}\right) \qquad M_{WL\_X} = 2.736 \cdot \frac{kft}{beam}$$

$$M_{WL\_Z} := WL_X \cdot \left(h_6 + \frac{hCap}{2}\right) \qquad M_{WL\_Z} = 6.84 \cdot \frac{kft}{beam}$$

Vertical Wind Pressure: *(AASHTO LRFD 3.8.2)*

$DeckWidth := FDeckW$ Bridge deck width including parapet and sidewalk

$P_{uplift} := -(0.02ksf) \cdot DeckWidth \cdot TribuLength$ (Acts upword Y-direction) $P_{uplift} = -66.033 \cdot kip$

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

$STRENGTH\_I = 1.25 \cdot DC + 1.5 \cdot DW + 1.75 \cdot (LL + BR + CF) + 1.0 \cdot WA$

$STRENGTH\_IA = 0.9 \cdot DC + 0.65 \cdot DW + 1.75 \cdot (LL + BR + CF) + 1.0 \cdot WA$

$STRENGTH\_III = 1.25 \cdot DC + 1.5 \cdot DW + 1.4 \cdot WS + 1.0 \cdot WA + 1.4 \cdot P_{uplift}$

$STRENGTH\_IIIA = 0.9 \cdot DC + 0.65 \cdot DW + 1.4 \cdot WS + 1.0 \cdot WA + 1.4 \cdot P_{uplift}$

$STRENGTH\_V = 1.25 \cdot DC + 1.5 \cdot DW + 1.35 \cdot (LL + BR + CF) + 0.4 \cdot WS + 1.0 \cdot WA + 1.0 \cdot WL$

$STRENGTH\_VA = 0.9 \cdot DC + 0.65 \cdot DW + 1.35 \cdot (LL + BR + CF) + 0.4 \cdot WS + 1.0 \cdot WA + 1.0 \cdot WL$

$SERVICE\_I = 1.0 \cdot DC + 1.0 \cdot DW + 1.0 \cdot \left(LL_{no\_Impact} + BR + CF\right) + 0.3 \cdot WS + 1.0 \cdot WA + 1.0 \cdot WL$

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}):

$F_{FDC} := (FNofBm - 2)\cdot FSuperDC_{Int} + 2\cdot FSuperDC_{Ext}$ $\quad F_{FDC} = 259.607\cdot kip$

$F_{FDW} := (FNofBm)\cdot FSuperDW$ $\quad F_{FDW} = 38.5\cdot kip$

Backward Span Superstructure DC (F_{BDC}) & DW (F_{BDW}):

$F_{BDC} := (BNofBm - 2)\cdot BSuperDC_{Int} + 2\cdot BSuperDC_{Ext}$ $\quad F_{BDC} = 259.607\cdot kip$

$F_{BDW} := (BNofBm)\cdot BSuperDW$ $\quad F_{BDW} = 38.5\cdot kip$

Total Cap Dead Load Weight (TCapDC):

$TCapDC := (CapDC)\cdot(CapL) + (NofBm)\cdot(BrgSeatDC) + EarWallDC$ $\quad TCapDC = 112.8\cdot kip$

Total DL on columns including Cap weight (F_{DC}):

$F_{DL} := \left(F_{FDC} + F_{FDW}\right) + \left(F_{BDC} + F_{BDW}\right) + TCapDC$ $\quad F_{DL} = 709.015\cdot kip$

Column & Drilled Shaft Self Weight:

DSahft Length, $DsHt := 0\cdot ft$ $\quad$ if Rounded Col, $ColDia := 0\cdot ft$

$ColDC := if\left[ColDia > 0ft, \frac{\pi}{4}\cdot(ColDia)^2\cdot(HCol)\cdot\gamma_c, wCol\cdot bCol\cdot HCol\cdot\gamma_c\right]$ $\quad$ Column Wt, $ColDC = 40.425\cdot kip$

$DsDC := \frac{\pi}{4}\cdot(DsDia)^2\cdot(DsHt)\cdot\gamma_c$ $\quad$ Dr Shaft Wt, $DsDC = 0\cdot kip$

Total Dead Load on Drilled Shaft (DL_on_DShaft):

$DL\_on\_DShaft := F_{DL} + (NofCol)\cdot(ColDC) + (NofDs)\cdot(DsDC)$ $\quad DL\_on\_DShaft = 789.865\cdot kip$

Live Load on Drilled Shaft:

$m = 0.85$ (Multile Presence Factors for 3 Lanes) $\quad$ (AASHTO·LRFD·Table·3.6.1.1.2 − 1)

$R_{LL} := (Lane + Truck)\cdot(NofLane)\cdot(m)$ (Total LLRxn without Impact) $\quad R_{LL} = 277.44\cdot kip$

Total Load, DL+LL per Drilled Shaft of Intermediate Bent:

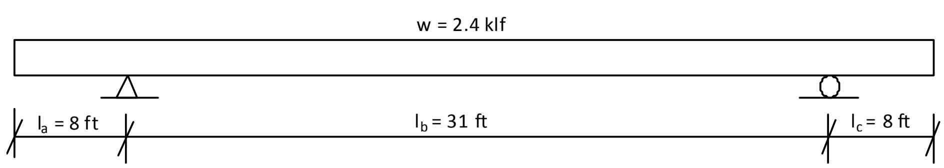
$$Load\_on\_DShaft := \frac{DL\_on\_DShaft + R_{LL}}{NofDs}$$ $Load\_on\_DShaft = 266.8 \cdot ton$

5. PRECAST COMPONENT DESIGN

Precast Cap Construction and Handling:

$w := b \cdot h \cdot \gamma_c$ (Cap selfweight) $w = 2.4 \cdot klf$

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.



$l_a := 8 \cdot ft$ $l_b := 31 \cdot ft$ $l_c := 8ft$

Construction factor:

$\lambda_{cons} := 1.25$ $\lambda_{cons} = 1.25$

Maximum Positive Moment (M_{maxP}) & Negative Moment (M_{maxN}):

$$M_{maxP} := \frac{w \cdot CapL}{2} \cdot \left(\frac{CapL}{4} - l_a\right)$$ $M_{maxP} = 211.5 \cdot kft$

$$M_{maxN} := \frac{w \cdot {l_a}^2}{2}$$ $M_{maxN} = 76.8 \cdot kft$

Factored Maximum Positive Moment (M_{uP}) & Negative Moment (M_{uN}):

$M_{uP} := \lambda_{cons} \cdot M_{maxP}$ $M_{uP} = 264.375 \cdot kft$

$M_{uN} := \lambda_{cons} \cdot M_{maxN}$ $M_{uN} = 96 \cdot kft$

$S := \frac{b \cdot h^2}{6}$ (Cap Section Modulus) $S = 18432 \cdot in^3$

Maximum Positive Stress (f_{tP}) & Negative Stress (f_{tN}):

$f_{tP} := \frac{M_{uP}}{S}$ $f_{tP} = 172.119 \cdot psi$

$f_{tN} := \frac{M_{uN}}{S}$ $f_{tN} = 62.5 \cdot psi$

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

$f'_c := 5 \cdot ksi$ (Compressive Strength of Concrete) Unit weight factor, $\lambda := 1$

$f_r := 5 \cdot \lambda \cdot \sqrt{f'_c \cdot psi}$ (PCI EQ 5.3.3.2) $f_r = 353.553 \cdot psi$

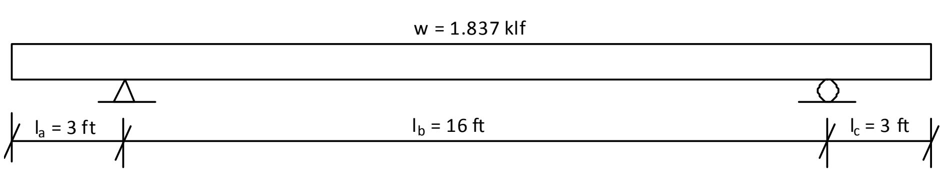
$f_{r\_check} := \text{if}\left[\left(f_r > f_{tP}\right) \cdot \left(f_r > f_{tN}\right), \text{"OK"}, \text{"N.G."}\right]$ $f_{r\_check} = \text{"OK"}$

Precast Column Construction and Handling:

$wCol = 3.5 \cdot ft$ (Column width) Column breadth, $bCol = 3.5 \cdot ft$

$w_{col} := wCol \cdot bCol \cdot \gamma_c$ (Column self weight) $w_{col} = 1.837 \cdot klf$

Due to the location of girder bolts on column, pickup 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.



$l_a := 3 \cdot ft$ $l_b := 16 \cdot ft$ $l_c := 3 \cdot ft$

Maximum Positive Moment (M_{maxP}) & Negative Moment (M_{maxN}):

$$M_{maxP} := \frac{w_{col} \cdot HCol}{2} \cdot \left(\frac{HCol}{4} - l_a \right)$$ $M_{maxP} = 50.531 \cdot kft$

$$M_{maxN} := \frac{w_{col} \cdot l_a^{\,2}}{2}$$ $M_{maxN} = 8.269 \cdot kft$

Factored Maximum Positive Moment (M_{uP}) & Negative Moment (M_{uN}):

$M_{uP} := \lambda_{cons} \cdot M_{maxP}$ $M_{uP} = 63.164 \cdot kft$

$M_{uN} := \lambda_{cons} \cdot M_{maxN}$ $M_{uN} = 10.336 \cdot kft$

$$S_{col} := \frac{wCol \cdot bCol^2}{6}$$ (Column Section Modulus) $S_{col} = 12348 \cdot in^3$

Maximum Positive Stress (f_{tP}) & Negative Stress (f_{tN}):

$$f_{tP} := \frac{M_{uP}}{S_{col}}$$ $f_{tP} = 61.384 \cdot psi$

$$f_{tN} := \frac{M_{uN}}{S_{col}}$$ $f_{tN} = 10.045 \cdot psi$

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

$f'_c := 5 \cdot ksi$ (Compressive Strength of Concrete) Unit weight factor, $\lambda := 1$

$f_r := 5 \cdot \lambda \cdot \sqrt{f'_c \cdot psi}$ (PCI EQ 5.3.3.2) $f_r = 353.553 \cdot psi$

$f_{r\_check} := \text{if}\left[(f_r > f_{tP}) \cdot (f_r > f_{tN}), \text{"OK"}, \text{"N.G."} \right]$ $f_{r\_check} = \text{"OK"}$

DEVELOPMENT LENGTH: AASHTO LRFD 5.11

$A_b := 1.56 \cdot in^2$ (Area of Bar) $d_b := 1.41 \cdot in$ (Diameter of Bar) $f'_c := 5 \cdot ksi$

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

$$l_{db} := \max\left[1.25\cdot\left(\frac{A_b}{in}\right)\cdot\frac{f_y}{\sqrt{f'_c\cdot ksi}}, 0.4\cdot d_b\cdot\frac{f_y}{ksi}, 12\cdot in\right] \quad \text{(AASHTO LRFD 5.11.2.1.1)} \qquad l_{db} = 52.324\cdot in$$

$$l_d := (\lambda_{mod})\cdot l_{db} \qquad l_d = 4.36\cdot ft$$

Basic Compression Development: AASHTO LRFD 5.11.2.2

$$l_{db} := \max\left(\frac{0.63\cdot d_b\cdot f_y}{\sqrt{f'_c\cdot ksi}}, 0.3\cdot d_b\cdot\frac{f_y}{ksi}, 8\cdot in\right) \quad AASHTO\cdot LRFD\cdot EQ\cdot(5.11.2.2.1-1,2) \qquad l_{db} = 25.38\cdot in$$

$$l_d := (\lambda_{mod})\cdot l_{db} \qquad l_d = 2.115\cdot ft$$

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 and cross-slope 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 minimum thickness of 0.5 in. to permit correction of the deck profile by grinding, and to compensate for thickness loss due to abrasion. 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. Overlay could be considered as an alternative to address the effects of camber.

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 $0.19\sqrt{f'_{cm}}$ (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%, 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 seismic loadings (Extreme Event Limit State) while under construction.
9. 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	$0.24 \sqrt{f'_{cm}}$ (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 methods 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 in. 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 sections 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 pre-cast 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 subsection addresses requirements for the design of composite steel modular systems. The following sections modify applicable sections 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.

D RECOMMENDED ABC CONSTRUCTION SPECIFICATIONS

XX SPECIAL REQUIREMENTS FOR PREFABRICATED ELEMENTS AND SYSTEMS FOR ACCELERATED BRIDGE CONSTRUCTION

Table of Contents

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 offsite and/or sub-assembled and erected as a unit to facilitate faster construction onsite 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 offsite (or, onsite 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 alone. 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 be 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

1. 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.
2. 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;
- Super plasticizer;
- Accelerator; and
- Steel fibers, specifically made for steel reinforcement with a minimum tensile strength 360,000 psi (2,500 MPa).

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 two 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 C 78. Only a concrete mix design that passes these tests may be used to form the joint.

XX.3.5 Grout

A structural non-shrink grout shall be applied at all pier column joints to ensure uniform bearing, as shown on the plans. Non-shrink 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 non-shrink grout that meets a minimum compressive strength of 4,000 psi within 24 hours when tested as specified in AASHTO T106.

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 percent of the specified minimum tensile strength of the connecting Grade 60 reinforcing bar. This equates to 90 ksi for reinforcing 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 holdups 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 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 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 in. in 10-ft 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 in. 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 units 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 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 is 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 Inspectors 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 joint.
   - 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 Number 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 60°, 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 is 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 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 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 in. Corrections and adjustments for grade shall be done only when approved by the engineer.

Bridge cross slope up to 4° 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 1/8 in. 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 in. 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 points. Differences in camber between adjacent modules shall not exceed ⅛in. 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 lbs. 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 non-shrink 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 ½ 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 forgo 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 are 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, Ec, 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 in., in conformance with the LRFD Construction Specifications. An additional thickness of ½ in. (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 Decks

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.

Mockups 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 mockup 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 should 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 Post-Tensioned (PT) Connections

Requirements for post-tensioning 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 where 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 Carriers (ADDCs). Following assembly onsite, 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 Bridges (LTTBs). 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 two 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 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 are cast in the deck, centered over the beam's web. A minimum tension capacity of 5,500 lbs 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)

TRB OVERSIGHT COMMITTEE FOR SHRP 2*

Chair: Kirk T. Steudle, *Director, Michigan Department of Transportation*

MEMBERS

H. Norman Abramson, *Executive Vice President (retired), Southwest Research Institute*
Alan C. Clark, *MPO Director, Houston–Galveston Area Council*
Frank L. Danchetz, *Vice President, ARCADIS-US, Inc.*
Stanley Gee, *Executive Deputy Commissioner, New York State Department of Transportation*
Michael P. Lewis, *Director, Rhode Island Department of Transportation*
Susan Martinovich, *Director, Nevada Department of Transportation*
John R. Njord, *Executive Director, Utah Department of Transportation*
Charles F. Potts, *Chief Executive Officer, Heritage Construction and Materials*
Ananth K. Prasad, *Secretary, Florida Department of Transportation*
Gerald M. Ross, *Chief Engineer, Georgia Department of Transportation*
George E. Schoener, *Executive Director, I-95 Corridor Coalition*
Kumares C. Sinha, *Olson Distinguished Professor of Civil Engineering, Purdue University*
Paul Trombino III, *Director, Iowa Department of Transportation*

EX OFFICIO MEMBERS

John C. Horsley, *Executive Director, American Association of State Highway and Transportation Officials*
Victor M. Mendez, *Administrator, Federal Highway Administration*
David L. Strickland, *Administrator, National Highway Transportation Safety Administration*

LIAISONS

Ken Jacoby, *Communications and Outreach Team Director, Office of Corporate Research, Technology, and Innovation Management, Federal Highway Administration*
Tony Kane, *Director, Engineering and Technical Services, American Association of State Highway and Transportation Officials*
Jeffrey F. Paniati, *Executive Director, Federal Highway Administration*
John Pearson, *Program Director, Council of Deputy Ministers Responsible for Transportation and Highway Safety, Canada*
Michael F. Trentacoste, *Associate Administrator, Research, Development, and Technology, Federal Highway Administration*

RENEWAL TECHNICAL COORDINATING COMMITTEE*

Chair: Cathy Nelson, *Technical Services Manager/Chief Engineer, Oregon Department of Transportation*

MEMBERS

Rachel Arulraj, *Director of Virtual Design & Construction, Parsons Brinckerhoff*
Michael E. Ayers, *Consultant, Technology Services, American Concrete Pavement Association*
Thomas E. Baker, *State Materials Engineer, Washington State Department of Transportation*
John E. Breen, *Al-Rashid Chair in Civil Engineering Emeritus, University of Texas at Austin*
Daniel D'Angelo, *Recovery Acting Manager, Director and Deputy Chief Engineer, Office of Design, New York State Department of Transportation*
Steven D. DeWitt, *Chief Engineer, North Carolina Turnpike Authority*
Tom W. Donovan, *Senior Right of Way Agent (retired), California Department of Transportation*
Alan D. Fisher, *Manager, Construction Structures Group, Cianbro Corporation*
Michael Hemmingsen, *Davison Transportation Service Center Manager (retired), Michigan Department of Transportation*
Bruce Johnson, *State Bridge Engineer, Oregon Department of Transportation, Bridge Engineering Section*
Leonnie Kavanagh, *PhD Candidate, Seasonal Lecturer, Civil Engineering Department, University of Manitoba*
John J. Robinson, Jr., *Assistant Chief Counsel, Pennsylvania Department of Transportation, Governor's Office of General Counsel*
Michael Ryan, *Vice President, Michael Baker Jr., Inc.*
Ted M. Scott II, *Director, Engineering, American Trucking Associations, Inc.*
Gary D. Taylor, *Professional Engineer*
Gary C. Whited, *Program Manager, Construction and Materials Support Center, University of Wisconsin–Madison*

AASHTO LIAISON

James T. McDonnell, *Program Director for Engineering, American Association of State Highway and Transportation Officials*

FHWA LIAISONS

Steve Gaj, *Leader, System Management and Monitoring Team, Office of Asset Management, Federal Highway Administration*
Cheryl Allen Richter, *Assistant Director, Pavement Research and Development, Office of Infrastructure Research and Development, Federal Highway Administration*
J. B. "Butch" Wlaschin, *Director, Office of Asset Management, Federal Highway Administration*

CANADA LIAISON

Lance Vigfusson, *Assistant Deputy Minister of Engineering & Operations, Manitoba Infrastructure and Transportation*

* Membership as of August 2012.