Engineering Design, Fabrication, and Erection of Prefabricated Bridge Elements and Systems

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Federal Highway Administration

HIGHWAYS FOR LIFE
Accelerating Innovation for the American Driving Experience.

Foreword

This manual has been developed for the purposes of enhancing the use of Prefabricated Bridge Elements and Systems (PBES) as part of accelerated construction projects. FHWA continues to focus on a need to create awareness, inform, educate, train, assist and entice State DOT's and their staff in the use of rapid construction techniques. This manual is a follow-up of a previously completed manual on Accelerated Bridge Construction (ABC). The purpose of this manual is to provide more in-depth information on the design, fabrication and erection of PBES. Significant portions of the previous ABC manual will be reproduced in this manual in order to provide a stand-alone document with minimal cross references.

Users of this manual will be able to perform the following tasks:

- Understand the different materials used in the construction of prefabricated bridge elements and systems.
- Understand the design requirements for the most common PBES technologies in use today.
- Understand how to design for durability using PBES.
- Understand how to specify PBES and design them for shipping and erection.

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Director, Office of Bridges and Structures

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16. Abstract

Through the Every Day Counts (EDC) program, the Federal Highway Administration (FHWA) has been taking initiatives to bring Accelerated Bridge Construction (ABC) technologies into the market that can help highway agencies build bridges faster and reduce adverse work zone impacts. The manual focuses on the design aspects of ABC using Prefabricated Bridge Elements and Systems (PBES). The purpose of the PBES/ABC design manual is to provide an overview of design, materials, and construction aspects of PBES; explore typical design, planning, and construction processes; identify how they relate to ABC; and offer guidance on where to obtain more information on each subject.

The manual reviews the 4th Edition American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications for their applicability to the design of PBES elements. The manual covers the design aspects of prefabricated bridge superstructure, substructure and foundation elements. While the general design concepts are the same for both cast-in-place and prefabricated elements, this manual discusses subtle differences in the design approach and provide recommendation that are not are not well covered in the AASHTO LRFD Bridge Design Specifications. The design recommendations provided herein are based on research data, engineering judgment, and the concept of emulative design. The manual provides a discussion on the design of jointless decks and critical connections for durability, other precast elements including culverts, arches and frames and other miscellaneous elements. The manual covers the tolerance and quality control requirements for various elements. The manual also explores the effect of shipping, handling, and assembly on the design process.

ABC projects often require the use of a wide variety of materials including conventional materials (e.g. steel, aluminum), new materials (e.g. polymers), or higher performance versions of conventional materials (e.g. ultra-high performance concrete). The manual explores the changes that can be made to materials that are commonly used in PBES/ABC projects. In addition, the manual also addresses the materials and construction related specification requirements, including sample specification language, for these materials.

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SI* (Modern Metric) Conversion Factors

APPROXIMATE CONVERSIONS TO SI UNITS				
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2,000 lbs)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

SI* (Modern Metric) Conversion Factors

APPROXIMATE CONVERSIONS FROM SI UNITS				
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
LENGTH				
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
AREA				
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
VOLUME				
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
MASS				
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2,000 lbs)	T
TEMPERATURE (exact degrees)				
^oC	Celsius	1.8C+32	Fahrenheit	^oF
ILLUMINATION				
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
FORCE and PRESSURE or STRESS				
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003).

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ACRONYMS

The following is a listing of typical acronyms that may be found in this document.

Acronym	Definition
ABC	Accelerated Bridge Construction
ACCT	Accelerated Construction Technology Transfer (FHWA Program)
ADT	Average Daily Traffic
AMVA	American Association of Motor Vehicle Administrators
AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
AISC	American Institute of Steel Construction, Inc.
AISI	American Iron and Steel Institute
AMTRAK	National Railroad Passenger Corporation (Amtrak is not a governmental agency; it is a private company.)
ANSI	American National Standards Institute
ASBI	American Segmental Bridge Institute
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
BCA	Benefit-Cost Analysis
C SHRP	Canadian Strategic Highway Research Program
CD	Compact Disc
CERF	Civil Engineering Research Foundation
CFLHD	Central Federal Lands Highway Division
CFR	Code of Federal Regulations
CIP	Cast-in-Place
CRP	Cooperative Research Program (TRB)
CSD	Context Sensitive Design
DOT	Department of Transportation
ECMT	European Conference of Ministers of Transportation
EFLHD	Eastern Federal Lands Highway Division
EIT	Electronic Information and Technology
EPS	Expanded Polystyrene
EU	European Union
EUREKA	European Research Coordination Agency
F SHRP	Future Strategic Highway Research Program (now known as SHRP 2)
FAA	Federal Aviation Administration
FAQs	Frequently Asked Questions
FHWA	Federal Highway Administration
FRP	Fiber-Reinforced Polymer
FY	Fiscal Year
GIF	Graphic Interchange Format
GSA	U.S. General Services Administration
GRS IBS	Geosynthetic Reinforced Soil Integrated Bridge System
HBP	Highway Bridge Program

HBRRP	Highway Bridge Replacement and Rehabilitation Program
HITEC	Highway Innovative Technology Evaluation Center
HRTS	Office of Research and Technology Services
HSIP	Highway Safety Improvement Program
HSS	Hollow Structural Section
HTML	Hyper-Text Markup Language
IBRD	Innovative Bridge Research and Deployment
IBTTA	International Bridge, Tunnel and Turnpike Association
ISTEA	Intermodal Surface Transportation Efficiency Act of 1991
ITE	Institute of Transportation Engineers
JPEG	Joint Photographic Experts Group
LCCA	Life Cycle Cost Analysis
LRFD	Load and Resistance Factor Design
NAS	National Academy of Sciences
NBI	National Bridge Inventory
NBIS	National Bridge Inspection Standards
NCHRP	National Cooperative Highway Research Program
NCSRO	National Conference of State Railway Officials
NDE	Non-Destructive Evaluation
NEXTEA	National Economic Crossroads Transportation Efficiency Act of 1997
NHI	National Highway Institute
NHS	National Highway System
NHTSA	National Highway Traffic Safety Administration
NIST	National Institute of Standards and Technology
NRC	National Research Council
NSF	National Science Foundation
NTSB	National Transportation Safety Board
OSHA	Occupational Safety and Health Administration
PBES	Prefabricated Bridge Elements and Systems
PCA	Portland Cement Association
PCC	Portland Cement Concrete
PCI	Precast/Prestressed Concrete Institute
PDF	Portable Document Format
PI	Principal Investigator
QC/QA	Quality Control/Quality Assurance
R&D	Research and Development
ROI	Return on Investment
SAFETEA	Safe, Accountable, Flexible, and Efficient Transportation Equity Act of 2003
SCOBS	Subcommittee on Bridges and Structures (AASHTO)
SCOH	Standing Committee on Highways (AASHTO)
SCOR	Standing Committee on Research (AASHTO)
SHA	State Highway Administration
SHRP	Strategic Highway Research Program
TIFF	Tagged Image File Format
TIG	Technology Implementation Group
TRB	Transportation Research Board

TRIS	Transportation Research Information Services (TRB)
TRL	Transportation Research Laboratory
UHPC	Ultra High Performance Concrete
USACE	U.S. Army Corps of Engineers
USDOT	U.S. Department of Transportation
WFLHD	Western Federal Lands Highway Division

CHAPTER 1. PURPOSE AND USE OF THE MANUAL

1.1. INTRODUCTION

The use of Accelerated Bridge Construction (ABC) using Prefabricated Bridge Elements and Systems (PBES) has become more commonplace over the last 20 years in the United States. In the past, the use of ABC for the most part has been driven by acute traffic control issues at specific sites. The concepts of accelerated bridge construction may be new to many agencies; however, the use of prefabricated elements has been applied many times in the past, dating back to the very early days of modern bridge construction.

The Federal Highway Administration (FHWA) has been at the forefront of ABC and PBES for many years. Part of their efforts focused on the development of several manuals on the subject that have been used by many agencies. They include:

- Decision-Making Framework for Prefabricated Bridge Elements and Systems (PBES), Publication Number FHWA-HIF-06-030, May 2006 [1].
- Manual on the Use of Self-Propelled Modular Transporters to Remove and Replace Bridges, Publication Number FHWA-HIF-07-022, June 2007 [2].
- Connection Details for Prefabricated Bridge Elements and Systems, Publication Number FHWA-IF-09-010, March 2009 [3].
- Accelerated Bridge Construction – Experience in Design, Fabrication and Erection of Prefabricated Bridge Elements and Systems, November 2011 [4].

The last manual listed above was developed as an overview of all aspects of accelerated bridge construction using PBES. The approach for this manual was to give an overview of ABC technologies and processes and to explore typical design, planning, and construction processes, identify how they relate to ABC, and offer guidance on where to obtain more information on each subject.

More recently, FHWA has undertaken an initiative called Every Day Counts (EDC). The goal of EDC is to bring new technologies into the market that can have an immediate impact on the way that we build our infrastructure. ABC and PBES are significant parts of EDC. The desire to reduce impacts to the traveling public is the primary driver of this effort.

The EDC program included many workshops, meetings, and exchanges between transportation agencies and FHWA. The EDC program has been very successful in encouraging agencies to take a serious look at ABC and PBES. During many of these meetings, participating agencies expressed a need to have more information on design of projects using ABC and PBES. Previous manuals had some information on this subject, but not sufficient detail. The purpose of this manual is to expand on the previous work.

It should be understood that all potential designs cannot be covered in any one manual. The FHWA has published two load and resistance factor design (LRFD) bridge design examples [74 and 75]. These examples cover typical bridges, but not nearly all of the different types of bridges in the nation's inventory.

These examples were chosen because they include a variety of common structure types, including:

- Integral abutments.
- Cantilever abutments.
- Pier bents.
- Wall piers.
- Spread footings.
- Pile supported footings.
- Steel girders.
- Prestressed concrete girders.

The goal of the design examples is to establish the general approach for the design of a typical bridge, which can help establish a general design procedure for all bridges. This manual will follow a similar approach. The most common forms of ABC and PBES will be covered in some detail; however, all design possibilities in use in the U.S. will not be covered. As the technology expands, it is anticipated that the creativity of the bridge engineering community will produce new concepts and innovations.

Several design examples were developed and presented in Chapter 12. The approach for this is to use previously published LRFD design examples [74 & 75]. Portions of these examples were amended and modified to account for PBES construction. The following is a list of the design examples in Chapter 12:

- Precast concrete integral abutment.
- Precast "flying" wingwall.
- Precast concrete cantilever abutment.
- Precast concrete multi-column pier bent.
- Precast concrete wall pier.
- Modular deck beam element (MDB) (development of software input parameters).
- Longitudinal deck connection between MDBs.
- Link slab connection at pier for MDBs.

1.2. INTENDED USERS

1.2.1. Design Engineers

Bridge design engineers are asked to take the concepts that are developed in the project planning process and turn them into plans and specifications that are suitable for project bidding and construction. Projects built using PBES technology require a slightly different approach when compared to bridges built using conventional methods. In particular, the engineer is required to integrate the design into the construction process. Traditionally, the realm of means and methods is left to the contracting community to determine.

Design engineers have expressed concerns over the use of prefabricated elements. The major focus of concerns is the issue with joints and connections between elements. It is important to note that all bridges built with conventional methods have joints and connections. For instance, a concrete pier built with conventional cast-in-place (CIP) concrete is not monolithic, but it is built with multiple joints and connections. They are referred to as "construction joints." The construction of a pier required the casting of a footing, followed by the casting of a column several days later. The joint between the footing and the column is a connection that is normally made with lapped reinforcing bars. PBES technologies do not necessarily mean that we need to build a bridge differently. This construction will be detailed as a precast connection made with some form of mechanical coupler. The AASHTO LRFD Bridge Design Specifications [5] contain specifications for mechanical reinforcing steel connectors. Substituting a mechanical coupler for a lapped bar is acceptable and within the purview of the specifications. This approach is referred to as "emulative design," in that the connection is designed to emulate a CIP connection with a precast connection. The American Concrete Institute (ACI) has published a specification on emulative design, which covers this approach [6].

The majority of PBES designs completed to date use some form of emulative design. While this approach may seem unsophisticated to some, it offers a level of comfort to the design engineer in that there will not be a significant change to the way that engineers design our structures. This approach will most likely continue in the near future.

In some cases, pure emulation of conventional construction is not used. The PBES approach may require a change to the structure geometry to facilitate the installation. For example, the first ABC bridge system installation in Utah (4500 South over I-215 in Salt Lake City) involved the replacement of a four-span bridge with a single-span bridge. Had the design been executed with conventional methods, the new bridge might have been a two-span structure. Switching the design to a single-span facilitated the span installation using SPMTs, which ultimately reduced time and cost.

In the future, other construction methods and structural details with PBES may emerge that will produce designs that are different from current design philosophies. Virtually all new technologies are given very careful thought and scrutiny by engineers before being adopted as normal. For example, it took years before the use of prestressed concrete became commonplace.

The majority of the technologies outlined in this manual are considered "market ready." The few items that are still considered to be experimental will be noted as such. Fortunately, PBES technologies have risen to the level of being more common; therefore, the limited use of these technologies by only a few agencies is becoming a thing of the past. There have been hundreds of successful projects built using PBES techniques. This has fostered a level of comfort in the design community as well as with owner agencies.

1.2.2. Materials Engineers

Materials typically used in conventional construction were developed based on the assumed timeline that often is not limited or compressed. Construction of the interstate highway system was primarily done on "greenfields" that did not have a traffic management component. The construction of a highway in the 1960s typically included many miles of roadway construction with bridges interspersed throughout. The bridge often was not on the critical path for the overall construction of the project. For example, on projects that involve significant roadway construction, the bridge can be built many months prior to the completion of the roadway.

The materials and construction methods that were developed during the construction of the interstate system were developed with this scenario in mind. For instance, the use of portland cement concrete (PCC) is based on material that is specified to develop full strength in 28 days. On many ABC projects, 28 days represents the entire construction window for the bridge; therefore, there may be a need for new materials that can gain strength more quickly on very rapid construction processes. These materials also need to have sufficient durability to ensure a service life with minimal or no maintenance.

Some forms of ABC and PBES require the use of new materials that can reduce the time for construction. Out of necessity, the bridge construction industry has been exploring ways to reduce construction time. The prestressed concrete industry has developed ways to accelerate concrete strength gain and reduce cure times, such as through steam curing.
Other products such as grouts also have been developed by the industry for use on bridge preservation projects. These proprietary materials also offer accelerated strength gain, which may be applicable to ABC projects. Fast-curing epoxy and polymer grouting materials have been successfully replacing cementitious grout in many applications.

PBES projects also bring about a need to use mechanical connectors and proprietary items such as reinforcing connectors and post-tensioning systems, commonly used in PBES applications.

Materials engineers need to be engaged with the design community to establish new ways of specifying materials for these projects. The use of prequalified materials lists should be expanded for specialized products. The use of prescriptive specification language may need to change to performance specification language so that the performance measures of ABC construction can be met.

This manual—especially Chapter 2—will offer guidance on the role that materials and products play in ABC and PBES projects. Materials engineers need to change the approach that historically has been used in the same way that design engineers need to re-evaluate their approaches. The goal of this manual is to identify the resources and materials that are available and make recommendations for the incorporations of these items into ABC and PBES projects.

1.2.3. Construction Engineering and Inspection Staff

The main focus of ABC is reduced construction time. This means that construction staff members play a key role in the success of any ABC project. ABC does not necessarily mean just building a bridge the same as before, but with more construction workers or double shifts. ABC technologies sometimes change the entire approach to the construction and quality control inspection for a bridge. FHWA is developing a separate construction manual for ABC and PBES. Even with this, much of the information presented in this manual will be beneficial to construction engineering and inspection staff.

Prefabrication of bridge elements is not new. Bridge contractors have been prefabricating bridge beams for over 50 years on virtually all projects. Prefabrication of other bridge elements has also been used for many years on select projects. The goal of PBES is to expand the use of prefabrication to all elements in the bridge on a regular basis. This will place more emphasis on erection and rigging and less emphasis on field casting of concrete. Construction engineers need to be more versed in these techniques and some of the equipment used to construct ABC bridges. Construction staff can help design staff by providing knowledge and perspective on construction equipment capabilities and techniques.

This manual and its companion manual entitled, “Contracting and Construction of Accelerated Bridge Construction Projects with Prefabricated Bridge Elements and Systems,” [86] present recommended changes to the roles of the construction engineering staff. In addition to new methods and equipment, the overall management of the projects may need to change in order to expedite the construction of the bridge. In some cases, more emphasis may need to be placed on document management, project scheduling, and decision-making processes.

CHAPTER 2. MATERIALS FOR ABC AND PBES

ABC projects often require the use of new materials, or higher performance versions of conventional materials. For example, the use of 28-day strength gain for cementitious materials may no longer be acceptable for very rapid construction projects where entire construction sequences are measured in hours. There is a need for new materials that can gain strength rapidly, without sacrificing quality and durability.

The use of proprietary products is more common in PBES projects. This chapter focuses on the most common materials used in ABC and PBES projects. It also explores the changes that can be made to more common materials to reduce the time for construction.

2.1. CEMENTITIOUS MATERIALS

2.1.1. Concrete

Concrete is the most common material used in ABC projects, especially for substructure elements. Concrete can be formed into an infinite number of shapes and sizes and can provide a durable long-lasting structure. One negative attribute to the use of concrete is the mass of the elements. This may be mitigated in some instances through the use of lightweight concretes, which will be covered in Section 2.1.2.

Transportation agencies have standard or pre-approved mix designs for conventional concrete provided by concrete batch plants. These are well tested and proven mixes. Precast plants make use of in-house mixes that are designed to produce high early strength gains. The reason for this is to reduce the amount of time that an element needs to gain sufficient strength so it can be removed from the casting bed or forms so they can be reused.

Agencies have different approaches to the approval of mix designs for precast facilities. Most use prescriptive specifications, allowing the fabricator to develop a mix that meets the specifications. These mixes are submitted to the agencies for testing and approval. Designers of prefabricated elements need to understand that conventional standard mix designs may not be appropriate for prefabrication.

It is recommended that agencies use a performance specification for prefabricated elements (similar to girders) and let the fabricator meet the requirements of the specification. Typical design strengths range from four to eight ksi. Owners may choose to specify durability requirements in the performance specifications as well.

Accelerators can be used to improve early strength gain. Accelerating the strength gain of concrete is especially important for closure pours, which can be the critical path for construction of a prefabricated bridge. There are several approaches that can be used to accelerate the strength gain of concrete. The first method is through the use of Type III High Early Strength Cement in the concrete mix. The second method is to use a lower water-cement ratio. The third is to use a calcium chloride admixture. Calcium chloride is not recommended because it increases the amount of drying shrinkage, which can lead to increased cracking. It also can potentially lead to corrosion of the reinforcing steel. If calcium chloride is used, the dosage should be limited to the values published in the ACI Structural Building Code [46].

Closure Pour Concrete: Closure pours can be constructed with standard concretes if construction schedules permit; however, the restraint of the adjacent concrete can cause shrinkage cracks to occur. These cracks are common in stage construction joints in bridge decks. They can be more pronounced in narrow closure pours due to the restraint from both sides of the pour and the high length-to-width ratios. There are ways to reduce the cracking tendency of closure pour concrete. The Washington State Department of Transportation (WSDOT) sponsored a research project to investigate mitigation strategies for early age shrinkage cracking in bridge decks [47]. The results of the study indicated that it is possible to significantly reduce drying shrinkage in standard mix designs. The following are the major points of this study:

- Both the size and source of coarse aggregates play a very important role in determining the properties of concrete, including drying shrinkage.
- Paste volume plays an important role in the free shrinkage of concrete. Concrete mixes with a small paste volume have lesser tendency of shrinkage cracking. The use of larger size aggregates reduces the paste volume in concrete mix. Mix designs for closure pours should use the largest feasible aggregate size.
- Concrete cracking resistance is the combined effects of both its flexural (tensile) strength and its free shrinkage property. The concrete mix that has an acceptable tensile strength and low free shrinkage strain is anticipated to have relatively good restrained shrinkage cracking resistance. The goal is to have the tensile strength gain outpace the buildup of tension stresses caused by shrinkage.
- Shrinkage Reducing Admixtures (SRA) can decrease drying shrinkage by reducing the surface tension of water, which causes a force pulling in on the walls of the pores in concrete.

Another source of information on reducing shrinkage in concrete is the ACI 223-83 “Standard Practice for the Use of Shrinkage-Compensating Concrete,” and ACI 223R-10 “Guide for the Use of Shrinkage-Compensating Concrete.”

High early strength concretes also have been used for closure pours. It is possible to attain strengths of four ksi in six hours; however, this comes at a price. Normally the cementitious material content is raised, which can increase shrinkage cracking. Massachusetts Department of Transportation (MassDOT) has successfully developed a performance specification that requires rapid strength gain with minimal shrinkage using multiple admixtures. This specification is included in the appendix of the FHWA ABC manual entitled “Accelerated Bridge Construction – Experience in Design, Fabrication, and Erection of Prefabricated Bridge Elements and Systems” [4]. This specification requires an interim strength of 2,000 psi at 4 hours after initial set. This approach greatly reduces the risk of developing a mix that can gain full strength very rapidly. It is relatively easy to design a mix that can attain a compressive strength of 2,000 psi in a few hours, but obtaining a compressive strength of 4,000 psi is much more difficult to attain in short order. This type of performance increases construction costs; therefore, the use of high early strength concretes should only be used where rapid strength gain is critical. Increasing the cure time to 18 hours will dramatically change the mix design for the closure pour concrete.

As previously stated, cracking in closure pour concrete is typically a result of shrinkage of the restrained concrete. The surrounding precast concrete will restrain the closure pour concrete. The key to minimizing cracking is to have the concrete gain tensile strength faster than the development of internal shrinkage stresses. Typical AASHTO and ASTM shrinkage tests measure the shrinkage of unrestrained concrete. A simple specimen is cast and its length change is measured over time. However, this test method is not an accurate measure of the potential for cracking in restrained concrete.

In order to determine the susceptibility of a concrete mix to confined shrinkage cracking, a test must be used that includes confined concrete. The AASHTO construction specifications include a confined shrinkage test entitled "AASHTO T334-08 Practice for Estimating the Crack Tendency of Concrete" [50]. Figure 2.1.1-1 is a sketch of an AASHTO ring test setup. This test consists of a 3-inch-thick ring of concrete that is cast around a 12-inch diameter, ½-inch thick steel pipe that is 6-inch tall. Strain gages are installed on the inside face of the steel ring to measure the buildup of compressive stress in the ring as the concrete shrinks. When cracking occurs in the concrete, the stresses in the steel ring will drop off, indicating the exact time of the crack.

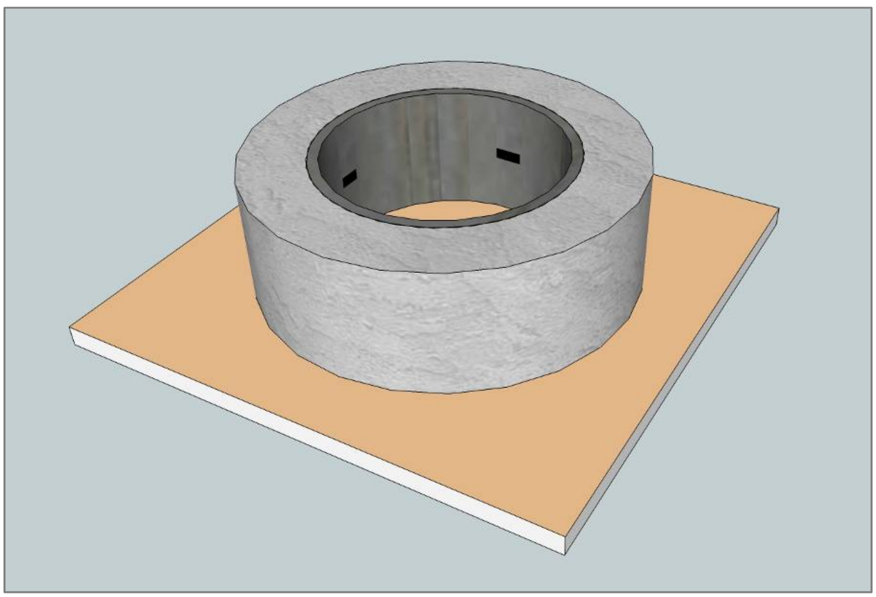


Figure 2.1.1-1 AASHTO Ring Test for Confined Concrete Shrinkage.

It should be noted that this test will not guarantee that a confined concrete closure pour will not crack. However, it will measure the relative cracking resistance of different mixes. A goal for this test may be to develop a mix that does not crack within a specified timeframe, such as 14 or 28 days. MassDOT's specifications use the ring test to identify potential candidate mixes. The final acceptance is made after a full-scale closure pour is cast and cured. Figure 2.1.1-2 shows a detail of the test setup. The test is a full-scale mockup of the deck, including a full-scale portion of the intended closure pour. The closure pour area is restrained by the rectangular outline of the test panel. The details of this test panel were set to mimic the actual dimensions of the details on the plans. If a similar test is proposed for a project, the details should be adjusted to match the proposed closure pours. Acceptable cracking limits, similar to conventional deck concrete, were specified by the agency that were measured at 14 days after placement of the closure pour concrete.

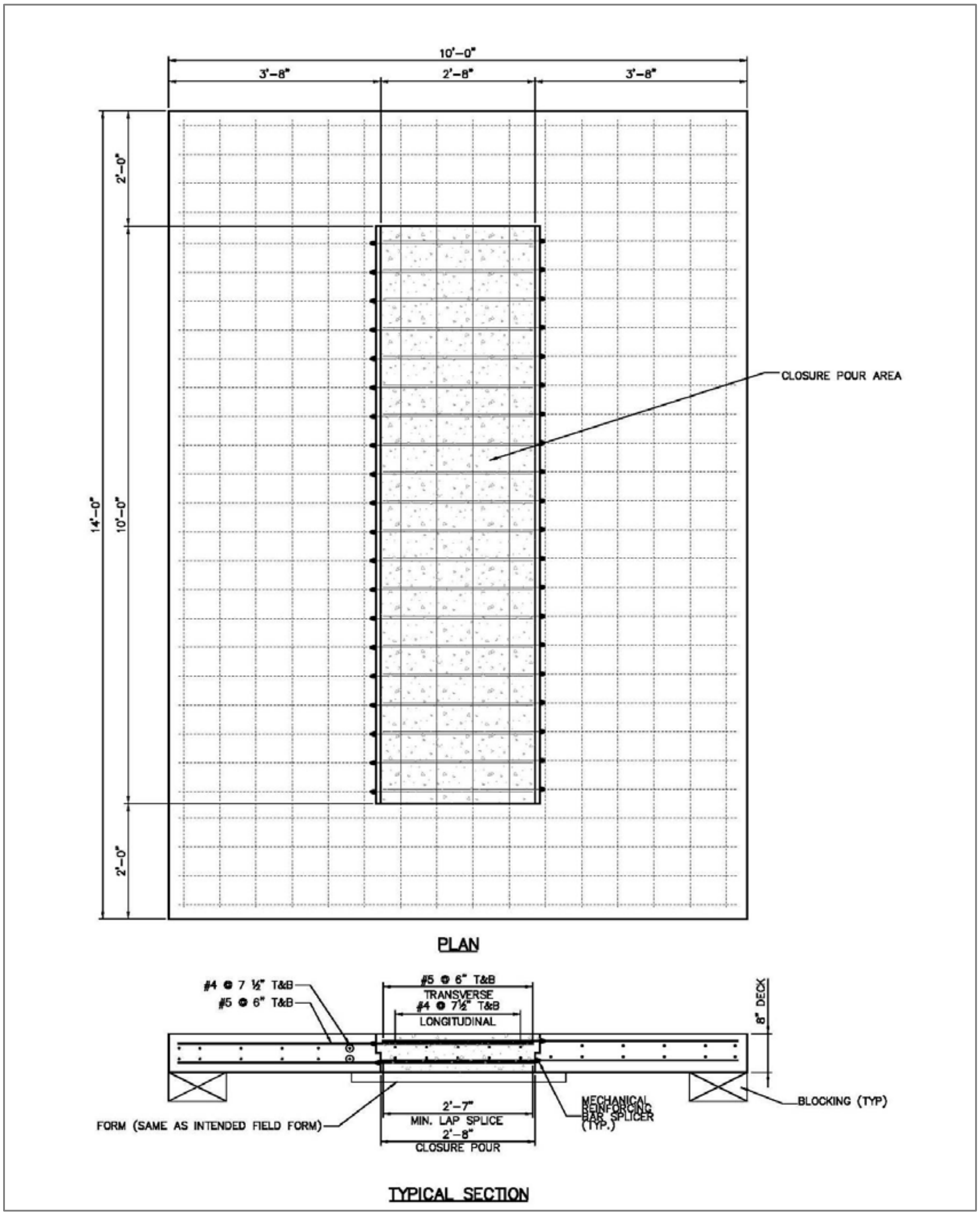


Figure 2.1.1-2 Confined Shrinkage Test used on the Massachusetts DOT 93 Fast 14 Project.

Minor cracking may occur in closure pours even when the best concrete mixes with a low shrinkage design are used. The key is to minimize the number of cracks and crack widths. The number of cracks is a function of the mix design. The width of the cracks is a function of the internal reinforcing. Many states have specification requirements for sealing and repair of cracked concrete in bridge decks. The most common repair procedures include the use of methacrylate crack sealers, or epoxy injection methods.

There have been several research projects that investigated various grouts and concretes that can be used in voids and closure pours. Several of the more significant projects are "Recommendations for the Connection between Full-Depth Precast Bridge Deck Panel Systems and Precast I-Beams" [22] and "Cast-in-Place Concrete Connections for Precast Deck Systems" [7].

The PCI Northeast Bridge Technical Committee has developed a set of recommendations and guidelines for site cast concretes used in ABC projects. Figure 2.1.1-3 contains PCI recommendations. These guidelines can be used by an agency that is considering the development of a program of ABC projects. The approach is to develop a set of performance parameters for a group of concretes that could potentially be used by the agency. Once the parameters are set, the agency can work with the local concrete producers to develop a list of approved mixes that meet the specified performance measures.

Once these mixes are developed, the design engineer needs to specify the required final strength of the closure pour concrete. The contractor could then choose any of the approved concretes for the closure pour areas based on the intended construction schedule for the elements. In many cases, the closure pour concretes are not loaded for days or weeks after placement; therefore, the use of specialized materials may not be necessary. This approach can reduce costs by using appropriate materials in connections that are not time-critical.

SITE CAST CONCRETE AND GROUT:

THE DESIGNER SHALL SPECIFY THE MINIMUM CONCRETE PROPERTIES FOR THE FINAL CONSTRUCTION (STRENGTH, CURE TIME, ETC.). THE ENGINEER RESPONSIBLE FOR THE ASSEMBLY PLAN SHALL SPECIFY THE REQUIRED CONCRETE STRENGTHS FOR VARIOUS STAGES OF THE ASSEMBLY BASED ON CALCULATIONS DEVELOPED FOR THE ASSEMBLY PLAN. FOR EXAMPLE: THE ASSEMBLY PLANS COULD SPECIFY A CONCRETE STRENGTH IN A CLOSURE POUR OF 2000 PSI FOR A CERTAIN STAGE OF CONSTRUCTION, PROVIDED THAT THE CONCRETE GAINS THE FULL DESIGN STRENGTH PRIOR TO OPENING THE BRIDGE TO TRAFFIC.

RECOMMENDATIONS FOR SITE CAST CONCRETE CONCRETE MIXES:

MOST STATES HAVE STANDARD CONCRETE MIXES FOR BRIDGE CONSTRUCTION USING CONVENTIONAL CONSTRUCTION. ACCELERATED BRIDGE CONSTRUCTION PROJECTS OFTEN REQUIRE CONCRETE THAT CAN GAIN STRENGTH AND CURE IN A RAPID MANNER. MATERIAL PERFORMANCE SPECIFICATIONS ARE RECOMMENDED IN LIEU OF RIGID PRESCRIPTIVE SPECIFICATIONS. THE FOLLOWING CONCRETE STRENGTH PARAMETERS ARE SUGGESTED FOR USE ON PREFABRICATED BRIDGE PROJECTS.

- VERY EARLY STRENGTH CONCRETE:
  - CONCRETE THAT WILL ATTAIN THE DESIGN STRENGTH IN LESS THAN 12 HOURS
- EARLY STRENGTH CONCRETE:
  - CONCRETE THAT WILL GAIN THE DESIGN STRENGTH IN LESS THAN 24 HOURS
- NORMAL CONCRETE:
  - CONCRETE THAT WILL GAIN THE DESIGN STRENGTH IN LESS THAN 7 DAYS

SHRINKAGE OF EARLY STRENGTH CONCRETE CAN LEAD TO CRACKING. FOR THIS REASON, SHRINKAGE COMPENSATING ADMIXTURES SHOULD BE CONSIDERED. LIQUID ADMIXTURES SHOULD BE USED IN LIEU OF EXPANSIVE METALLIC POWDERS.

IT IS RECOMMENDED THAT THE STATES WORK WITH LOCAL READY MIX PRODUCERS TO DEVELOP ACCEPTABLE MIX DESIGNS THAT CAN MEET THE REQUIRED PARAMETERS. IDEALLY, THESE MIXES SHOULD BE DEVELOPED PRIOR TO BIDDING AN ACCELERATED BRIDGE CONSTRUCTION PROJECT.

Figure 2.1.1-3. Recommendations for Site Cast Concrete (Source: PCI Northeast).

2.1.2. Lightweight Concrete

The use of lightweight concrete has been somewhat limited on bridge construction projects, primarily due to the perceived concern over durability and cost; however, there still have been numerous uses of lightweight concrete on all types of bridges in the U.S., including the upper deck on the suspension spans of the San Francisco-Oakland Bay Bridge, which was built in 1936 and is still in service.

The major advantage of the use of lightweight concrete on ABC projects is to reduce the weight of the elements in order to facilitate shipping and handling. An added benefit is that lightweight concretes can reduce the overall structure weight, which can be used to economize the design of substructures and foundations (especially in high seismic zones). Lightweight concrete can also be used to extend span ranges and facilitate the rehabilitation of old substructures. To date, most lightweight concrete used on bridges has been limited to bridge decks. Some agencies have made use of the material for other elements, such as girders.

Lightweight concrete is simply normal concrete manufactured using lightweight aggregates (both coarse and/or fine). These aggregates are manufactured by expanding certain types of shale, clay, and slate. The aggregates are produced in high temperature rotary kilns in which gas bubbles are formed within the aggregates that remain upon cooling. These bubbles reduce the weight of the expanded aggregate without sacrificing significant strength.

Replacing normal weight aggregate in concrete with lightweight aggregate produces lightweight concrete. The use of lightweight aggregate can be organized into three main categories of lightweight concrete:

- All lightweight: All aggregates (coarse and fine) are made up of lightweight materials.
- Sand lightweight: Lightweight coarse aggregate with normal-weight fine aggregate.
- Specified density: A blend of lightweight and normal-weight aggregates to achieve a target density.

The most common form of lightweight concrete is sand lightweight. This is due to the economics of the materials. Sand lightweight generally produces relatively light concrete at a relatively low cost. Specified density concrete is an approach to lightweight concrete where the designer specifies the required density in a performance specification. The contractor's supplier then formulates a mix that meets the specification requirements. This is helpful since it allows the supplier the flexibility to develop a mix for the specific lightweight and normal-weight aggregates being used. Allowing the contractor the leeway to adjust the mix will help to produce the material at a reasonable cost by only requiring enough lightweight aggregate to meet the required density. It should be noted that the specified concrete compressive strength will also have an effect on the density, which is another reason to use a performance-based specification. Contractors or precast element fabricators may also propose the use of reduced density concrete to address issues related to handling or shipping.

Table 2.1.2-1 presents the range of different types of concrete mixtures that can be made. This data was provided by the Expanded Shale, Clay and Slate Institute (ESCSI), which is comprised of companies that produce lightweight aggregate for use in concrete and other applications.

Table 2.1.2-1. Spectrum of Concrete Densities for Lightweight Concretes.
Note: LW = Lightweight; NW = Normal weight (Source: Expanded Shale, Clay and Slate Institute).

All LW Concrete	Sand LW Concrete	NW Concrete	Specified Density Concrete
90-105 pcf	110-125 pcf	135-155 pcf	90-155 pcf
LW Fine Aggregate	NW Fine Aggregate	NW Fine Aggregate	LW and/or NW Fine Aggregate

LW Coarse Aggregate	LW Coarse Aggregate	NW Coarse Aggregate	LW and/or NW Coarse Aggregate

A review of the data in the table shows the ranges of unit densities that can be obtained with lightweight concrete. The densities that can be achieved for ranges of unit weights can vary depending on the aggregates chosen and the mix designs. The key data point to take from this table is the fact that a wide range of densities can be achieved using lightweight aggregates. Simply specifying lightweight aggregates will not guarantee a given unit density. The recommended approach is for the designer to design to a specified density of concrete, and write construction specifications to guide the concrete supplier to design a mix that will produce that density.

Design calculations are usually based on the “equilibrium density” of lightweight concrete, which is the density after moisture loss has occurred over time. The unit weight fresh density of the concrete that is checked during placement, which is required for quality control is not the same as the equilibrium density. Contract documents may specify either or both of these densities (equilibrium or fresh), but the type of density that is specified must be clearly stated. It is conservative to calculate dead loads based on fresh density. Density is measured according ASTM C 567 – Standard Test Method for Determining Density of Structural Lightweight Concrete [51]. The density of the concrete with reinforcement should also be given in the contract documents so it is clear how dead loads have been computed.

An added benefit to lightweight concretes is that the porous lightweight aggregates are pre-wetted prior to batching, since they have higher absorption than normal weight aggregate, and provide internal curing moisture. Once the concrete is placed and the hydration process begins, the water absorbed in the lightweight aggregates is released to provide an internal source of moisture to improve the curing of the concrete. There have been several studies of this phenomenon, including a paper entitled “Internal Curing: A 2010 State-of-the-Art Review” [89]. This paper includes the results of trial bridge projects where the viability of this method was studied. Several other informal studies have been done by agencies where a fraction of the normal weight sand in a mixture has been replaced with pre-wetted lightweight fine aggregate. Adjacent panels were cast with and without lightweight aggregate. There were noticeable differences in the moisture content of the panels, with the panel containing lightweight aggregate providing more curing moisture. Internal curing has also been shown to reduce permeability and decrease shrinkage, both of which are beneficial to ABC projects. These benefits are attractive; however, the most significant benefit of internal curing for ABC may be the potential to reduce the amount of time for external curing or to provide curing where accelerated construction techniques do not allow time for external curing. By using internal curing, the construction time for a concrete closure pour can be reduced or the properties of the concrete enhanced to provide better long-term performance.

Lightweight concretes have been used on ABC projects in Utah that were constructed using SPMTs. The material was used to reduce the overall weight of the superstructures, which reduced the number of axles required for the SPMTs, thereby reducing the overall construction cost. Similar savings could be realized on other projects that use prefabricated elements due to reduced crane sizes, shipping trailers, or hauling permit costs.

The ESCSI has produced a number of documents on the use of lightweight concrete. This information can be found at www.escsi.org.

2.1.3. Grout for Voids

Grouts are materials that are mixed and placed in relatively small voids that cannot be filled with concretes. Some grouts provide very high early strength gain, are flowable, have low shrinkage characteristics, and have high bond strength. All of these properties are attractive for ABC projects; however, this performance comes at a price. The unit cost of grout materials is 5 to 10 times greater than concretes. For this reason, designers should only use grouts for voids that:

- Require high early strength.
- Need non-shrink properties.
- Require high bond strength.
- Are relatively small in size.
- Have reinforcement congestion.

It is common to connect prefabricated elements with grout in order to fill the void between the adjoining elements, and in many cases, structurally connect the adjacent elements.

Non-shrink cementitious grout is most often used to easily and efficiently provide a durable, structurally stable connection between precast concrete elements. Epoxy grouts can be used; however, they tend to have a low modulus of elasticity and are expensive when compared to cementitious grouts. In order to achieve desired results, careful selection and specification is required when using non-shrink grouts. Unfortunately, there is not simple uniform specification for grout materials. ASTM C1107 — 11 Standard Specification for Packaged Dry, Hydraulic-Cement Grout (Nonshrink) [68] covers many cementitious grouts, but it does not cover all grouts such as epoxy grouts and magnesium phosphate grouts, which have also been found to perform well. The majority of grouts in the market are proprietary and each has specific formulations to achieve the various properties that are desired.

There have been several recent research studies on the properties and performance of various grout materials [22, 7]. These studies investigated properties such as strength gain, shrinkage, and bond strength. The FHWA also has studied various products for closure pours, including the use of Ultra High Performance Concrete [87]. The FHWA does not endorse or recommend specific materials. Agencies should either investigate the results of research projects, or test products themselves. Many of the products are manufactured with different materials; therefore, it is difficult to write construction and materials specifications. Some agencies have solved this problem by developing pre-qualified product lists. The development of these lists should account for the intended use of the grouts and the required material properties that are desired. Multiple product lists may be required for various ABC applications.

The following paragraphs contain information on different grout types. These are taken from the previously published FHWA ABC Manual [4]. The FHWA manual entitled “Connection Details for Prefabricated Bridge Elements and Systems" [3] has more information on grout placement, curing, and specifications.

2.1.3.1. Grout Types

There is no such thing as a generic non-shrink grout. There are several different types of grouts, each with its own advantages and disadvantages. Cementitious non-shrink grouts are less expensive than other grouts, such as epoxy grout and magnesium phosphate grout. Cementitious grouts are generally easy to work with, and develop adequate strengths in a reasonable timeframe. These grouts are often pre-packaged and can be extended using small diameter stones for larger pours. Cementitious grouts are ideal for static and light dynamic loadings. This section will focus on cementitious non-shrink grouts since they are adequate for virtually all prefabricated connections. Other grout types may be acceptable for precast connections but require additional specification and suitability considerations on the part of the designer.

Ideally, a non-shrink cementitious grout will not exhibit dimensional change in the plastic or hardened state. To achieve non-shrink characteristics in grout, additives are mixed into the grout to counteract the natural tendency of grouts to shrink. There are different types of additives for cementitious grouts in the market. The additives have certain advantages and disadvantages.

Several common additives are as follows:
Gas Generating: This is the most common grout type. A chemical substance is added to the grout mixture to control shrinkage. In most cases, an aluminum powder is used. A chemical reaction occurs with the aluminum powder and the alkalis in the cement during the plastic phase to form hydrogen gas. The generated gas is used to promote expansion.

However, because small amounts of aluminum powder are used, the expansion can be difficult to control under various conditions. This uncontrolled expansion can cause bleed water to form at the grout surface that can cause loss of support and bond. This has led to the development of alternative compounds that can ensure a quality grout under varying conditions.

Ettringite: Ettringite expansive grout relies on the growth of ettringite crystals during the hardened state to counteract shrinkage.

Air Release: Air release additives react with water to release air and cause expansion.

2.1.4. Grout for Post-Tensioning Ducts

Post-tensioning has been used by many agencies for connecting prefabricated elements. The FHWA has previously published a manual entitled "Post-Tensioning Tendon Installation and Grouting Manual" [80]. This manual contains significant information on post-tensioning and grouting of ducts. Portions of this manual are reprinted in this section.

Cement grout is chemically basic and provides a passive environment around the post-tensioning bars or strands. In addition, grout serves to bond internal tendons to the structure. In the free lengths of external tendons, the principal role of the grout is to provide an alkaline environment inside the duct. Nevertheless, complete filling of the duct with grout is essential for proper protection.

The primary constituent of grout is ordinary portland cement (Type I or II). Other cementitious material may be added to enhance certain qualities of the final product. For example, fly ash improves corrosion resistance in aggressive environments. The addition of dry silica fume (micro-silica) also improves resistance to chloride penetration because the particles help fill the interstices between hydrated cementitious grains, thus reducing the permeability. The water-cementitious material ratio should be limited to a maximum of 0.45 to avoid excessive water retention and bleed and to optimize the hydration process. Any temptation to add water to improve fluidity on-site must be resisted at all times. Fluidity may be enhanced by adding a high range water-reducer (HRWR) (Type F or G).

2.1.4.1. Thixotropic Grouts

A thixotropic grout is one that begins to gel and stiffen in a relatively short time while at rest after mixing, yet when mechanically agitated, returns to a fluid state with much lower viscosity. Most grouts made with cementitious materials, admixtures, and water are non-thixotropic. Thixotropy may be exhibited by some, but not necessarily all, pre-bagged grouts.

A critical feature of a grout is that it should remain pump-able for the anticipated time to fully inject the tendon. This may be significant for long tendons or where a group of several tendons is to be injected in one continuous operation. Some thixotropic grouts can have very low viscosity after agitation, becoming easy to pump.

2.1.4.2. Admixtures

Like concrete, admixtures may be used to improve workability and reduce the water required, reduce bleed, improve pumping properties, or entrain air. Care must be exercised to use the correct quantities in the proper way according to manufacturer's instructions, and to remain within the mix properties established by qualifying laboratory tests.

Calcium nitrite may help improve corrosion resistance in some situations by bonding to the steel to form a passive layer and prevent attack by chloride ions.

HRWR improves short term fluidity. However, a grout with HRWR may lose fluidity later when being injected through hoses and ducts. Unlike a concrete mix, it is not possible to re-dose a grout especially when it is in the, pump, hoses, and ducts. Also, HRWR tends to cause bleed in grouts. On-site grout mixing with HRWR is not recommended.

Other admixtures include:

- Shrinkage compensating agents.
- Anti-bleed admixtures.
- Pumping aids.

- Air-entraining agents.

The addition of these should be strictly in accordance with manufacturer's recommendations. Furthermore, the mix should be qualified by appropriate laboratory testing. On site, daily grout production must be monitored by various field tests in order to maintain quality control and performance.

2.1.5. Flowable Fill

PBES projects normally include the installation of fill materials behind and under the elements. Flowable fill is a cementitious material that has been developed that can be used to rapidly fill areas without the need for compaction.

Flowable fill is a material that goes by several different names, depending on the region. Other names for the material include "controlled low-strength material," "controlled density fill," "plastic soil-cement," "soil cement slurry," "flowable mortar," etc. The common components in these materials are cement, water, aggregate (normally sand), and fly ash.

The key attribute of flowable fill is that it can be used to fill an embankment without compaction. Fly ash is used to enhance flowability, so it can be used to fill in confined areas, such as under slabs and footings. It can be pumped into place using standard concrete pumping equipment.

Even though flowable fill is a cementitious material, the strength of the material is significantly less than concrete. Flowable fill typically has a compressive strength of 1,200 psi or less. Some agencies use "excavate-able" flowable fill with a strength of 200 psi or less. Values as low as 200 psi may seem very low and "non-structural"; however, a 200 psi strength equates to a compressive strength of 28,800 pounds per square ft. This value is significantly larger than typical compacted granular fills, even when factored forces are considered.

Flowable fill should not be used for void grouting that will undergo significant forces such as keyways and connections. It should also not be used for voids exposed to traffic, since it has relatively low strength and freeze thaw durability. Care should be used with the use of flowable fill behind integral abutments. The high strength of the material (when compared to compacted soil) can generate very large passive earth pressures as the bridge expands. Based on this, compacted granular fill should be considered for integral and semi-integral abutments.

2.2. METALS

2.2.1. Steel

Steel is a material that historically has been used primarily for beam/girder elements. Steel does have properties that make it attractive for other elements as well. The key property of steel as a prefabrication material is that it has a high strength-to-weight ratio. This means that steel elements can be made lighter than an equivalent concrete element in most cases. This becomes an attractive option when the shipping and installation weight of an element becomes critical.

Steel elements should be specified as unpainted weathering steel where appropriate. This will eliminate the need for shop painting and field touch-up painting. In addition to beams and girders, steel can be used for lightweight decks. Lightweight decks such as orthotropic decks and grid decks can be used to reduce the size of substructure elements and allow for longer spans.

Steel can also be combined with concrete to form a composite element. The Massachusetts 93 Fast 14 project made use of prefabricated deck/beam elements that consisted of two steel beams fabricated with a composite concrete deck. Figure 2.2.1-1 shows one of these elements being installed. The use of steel elements on this project allowed for shipping and erection of large elements (over 100 ft long), and allowed for the re-use of the original substructure.

Figure 2.2.1-1. Modular Steel Deck/Beam Element Construction.

Other prefabricated elements for superstructures and substructures constructed with steel are discussed in chapters 4 and 5 of this manual.

2.2.2. Aluminum

The most common use of aluminum in bridges has been in bridge decks. Aluminum can also be used for beams and girders. Aluminum has excellent strength-to-weight ratios, which makes it ideal for reconstruction of bridges with limited load carrying capacities or for bridges where weight is critical (such as movable bridges).

As with lightweight concrete, aluminum bridge elements can be used to reduce the weight of the superstructure, thereby reducing the cost of substructures. Aluminum is often considered by designers to be a corrosion-free material. It is a highly corrosion-resistant material that can often be used in an un-coated condition; however, significant corrosion of aluminum can occur when it is placed in contact with other materials such as concrete. This can be resolved by isolating the concrete from the aluminum through the use of coatings or isolation materials.

To date, most bridge deck designs have been based on extrusion technologies. These designs are unique to each manufacturer; therefore, the systems fall under the category of proprietary products. Designers should work with the manufacturers and the owner agencies on the use of the products and the specific details. The design of aluminum structures should follow the requirements of the AASHTO LRFD Bridge Design Specifications [5].

2.3. POLYMERS

Polymer and polymeric material encompass very large, broad classes of compounds, both natural and synthetic, with a wide variety of properties. The most common polymers that are used in bridge construction are epoxy compounds. Epoxy in itself is akin to cement in concrete. It is the glue that joins other materials to form a structural material.

The following sections contain information on the most common form of polymers in use on ABC projects.

2.3.1. Fiber Reinforced Polymers

Fiber reinforced polymers (FRP) are a group of products that combine epoxy resins with high strength fibers such as glass fiber and carbon fiber. Glass fiber reinforced polymers (GFRP) are commonly used in reinforcing bars. This is due to the lower cost when compared to other FRP products. Other FRP products, such as carbon fiber polymers, are used in applications that require more strength.

FRP products include prefabricated deck panels, prefabricated box beams, double tees, stay-in-place deck forms, and arch elements. The FHWA maintains a website with significant information on FRP products and applications. This can be found at: www.fhwa.dot.gov/bridge/frp/frppract.cfm.

The American Composites Manufacturers Association (ACMA) is the world's largest trade group representing the composites industry. It is a good source of information on FRP products. The ACMA website can be found at www.acmanet.org.

The design of bridge structures using FRP products is not covered in the AASHTO LRFD Bridge Design Specifications [5]; however, there is significant design-related information in other documents. In 2009, AASHTO published the LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete Bridge Decks and Traffic railings [52]. The use of GFRP bars in bridge decks and precast bridge decks can extend the service life of structures, since GFRP is essentially an inert material.

FRP bridge decks can provide extremely lightweight elements that can be used to extend the service life of load restricted bridges such as historic trusses. FRP bridge decks are typically constructed with the pultrusion process. The American Society of Civil Engineers (ASCE) has recently published a pre-standard for LRFD design of pultruded FRP structures [53]. This document covers the design of common elements and connections.

2.3.2. Expanded Polystyrene

Expanded Polystyrene (EPS) is formed from polystyrene resins, beads, or granules. EPS is a product that was developed for other industries, where it is used for insulation and packaging. This material has found use in highway applications as lightweight fill in embankments and for forming concrete pours. An added benefit to this product is that it can be placed rapidly. EPS can be molded or extruded to form various shapes.

Initial specifications for EPS were based on the use of the product for insulation. Recently, an ASTM specification was adopted for use of the material in geotechnical applications. The current specification is ASTM D6817-11 Standard Specification for Rigid Cellular Polystyrene Geofoam [39]. Section 5.1.8.3 of this manual covers the use of this material for embankment fill, where it can aid in the construction of roadways over poor soil conditions.

EPS is commonly used to form areas for concrete placements. It is virtually inert; therefore, it can be left in place without concern for corrosion or degradation. It is also compressible; therefore, it can be used in areas that might experience thermal movement.

2.4. TIMBER

The use of timber as a bridge material is well documented. It has been used in bridges for centuries. This material, when properly treated with preservatives or coatings, can be a durable product for bridges. Timber is well suited for prefabrication through the glue lamination process. This process involves the gluing of layers of dimensional lumber to form larger shapes such as deck panels and girders.

The design of timber elements is covered in the AASHTO LRFD Bridge Design Specifications [5]. Materials and construction specifications are covered in the AASHTO LRFD Bridge Construction Specifications [29].

2.5. OTHER PROPRIETARY PRODUCTS

2.5.1. UHPC

Portions of the following text are taken from the previous FHWA ABC Manual [4]. It describes the material and its properties. Other information on the use of Ultra High Performance Concrete (UHPC) in prefabricated bridge projects is included in chapter 4 of this manual.

UHPC (sometimes referred to as Reactive Powder Concrete) is a class of cementitious products that is most commonly available as a proprietary product form. It combines high quality cement and stone products, along with either steel or organic fibers. The material can achieve very high compressive strengths of 18,000 to 33,000 psi. Another key feature of the material is that it can also achieve high tensile strength of approximately 1,000 psi depending on the type of fibers used. This leads to a material that is very ductile.

FHWA research has shown that under high tensile loading, the UHPC will develop tension cracks; however, the cracks are very small and tightly spaced, as opposed to wide intermittent cracks found in normal reinforced concrete elements that are exposed to high tensile stresses. The narrow cracks found in UHPC limit the transmission of water. UHPC also has very low permeability, which should lead to long service life. FHWA research has shown promising results for several applications (adjacent beam elements, precast full depth deck panel connections). The work focused on creep, shrinkage, resistance to chloride ion penetration, and freeze-thaw behavior.

This high performance material comes at a cost. The unit price for the material is significantly higher than conventional high performance concrete; therefore, replacement of high performance concrete with UHPC on major bridge elements is not financially viable at this time. UHPC has been successfully used on several bridges in the U.S. for girders and decks on an experimental basis.

Even with its high cost, UHPC has a high potential for use in ABC. Its high compressive and tensile strength makes it an ideal product for closure pour connections between adjacent elements. Ongoing testing at FHWA has shown that UHPC can develop reinforcing bars in very short distances. This opens up the potential to have very small closure pours that are fully reinforced. This is perhaps the best use of UHPC at this time, where designers can take advantage of the extremely high strength without incurring high overall costs for the bridge. This approach was studied under the SHRP2 Project R04, entitled “Innovative Bridge Designs for Rapid Renewal, ABC Toolkit” [85].

2.5.2. Thermoplastics

Rutgers University developed recycled plastic (also called “Thermoplastic”) technology for the U.S. Army, which is being used for bridges, railroad ties, fenders, etc. [69]. The recycled plastic specifically formulated with high density polyethylene or polypropylene coated glass fibers produces a sustainable engineering product.

The product is a green, sustainable, non-corrosive, and non-rotting construction material, requires low maintenance, reduces landfill dumping of plastic wastes, and is cost competitive. The product is well adaptable to ABC because of its suitable material characteristics such as lightweight prefabrication and flexibility of construction.

The properties of the material are as follows:

- Unit weight: 55 lb per cubic ft.
- Modulus of Elasticity: 250 ksi (static).
- Ultimate flexural stress: 3 ksi.
- Ultimate compressive stress: 2.5 to 4 ksi.
- Ultimate shear stress: Approximately 3 ksi.
- Coefficient of thermal exp.: 0.0000282 in/in/degrees Fahrenheit.

The ultraviolet degradation when fully exposed to sunlight is about 0.003 inch/year. The material is virtually impervious to moisture absorption, and its heat deflection temperature is between (+) 250 deg. Fahrenheit and (-) 250 deg. Fahrenheit. It is highly resistant to most acids and provides high abrasion resistance to salts and sand in marine environment. High factor of safety is used with respect to the ultimate stresses in order to lower the creep effect of the thermoplastic material.

Bridge applications of this material in the U.S. are as follows:

- Fort Leonard Wood, MO: 24 ft span for live load of 25,000 lbs (1998).
- Wharton State Forest, NJ: 56 ft total bridge length, HS-20 live load (2002).
- Fort Bragg, NC: Three bridges, M1 Abrams Tank (71 tons) (2009).
- Fort Eustis, VA: Two bridges, 38.5 (4 spans) and 84.08 (8 spans) ft long (see Figure 2.5.2-1), Cooper E-60 or 130 tons RR loads (2010).
- Birch Hill Road Bridge, ME: 13-ft span, HS-20 live load (2011).
- Onion Ditch Bridge, OH: 24-ft span, HS-20 live load (2012) (see Figure 2.5.2-2).

The bridges at Fort Bragg, Birch Hill Road and Onion Ditch were constructed entirely with recycled plastic (superstructure, substructure and including pilings). The major bridge elements were all prefabricated and shipped in standard trucks, and erected using lighter equipment that was easily available. The erection can even be made by county maintenance forces, as was done on the Onion Ditch Bridge in Ohio. Figure 2.5.2-1 is a photograph of the completed Fort Eustis Bridge. Figure 2.5.2-2 is a photograph of the Onion Ditch Bridge.

Figure 2.5.2-1. Prefabricated Thermoplastic Railroad Bridge.

Figure 2.5.2-2. Prefabricated Thermoplastic Roadway Bridge.

The Fort Eustis Bridges were the first thermoplastic railroad Bridges built in the world. Both the Birch Hill and Onion Ditch Bridges were the first vehicular bridges located in the U.S. highway system using recycled plastic components for both the superstructure and the substructure.

The recycled plastic products are well suited for marine applications, railroad ties and switch sets, board walks, culverts, jetties and piers, temporary bridges, and many other that demand rapid and sustainable products.

CHAPTER 3. CURRENT LRFD DESIGN PROVISIONS

This chapter includes information regarding the current AASHTO LRFD Bridge Design Specifications [5] and the applicability of these specifications to the design of ABC bridges using PBES.

3.1. PRECAST FULL DEPTH DECK UNITS

The design of decks and deck systems is covered in Section 9 of the AASHTO LRFD Bridge Design Specifications [5]. Design provisions for precast concrete full depth deck panels are included in this section under Article 9.7.5 (see Figure 3.1-1). This article covers certain features of precast full-depth deck panels, but not all design provisions.

AASHTO Article 9.7.5 contains certain limitations and provisions for deck panels. The article has a number of recommendations for deck panels, including the use of longitudinal post-tensioning combined with grouted shear keys to join the panels. The recommendation for the use of longitudinal post-tensioning was based on current practice at the time of the writing of this article. History has shown that these systems were performing well after a number of years in service. The minimum level of post-tensioning was not well defined when this article was written. At the time, there had not been significant research into this connection. A value of 250 psi (average effective prestress) was set based on research on the connection of precast concrete double tees [54, 55]. The research investigated the connection of flange edges of double tees. More recent research has verified that this level of post-tensioning is adequate for transverse deck panel connections [56, 57].

AASHTO Article 9.7.5 does not specifically address the design for composite action; however, the provisions do reference blockouts for shear connectors. This implies that standard design provisions for composite action are acceptable, and that the shear connectors can be placed within blockouts in the panel. This approach has also been verified through a number of research projects [8-27].

There are no specific design provisions for the internal reinforcing of full depth deck panels; therefore, it can be surmised that the provisions for CIP decks can be used for the design of precast deck panels. The PCI “State of the Art Report on Full-Depth Precast Concrete Bridge Deck Panels” [27] recommends this approach. AASHTO Article 9.7.2 covers the design of concrete bridge decks using the empirical design method. Article 9.7.2.4 specifically does not permit the use of the empirical design method for precast concrete decks.

9.7.5 Precast Deck Slabs on Girders

9.7.5.1 General

Both reinforced and prestressed precast concrete slab panels may be used. The depth of the slab, excluding any provision for grinding, grooving, and sacrificial surface, shall not be less than 7.0 in.

9.7.5.2 Transversely Joined Precast Decks

Flexurally discontinuous decks made from precast panels and joined together by shear keys may be used. The design of the shear key and the grout used in the key shall be approved by the Owner. The provisions of Article 9.7.4.3.4 may be applicable for the design of bedding.

C9.7.5.2

The shear keys tend to crack due to wheel loads, warping, and environmental effects, leading to leaking of the keys and decreased shear transfer. The relative movement between adjacent panels tends to crack the overlay, if present. Therefore, this construction is not recommended for the regions where the deck may be exposed to salts.

9.7.5.3 Longitudinally Post-Tensioned Precast Decks

The precast components may be placed on beams and joined together by longitudinal post-tensioning. The minimum average effective prestress shall not be less than 0.25 ksi.

The transverse joint between the components and the block-outs at the coupling of post-tensioning ducts shall be specified to be filled with a nonshrink grout having a minimum compressive strength of 5.0 ksi at 24 hours.

Block-outs shall be provided in the slab around the shear connectors and shall be filled with the same grout upon completion of post-tensioning.

C9.7.5.3

Decks made flexurally continuous by longitudinal post-tensioning are the more preferred solution because they behave monolithically and are expected to require less maintenance on the long-term basis.

The post-tensioning ducts should be located at the center of the slab cross-section. Block-outs should be provided in the joints to permit the splicing of post-tensioning ducts.

Panels should be placed on the girders without mortar or adhesives to permit their movement relative to the girders during prestressing. Panels can be placed directly on the girders or located with the help of shims of inorganic material or other leveling devices. If the panels are not laid directly on the beams, the space therein should be grouted at the same time as the shear connector block-outs.

A variety of shear key formations has been used in the past. Recent prototype tests indicate that a "V" joint may be the easiest to form and to fill.

Figure 3.1-1. AASHTO LRFD Bridge Design Specifications – Article 9.7.5.

3.2. PARTIAL DEPTH DECK PANELS

AASHTO Article 9.7.4.3 includes provisions for the design of partial depth deck panels. The specifications refer to this system as "Concrete Formwork," which technically is a misnomer. The term "partial depth deck panels" is a more accurate description of these elements. The panels do support a layer of concrete that is placed in the field, but they also provide structural resistance for positive flexure between the girders.

3.3. DESIGN OF OTHER DECK SYSTEMS

The designs of other prefabricated deck systems are also included in the AASHTO LRFD Bridge Design Specifications [5]. The following sections cover typical prefabricated systems.

3.3.1. Metal Grid Deck Panels

Metal grid decks have been in used for many years. These prefabricated elements provide a lightweight deck that is good for load restricted bridges and moveable bridges. These modular systems are also appropriate for ABC projects, since the deck can be prefabricated.

There are several different types of metal grid decks in use, including open grid, grids filled with concrete, and grids that are partially filled with concrete.

The design of metal grid decking systems is covered in Section 9.8.2 of the AASHTO LRFD Bridge Design Specifications [5].

3.3.2. Timber Decks

Prefabricated timber deck panel systems have been developed by the United States Department of Agriculture (USDA) Forest Products Laboratory. The USDA has published manuals and standards for prefabricated laminated timber decks [58]. These systems typically consist of glue laminated deck panels that are either bolted or post-tensioned together to form the deck of the bridge.

The design of timber deck systems, including the design for post-tensioning, is covered in Section 9.9.4 of the AASHTO LRFD Bridge Design Specifications [5].

3.3.3. Aluminum Decks

Aluminum decks are not as common as other deck systems; however, they are a lightweight deck system that is good for load restricted bridges and moveable bridges. These modular systems are also appropriate for ABC projects, since the deck can be prefabricated.

The AASHTO LRFD Bridge Design Specifications [5] contain provisions for orthotropic aluminum deck systems in Section 9.8.4.

At this time, there are not many producers of aluminum bridge decks. The detailing of the decks is controlled by the manufacturing process of the deck system. For this reason, designers should contact the aluminum industry for more information on aluminum decks (www.aluminum.org).

3.3.4. Fiber Reinforced Polymer Decks

The AASHTO LRFD Bridge Design Specifications [5] do not contain provisions for the design of FRP decks. To date, most FRP deck designs have been developed through research projects or through consultation with the manufacturers. In 2009, AASHTO published the LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete Bridge Decks and Traffic Railings [52].

3.4. ADJACENT BEAM/DECK ELEMENTS

The design of adjacent beam elements follows the applicable design procedures of any beam element. The following are the sections of the AASHTO LRFD Bridge Design Specifications [5] that are applicable to various beams designs:

- Steel Beams: Section 6
- Concrete Beams: Section 5
- Prestressed Concrete Beams: Section 5
- Timber Beams: Section 8

The design of decking systems that are integral with the adjacent beam element would typically follow the provisions in Section 9. The distribution of loads to these elements (dead load and live load) and the design of the deck portion of the elements are described in chapter 4 of this manual.

3.5. MECHANICAL CONNECTIONS—REINFORCING STEEL

The use of mechanical connections is an important part of designing bridges with precast elements. There is a need to connect the elements in the field with durable connectors that can provide adequate resistance to loads.

The AASHTO LRFD Bridge Design Specifications [5] includes provisions for mechanical reinforcing connections. Mechanical connections are akin to a bar splice in a concrete element; therefore, the provisions for mechanical connectors are included in Section 5.11.5 – Splices of Bar Reinforcement. Article 5.11.5.2.2 specifically covers the requirements for mechanical connectors. These devices are required to develop 125 percent of the specified yield strength of the connected bar. This does not represent the full strength of the bar, but an acceptable level of resistance for most concrete elements. The ACI Building Code [46] contains provisions that are similar to AASHTO, but it also contains provisions for mechanical connectors that can produce a higher level of resistance. The higher level connector, or Type 2 connector, is required to develop 100 percent of the specified minimum ultimate strength of the connector. Future provisions, prefabricated substructure elements may include provisions for this type of connector, especially for bridges located in moderate to high seismic zones.

3.6. POST-TENSIONING

Post-tensioning is used to prestress concrete structures to counteract the tension stresses caused by applied loads. Post-tensioning can be used in bridges constructed with CIP concrete and bridges constructed with precast concrete (segmental construction). The use of post-tensioning for ABC projects built with prefabricated elements is akin to segmental construction in that the elements are joined together with narrow un-reinforced grouted connections that have post-tensioning tendons passing through.

The AASHTO LRFD Bridge Design Specifications [5] contain numerous provisions for post-tensioned structures. Provisions for element design as well as tendon anchorage zone design can be found in Section 5.

3.7. SEISMIC DESIGN

Chapter 5 of this manual contains information on the design of prefabricated bridges for seismic events. In general, the seismic design of prefabricated bridges is the same as for conventional bridges. The detailing of internal reinforcement for seismic forces should follow the typical AASHTO LRFD design provisions for seismic design in Section 5.10.11. The design also can be completed using the provisions of the AASHTO Guide Specifications for LRFD Seismic Bridge Design [44], if specified by the owner agency.

There is one seismic design specification that has been the subject of concern with regards to precast elements. Article 5.10.11.4.1c calls for “end region” reinforcement to extend from columns into the connected elements by a distance that is greater than: the maximum cross-sectional dimension of the column, one-sixth of the clear height of the column, or 18.0 inch.

End region reinforcement is primarily transverse reinforcement used for confinement of column ends. This reinforcement is carried into the connected elements to limit the potential for the development of large spalls. Many designers specify spiral reinforcement for columns. Some designers have the interpretation that ONLY spiral reinforcement is allowed, and that it is impossible to have a spiral through a joint. The latter part of this interpretation is true; however, the first part of this statement is not. Both the AASHTO LRFD Bridge Design Specifications [5] and the AASHTO Guide Specifications for LRFD Seismic Bridge Design [4] allow the use of individual ties for seismic design. In fact, many designers in seismic regions prefer the use of individual ties or hoops. The concern is that the entire spiral ties could become unraveled if one leg were to break during a seismic event, while the failure of one tie would result in minimal loss of lateral confinement. If hoops or spirals are used, the ends of the bars must be properly anchored within the element core. The detailing of hoops and ties is specified in Article 5.10.11.4.1d of the AASHTO LRFD Bridge Design Specifications [5]. The AASHTO Guide Specifications for LRFD Seismic Bridge Design [4] has slightly different requirements for detailing of ties in Section 8.

3.8. LRFD LOAD RATINGS

The current procedures for inspecting and evaluating bridge elements are included in the AASHTO Manual for Bridge Evaluation (MBE) [59]. This manual covers bridge inspection methods as well as load rating procedures using the Load and Resistance Factor Rating procedure.

Article 4.8.4.2 of the MBE covers the inspection recommendations for precast concrete full depth deck panels. In general, these decks are inspected in the same way as a CIP concrete deck. Special attention is given to the anchorage zones of post-tensioned decks and the connections between panels. Inspectors are still directed to look for spalls, cracking, leaching, scaling, or other evidence of deterioration.

The basis of most ABC projects built with PBES is to emulate conventional construction. Chapter 4 of this manual discusses that the design of most prefabricated elements follows the same provisions as with conventional construction. The major change in the design and detailing process is the connections between the elements. Based on this, a rating of a prefabricated element would follow normal rating procedures. The main elements in a bridge structure that are routinely rated are the beams and girders. Beams and girders built as part of an ABC project would have the same rating procedure as conventional construction.

Most prefabricated elements in use today are decks, piers, walls, and abutments. In general, these elements would not normally be rated unless the owner has reason to believe that the load capacity of the elements may govern the load capacity of the entire bridge. If prefabricated elements do need to be rated, the rating procedure would follow the same procedure as a conventional bridge, since the original design was based on an emulative approach.

CHAPTER 4. DESIGN OF SUPERSTRUCTURE ELEMENTS

This chapter will cover the design of Prefabricated Bridge Superstructure Elements. The design of bridge systems will be covered in chapter 11. The bridge elements covered in this chapter are the most common ones listed in the FHWA definitions listed in Appendix A of this manual. Other less common elements can be designed with careful planning and the application of general engineering principles. Many of the design parameters in this chapter are applicable to these other elements.

For many types of PBES projects, the general design of the elements is the same as for conventional construction. There are subtle differences in the design of certain elements and there are certain items that are not well covered in the AASHTO LRFD Bridge Design Specifications [5]. This chapter offers recommendations for the design of elements that are not well covered. In cases where the AASHTO specifications are not clear, the design recommendations are based on a variety of sources. These sources include research data, engineering judgment, and the concept of emulative design.

4.1. ADJACENT BEAM/DECK ELEMENTS

Prefabricated beam elements are composed of two types: "deck" beam elements and "full-width" beam elements. Deck beam elements eliminate conventional on-site deck forming by placing the deck beam element flange to flange for I-shapes and web to web for box shapes. This approach offers several advantages over conventional CIP concrete work:

- They provide an instant safe work platform over the majority of the bridge for construction and inspection staff.
- They reduce the need for temporary safety railings and fall protection on all beams (fascia beams only).
- They reduce or eliminate the need for on-site deck forming and the subsequent form stripping.
- They reduce the impacts to the traveling public by reducing construction time.
- They reduce the need to work near live traffic.

The AASHTO LRFD Bridge Design Specifications [5] include provisions for some adjacent beam elements such as adjacent precast concrete box beams; however, some design considerations are not well covered. The following sections describe some of these design considerations.

4.1.1. Live Load and Dead Load Distribution

One of the most important factors in the design of a beam is the amount of dead and live load applied to each beam. The basic premise of the majority of beam designs in the LRFD specifications is that a typical bridge superstructure can be simplified to a "line girder" analysis, or a 2D problem. Line girder analysis requires the distribution of loads to girders by means of common statistics or empirical equations.

The distribution of dead load can vary depending on the details and type of dead load in question. The distribution of the self-weight of a beam is simple, but the distribution of composite dead loads applied after the beams are connected with a composite deck is more difficult. Once beams are connected, they respond to load as a unit. Finite, 3D element models can calculate the response of the entire bridge, but the models are difficult and expensive to build, and prone to modeling errors. Also, in the majority of cases, the increase in accuracy is not required. The reality is that bridge design has been streamlined to reduce design costs and provide uniformity in results. Bridge design is streamlined through the development of empirical equations and simple rule of thumb approaches. For instance, on typical beam slab bridges, the composite dead load applied to the superstructure is distributed equally to each beam based on the assumption that the deck and cross frames force the bridge to respond as a unit. A portion of Article 4.6.2.2 of the AASHTO LRFD Specifications states:

"*Where bridges meet the conditions specified herein, permanent loads of and on the deck may be distributed uniformly among the beams and/or stringers*."

A detailed finite element analysis typically shows that the fascia beam carries a larger portion of composite concrete barrier load but the same analysis typically overestimates the live load on the fascia beam. Simplified methods of load distribution do not reduce safety and allow uniform beam details for all girders. This reduces costs and allows for future bridge widening.

Many state agencies have state specific guidance on the distribution of composite dead loads such as barriers and sidewalks. Designers should consult and follow any state specific guidance for the design of adjacent beam/deck elements.

Live load distribution on typical beam/girder bridges is determined through the use of empirical equations. These equations have been changed and refined over the years as the bridge design specifications are updated. Additional refinements are also likely in the future. The following sections offer specific guidance for load distribution of common PBES beam/deck elements.

4.1.1.1. Double Tees

Designers use double tee sections in parking structures due to their efficiencies and ease of construction. The double tees used in garage structures typically do not have the capacity to accommodate modern highway loadings. They are usually designed for passenger car loading. Several states use double tee and triple tee shapes for county and rural bridges; however, their use is rare. This is because these sections have relatively limited span range capabilities. Figure 4.1.1.2-1 depicts a typical double tee deck/beam bridge built with Florida Double Tee beams.

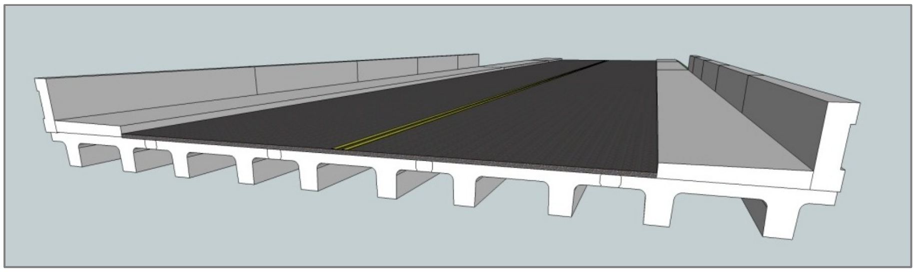


Figure 4.1.1.2-1. Double Tee Bridge.

In recent years, the PCI Northeast Bridge Technical committee has developed a new double tee section called the NEXT Beam, which stands for the Northeast Extreme Tee. The “extreme” designation refers to the span capabilities of this robust section. The beam is currently configured in two ways. The first uses the top flange of the beam as a form to support a CIP concrete deck. This is referred to as a NEXT F beam. The F stands for “form.” This element is considered an ABC method since the forming and subsequent stripping of the concrete deck slab forms is eliminated, saving time. Figure 4.1.1.2-2 depicts a NEXT F beam bridge. The NEXT F beam is cast in widths ranging from 8 ft to 12 ft, allowing numerous configurations that precisely match most bridge cross sections.

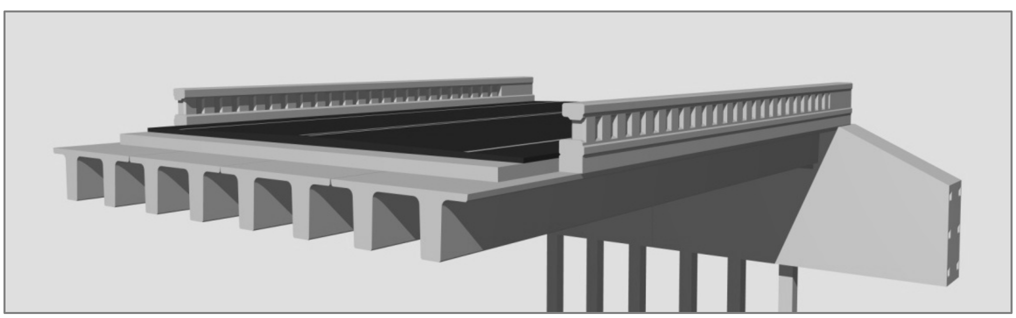


Figure 4.1.1.2-2. NEXT F Beam Bridge.

The second uses the NEXT beam in a beam/deck configuration, similar to what is shown in Figure 4.1.1.2-1. This is referred to as a NEXT D Beam. The D stands for “deck.” This beam is fabricated in widths ranging from 8 ft to 10 ft wide.

The PCI committee has been studying the design approach for this section, including the dead load and live load distribution methods. This work includes limited research using finite element studies and field verification of behavior. The following outlines the current recommendations of the PCI Northeast Bridge Technical committee.

The distribution of dead load in double tee bridges is accurately estimated by assuming that each stem and tributary flange is an individual stringer/beam. By using this assumption, the designer can use the guidance of the AASHTO LRFD Article 4.6.2.2 or the requirements of the state bridge manual (if applicable).

The AASHTO Table 4.6.2.2.1-1 shows typical bridge sections that are applicable to the live load distribution equations in subsequent tables. The table depicts figures of two similar structures, a double tee bridge and a deck bulb tee bridge. Figure 4.1.1.2-3 shows the two sections.

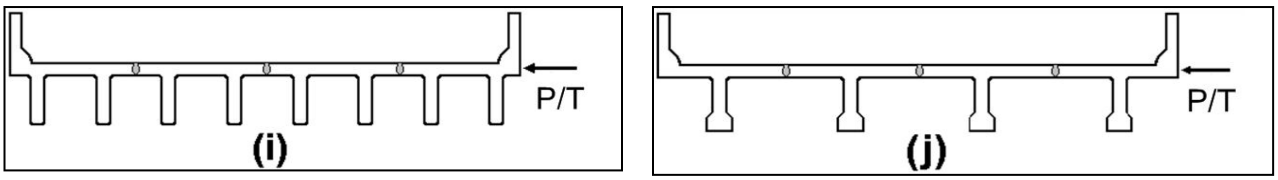


Figure 4.1.1.2-3 Portion of AASHTO Table 4.6.2.2.1-1

There are two methods of calculating the distribution factors for these two bridge cross sections. One with beam flange connections designed to resist shear only. The other uses a connection for shear and moment allowing the beams to act as a unit. The shear and moment connection normally uses transverse post-tensioning, or a reinforced concrete or reinforced grout closure pour.

When the beams are connected with a shear connection only, the AASHTO equations for live load distribution are straightforward. When the beams are connected to act as a unit, issues arise with respect to the use of the AASHTO equations. The two similar bridges shown in Figure 4.1.1.2-3 use the same live load distribution factor equations. The definition for S is “spacing of beams or webs.” The spacing S could be defined as the spacing from centerline of the double tee to the centerline of the adjacent double tee or it could be defined as the spacing between the webs of a single double tee.

The approach the developers of the NEXT beam recommend is to treat the double tee in a similar manner as the deck bulb tee, treating each stem as an independent beam. The spacing term would be the stem spacing. The AASHTO equations for live load distribution are for the percentage of lanes per beam. The live load distribution factor is calculated for each stem, and the answer doubled for application to the entire double tee section. An example of this calculation is included in chapter 12. This method results in a more conservative distribution of live load to the double tee girder, when compared to the approach that uses the fill double tee spacing.

Figure 4.1.1.2-4 includes text outlining the current PCI Northeast Bridge Technical Committee recommendations for live load distribution. These recommendations are based on the assumption that the flanges are connected to act as a unit.

Load Distribution for NEXT Deck Beams

- Parapet Dead Load
- If a concrete barrier is placed after the beam flange connections are made, the distribution of the barrier load can be treated as a composite dead load. The following procedure would then be appropriate.
- The goal is to have the same number of strand in each stem
- Treat each stem as an individual stringer
- Calculate the composite dead load on each stem using normal design procedures for composite dead loads in stringer bridges.
- Add the composite dead loads from each stem together and apply them to the design of the entire beam.
- If the barrier is placed before the longitudinal beam flange connections are made, the dead load of the parapet would need to be applied to the fascia beam.
- Wearing surface and sidewalk loads
- Treat these the same as a composite load (see above)
- Live Load:
- Treat each stem as an individual stringer and use AASHTO Cross Section (i) (Table 4.6.2.2.1-1).
- Calculate the live load distribution factor (LLDF) on each stem according to

Figure 4.1.1.2-4. Recommended Design procedure for NEXT Deck Beam. (Source: PCI Northeast Bridge Technical Committee).

The design of NEXT F beams follows a similar approach. The difference is that the beam supports a composite concrete deck; therefore, AASHTO Cross Section "k" would seem appropriate. Figure 4.1.1.2-4 shows AASHTO Cross Section k.

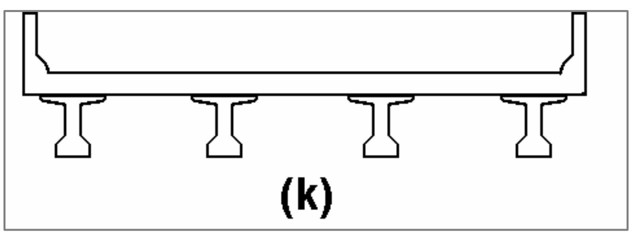


Figure 4.1.1.2-5. Portion of AASHTO Table 4.6.2.2.1-1.

The same issue with regard to the beam spacing term S arises in this scenario also. The PCI Northeast Bridge Technical Committee recommends using the same approach outlined in Figure 4.1.1.2-4 for the design of the beams shown in Figure 4.1.1.2-2.

4.1.1.2. Deck Bulb Tees

The calculation of composite dead load and live load distribution for deck bulb tees is covered by the AASHTO LRFD Bridge Design Specifications [5]. Section 4.6.2.2 covers the design of beam-slab bridges. Deck bulb tees fall under the design provisions for cross section “j” as shown in Table 4.6.2.2.1-1.

If a moment connection is made at the deck edge connection, the beams act as a unit and the appropriate equations should be used. If a simple shear connection is used, the live load distribution equations for beams “connected only enough to prevent relative vertical displacement at the interface” is appropriate.

The cross slope of the roadway can have an impact on the design of a deck bulb tee beam. Several approaches can be used to accommodate the roadway cross slope:

- The thickness of the top flange of the deck bulb tee can be varied across the beam width to accommodate the deck cross slope. The beam properties and beam response must be evaluated during design. The variable thickness top flange increases the section properties of the beam. The centroid of the beam shifts vertically and horizontally. The beam weight is increased. All these affect the internal stresses in the beam. The lifting methods must also be evaluated since the centroid will not be centered on the beam web.

- For narrow bridges, the deck bulb tees can be placed flat in the transverse direction and the crown can be made up in the overlay (if used).

- The deck bulb tee beams can be canted such that the entire beam is sloped to meet the cross slope of the roadway. This affects the internal stresses in the beam. Erection and stability concerns could arise for bridges with large cross slopes. Designers must carefully consider these effects and check the beam for all appropriate loads.

4.1.2. Transverse Design of Beam/Deck Elements

The design of beam/deck elements for major axis flexure and shear is straightforward once the dead load and live load participation of each beam is calculated as described in the previous sections. The portion of the design that is not covered well is the transverse design of the beam flange to flange connections. The design engineer needs to make certain assumptions for the design of this connection.

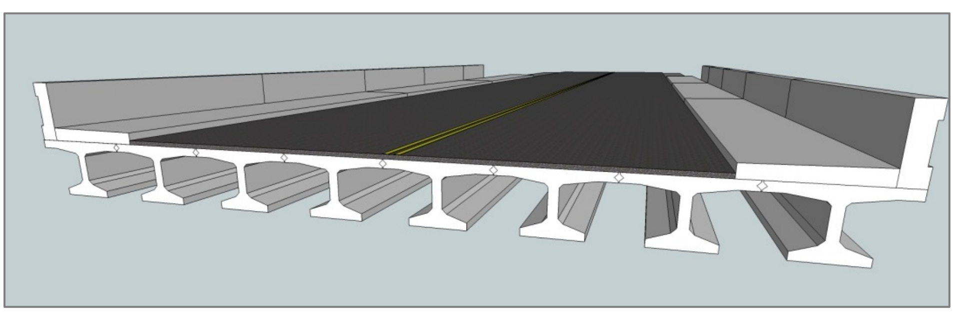


Figure 4.1.1.3-1. Typical Deck Bulb Tee Bridge.

Some state agencies have assumed that the connection is a simple shear connection, which is within the purview of the AASHTO specifications. These connections are typically made with a grouted shear key combined with welded ties spaced at approximately five ft on center. These connections have been used in several states, including Texas and Washington. There is concern within the bridge community that the connection is not robust enough to accommodate high-volume truck loadings; therefore, these bridge types have been limited to rural roadways with low truck traffic volumes.

4.1.2.1. Beam/Deck Edge Connections—Mild Reinforcement

Recently, research has been completed on full moment connections using a small closure pour of concrete or grout combined with mild reinforcement for this deck connection [7]. Various methods of connecting the deck edges were studied, including hooked bars and headed bars. The study also included a limited parametric study of the forces acting on the connections.

One approach for the design of this connection is to treat the top flange of the beam/deck element as a CIP concrete deck. The moments and shears can then be calculated using the "Strip Method" as specified in AASHTO Section 4.6.2.1. An example of this calculation is included in chapter 12. This method is thought to produce conservative transverse bending moments in slab (based on the fact that the AASHTO "Empirical Method" of deck design (Article 9.7.2) produces deck slabs with significantly less reinforcing).

4.1.2.2. Beam/Deck Edge Connections—Post-Tensioning

Another method for designing beam/deck connections is to use transverse post-tensioning. The AASHTO LRFD Bridge Design Specifications have guidance for this design method. The commentary in Article 4.6.2.2.1 recommends a post-tensioning stress of 250 psi for this situation. The Florida Double tee was developed with this amount of post-tensioning and has been found to perform adequately.

4.2. FULL DEPTH PRECAST CONCRETE DECK PANELS

The precast concrete full depth deck panel has been one of the most widely used prefabricated elements in the U.S. Full depth deck panels differ from partial depth panels in that they make up the entire deck section. Figure 4.2-1 shows a schematic deck panel layout on a steel girder bridge.

Significant research has been completed on the subject [8 - 27], including the connection to the beams and the connections between the panels. PCI recently published a report entitled "State of the Art Report on Full Depth Precast Concrete Deck Panels" [27]. This report contains significant information on the design of deck panels, including a design example. The following text contains recommendations for the design of precast concrete full depth deck panels based on this report and other research.

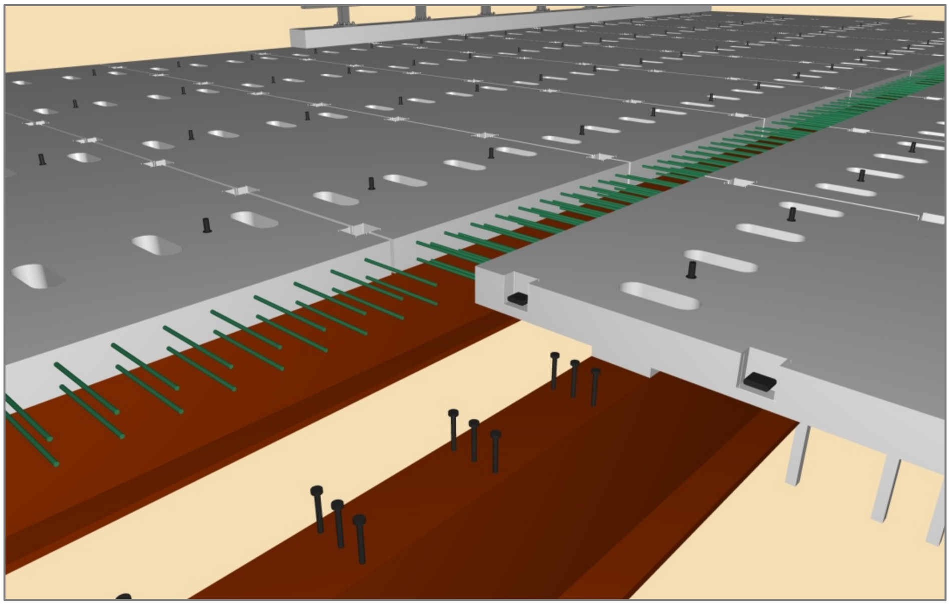


Figure 4.2-1. Precast Concrete Full Depth Deck Panels.

This section covers the design of typical full depth precast concrete deck panels. Prior to the start of a design, the engineer needs to establish certain features of the design. The PCI Northeast Bridge Technical Committee has developed answers to frequently asked questions about full depth deck panels. Portions of the text in this document have been taken from the PCI document with permission.

Some agencies specify overlays for deck panel projects. The overlays serve two purposes. First, they provide a final tolerance adjustment to the top surface of the panels. Even with the best fabrication and construction practices, the final riding surface can be rough. The installation of an overlay will provide a smooth riding surface.

Overlays can also be used to provide an additional layer of protection for the deck. Several overlay systems are in use in the U.S. The most common is a bituminous concrete pavement with a high-quality membrane waterproofing system. Some states are using thin concrete overlays. The Virginia DOT has developed a rapid setting thin latex modified concrete overlay that can be installed overnight. Both of these systems can provide a significant measure of deck protection as well as a smooth riding surface.

Another option is to use a thin bonded polymer overlay system. These systems are lightweight and provide protection from de-icing salts. This is normally used in conjunction with profile grinding of the deck in order to get a smooth riding surface. Tighter fabrication tolerances are recommended for this option since grinding is normally limited to approximately one-quarter inch. Match casting or line casting of the panels is recommended to achieve this level of quality.

4.2.1. Geometry of Full Depth Deck Panel Designs

There are no national standard deck panel sizes and shapes; therefore, different deck configurations can be accommodated with special pieces if necessary. The following sections contain information on typical bridge geometric configurations:

Skews: Deck panels can be skewed to accommodate moderate skew configurations. The AASHTO LRFD Bridge Specifications limit the maximum skew angle for main deck reinforcement to 25 degrees. This is a reasonable maximum skew angle for deck panels as well. If post-tensioning is used to join the deck panels, the designer should account for the skew of the tendons with respect to the angle of the connection. This will cause a reduction in the perpendicular stress across the connection brought on by skew.

Roadway Crowns: Roadway crowns complicate the fabrication process. Crowns can be cast into the panels, however, this requires a special form that will be custom to each project (as opposed to a simple flat form that can be reused), which tends to increase costs. If the panels are transversely prestressed or post-tensioned, the roadway crown complicates the matter even further. Section 4.2.4 has more information on this matter.

The most cost-effective way to accommodate crowns is to use a simple closure pour at the crown. The connection can be made with lapped bars or hooked bars for narrower gaps. Closure pours allow for more adjustment of the panels in the field during placement.

Horizontal Curves: Curves can be accommodated through the use of trapezoidal deck panels. The deck panel connections are normally run radially (perpendicular to the bridge baseline). If post-tensioning is used to join the deck panels, the designer should account for the loss of post-tensioning caused by the friction in the ducts brought on by the curvature. For most curves, the ducts can be run on tangents within each piece. Article 5.9.5.2.2 of the AASHTO LRFD Bridge Design Specifications [5] covers the friction losses of post-tensioning tendons with angle points.

Vertical Curves: Vertical curvature is normally accommodated by small angle points at each transverse connection. Deck overlays or deck grinding can normally accommodate the slight variation in the profile of the deck panels. For extreme vertical curvature combined with thin overlays (or bare decks), it may be necessary to profile the deck panels or grind the deck to the desired profile.

Splayed Framing: Splayed girder configurations can be accommodated with deck panels. The layout of the girder connection blockouts needs to be carefully coordinated with the deck framing during fabrication. Post-tensioning ducts (if used) can also be splayed. This type of geometry can lead to costly deck elements. Designers should account for this complexity in project estimates.

Superelevation Transitions: Superelevation transitions in bridge decks can be problematic with any decking system. The deck surface for a transition region is a warped section. The use of flat panels will most likely result in unacceptable geometry at the connections and lead to problems with post-tensioning systems (if used). For this scenario, it is suggested that match casting or line casting (match casting with small gaps between the pours) be used. This requires the fabrication of the entire bridge (or sections of the bridge) to be laid out in the bed with the required profile built into the forms. This method of fabrication is very expensive; therefore, designers should coordinate with the highway designers in order to attempt to locate the transition off of the bridge if possible.

Another option for superelevation transitions is to detail the panels with a normal cross slope and make up for the superelevation in the overlay system. This option will add weight to the bridge; therefore, the supporting structure should be designed to support the extra loads.

4.2.2. Size of Deck Panels

There is no standard panel size in the U.S. Maximum panel dimensions are a function of shipping and handling. Eight- to 10-ft wide panels are common. A reasonable maximum length of panel is 40 ft. The maximum length of panels is somewhat controlled by the length of flatbed trucks used to haul the panels. Longer panels may require the use of special cradles in order to prevent cracking during shipping, and also will require the use of special lifting hardware that may include spreader beams and multiple slings. If the bridge deck being constructed is wider than 40 ft, a simple closure pour is suggested between adjacent panel groups.

4.2.3. Minimum Deck Panel Thickness

The minimum thickness of precast concrete deck panels can be the same as an equivalent CIP concrete deck; however, there are certain factors that may require a thicker deck. The strength of the section is not normally the issue. Initial deck grinding and detailing of the internal reinforcing can lead to a reduction in cover. Several agencies require an additional ½-inch of cover when initial grinding is specified.

If post-tensioning is used, the anchorage assembly should be installed with the required minimum concrete cover over the anchorage assembly. Several manufacturers make special anchorages for slabs. The designer should investigate the layout of the anchorage assembly to determine the minimum panel thickness. The layout of the required general zone and local zone reinforcing layout should also be checked.

The layout of the post-tensioning duct and the internal reinforcing can also lead to detailing problems. The ducts are typically placed at mid-depth in the deck panel; however, it is reasonable to lower the duct slightly to accommodate reinforcing conflicts brought on the larger top cover.

Two types of duct are currently being used in the U.S. The first is a plastic oval duct that is approximately 1 inch tall by 3.5 inches wide. This duct can fit within most deck panels without any difficulties. There have been problems with sagging and compressing of the oval duct during casting, which leads to problems in construction during the strand installation process. Many agencies are moving toward a 2-inch diameter round duct. Round ducts do not have the same problems with deformations and compression. They also greatly facilitate tendon installation. The designer should check the layout of the duct and the internal reinforcing.

For a post-tensioned deck panel with 2 inches of top cover and 1inch of bottom cover (typical in northern environments), the minimum panel thickness is approximately 8.5 inches based on currently available anchorage assemblies and a 2-inch diameter round duct. Thinner slabs can potentially be used where the specified concrete cover is less.

Decks built without post-tensioning can potentially be thinner. The layout of the internal reinforcing and any potential hardware should be considered.

The minimum deck panel thickness based on strength is a function of several factors:

- Beam spacing and skew.
- Concrete strength.
- Overhang design (see Section 4.2.5 below).

4.2.4. Design of Main Reinforcing

The recommended design procedure for precast full depth deck panels is similar to the design of a CIP concrete deck. Moments and shears can be calculated using the "Strip Method" as specified in AASHTO Section 4.6.2.1. Since this method provides "per foot width" moments and shears, the main reinforcing (perpendicular to the supporting beams) can be designed for these moments, even with the presence of transverse grouted key connections.

The PCI State of the Art Report recommends the following limit state checks for the design of the main reinforcing:

- **Strength I:** For strength of the section under factored dead and live loads caused by normal vehicular traffic.
- **Service I:** For checking crack control in reinforced concrete sections.
- **Service III:** For checking tension in prestressed concrete decks with the objective of crack control.

Large, thin deck panels can present a challenge with respect to shipping and handling; however, these challenges can be addressed. Large panels may require more complex lifting hardware such as spreader beams and multiple slings. Typically, design engineers are not responsible for the calculation and checking of shipping and handling stresses. This is due to the fact that the designer does not know the exact methods for handling the panels. Shipping and handling procedures fall under the realm of "contractor means and methods." The project specifications should require that the contractor use a licensed professional engineer to prepare and submit lifting and handling calculations based on the anticipated lifting methods. The PCI Design Handbook for Precast and Prestressed Concrete [28] contains guidance on lifting and handling methods and calculations and should be specified for all precast concrete elements.

One of the most critical loadings for deck panels is immediately after the panel is set. Depending on girder camber and setting procedure, the panels may be cantilevered over the end girder or spanning an interior girder. A well-defined setting procedure developed by the contractor can mitigate this but will not eliminate it. The designer may wish to establish a requirement for analysis of the panel with the loss of a support location to cover this scenario. Section 4.2.10 contains more information on this subject.

It is possible to design the deck using prestressed concrete. It is relatively easy to produce a flat panel using pre-tensioned prestress. There can be difficulties with the design of panels that are not flat. On some occasions, designers have detailed precast panels with crowns to create the bridge cross slope. This is not an issue for a panel designed with mild reinforcing, but prestressing of a crowned panel can lead to production problems. An alternative way to achieve cross slope in a deck panel is to crown the top surface only. This may be possible for narrow bridges with minor cross slopes. Most designers accommodate roadway crowns by detailing two panels on each side of the crown point and connecting them with a closure pour at the crown.

Post-tensioning can also be used for deck panel designs. Many segmental concrete box girders are built with transverse post-tensioning in the decks. This is used to produce large cantilevers that are common in these bridges. The cost effectiveness of transverse post-tensioning on a typical stringer bridge is questionable; however, it may be viable for bridges with wide beam spacing and/or large overhangs.

4.2.5. Overhang and Barrier Design

The design of the overhang region of a deck panel can be one of the most challenging aspects of the deck design. Often, the maximum bending moment in a deck occurs at the base of a concrete barrier. The problem with this location is that it is very close to the edge of the panel. If prestressing is used for the main reinforcing of the panel, the strand will most likely not be properly developed at the base of the barrier. This will require the addition of mild reinforcing that is properly anchored.

All barriers used on projects built with Federal funding need to be crashworthy (proven through full-scale crash tests). This has led to most agencies relying on a limited number of standard barrier designs. The AASHTO LRFD Bridge Design Specifications contain several test levels of barriers and railings. Many agencies are moving toward higher test levels in order to improve safety and prevent vehicles from over-topping the barriers. Many designs have been completed for test level four barriers supported by with 8.5-inch-thick panels. Higher test level barriers can have a dramatic impact on the design of deck overhangs.

The FHWA has developed complete LRFD design examples for two bridges [74, 75]. These examples include a detailed set of calculations for the deck overhang. The overhang design is based on the premise that the barrier will fail before the deck overhang fails. This means that the higher test level barriers exert larger forces on the deck. This may lead to large reinforcing bars that are closely spaced, which can cause several potential problems with respect to precast deck panel designs.

- The layout of the transverse bars needs to accommodate pockets used for shear connectors. These pockets lead to the inevitable non-uniformity of bar spacing. Typical designs are based on a “per foot” analysis. The required reinforcing is then adjusted to pass around the pockets. Large overhang reinforcing can lead to the need to bundle bars in the overhang region. If the reinforcing layout gets too congested, the designer should consider thickening the panel.

- The large amount of reinforcing can lead to sections that are over-reinforced. The AASHTO specifications allow for highly reinforced sections; however, it comes at a cost. The phi factors need to be adjusted for this scenario, which reduces the section resistance. The solution to this problem is to thicken the section.

- It is important to properly develop the overhang bars. Some designers attempt to develop the bars within the precast deck panel. The bend radii of larger bars can be difficult to fit within the panel. The solutions to this problem are to hook the bars up into the barrier or curb, rotate the bars so that the hook is within the panel, or use smaller bars with smaller bend radii (this may not be an option). A thicker slab will also help to alleviate this issue.

The Utah DOT has used precast concrete deck panels with integral barriers cast in the fabrication facility. This requires a connection in the barrier at every deck panel connection. There are two ways to accommodate this connection. First, the reinforcing in the barrier can be designed as an “end zone,” using the same reinforcing that is typically used at the end of the bridge. It should be noted that if the barrier is not continuous, the designer cannot take advantage of the continuous barrier effective flange width factor specified in AASHTO Article 4.6.2.6.1.

Another issue with this approach is that the end zone reinforcing significantly increases the moment capacity at the base of the barrier, which increases the required deck reinforcing needed to confine the barrier failure mode to within in the barrier. Refer to A13.4.2 in the LRFD Specifications [5] for more information on this issue.

The Utah DOT has also used a detail to connect the ends of the barriers within each panel. This detail is based on a roadside barrier design that was successfully crash tested. Figure 4.2.5-1 shows the detail. As with many crash tested elements, there was no formal design used for the development of this joint. This detail, as well as other details for precast full depth deck panels, can be found at the Utah DOT ABC website (http://www.udot.utah.gov (search ABC)).

4.2.6. Design of Longitudinal Post-tensioning at Transverse Connections

Longitudinal post-tensioning is used to provide connections between individual deck panels on bridges with girders that are parallel to traffic. This method of connection has proven to provide very durable bridge decks without leakage.

There are two types of post-tensioning systems in the market—bonded and un-bonded. It is recommended that only bonded post-tensioning systems be used for precast concrete deck panels. The reason for this recommendation is the potential need to patch the deck in the future. If a post-tensioning duct and strand were ever compromised, the bonded strand stress could be lost at the damaged area; however, the stress would remain within a development length of the strand. Given the inherent redundancy of the installation, a localized loss of stress such as this can be considered acceptable.

The AASHTO LRFD Bridge Design Specifications include provisions for longitudinal post-tensioning in Article 9.7.5. Figure 4.2.4-1 shows AASHTO Article 9.7.5.3, including the corresponding commentary.

This provision recommends post-tensioning with a minimum average effective prestress of 0.25 ksi. This amount of prestress has proven to produce a bridge deck that is very durable. The prestress combined with grouted shear keys provides ample resistance to two-way slab action brought on by the presence of a wheel load in the center of a bay.

The 0.25 ksi value is a Service III limit state value. The design should account for all loading conditions, including the effects of live load continuity. For continuous bridges, the 0.25 ksi prestress should be increased to account for the tension in the deck caused by live load.

9.7.5.3 Longitudinally Post-Tensioned Precast Decks

The precast components may be placed on beams and joined together by longitudinal post-tensioning. The minimum average effective prestress shall not be less than 0.25 ksi.

The transverse joint between the components and the block-outs at the coupling of post-tensioning ducts shall be specified to be filled with a nonshrink grout having a minimum compressive strength of 5.0 ksi at 24 hours.

Block-outs shall be provided in the slab around the shear connectors and shall be filled with the same grout upon completion of post-tensioning.

C9.7.5.3

Decks made flexurally continuous by longitudinal post-tensioning are the more preferred solution because they behave monolithically and are expected to require less maintenance on the long-term basis.

The post-tensioning ducts should be located at the center of the slab cross-section. Block-outs should be provided in the joints to permit the splicing of post-tensioning ducts.

Panels should be placed on the girders without mortar or adhesives to permit their movement relative to the girders during prestressing. Panels can be placed directly on the girders or located with the help of shims of inorganic material or other leveling devices. If the panels are not laid directly on the beams, the space therein should be grouted at the same time as the shear connector block-outs.

A variety of shear key formations has been used in the past. Recent prototype tests indicate that a "V" joint may be the easiest to form and to fill.

Figure 4.2.4-1. AASHTO Article 9.7.5.3 Longitudinally Post-Tensioned Precast Decks.

4.2.6.1. Duct Size, Spacing, and Layout

The size, spacing, and layout of post-tensioning ducts should be established with the goal of providing a relatively low level of uniform post-tensioning force across the width of the bridge. This differs from the approach for main members that have a higher level of post-tensioning that is applied eccentrically to the section. The AASHTO LRFD Bridge Design Specifications offer some guidance on these parameters:

- **Duct Size:** AASHTO Article 5.4.6.2 sets certain limits on minimum duct size.
- **Duct Spacing:** There are provisions for duct spacing in the AASHTO Specifications; however, most of these are established for segmental construction or CIP concrete structures. AASHTO Article 5.10.3.4 covers "Maximum Spacing of Tendons and Ducts in Slabs." This limits the maximum spacing to four times the thickness of the slab. The commentary notes that this is for transverse post-tensioning. This provision is not considered to be applicable to the spacing of longitudinal post-tensioning used for distribution of panel connections.

The layout of longitudinal ducts within the slab should consider the goals of the system. The primary goal is to provide uniform prestress across the width of the bridge. It is not imperative to have precisely uniform spacing since the post-tensioning force will distribute across the width within a reasonable distance from the anchorage.

The PCI Northeast Bridge Technical Committee has set a recommended minimum distance of 18 inches from the centerline of a tendon to the edge of the deck panel or the centerline of the girder shear connector pocket. It is not necessary to have a tendon in the deck overhang region; however, it is acceptable, especially if a continuous concrete barrier or curb is used. The continuous barrier will help to connect the adjacent panels.

The location of the tendon within the panel should be at mid-depth or as close as possible to mid-depth, with the goal of providing uniform post-tensioning across the connection.

4.2.6.2. Design for Losses

The design for losses is dependent on a number of factors, including the exact hardware that will be used. In a design-bid-build (DBB) project, the design engineer does not know the exact hardware; therefore, the calculation of losses is approximate. It is recommended that the designer check the design with assumed values, detail the deck accordingly, but specify that the contractor hire an engineer to prepare the final jacking calculation based on the actual hardware chosen. The plans and specifications should clearly note this process.

The design of the deck panel post-tensioning should account for losses. The following short-term losses should be addressed in the design:

- Friction and wobble in the ducts.
- Friction due to curvature of ducts.
- Anchorage set.

- Elastic shortening.

The AASHTO LRFD Bridge Design Specifications offer guidance on the calculation of these losses based on assumed materials. However, as previously stated, the supplier of the selected system should be required to calculate values specific to the supplied materials.

The elastic shortening calculation should be based on the assumption that all strands are tensioned at the same time. This is not normally the case. Most contractors will tension one tendon at a time. This means that only the first tendon will experience the total elastic shortening of the deck, while the last tendon will see relatively little elastic shortening. In reality, these values are small for a typical bridge deck stressed to provide a final concrete compression force of 0.25 ksi. It is conservative to assume that the elastic shorting is experienced by all tendons.

There are specific elastic shortening equations in the AASHTO Specifications for post-tensioned members. Equation 5.9.5.2.3b-1 is written for beam members and may not be appropriate for deck panels. Equation 5.9.5.2.3a-1 is normally used for pre-tensioned members. This equation is recommended for deck panels since it is straight forward and will provide conservative results.

Other potential losses are more complicated due to the fact that the deck is typically made composite with the supporting girders soon after the application of the post-tensioning force. The restraint of the girders affects the losses in the deck in multiple ways. This is a very complex problem that is not easily solved.

There are several approaches that can be used for the determination of long-term losses due to creep in the deck. Some designers have chosen to neglect long-term losses due to creep. This is because the AASHTO post-tensioning recommendation of 0.25 ksi is somewhat arbitrary and is based on early research involving transverse post-tensioning of precast double tee bridges. Additionally, early projects built with initial post-tensioning of 0.25 ksi have performed very well over many years.

Another option is to simply use a lump sum estimate of losses. AASHTO Section 5.9.5.3 offers guidance on the calculation of Approximate Time-Dependent Losses.

The last approach would include the detailed time dependent calculation of losses. As previously stated, this can be a complicated process. Designers can use a procedure similar to the procedure defined in AASHTO Article C4.6.6, which addresses the effects of temperature differential across a structural system. The temperature differential terms can be replaced with the creep differential in order to identify this loss. For steel plate girders this calculation typically results in a loss of compression in the deck of approximately 50percent of the compression force after initial losses. The creep effects transfer post-tensioning forces into the girder. The increase in girder stresses is usually small and similar in magnitude to the forces from shrinkage of a CIP concrete deck; therefore, these stresses do not normally require special attention.

4.2.6.3. Effect of Cure Time on Losses

The AASHTO loss equations are based on the use of fresh concrete. In reality, precast concrete deck panels are typically built with fully cured concrete. The age of the concrete at the time of post-tensioning can reduce the amount of losses in the system.

Longer curing times normally have a beneficial effect on the deck panels. Typically, a longer cure times reduces the amount of total creep in the deck. The magnitude of effects on the girder is similar to that of shrinkage in a CIP deck. Some agencies have specified curing times of 45 days in order to reduce the effects of long-term creep. This can have a negative impact on the construction schedule and cost; therefore, indiscriminately specifying this cure time should be carefully considered and, if required, clearly stated in the project specifications and documents.

4.2.6.4. Post-Tensioning Anchorage Zones

The design of post-tensioning anchorage zones is affected by the anchorage hardware chosen. Special reinforcement is required in anchorage zones. There are two specific types of reinforcement that must be designed.

Local Zone Reinforcement: The local zone is defined by AASHTO as the volume of concrete that surrounds and is immediately ahead of the anchorage device and that is subjected to high compressive stresses.

The design of the local zone reinforcement is highly dependent on the properties of the anchorage hardware. The specifications in the AASHTO LRFD Bridge Design Specifications are based on flat plate anchorage assemblies. In reality, most manufacturers of anchorage hardware use steel castings with special flanges to help distribute the post-tensioning forces into the concrete. Local zone failures have become very rare based on the specialized features of these assemblies.

It is recommended that the design of local zone reinforcement be completed by the manufacturer of the post-tensioning anchorage assembly. The design should follow the provisions of AASHTO Section 5.10.9.7 or be tested according to Article 10.3.2.3 of the AASHTO LRFD Bridge Construction Specifications [29]. The project plans and specifications should reflect this recommendation. Specifications should clearly state the specific delineation of responsibility for items that are to be designed by the supplier. Without a clearly defined delineation of responsibility, the projects can be confusing to the contractor and difficult to bid.

General Zone Reinforcement: The general zone is defined by AASHTO as the region adjacent to a post-tensioned anchorage within which the prestressing force spreads out to an essentially linear stress distribution over the cross-section of the component.

The design of the local zone should be completed by the bridge design engineer according to AASHTO Article 5.10.9.3.1.

4.2.6.5. Sequence of Construction for Decks with Longitudinal Post-Tensioning

The installation of deck panels designed with longitudinal post-tensioning requires a specific construction procedure. The goal is to stress the post-tensioning prior to making the composite connection with the deck. Otherwise, the post-tensioning force would create a positive moment on the girder that would be undesirable. The following sequence is typically used:

1. Erect and level panels.
2. Torque leveling bolts to equal values to provide equal dead load distribution.
3. Install tendons in post-tensioning ducts loosely. Seal duct couplings.
4. Grout transverse connections.
5. When transverse connection grout has attained a strength of 500 to 1000 psi, tension tendons.
6. Install shear connectors. (Note: this can be done any time after panel erection.)
7. Grout shear connector pockets and beam haunches.
8. Grout post-tensioning ducts. (Note: this can be done any time after tensioning of tendons.)

4.2.7. Design of Mild Reinforced Transverse Connections

There are several approaches that can be used to connect panels. Recent research has investigated the use of various connections for deck panels [7]. Several new concepts were developed and new materials were investigated for the closure pour concrete.

The following sections discuss the conventional options and several of the various new options that are available:

Conventional CIP concrete with lapped reinforcing bars: This is the simplest connection; however, it normally results in the widest closure pour. Reinforcing bars can be extended from the edges of the deck panels and lapped with similar bars in the adjacent panel. The design of the connection should be based on the requirements for lap splices in the AASHTO LRFD Bridge Design Specifications.

Conventional CIP concrete with hooked reinforcing bars: This connection is similar to the lapped bar connection except the lap splices are replaced with lapped hooked bars. This will reduce the width of the closure pour since the development length of a hooked bar is substantially less than a lapped bar. It is important to detail the lap such that the bars are fully developed within the lap region.

Grouted connections with headed reinforcing bars: NCHRP Project 10-71 [7] investigated the use of headed reinforcing bars in narrow closure pour connections. The recommended fill material was non-shrink grout. The results of the study indicated that it is possible to develop a #5 reinforcing bar lap splice in approximately 6 inches using headed reinforcing bars. These headed bars are readily available, relatively inexpensive, and manufactured by at least three companies. The report also includes test data on other narrow connections with hooked bars, and information on connection grout materials.

Grouted dowels in steel box inserts: NCHRP Project 12-65 [8] investigated the use of grouted reinforcing dowels placed in slotted connection cast into the precast deck panels. The research successfully developed a system that could be built without longitudinal post-tensioning. The bars are inserted through a narrow slot in the top of the deck panel and grouted into place. The grout is also used to fill a narrow shear key between the panels. Several of these decks have been successfully built by both Texas DOT and Utah DOT.

Ultra high performance concrete connections with reinforcing bars: Ultra high performance concrete (UHPC) is a material that has very high compressive strength as well as very high tensile strength. A key ingredient in the mix is high strength steel fibers. The resulting mix can attain compressive strength well over 20 ksi. Tests have shown that UHPC can develop reinforcing bars in very short distances. Lap lengths for typical deck reinforcing are typically 6 inches or less. This allows for the design of a full moment connection closure pour of approximately 8 inches in width.

FHWA has been studying this material for use in deck connections. The preliminary results are very promising. The material has been tested for long-term durability by studying fatigue and long-term exposure to salt water. New York DOT has built several projects to date and is planning more in the future.

UHPC is proprietary and only available in the U.S. from one manufacturer. The hope is that in the near future, more similar products will come into the market to allow more widespread use.

4.2.8. Design of Longitudinal Connections

There are several approaches that can be used to join panels along longitudinal connections. These connections are typically incorporated into the deck panel system for the following reasons:

- Simplify the accommodation of a roadway crown point.
- Allow the use of multiple panels for very wide bridges.
- Allow for accommodation of casting tolerances by allowing horizontal adjustment of the panels to provide a smooth outside edge on the deck.

Most of the same options discussed in Section 4.2.7 are applicable to longitudinal connections. The one exception is the grouted dowels placed in steel box inserts. The close spacing of the main deck reinforcing will most likely make this connection unfeasible.

The selection of a proper material for use in these connections is critical to the long-term performance of the system. Some high early strength concretes experience significant shrinkage, which can lead to excessive cracking and long-term deterioration. Chapter 2 contains information on the selection of appropriate connection materials.

Different materials gain strength at different rates. UHPC shows promise as an effective connection material; however, it requires several days to gain the ultimate strength. It does gain significant strength in the interim. Strength gain rate must be considered in the design of the connection accounting for the anticipated loads during the curing of the material. The Massachusetts DOT (MassDOT) 93 Fast 14 project used an innovative approach to reducing the cure time for the closure pours. The approach was to design the connection for a lower strength than the final specification strength. For example, deck concretes are typically designed with concrete that has a 28-day compressive strength of 4 ksi. The AASHTO LRFD Bridge Design Specifications do not specify a minimum design strength for deck concrete; therefore, the designer has options for each design. It is common knowledge that concretes gain a significant percentage of the final strength in the early hours of curing; therefore, it is feasible to design a closure pour area for a lower interim strength while specifying a higher strength for the final condition.

Example calculation 4.2.8-1 demonstrated a design modification for a deck closure pour connection using a lower interim strength. The specified 28-day strength of the deck concrete could be 4 ksi, but an interim concrete strength of 2.5 ksi could be specified for opening the bridge to traffic. The results of this exercise show that the effect of concrete strength on a flexural element like a bridge deck is minimal. The design of a closure pour could be based on this lower interim strength. The designer should note that this lower interim strength also will have an impact on the bar development and lap lengths.

Example Calculation 4.2.8-1: Modification to deck closure joint designed for lower interim strength

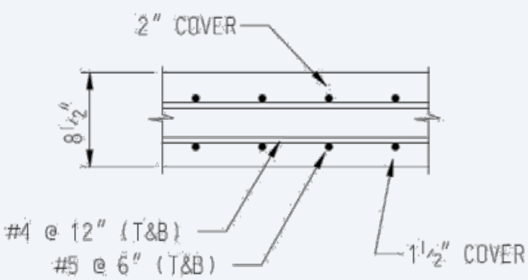
Approach: Take standard deck design (4 ksi) and modify the design for 2.5 ksi.

Given:

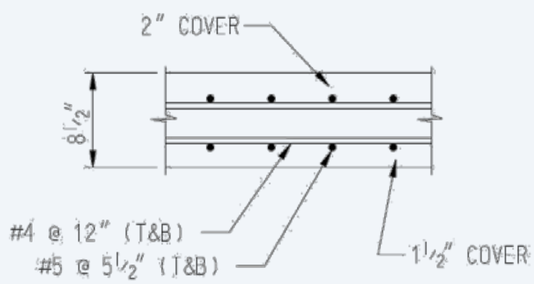
- Beam Spacing 10 ft.
- Deck Thickness: 8.5 inches.
- Top Cover: 2 inches.
- Bottom Cover: 1.5 inches.
- Deck section taken from MassDOT Standards (LRFD).

Using standard beam design calculations, the flexural resistance of the sections can be calculated. The bar size was kept constant and the spacing modified to achieve equal or greater flexural capacity. The following table shows the results of the redesign.

	Reinforcement	Flexural Bar Cover	Design d	ΦMn	Result
		in.	in.	in-kip	
Original Design (bottom)	#5@6 inches	1.5	6.69	209	
Redesign (bottom)	#5@5.5 inches	1.5	6.69	215	OK
Original Design top)	#5@6 inches	2	6.19	192	
Redesign (bot)	#5@5.5 inches	2	6.19	197	OK



4 ksi DESIGN



2.5 ksi DESIGN

DECK RE-DESIGN WITH LOWER CONCRETE STRENGTH

This example demonstrates that it is feasible to design closure pour areas with a lower design strength. The impact on bar spacing is minimal. This is only a check on flexural capacity. Serviceability checks in AASHTO Article 5.7.3.4 should also be made. This design was checked and found to be adequate for serviceability.

4.2.9. Design of Continuous Bridges with Full-Depth Deck Panels

Continuous span bridges often make use of longitudinal deck reinforcement to enhance the negative moment resistance of the girders. Integral connections to substructures in high seismic regions also place high negative moment demand on deck systems. Deck panels have transverse connections that make the use of mild reinforcement difficult.

There are two approaches to a solution to this issue. If the deck panels are post-tensioned, the post-tensioning system can be used to provide the desired resistance. The tendons can be used to provide ultimate moment resistance for the girders. If mild reinforcing bars are used in the deck panel connections, they can also be used for the ultimate moment resistance, provided that they are properly developed at the transverse deck connections.

4.2.10. Dead Load Distribution

This is an important aspect of deck panel design. The design of supporting framing is normally based on the assumption that the concrete is a fluid load that places tributary weight on each beam. If a deck panel is placed with support on only a few beams, the amount of dead load will not be uniform. To account for this, it is recommended that a support system be used at each beam that is under the panel.

The most common system in use is leveling bolts. These bolts are used to set panel grades; however, they can also be used to establish the proper dead load distribution in the bridge. It is common to specify that the torque in each leveling bolt be adjusted to within 15percent of each other. Minor variations in bolt load can be accommodated through the beam cross frames.

A panel setting procedure should be outlined on the plans to ensure that the panel is not damaged. The following procedure should suffice:

1. Estimate the extension of the leveling bolts in each panel based on the required setting grade of the panel and a survey of the top of the beams at the leveling bolts.
2. Set the bolts to the required extension.
3. During final setting, slowly set the panel checking for the contact of the leveling devices. Adjust leveling bolts as necessary to get uniform contact (before releasing the panel from the crane).
4. Prior to releasing the panel, check the torque on all bolts to achieve similar torques.
5. Release the panel form the crane.
6. Adjust the final torque on the leveling bolts to within 15 percent of each other.

Another approach to this procedure is to check the stresses in the panel for four-point support. If the deck stresses are within acceptable limits, the above procedure can be simplified. As previously stated, this work is under the purview of “contractor means and methods.” This calculation and the final setting procedure should be completed by the contractor as part of the assembly plan. Chapter 14 contains more information on specifications and assembly plans.

4.2.10.1. Leveling Devices

One of the benefits of using a precast concrete deck panel system is that the elevation of the top of the deck can be controlled and adjusted during construction. All precast deck panel designs should include provisions for grade adjustment leveling systems.

Most designers detail leveling bolt systems for the adjustment of deck grades. Typically two bolts per beam are specified. The bolts can easily raise or lower the panel to obtain the exact grades specified on the plans. This can normally be done without power tools.

Shims have also been used; however, the adjustment of shim heights can be problematic. In order to adjust grade, the panel would need to be lifted and reset after the shims are adjusted. Shims also do not offer the level of accuracy of dead load distribution when compared to leveling bolts.

4.2.10.2. Effect of Integral Barriers and Curbs

It is possible to cast a barrier integrally with the deck panel. Figure 4.2.10.2-1 shows an integral barrier on a deck panel project in Utah. This system eliminated the need to cast a barrier in the field while providing the detailing of a CIP crash tested barrier.

Figure 4.2.10.2-1. Precast Concrete Deck Panel with Integral Barrier (Source: Utah DOT).

This approach saves time in construction; however, it brings about certain design issues that need to be addressed. Most designers assume that barriers placed after the deck is cast are uniformly distributed to all beams as a composite dead load. Some agencies have other distribution assumptions; however, they are considered composite dead loads. If the barrier is cast into the deck panel, it will be a non-composite dead load, which will affect the design of the supporting girders.

There are two approaches to setting a panel with an integral barrier. The design can be based on uniform support on all beams and achieved by following the panel installation procedure described above. This also will require the analysis of the panel with equal support reactions. This analysis will need to be an iterative process that is executed by modeling the support points as springs. The spring coefficients will need to be adjusted manually until the reactions are relatively equal. Once this model is correct, the stresses in the deck can be calculated and checked. The installation of the panel could then follow the procedure outlined above.

The other approach is to analyze the panel with rigid supports (in the vertical direction). The stresses in the deck can be calculated and the amount of dead load distribution to each beam determined. This will require a complicated setting procedure where the torque in the leveling bolts is adjusted according to the relative difference in reaction at each beam.

The option for setting the panel will affect the design of the beams; therefore, it needs to be determined in the design phase and specified in the plans or specifications.

4.2.11. Shear Connector Design

This aspect of deck panel design has been thoroughly studied by multiple universities and agencies [8-27]. The connection is typically made by means of shear connectors placed within blockouts in the panels. The design of the shear connectors can be made using the same provisions that currently exist in the AASHTO LRFD Bridge Design Specifications. Pocket spacing is typically constant. Resistance to variable shear demand is accommodated by varying the number of shear connectors in each blockout.

The AASHTO LRFD Bridge Design Specifications limit the maximum spacing of shear connectors to 24 inches. Typically, designers use 24 inches as a maximum pocket spacing for this reason. Larger pocket spacing is typically beneficial with the design of composite prestressed concrete beams. Research has shown that precast deck panels can be built with pocket spacing up to 48 inches, with no detrimental effects to the performance of the composite connection [13].

Several configurations of the blockouts are currently in use. The most common is a full thickness blockout that provides an opening to insert the shear connectors. The blockout is normally filled with grout, which connects the panel to the girder so that horizontal shear is transferred.

Some agencies that use bare concrete decks have raised concerns with the durability of this detail, since it has exposed joints around the perimeter of the blockout after the grout is placed. There also is concern regarding the surface roughness of the completed bridge. A research project was completed to address these concerns [8]. The resulting details are a partial depth blockout that is only open to the underside of the deck panel. The blockout is formed by a galvanized steel box that is set into the panel. The blockout also is filled with non-shrink grout that is pumped through ports that project to the top of the panel. Two ports are used, one for the injection of grout and the other to vent the void and ensure that the blockout is filled. The makeup of this connection requires that the shear connectors be placed before the panel is set. This requires the incorporation of an accurate survey of the beams to ensure that there are no conflicts between the connectors and the blockouts. An advantage to the connection is that the top deck reinforcing can be placed over the blockout, which simplifies the layout of reinforcing in the deck panel. Figure 4.2.11-1 is a detail of this connection.

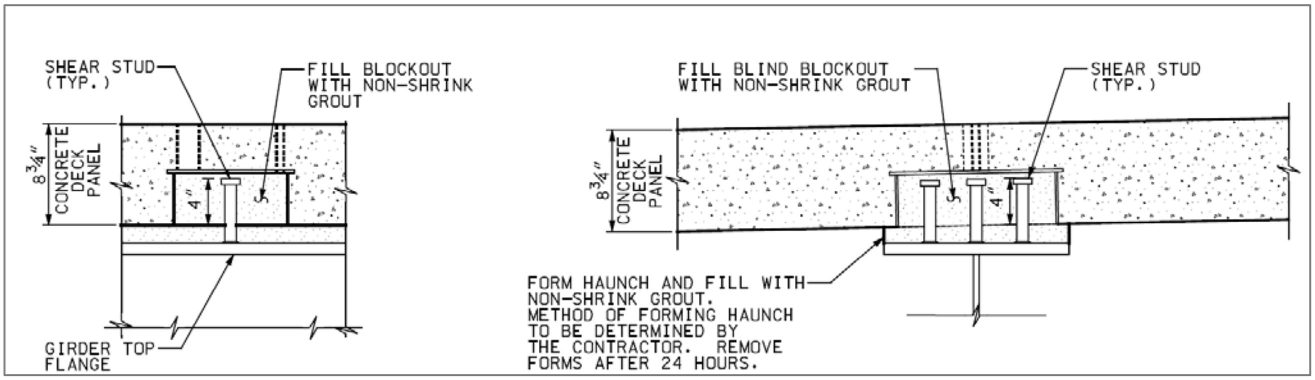


Figure 4.2.11-1. Transverse and Longitudinal Section of Partial Depth Blockout Connection (Source: Utah DOT).

The following sections include specific information on the design of shear connectors for concrete and steel beams.

4.2.11.1. Concrete Beams

On some composite concrete beams, the web shear reinforcement is extended from the beam into the CIP deck. In many cases, all of the shear reinforcement is extended even if it is not necessarily required for horizontal shear capacity. This is done to keep the shear reinforcement detailing consistent and to avoid problems with fabrication.

The design of concrete beams for composite action when precast concrete deck panels are used requires a different approach. As previously stated, the typical precast concrete deck panel is fabricated with uniform pocket spacing, with 24 inches on center being common. The required horizontal shear resistance of the connection is accommodated by varying the amount of reinforcement that is in each pocket, not by varying the spacing of the pockets. This complicates the fabrication process somewhat because the location of the shear reinforcing bars needs to be installed at specific locations and spacing during the fabrication of the beams. However, this complication is not beyond the ability of modern precast concrete girder fabrication plants. Figure 4.2.11.1-1 is a detail of the connection of a typical bulb tee girder to a precast concrete deck panel. Note that the horizontal shear reinforcing is separate and different from the vertical web reinforcing steel.

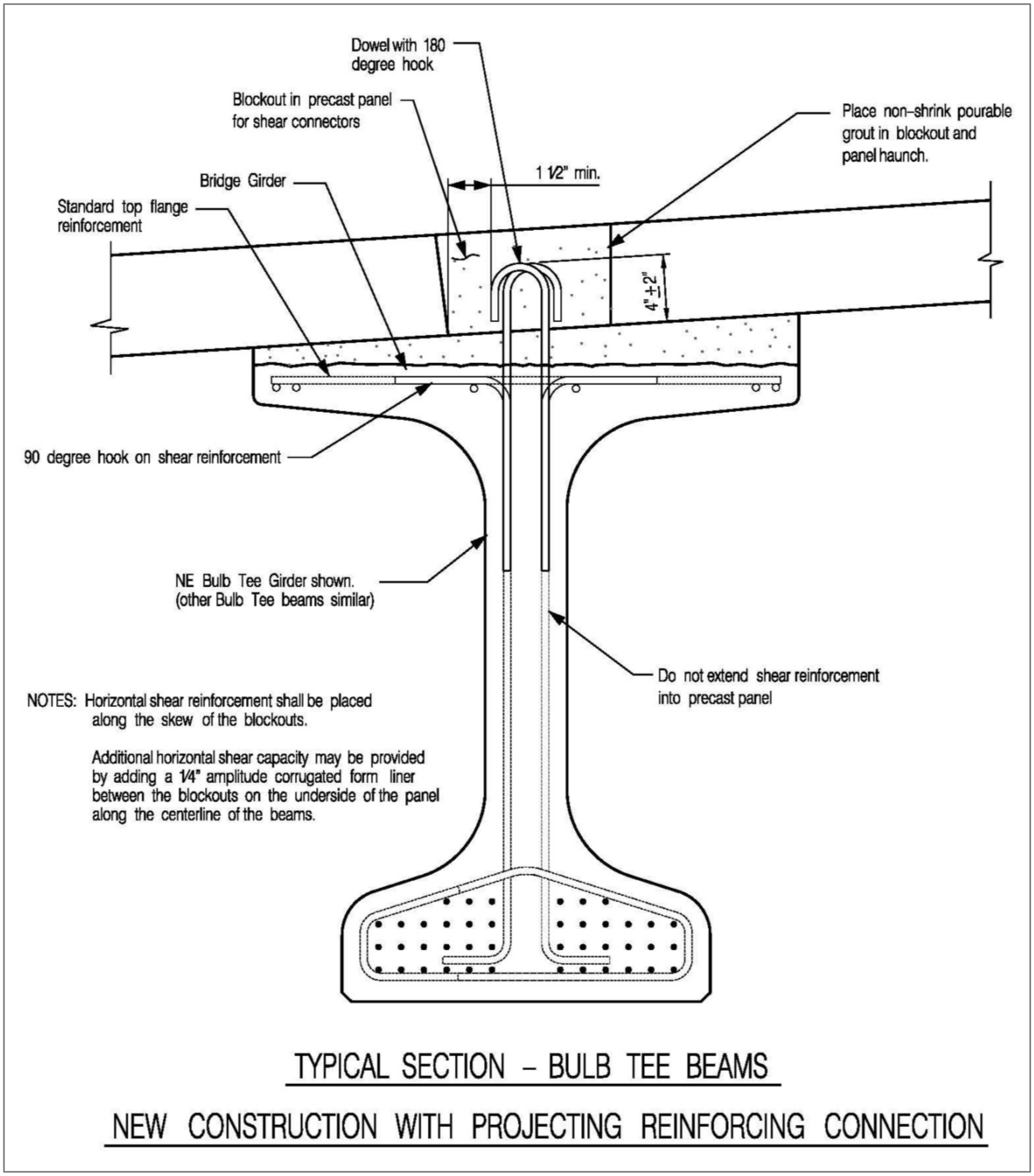


Figure 4.2.11.1-1. Connection of Precast Deck Panel to Concrete Beam (Source: PCI Northeast).

Another approach to connecting precast concrete deck panels to concrete beams has been developed that involves the use of welded stud shear connectors. The approach is to embed a steel plate in the top of the concrete beam. The plate is attached to the beam with welded stud shear connectors. Once the deck panels are set on the beam, a second set of studs is attached at the bridge site. Figure 4.2.11.1-2 is a detail of this connection. This method simplifies the construction and fabrication process. It is important to include a requirement that a grounding source be included in the fabrication process. Without a ground, the weld cannot be made. One way to do this is to install a grounding lug on each plate in the fabrication plant. In the field, the welder can ground to the plate to make the welds.

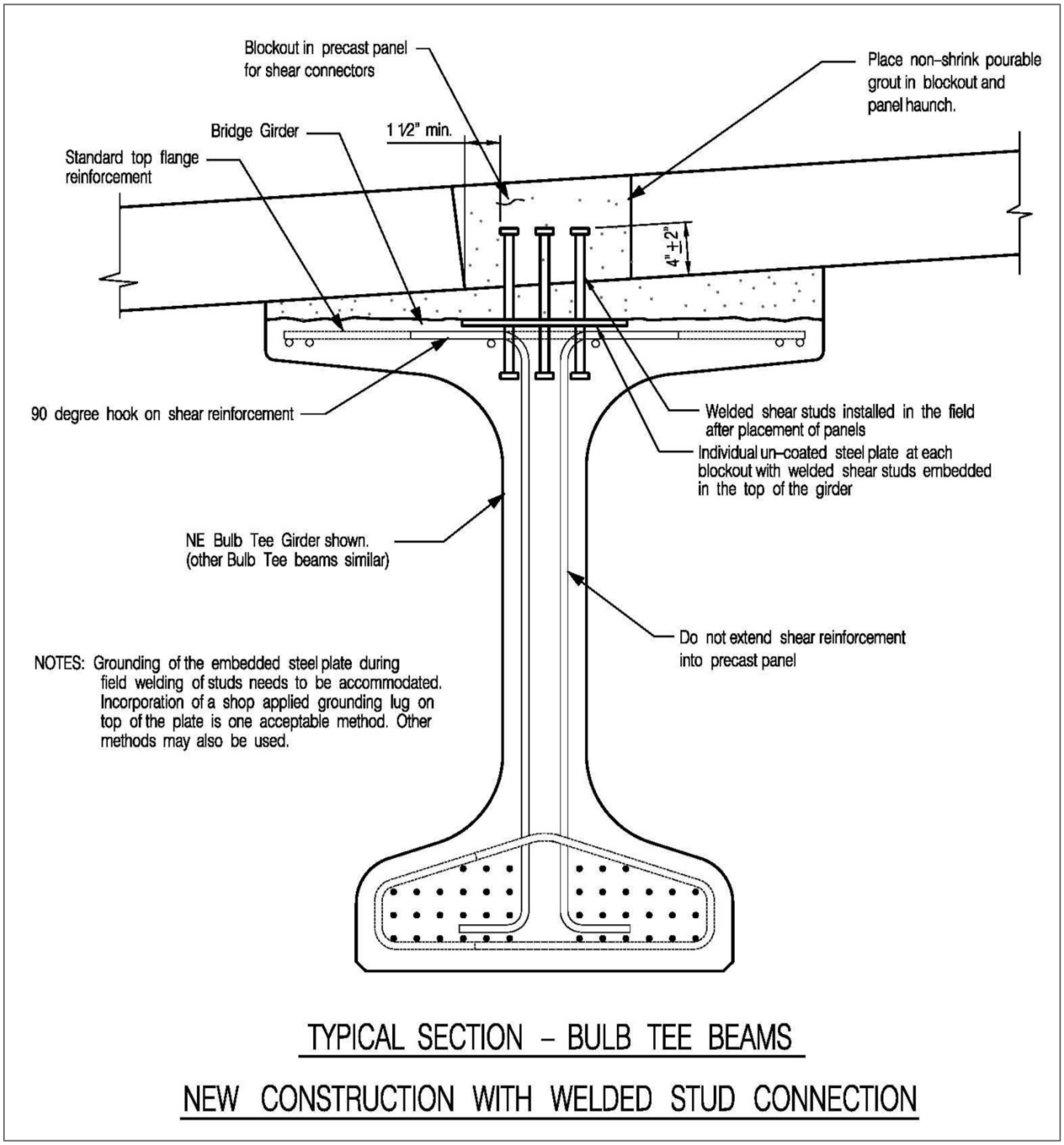


Figure 4.2.11.1-2. Connection of Precast Deck Panel to Concrete Beam Using Welded Studs (Source: PCI Northeast).

It is recommended that individual steel plates be used at each blockout. The use of continuous plates complicates the fabrication process in the plant.

A third approach that has been successfully used is to drill shear anchors into the beam after erection. This is especially useful for deck replacement projects. The installation of the shear connectors is similar to the welded stud details. The connectors can typically be installed after the deck panels are set. Figure 4.2.11.1-3 is a detail of this connection.

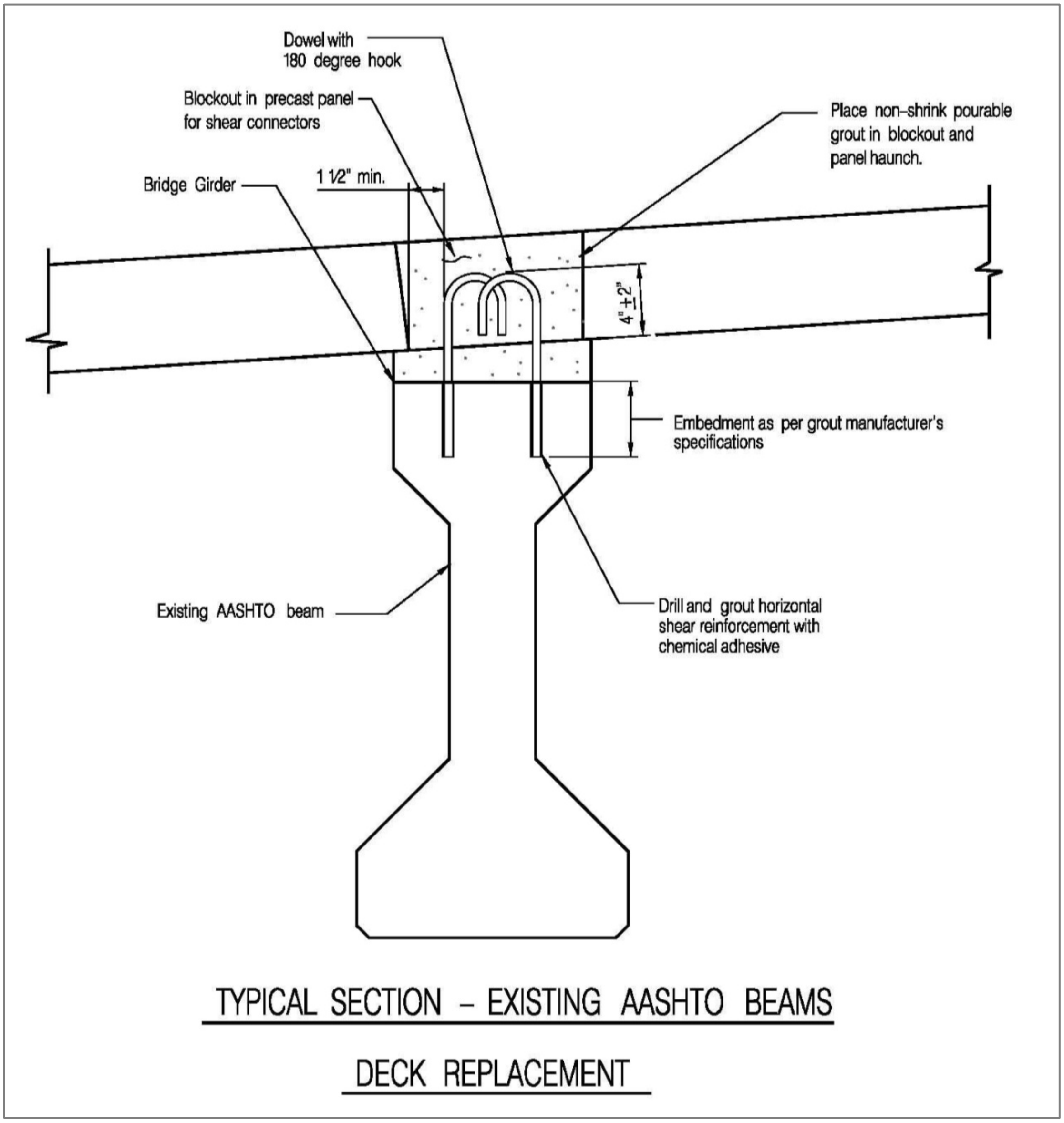


Figure 4.2.11.1-3. Connection of Precast Deck Panel to Existing Concrete Beams (Source: PCI Northeast).

4.2.11.2. Steel Beams

The connection of precast deck panels to steel beams is relatively straight forward. Most of the same details shown above for concrete beams are applicable to steel beams. Figure 4.2.11.2-1 is a detail of a typical steel beam connection.

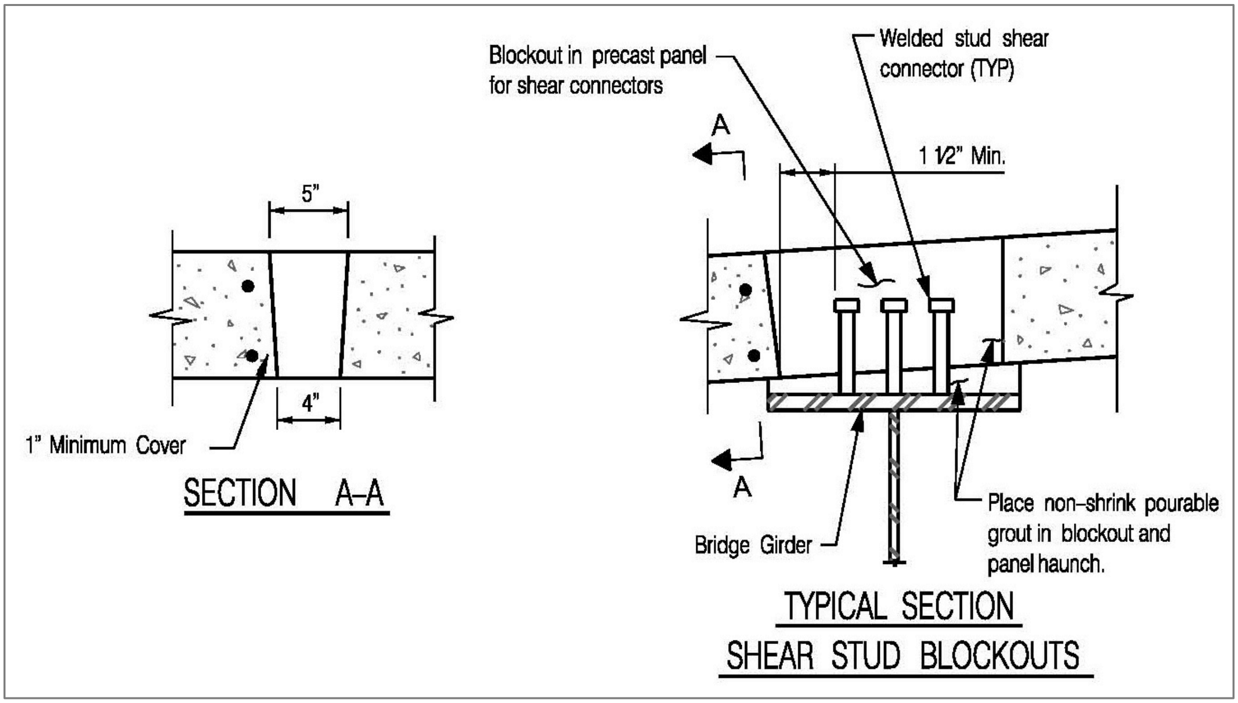


Figure 4.2.11.2-1. Connection of Precast Deck Panel to Steel Beams (Source: PCI Northeast).

Occasionally, the shear connector spacing requirements on steel girder bridges are difficult to meet. Designers have found that even with 24-inch pocket spacing, the number of studs per pocket can get quite large. An increase in the size of the shear connector diameter will decrease the number of connectors required. There are discussions underway with the AASHTO Bridge Sub-committee on Revisions to the Current Specification to potentially reduce the number of shear connectors for steel bridges. In the interim, the PCI Northeast Bridge Technical Committee has come up with larger reinforced pockets that can accommodate a large number of shear connectors. Figure 4.2.11.2-2 shows a sketch of this concept. Similar designs have been used in Nebraska [12]. Other methods of connecting deck panels to girders are being studied by FHWA, including continuous blockouts on the underside of the panels combined with UHPC. These new details will allow more closely spaced shear connectors.

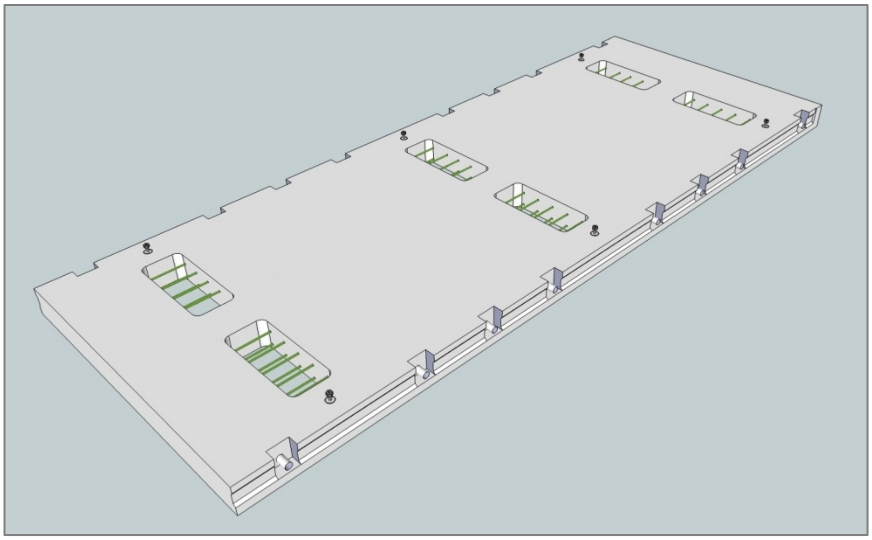


Figure 4.2.11.2-2. Large Pocket Panel (Source: PCI Northeast).

4.2.12. Haunch

The haunch is the gap between the bottom of the deck panel and the top of the supporting beam. The purpose of a haunch is to account for vertical camber tolerances in the beam as well as tolerances in beam seats. It is normally desirable to minimize the height of the haunch in order to keep the structure thickness to a minimum.

On CIP concrete decks, it is possible to have a zero haunch and even a slightly negative haunch. The deck forming can be adjusted to account for beams that are either set too high or beams that are over cambered.

Bolted splices on steel girders can be problematic with precast concrete deck panels. On a typical CIP deck, the concrete and reinforcing bars can be set around the splice plates and bolt heads. This is not possible with a precast system. There are several options that can be used. The haunch can be increased to accommodate the splice. The splice can be customized to minimize of eliminate the top splice plate (single shear connection). A CIP closure pour can be used at the splice location.

In general, it is recommended that additional haunch depth be provided for bridges with precast concrete deck panels. This will help to avoid dimensional issues and facilitate the flow of grout under the panels. It is recommended that the design haunch thickness be 1 inch greater than on the same bridge with a CIP deck. The design haunch height must account for the presence of bolted field splices on steel girder bridges.

4.2.13. Design of Integral Abutment Bridges With Full Depth Deck Panels

Integral abutments are very common across the country. Integral pier caps are also common in high seismic regions. On integral bridges, the deck is typically tied into the integral diaphragm pour.

On bridges constructed with precast concrete deck panels, the deck connection is made by incorporating the deck connection into the integral diaphragm pour. Reinforcing bars are typically extended from the ends of the panels in order to make the connection. If the deck panels include longitudinal post-tensioning, there can be difficulties with extending large amounts of reinforcing at the ends of the panel. The anchorage assemblies can take up a significant portion of the deck panel end. If the post-tensioning system is bonded, the designer can specify that the post-tensioning strands be extended into the diaphragm pour. This is not recommended if the post-tensioning system is unbonded.

4.3. MODULAR ELEMENTS

Modular elements are comprised of prefabricated beam and deck elements combined into one modular element. There are several modular elements in use in the U.S. at this time. The most common are modules that have steel girders combined with a concrete deck that is cast in the fabrication facility.

Steel girder modular elements offer advantages over similar modular elements made with precast concrete beams. Precast modular elements can require larger trucks for shipping and larger cranes for erection and handling. For short- and moderate-span bridges, the use of precast modular elements should be considered along with similar steel elements.

Several projects built with modular elements have been completed to date. The most significant project was the MassDOT 93 Fast 14 project. This project involved the replacement of 14 bridges (41 total spans) using modular steel beam elements. Traffic was diverted from one side of the expressway to the other side via two median crossovers. This left one entire side of the highway available for construction without the need to accommodate traffic adjacent to the work zone. The majority of this large DB project was completed in 10 weekends. Up to 6 spans were replaced during the 55-hour weekend period. Figure 4.3-1 shows one of the 252 modular steel elements being installed.

One of the benefits of modular elements is that elements do not require the installation of a deck after erection. This eliminated the complicated forming required on and around the actual girder. Modular elements are often simpler to build when compared to prefabricated deck panels since there is no need install shear connectors, place grout, or install post-tensioning. It is possible to construct a modular beam bridge span in less than two days. On certain bridges with appropriate circumstances, modular beam elements have been installed overnight.

Figure 4.3-1. Massachusetts DOT 93 Fast 14 Construction using Modular Elements.

A modular steel folded plate girder system also has been developed by the University of Nebraska. This consists of modular steel inverted tub girder elements topped with a precast concrete deck. The beams are made of a single folded steel plate. Figure 4.3-2 shows a folded plate modular element being installed on a bridge in Massachusetts. This system is useful for short-span bridges, since the machinery required to build the beams is currently limited to 60 ft due in length. It also is not possible to camber the beams; therefore, vertical curved roadway geometry would need to be accommodated in the deck haunch.

Another option that has been used is the “Inverset” system. This system was proprietary at one time; however, it is no longer patented. The system is innovative in that the deck cast on the beams in an inverted position. Once the deck is cured, the entire element is rotated, placing the deck into compression.

Figure 4.3-2. Folded Plate Modular Beam Element (Source: Massachusetts DOT).

The design of the elements is essentially the same as the design of a typical composite beam with a CIP concrete deck. There are two options available for multiple-span bridges. One option is to design the bridge as a simple span for live load, but design the deck as jointless. Link slab techniques can be used to provide a jointless deck at the piers without live load continuity. Some designers question the efficiency of designing a beam for simple-span action. In reality, this approach can be very cost-effective for short to moderate spans, when the cost of field splices is considered.

The concept of link slabs is to design the connection of the deck across the pier to accommodate the rotation of the beams without significant cracking. This is done by de-bonding a small portion of the deck near the pier to allow for a wider spread of the rotation strain. Link slab technology has been in use for many years. It can provide a jointless durable deck connection without the complexities of developing a continuity connection. Information on link slabs can found in the following papers:

- “Behavior and Design of Link Slabs for Jointless Bridge Decks” [30].
- “Combining Link Slab, Deck Sliding over Backwall, and Revising Bearings” [31].

Structural continuity can be achieved on multi-span structures by designing the beams as simply supported for dead load and continuous for live load. This approach has been used for many years in the prestressed concrete industry. By using this technique for modular steel beam units, the time consuming process of in-span bolted field splices can be reduced or eliminated.

The University of Nebraska has studied a CIP closure pour connection for continuity at the piers [32]. They have studied steel I-girders and box girders. The conclusion was that full live load continuity can be achieved using simple connection details.

The critical aspect of the design and the key to long-term durability is the deck connection between the elements. The methods described in Section 4.2.8 are applicable to this connection. It is very important to use non-shrink or low shrinkage materials in closure pours. Shrinkage of the closure pour concrete can result in transverse cracking or cracking along the precast interface. This cracking is caused by the restraint brought on by the adjacent modular units. Several states are using shrinkage modifying admixtures to reduce the amount of drying shrinkage in the closure pour concrete. Chapter 2 contains more information on concretes used for closure pours.

CHAPTER 5. DESIGN OF SUBSTRUCTURE ELEMENTS

Early prefabricated projects focused on superstructure elements. While prefabricated superstructure elements reduce overall project construction time, there are significant opportunities to reduce construction time through the use of prefabricated substructure elements.

In most cases, prefabricated substructure elements are designed to emulate cast-in-place concrete. The following sections contain information on the design requirements for prefabricated substructure elements.

5.1. DESIGN OF PREFABRICATED ABUTMENTS AND WALLS

Abutments and walls are normally similar structures. The primary purpose of retaining walls is to retain soils. Abutments are essentially retaining walls that also support the beams. For this reason, the discussion on the design of prefabricated abutments and walls are combined in the following sections.

5.1.1. Cantilever Structures

Cantilever walls are defined by AASHTO [5] as:

"…a concrete stem and a concrete base slab, both of which are relatively thin and fully reinforced to resist the moments and shears to which they are subjected."

Cantilever walls and abutments consist of footings that support a vertical wall element that is "cantilevered" from the footing. Gravity walls are not discussed in this manual because the design of a precast gravity wall is akin to the design of a masonry wall. The prefabricated elements can simply be stacked like blocks without significant connections. The AASHTO bridge design specifications [5] cover the design of block retaining walls.

The vast majority of cantilever wall structures are made with reinforced concrete. Walls can be made with steel sheeting or steel soldier piles. The design of these systems is well documented; therefore, it will not be covered in this manual. This section will focus on the more common concrete wall systems. Section 5.1.7 covers options for precast concrete soldier pile wall details.

5.1.1.1. Joints

CIP concrete wall stems contain joints. It is important to understand the purpose of the joints in order to design prefabricated wall elements. Three joints are commonly used in concrete wall structures.

Construction Joints: Construction joints are joints that are used to facilitate the construction of the CIP wall. Construction joints typically have reinforcement passing through them. The concrete from one side of the joint is cast against the face of the concrete of the previous pour without any special joint materials. Construction joints can run in any direction.

Precast concrete walls emulate CIP construction by providing systems to connect the rebar across the joint. Grouted joints are typical. Keyed joints are also possible by placing the wall stem in a recess in the footing that can be grouted to lock in the wall stem. Designers should ensure that the size and depth of the key provides ample room to make the connection between the elements. Figure 5.1.1.1-1 shows a stem-to-footing connection with a recessed keyed joint that was developed by the PCI Northeast Bridge Technical Committee.

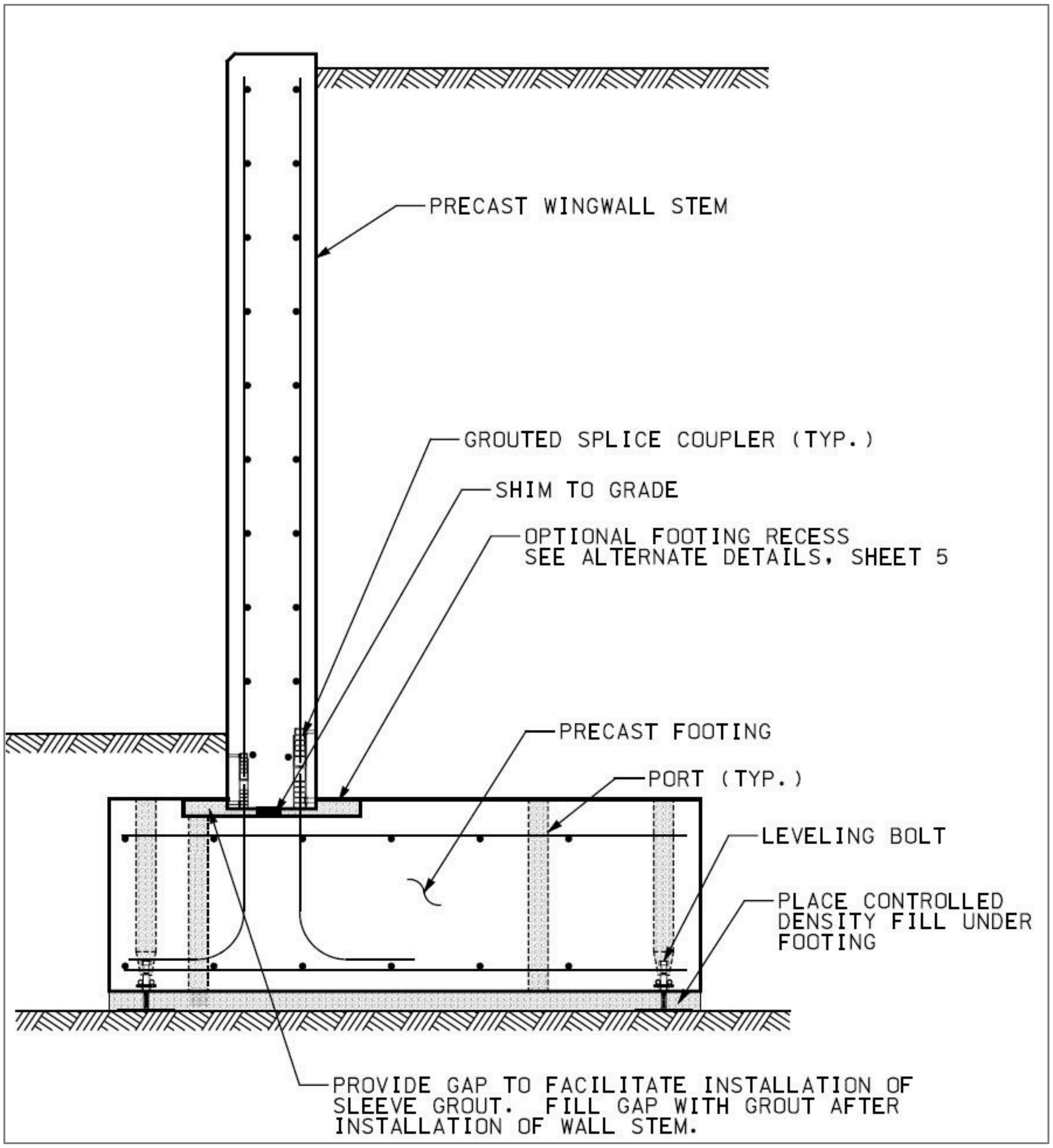


Figure 5.1.1.1-1. Precast Wall Stem to Footing Connection (Source: PCI Northeast).

Contraction Joints: Contraction joints are typically vertical joints running from the top of the footing to the top of the wall. Contraction joints are added to walls for two purposes. The first is to reduce cracking brought on by the internal tension that develops during the curing process. The restraint of the footing causes this tension stress as the stem concrete shrinks. The second purpose of contraction joints is to allow thermal contraction of the structure when it is exposed to low temperatures. Contraction joints are similar to construction joints, except that normally, no reinforcement runs through the joint.

Some agencies require vertical shear keys in contraction joints in order to transfer out-of-plane shear between stem sections and to maintain a flush wall surface. Some agencies use un-keyed contraction joints. Experience shows that un-keyed walls perform very well. Designers should consider the use of un-keyed contraction joints when designing prefabricated wall stems to simplify the fabrication and construction of the wall.

When designing and detailing precast wall elements, contraction joints can be detailed as narrow open joints. A thin rubber or neoprene sheet can be adhered to the rear face of the joint to keep soils from piping through the joint. This is common practice in modular block retaining wall construction. If a keyed joint is desired, a simple grouted female-female shear key can be used. Figure 5.1.1.1-2 shows contraction joints that were developed by the PCI Northeast Bridge Technical Committee (both keyed and un-keyed).

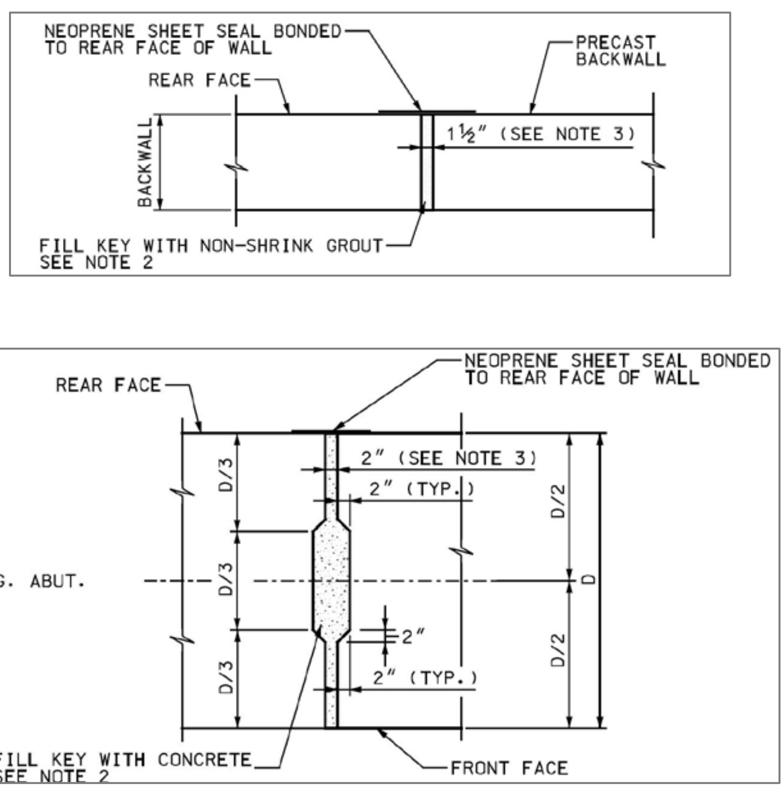


Figure 5.1.1.1-2. Wall Stem Contraction Joints (Source: PCI Northeast).

Expansion Joints: Expansion joints serve multiple purposes. They are similar to contraction joints, except they can accommodate thermal expansion as well as thermal contraction. This is done by placing a flexible filler material within the joint that can compress as the joint narrows. The joint can be keyed or un-keyed, as with contraction joints.

The AASHTO bridge design specifications [5] contain requirements for joints in walls; therefore, it is important to include these joints in prefabricated walls also. The AASHTO specifications call for a maximum spacing of contraction joints of 30 ft and a maximum spacing of expansion joints of 90 ft. The key factor in these specifications is that these are maximum values. It is perfectly acceptable to have more closely spaced joints in prefabricated substructures.

Expansion joints are typically not required in precast wall construction if un-grouted joints are used at contraction joints. Each contraction joint will act as a small expansion joint. If grouted contraction joints are used, consider open expansion joints. The use of flexible joint material can make construction difficult. The material can be adhered to one element; however, experience shows that the material is easily dislodged during placement of the adjacent element. To alleviate this potential problem, designers can investigate the use of spray in place of expansive insulation foam products. Another option is to use an open joint combined with a neoprene sheet seal on the rear face. This joint would be similar to the top detail in Figure 5.1.1.1-2 except that the grout could be replaced with chemical expansive insulation foam.

5.1.1.2. Design Approach

The design of a precast cantilever wall stem is typically the same as a CIP wall stem. The main vertical reinforcement is designed to resist the bending moments acting on the wall stem. The transverse reinforcement is used to distribute forces to the main bars. The AASHTO design specifications [5] can be applied to the design of both sets of bars.

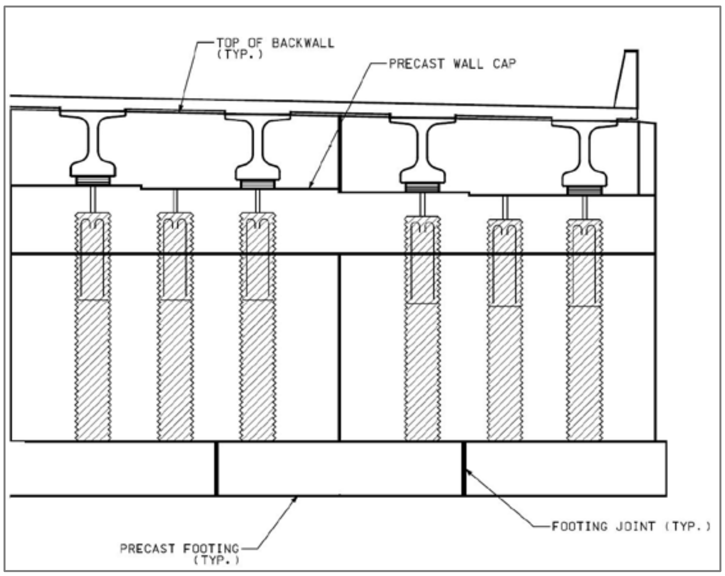
The design of the connection between the stem and the foundation is different than a CIP wall. CIP construction uses reinforcing lap splices to make the connection. Precast concrete walls use a mechanical connector to achieve the same result.

Several approaches have been developed for the design of this connection. Refer to section 5.2.3 for more information on this connection, since the designs are used for both walls and piers.

5.1.1.3. Caps

The top of an abutment can involve somewhat complex detailing. It is common to have steps and pedestals to accommodate the variable seat elevations brought on by cross slope and skew. If these features are incorporated into the wall stem elements, each element will most likely be different. One way to avoid this is to detail a separate wall cap element. The cap can be detailed as a relatively thin element that can span over several wall stem elements. This reduces the complexity of the wall elements, focusing the complexity of the bridge seat to one special element. This type of detailing also ties the stems together. Figure 5.1.1.3-1 shows details of abutment cap elements that were developed by the PCI Northeast Bridge Technical Committee.

The connection between the wall stems and the cap is a low moment demand connection. The connection is similar to the connection at the bottom of cap stones in stone masonry abutments. A simple dowel pocket connection combined with a grout bed is often adequate. Bars can be projected from the lower stem into voids in the cap, or dowels can be drilled and grouted through small voids in the cap. The voids can be formed using corrugated metal pipes. The pipes help to transfer forces from the dowels to the elements. Figure 5.1.1.3-2 shows a detail of the connection between the stem and cap that was developed by the PCI Northeast Bridge Technical Committee. This connection has ample room to accommodate erection tolerances of the wall stems. The backwall is shown as a separate element; however, it can be made integral with the cap element. Section 5.1.1.4 has more information on the use of corrugated pipe voids for connections.



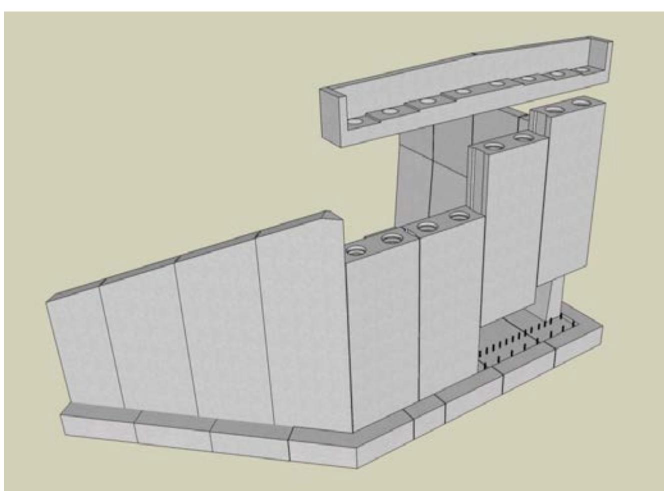


Figure 5.1.1.3-1. Wall Cap Element Details and Sketch (Source: PCI Northeast).

5.1.1.4. *Use of Corrugated Pipe Voids*

Several research projects have investigated the use of corrugated metal drainage pipes (CMP) to create voids in precast elements [35, 36].

The Iowa State University study [36] studied the use of corrugated pipe voids for the connection of piling to an integral abutment stem. The objective of the research was to verify that the pipe void could transfer loads from the piles to the cap and if the CMP pipe would have an adverse effect on the performance of the cap. Several of the conclusions of this study were:

- The CMP had no apparent effect on the pier cap section.
- Essentially no differential movement was detected between the precast concrete and the concrete in the CMP in any of the abutment specimens or pier cap specimen. The movement measured was less than 0.0033 inches in every test.
- No movement was detected in the shear tests; measured movement was less than 0.0005 inches, the LVDT precision.
- Shear failure was not detected between the H-pile in the abutment and the concrete in the CMP.

Based on this research, it can be assumed that the CMP void has no effect on the capacity or design of the section; therefore, the design of the cap can be the same as with a CIP cap.

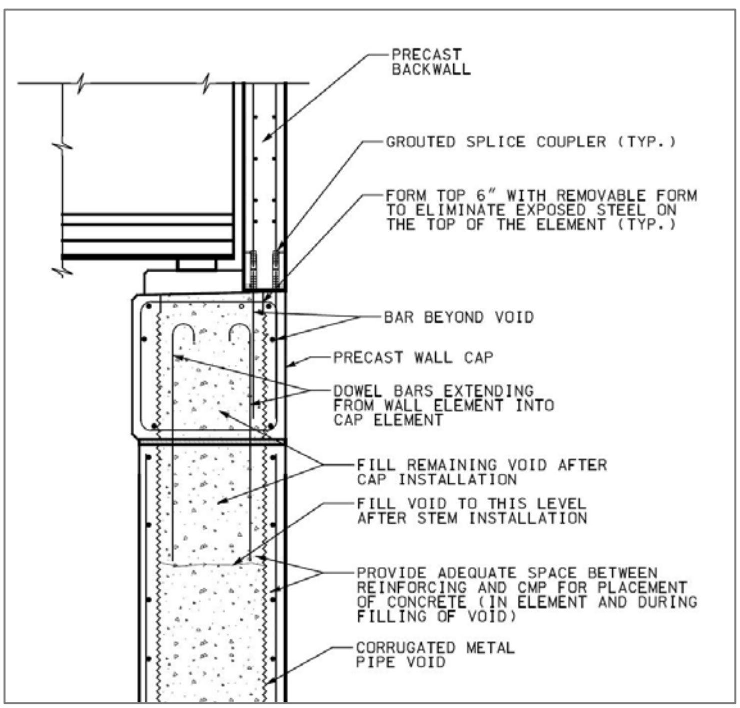


Figure 5.1.1.3-2. Abutment Cap Connection using Corrugated Metal Pipe Voids (Source: PCI Northeast).

These voids can serve two purposes in the design and detailing of elements. The first purpose is to aid in connecting elements. Dowel bars can be projected from an adjacent element into the void. A connection is completed when the void is filled with concrete. The force in the reinforced core is transferred to the adjacent precast concrete element through the corrugations.

The vertical shear resistance of this connection is not specifically covered in the AASHTO LRFD Bridge Design Specifications [5]. Using engineering judgment, a designer could consider using interface shear for the resistance to vertical shear. Using the AASHTO Equation 5.8.4.1-3, the nominal interface shear stress could be calculated as follows:

$$Vni = cAcv + \mu\ (Avf\ fy + Pc)$$

But not greater than the lesser of:

$$Vni \leq K_1\ f'c\ Acv$$
$$Vni \leq K_2\ Acv$$

Where:

c = cohesion factor
μ = friction factor
Acv = concrete area engaged in interface shear
Avf = Area of transverse reinforcement Crossing the shear plane
fy = nominal yield stress of reinforcement
Pc = permanent net compressive force normal to the shear plane
K_1 = fraction of concrete strength available to resist interface shear
K_2 = limiting interface shear resistance

AASHTO Article 5.8.4.3 defines several of the terms for various interface shear situations. The AASHTO provisions that most closely match this situation could be:

"For normal-weight concrete placed against a clean concrete surface, free of laitance, with surface intentionally roughened to an amplitude of 0.25 inch."

With this provision the following interface shear equation terms are defined as:

c = 0.24 ksi
μ = 1.0
K_1 = 0.25
K_2 = 1.5 ksi

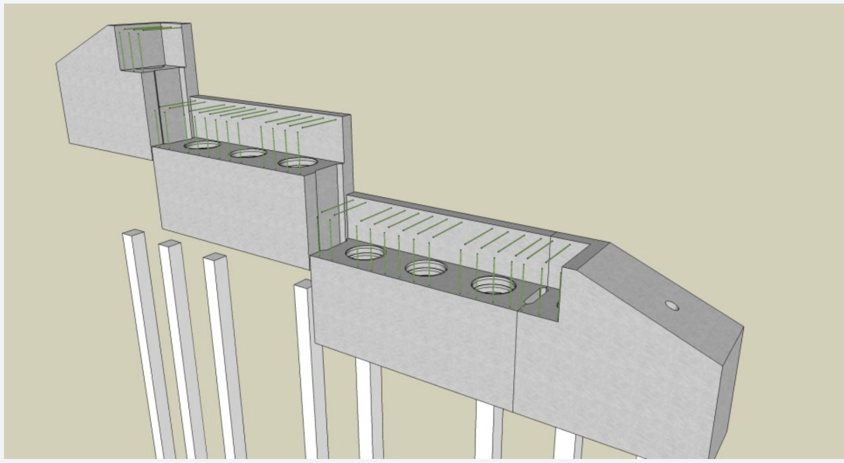
This provision is not specifically for this application because the AASHTO provision uses the wording "concrete surface." This may be acceptable since the surfaces of many formed concrete surfaces are similar to the surface of a corrugated steel surface. Some designers have expressed concern with the use of this provision, especially the cohesion term. If this is the case, it is possible to run reinforcement through the void to resist the vertical shear. The resistance of that reinforcement would also be calculated using interface shear equation. The area of the reinforcement would be used for the Avf term in the equation.

Another potential approach to this resistance is the use of punching shear theory. This theory is based on limiting the shear stress across a potential shear plane. The shear plane would be along the inside face of the void running from crest to crest of the corrugations, which equates to the inner circumference of the pipe multiplied by the height. The nominal punching shear resistance stress is defined in AASHTO Article 5.13.2.5.4 as $0.125(f'c)^{1/2}$. For a fill concrete of 4 ksi, the nominal punching shear value would be 0.25 ksi.

A comparison of these two approaches on a void with 4 ksi concrete yields similar results. If interface shear is used without transverse reinforcement, the nominal resistance across the failure plane would be 0.24 ksi. If punching shear theory is used, the nominal resistance would be 0.25 ksi. Both of these theories would seem to have merit, especially in light of the significant resistance found in the Iowa State research [36].

Example 5.1.1.4-1 is a calculation for punching shear resistance for a 24-inch diameter corrugated metal pipe connection subjected to vertical axial shear.

Example 5.1.1.4-1. Punching Shear Resistance on a 24-inch Diameter Corrugated Metal Pipe Void in a Precast Integral Abutment Stem.



Calculate the resistance of each void using the detail above.
Basis: AASHTO Article 5.13.2.5.4

Given:

f'c = 4 ksi (void concrete)
Inside Pipe diameter = 24 inches
Height of void above the pile top = 3 ft

Circumference of pipe = 24 inches(π) = 75.4 inches

Punching Shear area per linear foot of pipe

Acv = 75.4 inches(12 inches./ft) = 905 in^2

Nominal Interface Shear Resistance

$Vn = 0.125(f'c)^{1/2}\ Acv$
$\quad = 0.125(4)^{1/2}(905)$
$\quad = 226$ kips per linear foot of pipe

Total resistance of the Void

$Vn_{total} = 226\ (3) = 750$ kips per pile

The calculation in Example 5.1.1.4-1 demonstrates the significant resistance of this connection method. This resistance makes the use of CMP voids very attractive to a number of different connections within prefabricated elements, including integral abutment stems and pile caps.

Another option for the use of a CMP void is to connect two elements with reinforcement placed within the void spanning from one element to the next (dowels). Figure 5.1.1.3-2 shows an example of this approach. The transfer of force from the reinforced core to the surrounding concrete and reinforcing in the precast element is similar to the theory of lapped deformed reinforcing bars. If the bars are placed adjacent to the corrugations, the force in the bars can be transferred to adjacent precast concrete in a similar fashion as a non-contact lap splice. If the bars are spaced less than one-fifth the required lap length (or 6 inches), the transfer of force is equal to a lap splice.

The voids can be filled with standard concrete. Research has shown that shrinkage of the fill concrete is not detrimental to the performance of the connection [36].

An added benefit of CMP pipe voids is to reduce the shipping and handling weight of the element. The weight of wall stem elements can be reduced by eliminating concrete from the element. It is possible to detail more voids than are necessary for connections in order to reduce weight. The sketch in Example 5.1.1.4-1 shows an integral abutment steel with CMP voids. In the center two stems elements, there are three voids shown even though there are only two piles in the stem section. The voids can be filled with concrete after installation of the wall stem.

5.1.2. Integral Abutments

Integral abutments are cost effective and easy to construct. Integral abutments require fewer piles when compared to cantilever abutments, require fewer concrete pours, and often do not require excavation below water levels. CIP concrete integral abutments are relatively easy and fast to build; however, there are opportunities to reduce the construction time even more through the use of prefabricated elements.

5.1.2.1. Joints

The nature of integral abutments is that they do not have joints at the deck level. When building an integral abutment with precast stem elements, it is inevitable that a small closure pour be included in the construction. This is due to the fact that multiple elements are converging at the integral connection between the superstructure and the substructure. These elements can include the girders, deck, abutment stem, and in some cases, the approach slabs. It is virtually impossible to make these connections without a closure pour due to the fabrication and erection tolerances of all of the elements. The closure pour allows for correcting significant mis-alignment of each element without compromising the construction.

The connection of all of these elements is made by projecting reinforcing from the precast wall stem into the diaphragm pour between the girders. Reinforcement can also be projected from the deck into the diaphragm pour. The design of this reinforcement is essentially the same as with a CIP integral abutment. Figure 5.1.2.1-1 is an integral abutment detail that was developed by the

Utah DOT that shows the interaction between the various elements that are connected in this closure pour. The detail shows a separate backwall element. It is possible to make this backwall integral with the precast stem if the beam depths are not excessive.

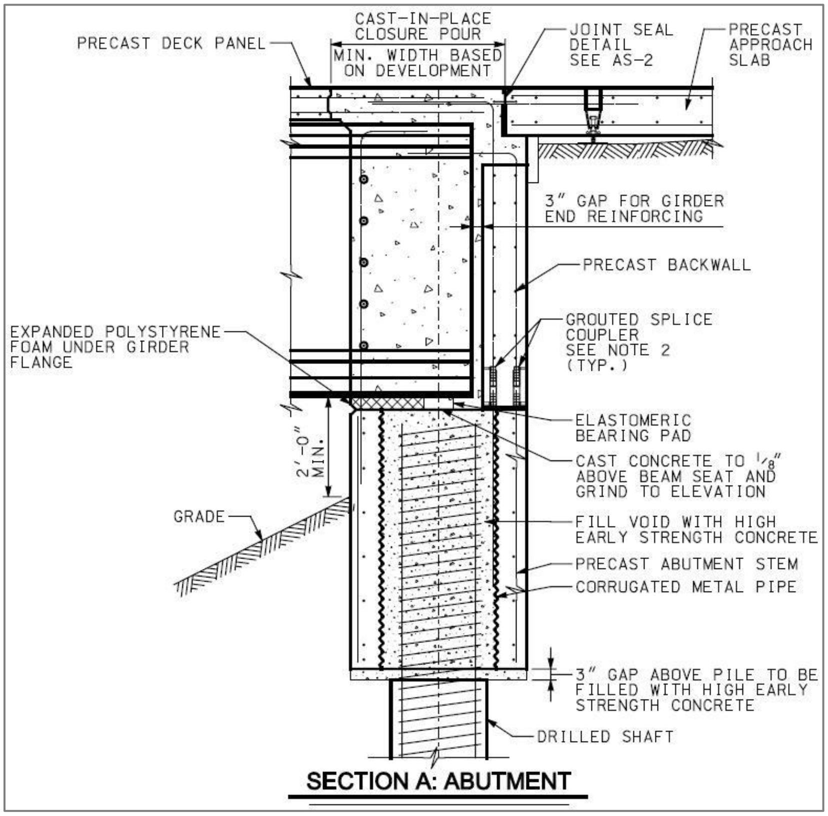


Figure 5.1.2.1-1. Precast Integral Abutment (Source: Utah DOT).

Figure 5.1.2.1-2 is a sketch of an integral abutment made with precast concrete stems. The sketch shows precast stem elements with an integral backwall. The purpose of the integral backwall is to allow for the installation of the approach slabs prior to completing the concrete closure pour. The sketch shows vertical joints between the stem elements. This is a detail that was first used in Utah and later adopted by the PCI Northeast Bridge Technical Committee. It differs from past integral abutment elements that were post-tensioned transversely. The joint is designed as a pinned joint about the vertical axis, with no reinforcement passing through. Even without continuity across this joint, the structure is stable. This is due to the fact that the main resistance to soil pressures acting on the stem is the vertical reinforcing in the stem that is connected to the superstructure via the diaphragm pour. This system is designed as a one-way panel, with the main reinforcing steel running vertically and the distribution steel running horizontally.

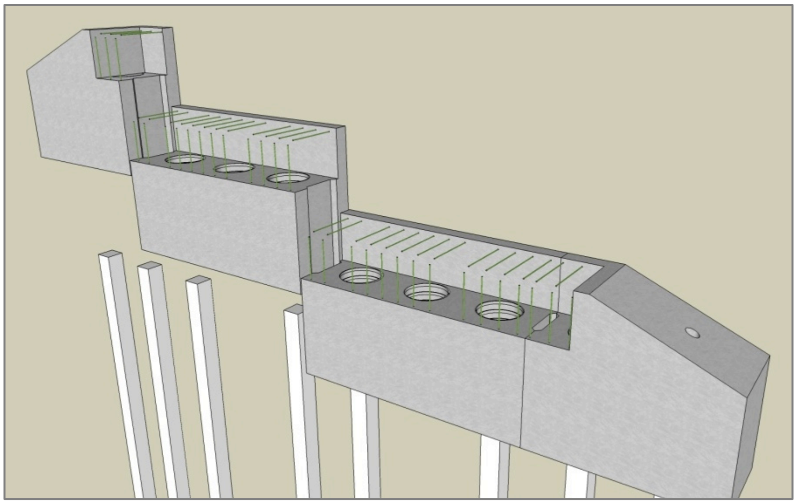


Figure 5.1.2.1-2. Precast Integral Abutment.

5.1.2.2. Use of Corrugated Metal Pipe Voids

The details shown in Figures 5.1.2.1-1 and 5.1.2.1-2 show circular voids in the stem elements made with corrugated metal pipe (CMP) sections. These voids are used to connect the stems to the piles. This technique was developed and studied by researchers at Iowa State University [36]. It is a versatile connection that allows for significant tolerance in the location of the piles.

The designer can size the CMP to accommodate the size of the pile and the specified pile installation tolerance. Many states specify liberal pile installation tolerances, typically plus or minus 6 inches in each direction. This is due to the fact that most pile caps made with CIP concrete are not sensitive to pile locations. In many cases, it may be possible to reduce the pile tolerances to plus or minus 3 inches. This is typically done through the use of pile installation frames or templates. Smaller pile tolerances will result in smaller voids and thinner stems. The bridge designer should coordinate the size of the void and the pile installation tolerances with the geotechnical engineer. The sizing of the CMP void will be set using the following equation:

CMP ID Pipe size:

D = Dpile + 2*(Tpile) + Dclear

Where:

D = Inside Pipe Diameter
Dpile = Diameter of round pile or maximum Diagonal dimension of rectangular pile
Tpile = Pile installation tolerance
Dclear = Minimum clear distance between pile edge and inside face of CMP (typically 2 inches for round piles and 1 inch for rectangular piles).

The value of “Dclear” can be adjusted for different piles. For instance, drilled shafts may need a larger dimension to accommodate reinforcement tolerances. Figure 5.1.2.2-1 includes an example of how to calculate the minimum diameter of a corrugated pipe for an integral abutment stem.

Example 5.1.2.2-1. Determine Minimum CMP Void for HP12x74 Pile.

Size of HP12x74 Pile:
Depth = 12.1 inches
Width = 12.2 inches

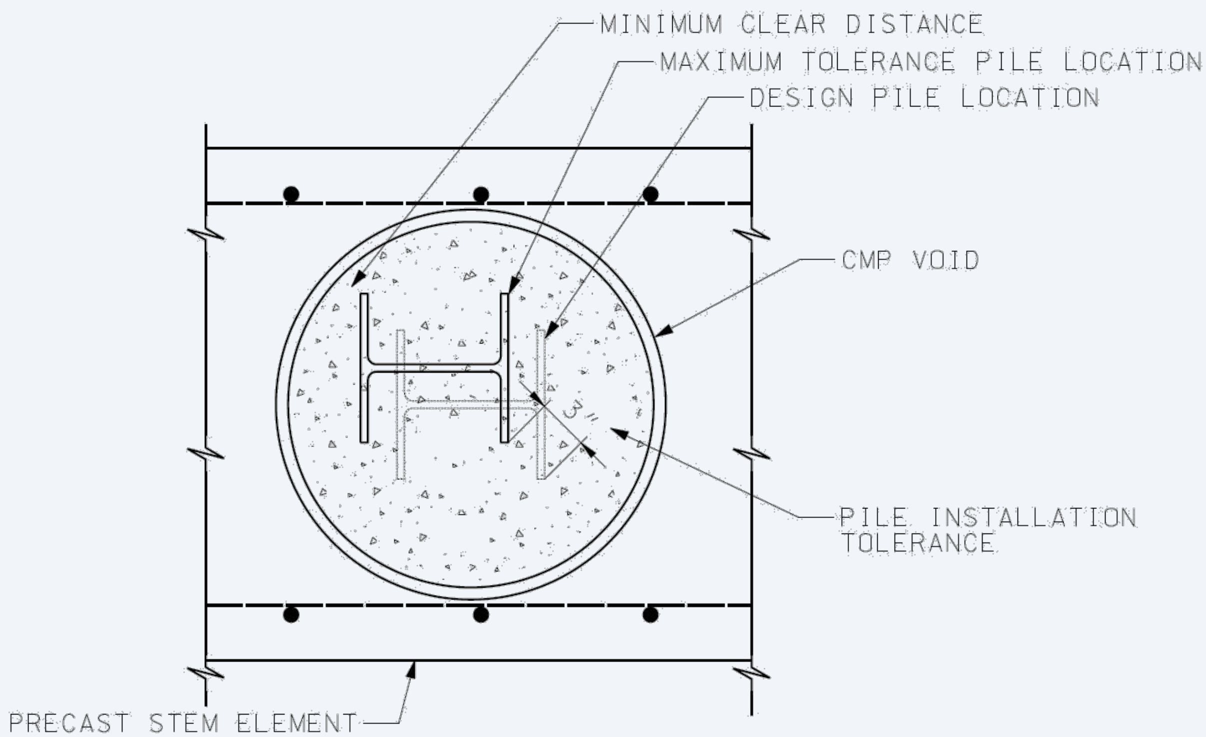
Assume Pile Tolerance = $\pm$ 3 inches
Assume Minimum Clear Distance = 1.5 inches

$$\text{Dpile} = \sqrt{12.1^2 + 12.2^2} = 17.2\ inches$$

Tpile = 3 inches

Dclear = 1.5 inches

Therefore: CMP ID Pipe size = 17.2 +2*(3) + 2*1.5 = 26.2 inches
Round up to the next largest 3-inch increment (pipe standard), use D = 27 inches.



PILE TOLERANCE DIAGRAM

Reference 36 and Section 5.1.1.4 contains more information on the design of this connection. A worked example of this connection is also included in Section 12.1.1.

5.1.3. Semi-Integral Abutments

Semi-integral abutments differ from integral abutment in that they do not contain a moment connection between the superstructure and the substructure. A joint is normally placed below the bottom of the beam. The joint can either be pinned (fixed against lateral movement) or an expansion joint (free to translate with thermal movement). The key to semi-integral abutments is that the joint in the system is located at the beam seat and not in the deck, where leakage could lead to deterioration. Figure 5.1.3-1 shows a detail of a semi-integral abutment that was developed by the PCI Northeast Bridge Technical Committee. This detail shows a CIP backwall that is attached to the end of the bridge girders. It is possible to design this detail with a precast backwall. The foam filler that is shown simply retains soil. It does not need to be water-tight. When combined with pervious backfill material, ground water will drain below the abutment stem and not through the joint.

Use of a very durable element such as a reinforced elastomeric pad will further minimize potential exposure damage. This abutment type is particularly advantageous for complete bridge superstructure systems that are installed on the foundation elements. The elimination of a cast connection can greatly speed the bridge installation.

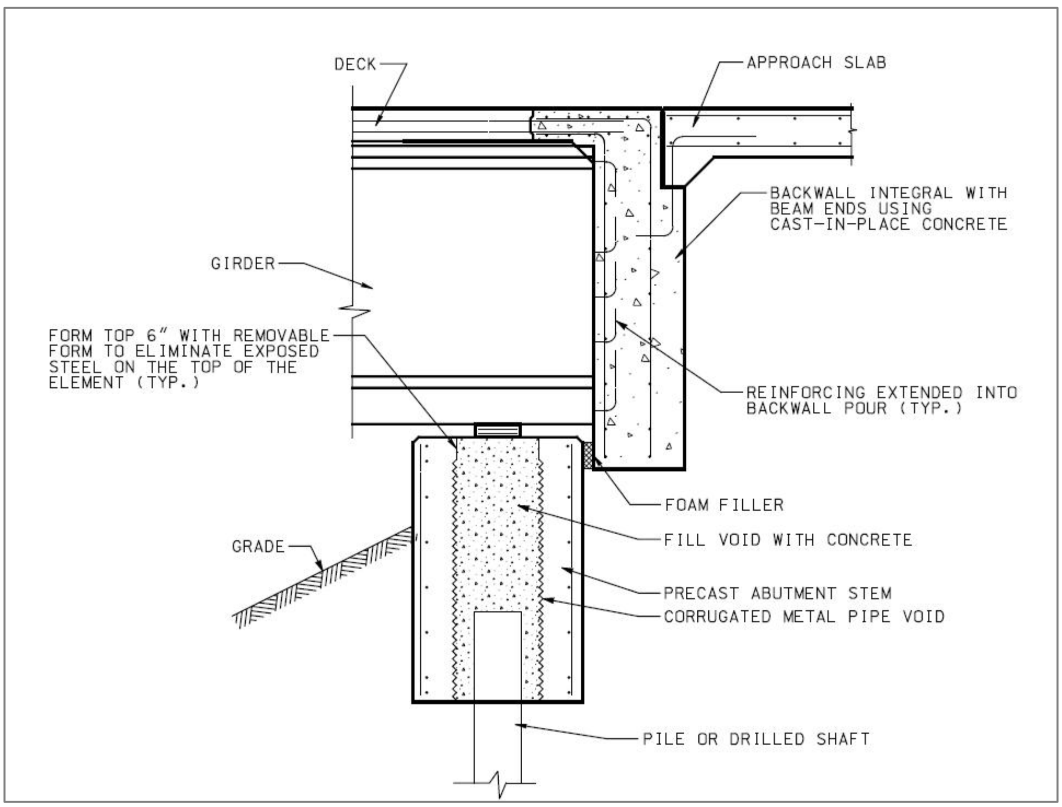


Figure 5.1.3-1. Semi-Integral Abutment Detail (Source: PCI Northeast).

5.1.3.1. Precast Backwalls

Figure 5.1.3.1-1 shows a similar detail, except that the backwall is precast. This is taken from a project that was built in Rhode Island. In this project, the precast backwall not only retained the soils behind the abutment, they also retained soils on the side of the abutment. The backwall was connected to the bridge deck through a closure pour, creating a three dimensional seal against water penetration to the bridge seat.

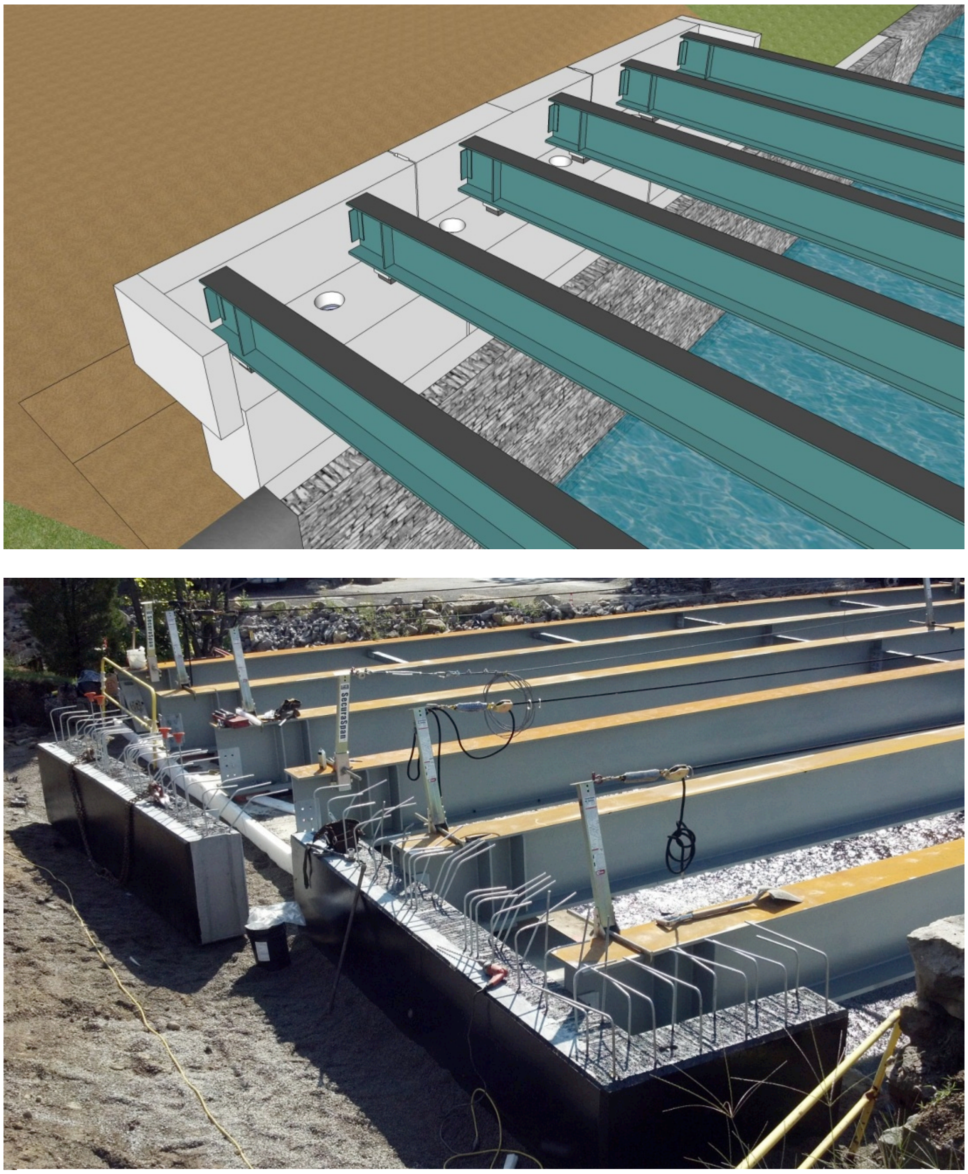


Figure 5.1.3.1-1. Precast Semi-Integral Abutment Backwall
Top: Sketch of Precast Backwall Concept
Bottom: Actual Bridge Construction.

The design of precast semi-integral backwalls is similar to the design of CIP semi-integral backwalls. The element needs to resist the soil forces acting on the backwall. The basic design assumption is that the superstructure resists the soil forces. The soil forces are transferred to the superstructure through the backwall. The precast backwall shown in Figure 5.1.3.1-1 resists these forces in several ways. First, the vertical reinforcing is anchored into the deck, providing resistance via a vertical cantilever. Second, the horizontal reinforcing in the precast backwall resists soil forces by spanning between beams. A simple shear key was used between the backwall elements. The horizontal reinforcing in the backwall can be designed based on the assumption that this is a pin.

Semi-integral abutments have some effect on the design of the beams and bearings. The weights of the backwalls and approach slabs are transferred to the beam end. In addition, the live load on the approach slabs are also transferred to the beam ends. This will need to be accounted for in the design of the bearing stiffeners and bearing. One benefit to this detail is that the backwall can act as the end diaphragm for the beams. It can be used to support the end edge of the deck and provide lateral stability to the beams. This greatly simplifies the detailing of the bridge and provides more room for inspection of the beam ends and bearings in the future.

5.1.3.2. Restraint (Longitudinal and Transverse)

There are several ways to provide restraint for longitudinal and transverse forces in semi-integral abutment bridges:

- Longitudinal forces can be resisted by the soil behind the backwalls via passive resistance.
- The overlap of the backwall shown in Figure 5.1.3-1 can be designed to resist longitudinal forces. Forces causing displacement away from the fill are transferred to the abutment stems via key action.
- CIP concrete keeper blocks or shear keys can transfer transverse forces into the abutment stem. These blocks can be cast after the beams and backwalls are erected. Figure 5.1.3.2-1 shows an elevation of a semi-integral abutment with a shear key.
- Fixed bearings with anchor bolts can be used to transfer longitudinal and transverse forces into the abutment stems.

Restraint in seismic regions is an important issue. The strength of the restraining system using precast elements can be difficult to achieve when compared to CIP concrete construction techniques. Published seismic retrofit schemes for existing bridges have served as a basis for many of the details used in ABC projects that involve full superstructure system installations. In these cases, CIP shear keys or keeper blocks can be installed after the bridge is set in position.

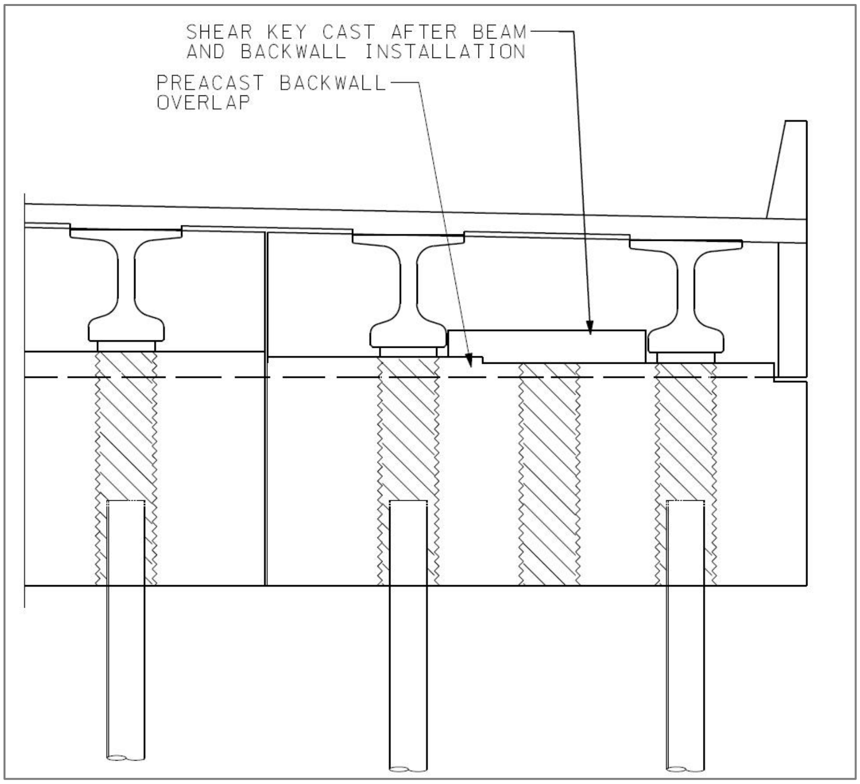


Figure 5.1.3.2-1. Semi-integral Abutment Elevation with Transverse Shear Key.

5.1.4. Geosynthetic Reinforced Soil Integrated Bridge System (GRS-IBS)

The FHWA has led the development of a bridge substructure method called Geosynthetic Reinforced Soil Integrated Bridge System (GRS-IBS). Figure 5.1.4-1 shows a cross section of a typical GRS-IBS bridge abutment.

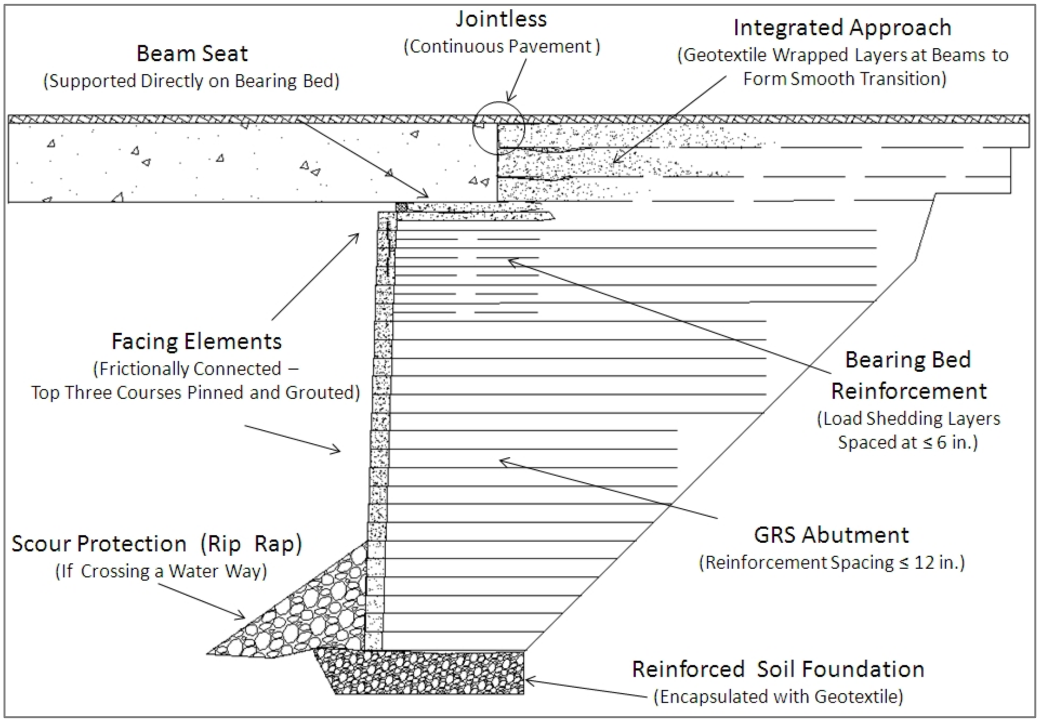


Figure 5.1.4-1. Geosynthetic Reinforced Soil Integrated Bridge System (GRS-IBS) (Source: FHWA).

The term “integrated” describes the fact that the bridge abutment and the approach embankment are integrated together to form a common structure. The idea with GRS-IBS is to have the bridge and the approach embankment supported on an integrated shallow foundation, that foundation being a geosynthetic reinforced soil mass. This system does not eliminate settlement, but it does eliminate “differential” settlement, which is the cause of the bump at the end of the bridge. The result is that the GRS-IBS bridge does not need approach slabs since the differential settlement is eliminated. The GRS-IBS system can significantly reduce the cost of the bridge.

GRS-IBS bridges are not for every location since they are not supported on deep foundations. The depth of the GRS system needs to account for anticipated scour. Additional scour protection can be added in front of the wall to minimize the scour. If this system is used on a multi-span bridge, the designer should carefully evaluate the potential for differential settlement that may occur between the abutment and the pier. Currently, FHWA is recommending that the maximum span length for this system be limited to 140 ft, which is the longest GRS-IBS bridge built to date. This span limitation may be increased in the future as more experience is gained. The current recommendation on the maximum GRS abutment height is 30 ft. As with the span length limitation, this may be increased in the future.

Virtually any type of superstructure can be supported on a GRS abutment. Figure 5.1.4-1 shows a precast concrete box beam; however, stringer bridges can also be used. A small footing can be cast under the beam seat. The beams can be set on this footing and an integral diaphragm can be cast to provide the backwall for the reinforced approach embankment. Figure 5.1.4-2 shows a steel girder bridge that was built using GRS-IBS.

The FHWA has developed a detailed guide for the design of GRS-IBS bridges [37]. This guide contains information on the design of the abutment and the approach embankment. Sample design calculations are also included. Guide specifications have also been developed by FHWA [38], which can be used for specifying this system on projects.

The bearing capacity of a GRS abutment is significant. The recommended maximum allowable bearing pressure is currently set at 4,000 lbs per square ft (service limit state). Figure 5.1.4-3 shows the design envelope for vertical capacity and strain with 8-inch reinforcement spacing. The graph shows that there is not failure plateau indicating that the ultimate capacity of the system is above 20,000 lbs per sq ft. Based on this, the recommended allowable bearing pressure of 4,000 lbs per sq ft is well below the ultimate capacity of the system.

Figure 5.1.4-2. GRS-IBS Bridge Built with Steel Girders.
Left: Concrete Footing Supporting Girders
Right: Backfilling after casting of Integral Diaphragm
(Source: FHWA).

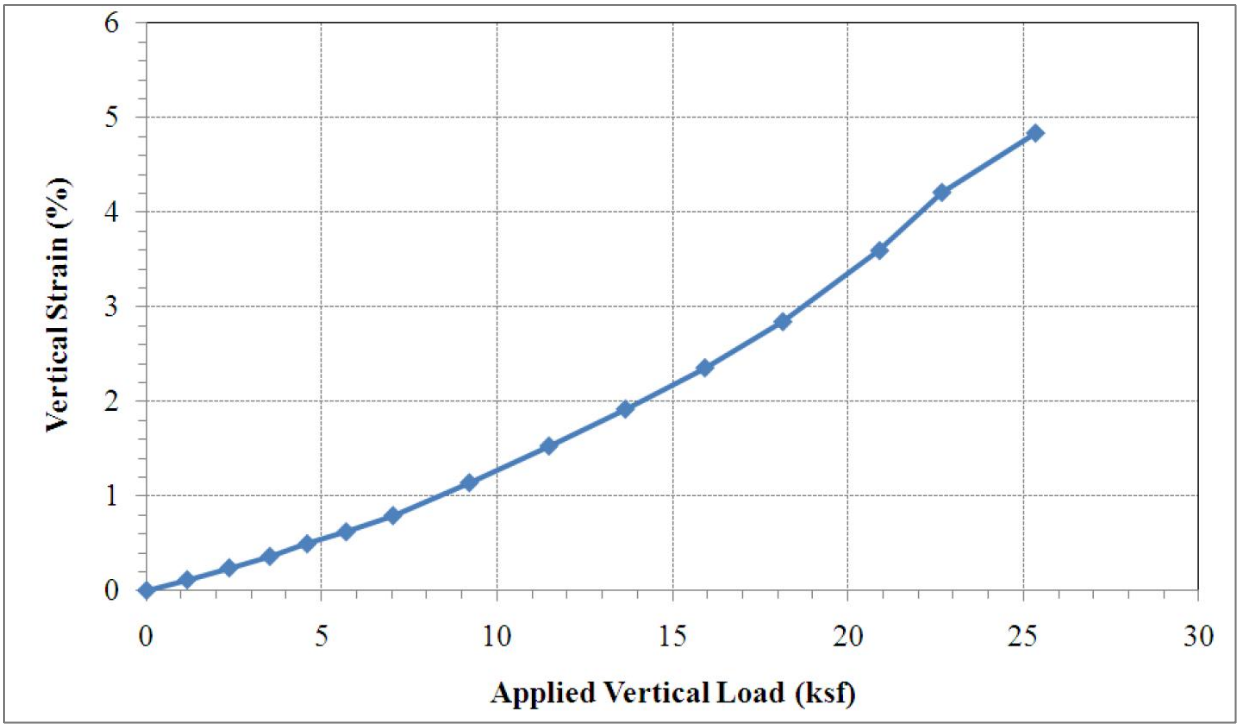


Figure 5.1.4-3. Design Envelope for Vertical Capacity and Strain of a GRS Structure with 8 Inch Reinforcement Spacing (source: FHWA GRS-IBS Implementation Guide [37]).

5.1.5. MSE Retaining Walls

Mechanically Stabilized Earth (MSE) retaining walls are very common in the U.S. They are comprised of precast concrete panels connected to reinforcing strips that are embedded into the backfill soils. Figure 5.1.5-1 shows a cross section of a typical MSE wall. The use of MSE accelerates the construction of walls since the curing of concrete is minimized (footing only), and backfilling and erection of the wall occur in parallel. MSE walls function by engaging the soil mass behind the wall face to form an earth gravity wall system.

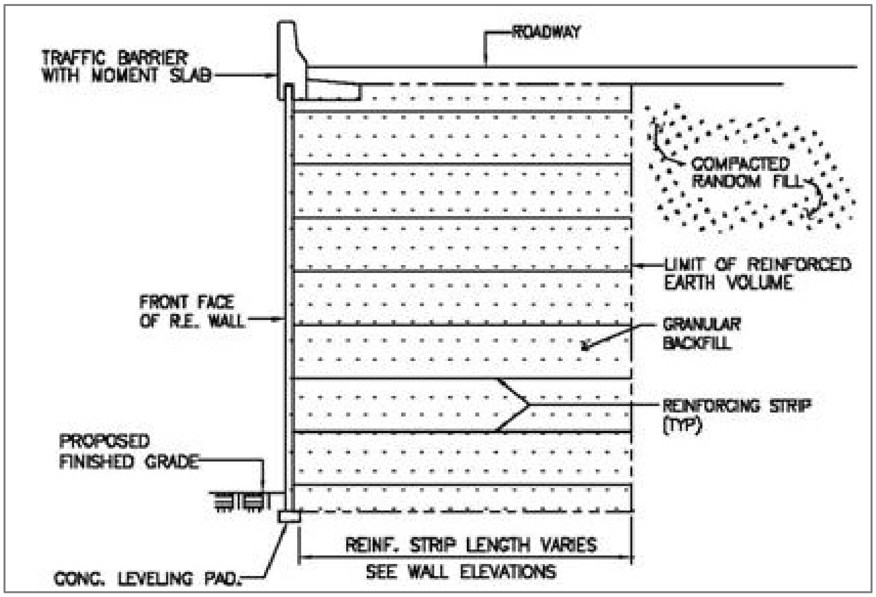


Figure 5.1.5-1. Mechanically Stabilized Earth Wall Section.

MSE walls are commonly used for retaining walls and wingwalls. Many states also use MSE walls for abutments; however, the walls typically do not support the bridge. Piles or drilled shafts are installed prior to wall construction. The MSE wall is then typically built in front of the piles with the reinforcing strips placed between the piles. Once complete, a concrete footing is installed on top of the piles, creating two separate structures. Some states have experimented with placing spread footings on top of MSE walls in a similar fashion to GRS-IBS bridges; however, this approach is not common.

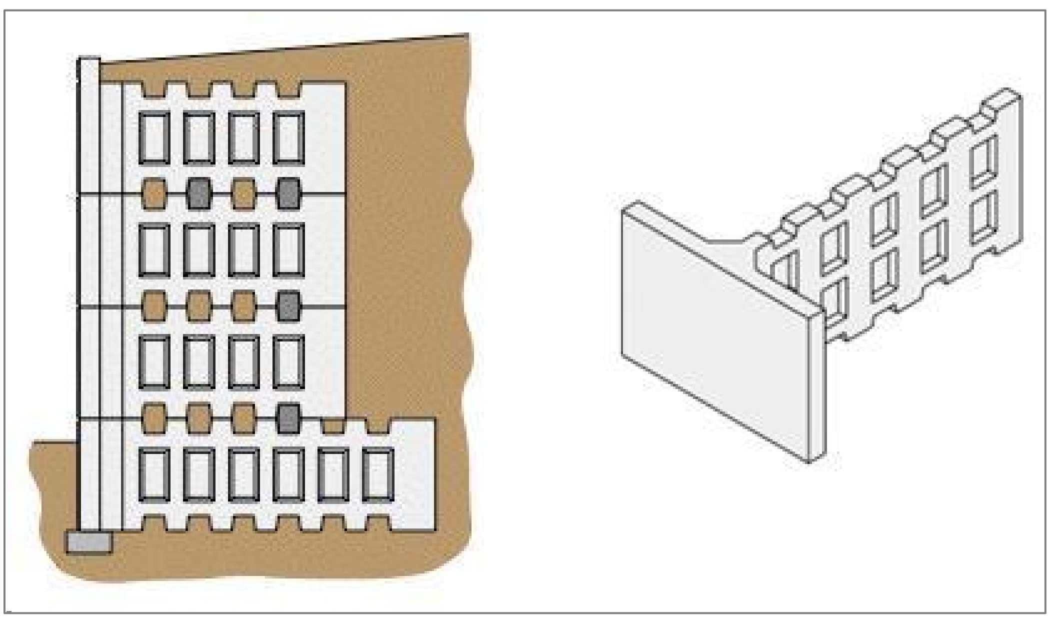


Figure 5.1.6-1. Precast Modular Wall Example (T-WALL®)
(Source: The Neel Company).

The design of MSE walls is well documented in Section 11.10 of the AASHTO LRFD Bridge Design Specifications [5]; therefore, it is not covered in this manual.

5.1.6. Prefabricated Modular Retaining Walls

One of the most basic wall construction techniques is to build mass gravity walls. These walls resist soil forces through the shear mass of the wall. Gravity walls can be constructed with prefabricated elements made of precast concrete modules, precast concrete cribbing, or steel bins. Figure 5.1.6-1 shows details of modular retaining walls.

Prefabricated modular retaining walls can be cost-effective since they use relatively little concrete and steel. The systems make use of backfill soils within the blocks and bins to create the mass for the structure.

The design of prefabricated modular retaining walls is well documented in the Section 11.11 of the AASHTO LRFD Bridge Design Specifications [5]; therefore, it is not covered in this manual.

5.1.7. Soldier Pile Retaining Walls

Soldier pile walls are constructed by installing driven piling or drilled shafts at relatively close spacing, and installing a structure between the piles to retain the soil behind the wall. The resistance of the wall is developed by the lateral resistance of the piles. For tall soldier pile walls, the resistance can be improved by the addition of tie back soil or rock anchors. It is possible to design and detail a soldier pile wall that makes use of precast concrete elements. Figure 5.1.7-1 shows a detail of a semi-integral abutment that was developed by the PCI Northeast Bridge Technical Committee. This detail shows a schematic precast concrete soldier pile wall that makes use of a corrugated metal pipe void for the connection of the pile or shaft to the wall element. Section 5.1.2.2 of this manual describes the use of CMP voids and the research that was used to develop this connection.

There are several ways to detail these walls. The stem can get wide depending on the size of the pile and the void. Section 5.1.2.2 includes information on sizing the pile void. In many cases, the wall stem will approach 4 ft thick. Additional CMP voids can be used to reduce the shipping weight of the wall stem. Once set, the voids can be filled with concrete or other fill material. Another option is to detail the wall with a stepped rear face. This will reduce the amount of concrete required, but increase the forming cost for the section. The stepped option may be desirable for long walls that require many elements. Figure 5.1.7-2 shows these two options. The joints between the wall elements are simple grouted keyways.

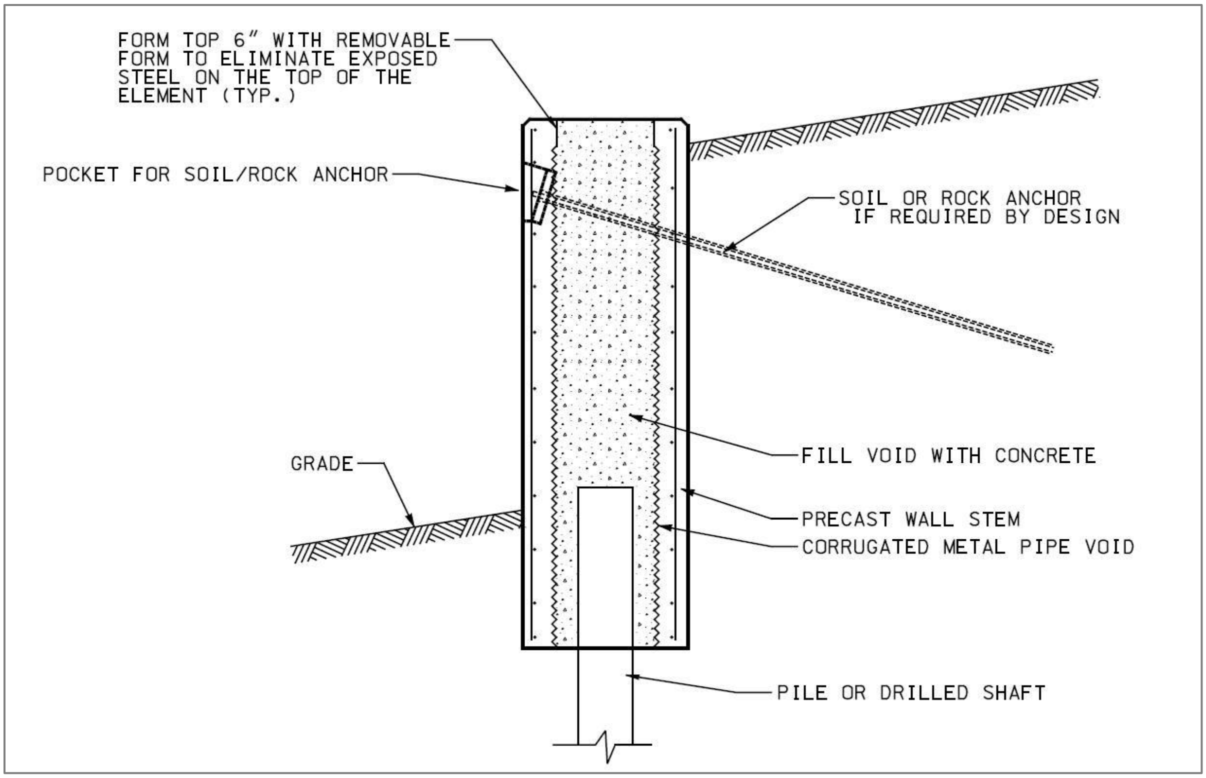
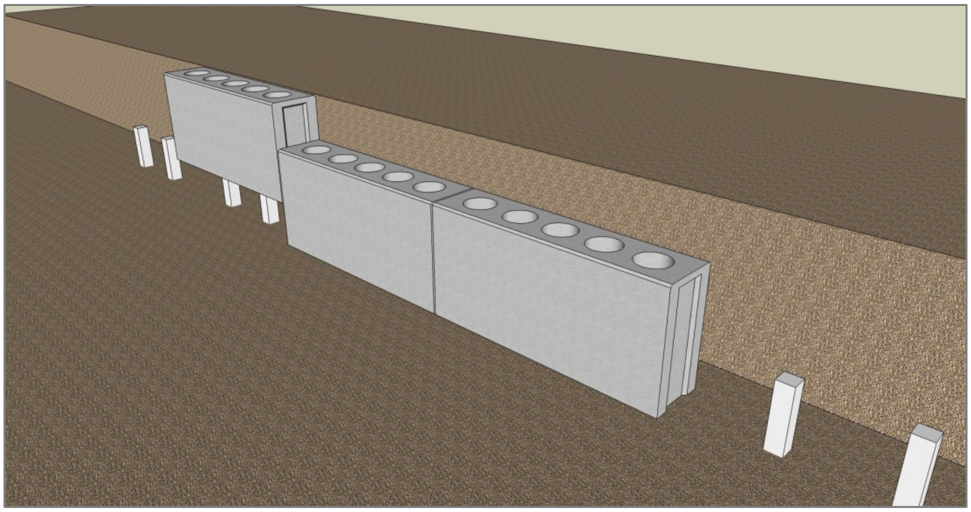


Figure 5.1.7-1. Precast Concrete Soldier Pile Wall (Source: PCI Northeast).

The design of precast soldier pile walls is covered in several sections of the AASHTO LRFD Bridge Design Specifications [5]. Chapter 11 can be used to determine the forces acting on the wall, and Chapter 10 can be used for the design of the piles. The design and detailing of the internal reinforcing is designed based on the assumption that the joint between the walls is a pinned connection.



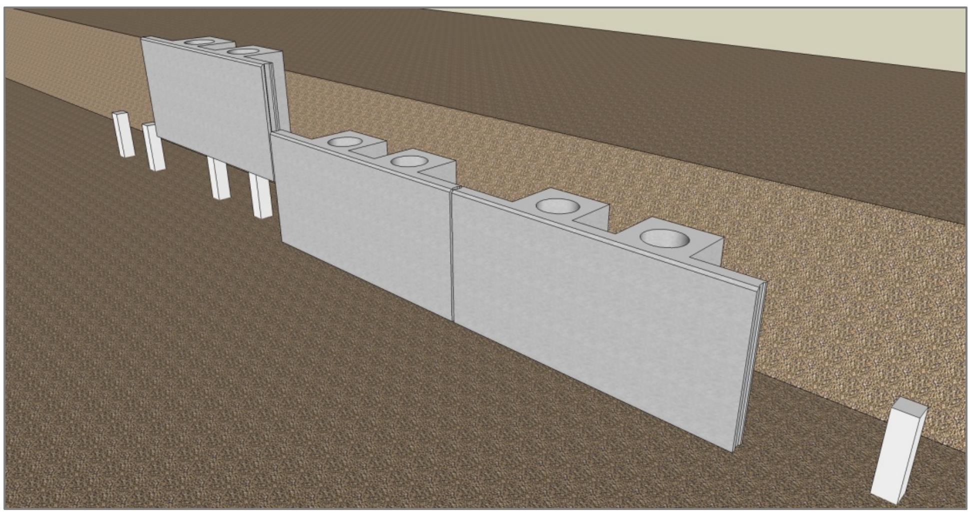


Figure 5.1.7-2 Precast Soldier Pile Wall.
Top: Wall with Extra Voids to Reduce Weight
Bottom: Wall with Stepped Rear Face to Reduce weight
(Source: PCI Northeast).

5.1.8. Backfilling

The use of prefabricated wall elements can accelerate the construction of substructures and walls by eliminating most of the site-cast concrete. This approach makes the backfilling of embankments the critical path to the completion of the project. There are several methods that have been used to accelerate the installation of structure backfill. Each has its own merits. The designer should evaluate each option and choose the best option to suit the conditions on each project.

5.1.8.1. Compacted Granular Fill

The most common form of backfill for structures is compacted granular fill. Granular fill offers structural integrity for loads placed above the fill. It has an added benefit in that it can allow for free drainage of groundwater. The groundwater can either be removed from behind the structure via weepholes in the wall structures, or through the use of a structure under the drain system.

Compacting granular fill can take considerable time. Most granular fills need to be installed in lifts of between 6 and 12 inches. Transportation agencies have long-standing specifications on the installation and compaction of granular fills. The simplest way to accelerate granular fill installation is to increase the amount of equipment that is used to place and compact the fill. There are limits to the use of more equipment, especially in confined areas such as the back of small abutments.

The New Hampshire DOT designed an accelerated construction project that specified options for compacted granular fill or flowable fill. The contractor chose compacted fill due to its lower cost and simplicity of installation. The contractor was able to backfill two abutments in one day with added work crews and equipment. Based on this, the use of granular fill should not be ruled out in an ABC project, especially if the volume of fill is not excessive.

Not all granular fills compact at the same rates. Standard specifications for granular fills have ranges of gradations. It is possible to have two granular materials that meet the specifications, but require different compaction needs in order to meet the specified compaction requirements. Contractors should investigate different sources of granular fills and consider a compaction test area prior to the start of construction. In some cases, a more expensive fill material will compact at a faster rate and be more attractive to a contractor working on an accelerated project.

Designers should consult with construction management staff to estimate the volume of granular fill that can be placed in a given area. Armed with this information and an anticipated construction schedule, the designer can make an educated decision regarding the use of granular fills.

The FHWA is studying the use of a new emerging technology called intelligent compaction (IC). This is a process where the compaction equipment monitors the work in progress through the use of vibration monitoring devices. The compaction machines are able to sense when the materials are properly compacted, which eliminates the need for re-compaction after a failed test. It also eliminates over-compaction of the materials being placed.

IC equipment has another added benefit in that the equipment has the ability to continuously monitor the compaction in progress and record the data in an on-board computer using global positioning technology. Once complete, the equipment can provide compaction data on every square foot of the compacted area, versus discrete compaction testing using conventional testing equipment. This leads to improved quality control of the materials being placed. IC can be used for sub-soils, sub-base, and bituminous pavement.

Intelligent compaction technologies are part of round two of the FHWA Every Day Counts initiative (EDC2). More information can be obtained at the FHWA Every Day Counts website at: www.fhwa.dot.gov/everydaycounts.

5.1.8.2. Flowable fill

Backfilling of structures can be a time consuming process. Compaction and compaction testing can slow the progress of the work. The process of backfilling structures can be expedited through the use of flowable fill materials. Section 2.1.4 contains information on this material and how it is formulated.

Flowable fill will not settle or rut under loads, making the material an ideal pavement base or fill under bridge approach slabs. It can be placed quickly and can support traffic within hours after placement. Setting and early strength gain time can be as short as one hour, but can take up to 8 hours, depending on the mix design. Some agencies have reported excessive settlement of thick flowable fill placements during curing. This should not be a problem for voids that are relatively thin, such as bedding for spread footings. Thicker placement may require multiple lifts. The first lift can be the primary thick lift, with the second lift being the leveling course (thin).

If flowable fill is used for backfilling, there will be a need to form the extents of the pour in order to retain the material. The time required to form the area may be detrimental to the production rate of backfilling. In these cases, compacted fill may be a better solution.

5.1.8.3. Expanded Polystyrene Geofoam Fill

Expanded Polystyrene (EPS) is formed from polystyrene resins, beads, or granules. EPS is a product that was developed for other industries, where it is used for insulation and packaging. It was determined that the material could be used for lightweight fill in embankments. An added benefit to this product is that it can be placed rapidly. EPS also can be molded or extruded to form various shapes. The material is typically formed into solid blocks for highway applications.

Initial specifications for EPS were based on the use of the product for insulation. Recently, an ASTM specification was adopted for use of the material in geotechnical applications. The current specification is ASTM D6817-11 Standard Specification for Rigid Cellular Polystyrene Geofoam [39]. The industry is working with AASHTO to develop material and design guidance. Table 5.1.8.3-1 shows the initial recommendations for inclusion in AASHTO LRFD Bridge Design and/or Construction Specifications [5, 29]. Future revisions to these specifications may include this and more information on the use of EPS Geofoam for highway applications.

Table 5.1.8.3-1. Proposed AASHTO Geofoam Specifications
(Source: AFM Technologies).

Property		Type			
		EPS40	EPS50	EPS70	EPS100
Block Density Minimum	lb/ft^3	1.00	1.25	1.50	2.00
Elastic Limit Stress minimum	psi	5.8	7.2	10.1	14.5
	psf	835	1036	1454	2088
Initial Tangent, Young's Modulus	psi	580	725	1015	1450
ASTM D6817 Type recommended to meet proposed AASHTO Geofoam Specification		EPS19	EPS22	EPS29	EPS39

A review of this information clearly indicates that the material cannot support large point loads. Geofoam fills require a load distribution slab or layer of fill to distribute point loads to the geofoam. The load distribution slab or layer of fill will also protect the EPS from ultraviolet light and chemical attack.

The information also shows the minimal weight of the materials. A typical block would be 4 ft x 8 ft x 3 ft. This would only weigh 192 lbs for the highest density product. This means that a small work crew could place the material without equipment,leading to very fast construction.

Care should be taken for fill areas that are subjected to high water tables. With a unit density of less than 2 pcf, the material can generate significant buoyancy forces when subjected to high water. This should be investigated by the designer. If high water tables are present, then the material either needs to be weighted down or not used below the anticipated high water level.

Figure 5.1.8.3-2 is a photograph of a typical EPS Geofoam embankment prior to placement of the roadway structure and side slopes.

Figure 5.1.8.3-2. EPS Geofoam Embankment (Source: ACH Foam Technologies).

The lightweight nature of this material makes it ideal for areas where long-term settlement potential is high. The embankment weight can be reduced by a factor of up to 100, which will greatly reduce the amount of long-term settlement and the need for embankment pre-load time. More information on this subject can be obtained at the FHWA website: www.fhwa.dot.gov/research/deployment/geofoam.cfm.

5.1.8.4. Open Graded Fill

Open graded materials consist of clean, crushed angular stone (not round). The maximum grain size to efficiently achieve compaction behind the abutment wall face is 0.5 inches. An example of open graded fill is AASHTO No.89 stone fill. Table 5.1.8.5-1 shows information on this material.

Gradation (AASHTO M-43)	U.S. Sieve Size	Percent Passing
	1/2 inch	100
	3/8 inch	90-100
	No. 4	20-55
	No. 8	5-30
	No. 16	0-10
	No. 50	0-5
Plasticity Index (PI) (AASHTO T-90)	PI $\leq$ 6	
Soundness (AASHTO T-104)	The backfill shall be substantially free of shale or other poor durability particles. The material shall have a magnesium sulfate loss of less than 30 percent after four cycles (or a sodium value less than 15 percent after five cycles).	

Figure 5.1.8.5-1. Typical Open Graded Fill Material
(Source: FHWA GRS-IBS Interim Implementation Guide [37]).

Compaction of open graded fill is easier, and thereby faster than conventional well graded fills. An added benefit is that the material has better drainage characteristics when compared to other fill materials.

5.2 DESIGN OF PREFABRICATED PIERS

The design of bridge piers constructed with prefabricated elements is similar to the design of piers constructed with CIP concrete. Most designs are based on emulative design concepts, where designers emulate CIP concrete with precast concrete. Some agencies have designed bridge piers with prefabricated steel elements. This type of construction will be faster than construction with grouts and concrete pours (curing of materials is not required). The design of steel piers should follow normal steel design criteria as specified in the AASHTO LRFD Bridge Design Specifications [5].

Bridge designers have expressed concern over the performance of joints in precast structures. The reality is that virtually all concrete structures have joints. For instance, the footing of a pier is cast first with projecting reinforcing bars. Once the concrete has cured, the column or wall stem is cast on top. The interface between the two pours is a construction joint. Emulation design replaces this construction joint with a connection that may include a mechanical connector, a grouted void, or a closure pour.

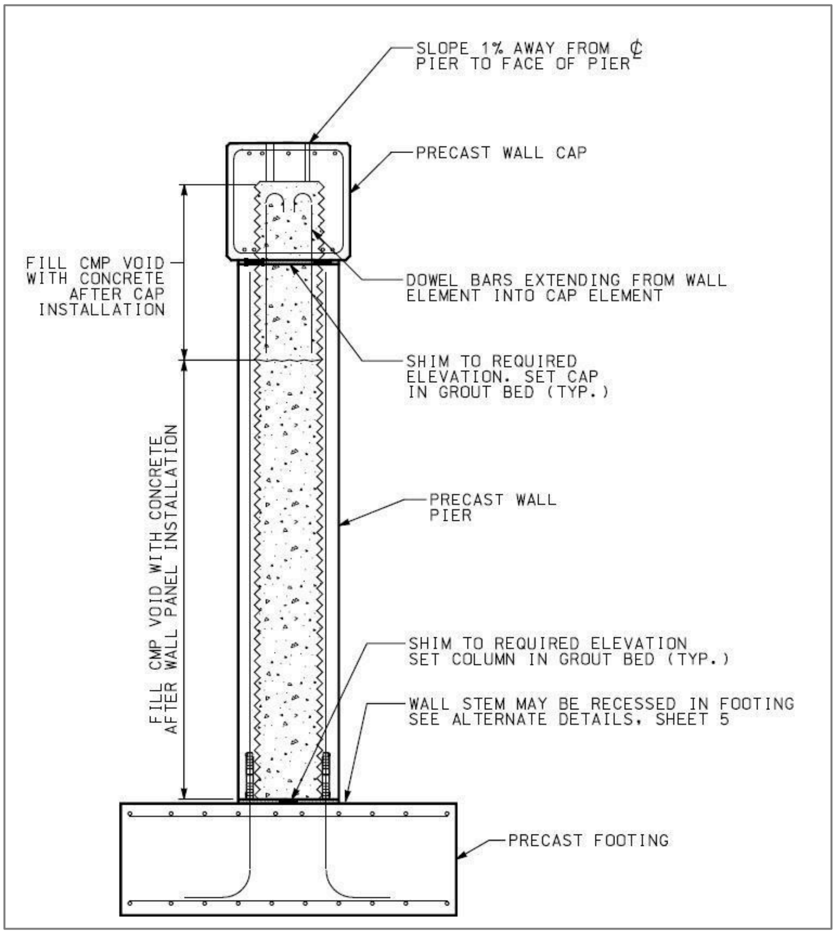
Designers have also expressed concern over the use of mechanical connectors in joints. The concerns are with respect to the structural behavior of the connector that is used to transfer force from one reinforcing bar to another. This concern should not be an issue because most concrete structures contain transfer mechanisms for reinforcing bars. The most common mechanism is a lap splice. The theory of lap splices is that the force from one bar is transferred to the adjacent bar via the surrounding concrete. Mechanical connectors use similar but different means to transfer these forces; however, the result is the same. Force is transferred from a bar to the surrounding concrete or grout and then to an adjacent bar. Section 5.2.3 contains more information on some of the most common element connections used in piers.

5.2.1. Pier Bents

The design of pier bents is typically accomplished through the use of frame analysis software. The forces acting on the superstructure are transferred to the substructure via bridge bearings or integral diaphragms. The direction of each force needs to be carefully accounted for and then grouped according to the appropriate load combinations as specified in the AASHTO LRFD Bridge Design Specifications [5]. The frame analysis software will produce forces in each member and at each connection. The design engineer then applies the principles of concrete design (or steel design in the case of a steel pier bent) to design each member.

5.2.2. Wall Piers

Wall piers are similar to cantilever abutments in that they have a footing and a vertically cantilevered stem that is reinforced to resist vertical and lateral forces. The difference is that they typically do not support soil, but they do resist lateral forces such as vehicle collisions. The design and detailing of wall piers follows the same procedure as abutments. Figure 5.2.2-1 shows details of a precast wall pier that was developed by the PCI Northeast Bridge Technical Committee. The details show precast footings, precast stems, and a precast concrete cap. The connections include grouted reinforcing splice couplers at the base and dowels cast into a closure pour within the corrugated metal pipe voids at the top of the stems. See Section 5.1.1 for more information on the design of cantilever wall structures.



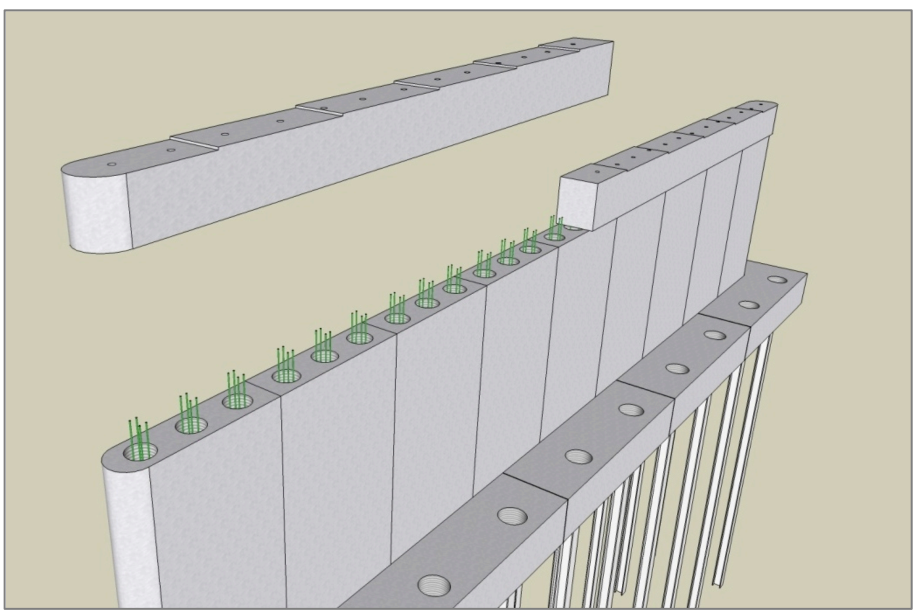


Figure 5.2.2-1. Precast Wall Pier Details and Sketch
(Source: PCI Northeast).

5.2.3. Design of Pier Connections

The only design change with respect to prefabricated concrete elements is if the mechanical connections have an impact on the location and spacing of the internal reinforcing bars. Two common reinforcing bar splice devices are grouted post-tensioning ducts and grouted reinforcing splice couplers. Each of these has devices that are larger than the connected bars. Typically, designers strive to have adequate concrete cover on all metallic devices (including mechanical couplers). In order to achieve this, the connected reinforcing bars must be placed deeper within the element. The result is that the effective depth of the concrete section is reduced, which will also reduce the resistance of the element. To resolve this, the designer will need to add more reinforcement for a given section. In some cases, the element depth may need to be increased in order to accommodate the forces. Figure 5.2.3-1 contains a calculation for simple moment connection with both standard reinforcing and a mechanical connector. Emulation design of connections with grouted post-tensioning ducts follows a similar approach. Designers should ensure that all AASHTO provisions be checked with the use of larger bars, including serviceability requirements.

When using mechanical connections in pier caps, the designer also should investigate the size and spacing of the connections and their impact on the layout of reinforcing in the pier cap. Figure 5.2.3-2 shows a precast pier cap being installed on a bridge in Utah. This connection made use of grouted reinforcing splice couplers. An inspection of this photograph shows that the spacing of the couplers near the side face of the cap was adjusted to allow for the passage of the internal cap reinforcement (red circle). The round pattern of the couplers created an interference with the longitudinal bars. The pattern of the couplers on the interior of the cap was adequate to allow for passage of the longitudinal cap bars between the couplers (yellow circle). In this case, the designer had to adjust the design of the column to account for this bar pattern. This example is a hammerhead pier cap with minor amounts of reinforcing in the bottom of the cap. For multi-column bents with continuous cap beams and more bottom reinforcing, this potential interference issue may become more pronounced. As with the beam example in Figure 5.2.3-1, the use of fewer larger bars is preferred.

Example Calculation: Emulative Detailing of Reinforcement with Grouted Splice Couplers

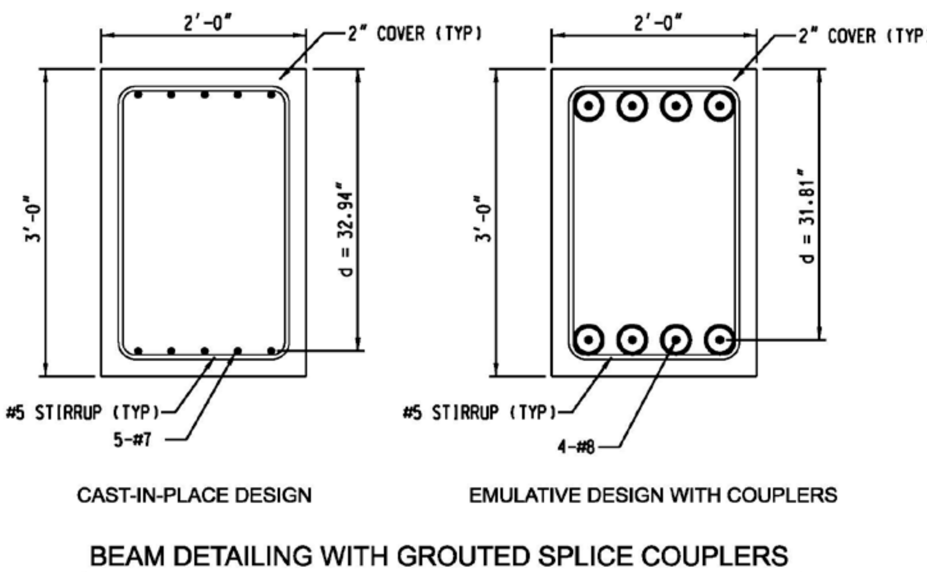
Approach: Calculate equivalent beam sections

Note: A Beam Section is shown for simplicity. Emulative column calculations are more complex, but follow a similar approach.

Given: Section shown to the left below
Factored applied moment = 5100 in-kips

Using standard beam design calculations, the flexural resistance of the sections can be calculated. Adding grouted couplers affects the location of the flexural reinforcing, which affects the bending resistance. The following table shows the key variables that are affected by this change and several options for the emulative design:

	Reinforcement	Coupler dia.	Flexural Bar Cover	Design d	ΦMn	Result
Original Design	#5-#7	N/A	2.625	32.94	5157	OK
Trial 1	5-#7 w/ coupler	3	3.6875	31.88	4985	NG
Trial 2	4-#8 w/ coupler	3.5	3.875	31.81	5198	OK



This example demonstrates the common practice of using smaller numbers of larger reinforcing bars when employing emulative design with mechanical couplers.

Figure 5.2.3-1. Emulation Design Example.

Figure 5.2.3-2. Layout of Bar Couplers in Pier cap
(Source: Hanson Structural Precast, Salt Lake City).

5.2.3.1. Grouted Reinforcing Splice Couplers

Grouted reinforcing splice couplers were developed in the building construction industry many years ago. There are at least three companies with similar sleeves in the U.S. The couplers consist of a cast steel sleeve that connects to the reinforcing steel on either side of the coupler. The couplers are typically placed on one side of the joint between elements. The connection is made by inserting the bar in the sleeve and grouting the void in the sleeve. (One company uses a threaded bar connection on the side of the sleeve that is embedded in the element.) Figure 5.2.3.1-1 shows a typical coupler connection.

Figure 5.2.3.1-1. Grouted Reinforcing Splice Coupler
(Source: Splice Sleeve North America).

Figure 5.2.3.1-2 shows a detail of a typical column-to-cap connection using grouted reinforcing splice couplers that was developed by the PCI Northeast Bridge Technical Committee. The detail shows a sloping cap with a recessed joint. It is not necessary to recess the joint, especially if the cap is flat. The recess does improve the shear resistance of the connection. Without a recess, the designer should check the shear resistance of the connection using the AASHTO LRFD Provisions [5] for interface shear (Article 5.8.4).

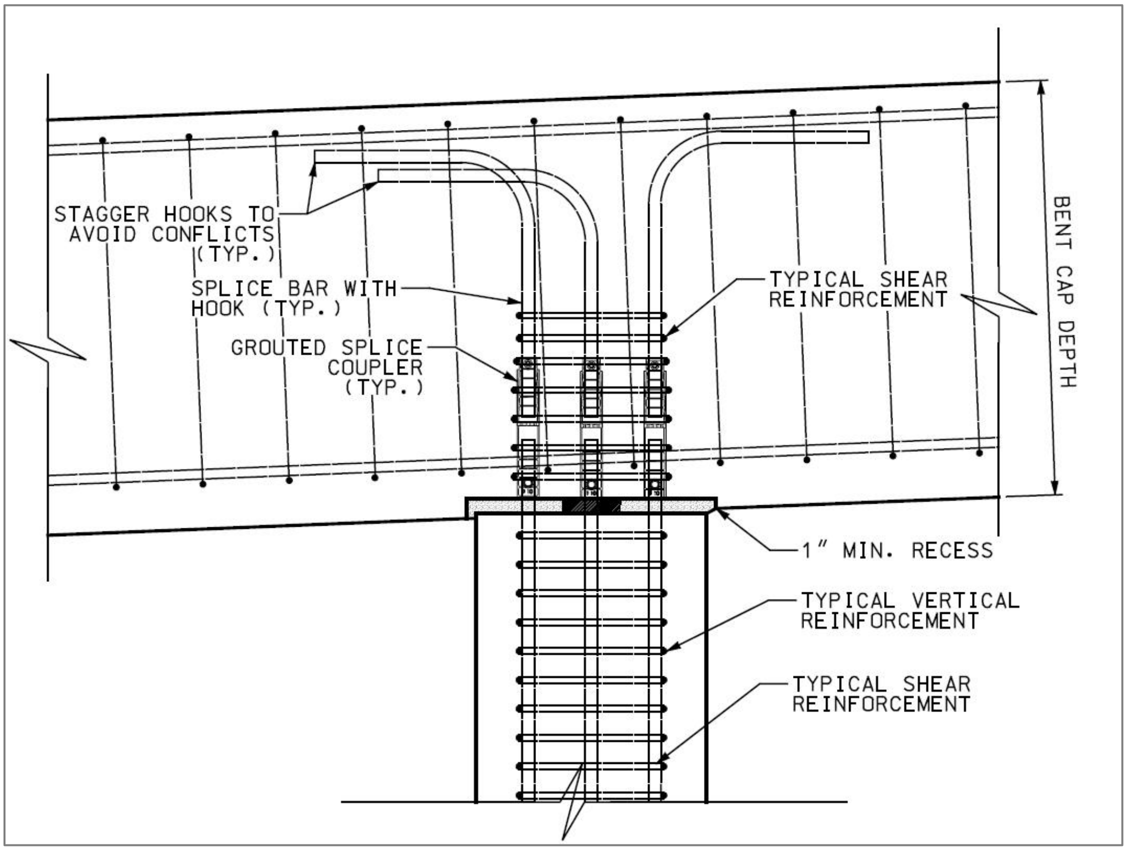


Figure 5.2.3.1-2. Column to Cap Connection using Grouted Reinforcing Splice Couplers (Source: PCI Northeast).

No special design calculations are required for these connectors, which makes their use appealing to many designers. The force from one bar is transferred to the grout via the deformations on the bar, then the force is transferred to the sleeve via deformations on the inside face of the sleeve, then the force is transferred to the adjacent bar through the same mechanism.

In theory, this connection is akin to a bar lap splice, where the bar force is transferred to the surrounding concrete and then to the adjacent spliced bar. This makes this connector a true “emulative” connection, which will behave the same as a CIP construction joint. Figure 5.2.4-1 shows test results of a grouted reinforcing splice coupler joint subjected to seismic loads. The top plot is a CIP concrete specimen and the bottom plot is a specimen with grouted couplers. An examination of these plots shows that the connection behaves in a true emulative fashion.

The AASHTO design specifications allow the use of mechanical bar connectors as long as the connector can develop 125 percent of the specified bar yield strength with minimal slippage (AASHTO Article 5.11.5.2.2). Most couplers in use in the U.S. can comply with these requirements; therefore, grouted couplers can be substituted for reinforcing bar lap splices. The AASHTO specifications do have limitations on the use of these connectors in seismic regions, however. Section 5.2.4 contains more information on this topic.

The only design change required when using grouted reinforcing splice couplers is that the reinforcing cage should be recessed farther into the concrete core in order to achieve adequate cover over the couplers. This will affect the capacity of the element since the effective depth of the member will be reduced. The designer should specify the location of the reinforcing cage used in the calculation in the details. This will be based on the diameter of the couplers. Different manufacturers have different dimensions; therefore the designer should specify dimensions that can be met by all manufacturers. Figure 5.2.3.1-3 is a table of dimensions that can be met by the three most common manufacturers in the United States.

BAR SIZE	OUTSIDE DIAMETER (IN.)	LENGTH OF COUPLER (IN.)
4	2.625	14.125
5	3.000	14.125
6	3.000	14.125
7	3.000	18.75
8	3.500	18.75
9	3.500	18.75
10	3.500	23.5
11	4.000	23.5
14	4.000	28.375
18	4.500	39.625

USE THIS TABLE FOR DETAILING OF ELEMENT REINFORCEMENT INCLUDING SPACING, COVER, AND EMBEDMENT LENGTHS. IN MOST CASES, THESE DIMENSIONS WILL WORK FOR OVERSIZED COUPLERS. IF THE FABRICATOR ELECTS TO OVERSIZE A COUPLER, THESE REQUIREMENTS SHALL BE CHECKED DURING THE DEVELOPMENT OF SHOP DRAWINGS.

SOURCES: MATERIAL SPECIFICATIONS FROM THE THREE MOST COMMON SUPPLIERS (NMB SPLICE SLEEVE, LENTON-ERICO, DAYTON SUPERIOR)

Figure 5.2.3.1-3. Typical Sizes of Grouted Reinforcing Splice Couplers (Source: PCI Northeast).

Some of the companies have proven through testing that it is possible to achieve the required AASHTO performance using oversize couplers. For instance, a number seven coupler may be able to be used on a number six reinforcing bar. One company is now producing standard couplers with a larger end opening on one side. These approaches give more flexibility in tolerances for the fabrication and erection of the elements. Designers should specify that this approach will only be allowed if supported by acceptable testing data.

One of the key aspects of these products is the grout used in the coupler. Designers need to know that the connector is a system, and the grout is part of the system. It is not necessary to specify the type of grout in the coupler. It is important to specify that the grout used in the field matches the certified test report for the coupler.

5.2.3.2. Grouted Post-Tensioning Ducts

Several research projects have been completed that investigated the use of corrugated metal post-tensioning ducts for connections in piers [40-43]. Reinforcing bars are inserted into the ducts and grouted into place. The basic concept is somewhat similar to grouted reinforcing splice couplers. The main difference is that the post-tensioned ducts do not "splice" the bars; they simply "develop" the bars in the mass concrete. If splicing is required, it is achieved through the use of a non-contact lap splice. The spliced bar is set outside the duct in the precast element within the requirements specified in Article 5.11.5.2.1 of the AASHTO LRFD Bridge Design Specifications [5] for non-contact lap splices. The following is the text from this article:

"Bars spliced by noncontact lap splices in flexural members shall not be spaced farther apart transversely than one-fifth the required lap splice length or 6.0 inches."

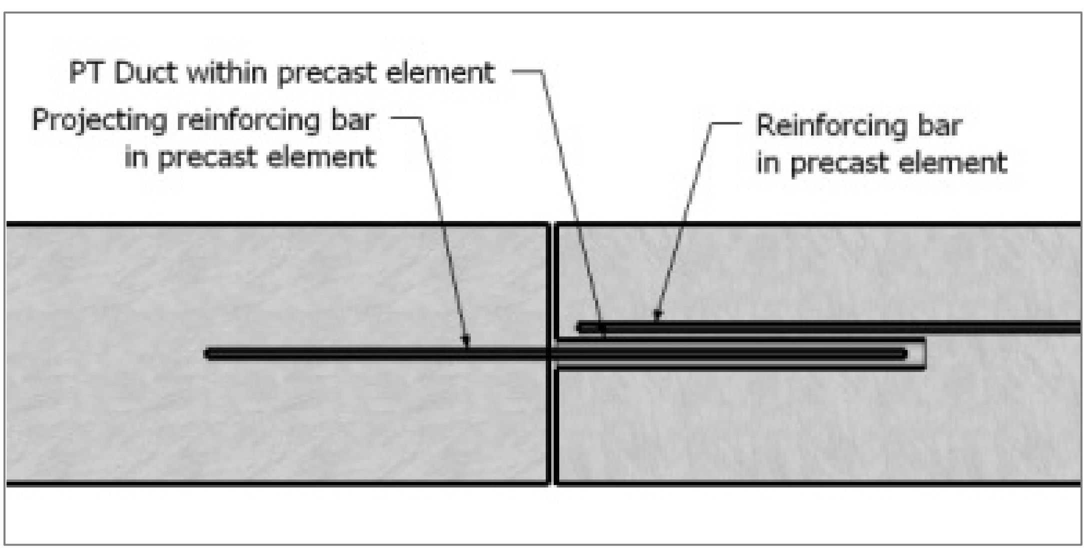


Figure 5.2.3.2-1. Grouted Post-Tensioning Duct Connection.

If the bar detailing meets this specification, then the detail should be considered the same as a conventional lap splice.

The connection is made by injecting grout into the embedded post-tensioned duct through standard post-tensioned duct grout ports. The space between the elements also needs to be grouted to complete the connection.

Grouted duct connections can be used in any connection where projecting bars are inserted into adjacent elements. The most common connections are “column to cap” connections, where the ducts are placed in the cap, and “column-to-column” connections, where the ducts are placed in one of the connecting column segments. Figure 5.2.3.2-2 shows a pier cap connection that uses a grouted post-tensioned duct connection. The photo shows the test cap element prior to casting.

Figure 5.2.3.2-2. Column to Cap Connection using Grouted PT Ducts Connection (Source: NCHRP Project 12-74).

The design of pier elements when using corrugated metal duct connections is essentially the same as with conventional CIP concrete elements. This is due to the fact that the connections are emulative of construction joints. It is important for the designer to investigate the detailing of the connections during the design phase. This is due to the size of the ducts. As with grouted reinforcing splice couplers, the location of the bars needs to be set deeper within the element in order to achieve adequate cover over the metal duct. This will affect the design of the element.

Section 5.2.3 includes an example of the effect of this connection on the design of the element. The size of the duct will have an impact on the reinforcing design, and possibly the size of the element. Designers should investigate the use of fewer larger bars as opposed to many smaller bars.

Grouted post-tensioned duct connections have been tested for seismic resistance. The performance of the connection, when combined with adequate supplemental reinforcement, provides excellent performance as a ductile column connection. This will be discussed in more detail in Section 5.2.4. The research documents listed above [40-43] contain much more information on the design of connections using grouted post-tensioned ducts. The work also contains recommendations for construction specifications.

5.2.3.3. Corrugated Metal Pipe Voids

Section 5.1.2.2 contains information on the use of corrugated metal pipe voids (CMP) for connections in precast integral abutment elements. These connections have also been studied by researchers for use in precast pier caps [35]. The CMP pipe is cast into the pier cap directly over the columns. The vertical reinforcing in the column is projected from the column into the void. Supplemental reinforcing is added to the cap within and around the void. Once set in place, the void is filled with concrete to complete the connection. The resulting connection emulates a conventional CIP concrete connection.

Concrete-filled CMP pipe void connections have been tested for seismic resistance [35]. The performance of the connection, when combined with adequate supplemental reinforcement, provides excellent performance as a ductile column connection. This will be discussed in more detail in Section 5.2.4.

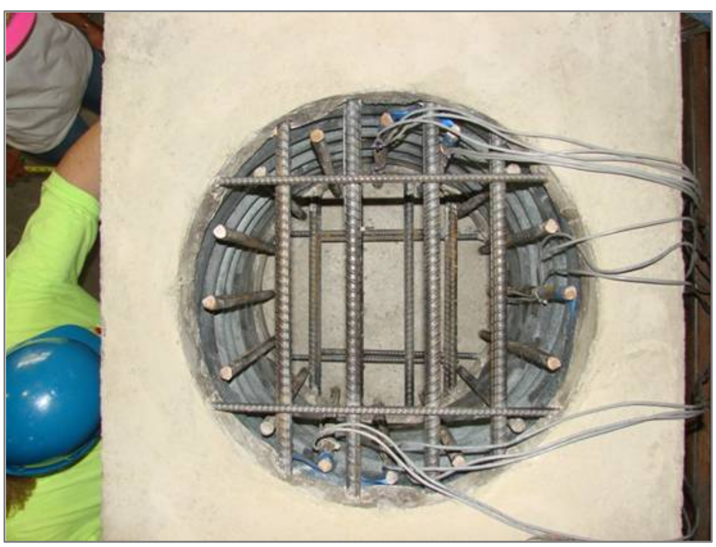


Figure 5.2.3.3-1. CMP Pipe Void Connection
(Source: NCHRP Project 12-74).

The design of this connection is well documented in the research report [35]. The report includes design examples and specification recommendations. The designer should be aware that the CMP void may require changes in the detailing of the cap beam element. Spacing of the longitudinal reinforcing may need to be adjusted to avoid conflicts with the column reinforcing. This reinforcement can be passed through the void through small holes in the pipe. Figure 5.2.3.3-2 shows the assembly of this connection. The photo shows the bottom longitudinal cap reinforcement and its interaction with the vertical column bars. The top of the void is typically formed with a removable form so that the metal pipe is not exposed to the elements in the completed connection.

One significant advantage of this connection is that the metal pipe provides the confinement reinforcement for the connections. Additional hoop reinforcement is typically not required provided that the pipe has adequate sectional strength. This allows for the simple projection of only the column vertical reinforcement into the void. Figure 5.2.3.3-1 shows the layout of the reinforcement within the void.

Figure 5.2.3.3-2. CMP Pipe Void Connection Installation
(Source: NCHRP Project 12-74).

5.2.3.4. Post-Tensioning Systems

Post-tensioning can be used to connect precast elements. Segmental bridge design specifications have been incorporated into Section 5 of the AASHTO LRFD Bridge Design Specifications.

Books are also available that document the design process for segmental construction. For this reason, the subject of design of segmental elements is not covered in this manual.

Many of the same issues regarding detailing and spacing of internal reinforcement that are discussed in Section 5.2.3 apply to post-tensioned segmental elements. The designer needs to account for the detailing of the ducts, anchorages, and the internal reinforcement during the design process.

5.2.3.5. Corrugated Column Ends

There is ongoing research into the use of corrugated column ends for the connection of a precast column element to a CIP footing. The research is sponsored by the FHWA Highways for LIFE program, and is entitled "Precast Bent System for Use in High Seismic Regions." The research studied this footing connection in addition to further studies of the use of grouted post-tensioning ducts for column-to-column and column-to-cap beam connections.

The approach for this connection is to resist the vertical forces in the column using interface shear, while resisting the ductile column forces through a socket type connection. The column is simply set on the ground within the footing forms and reinforcing. The connection is completed by casting the footing concrete around the column end. CIP concrete is not required under the column end with this connection. The connection is also ideal for foundations with large numbers of piles that make precast pile caps unfeasible. The use of CIP concrete may not seem to meet the goal of accelerated construction; however, the simplicity of this connection makes the construction significantly faster than conventional construction.

Figure 5.2.3.5-1 is a sketch of the proposed detail. The portion of the column end that is embedded in the footing is formed with corrugations. It is important to note that there is no reinforcing projecting from the column into the footing, which greatly simplifies the construction since all layers of the footing reinforcing can be placed prior to erection of the column. Once set, the column is braced and the footing is poured. Testing has demonstrated that this connection can achieve the full moment capacity of the column with emulative ductility and little or no damage to the footing. Testing also has demonstrated that the corrugations can support the vertical loads on the columns. The research is essentially complete. It is anticipated that the full research report will be available in 2013. One demonstration project also has been completed in Washington State.

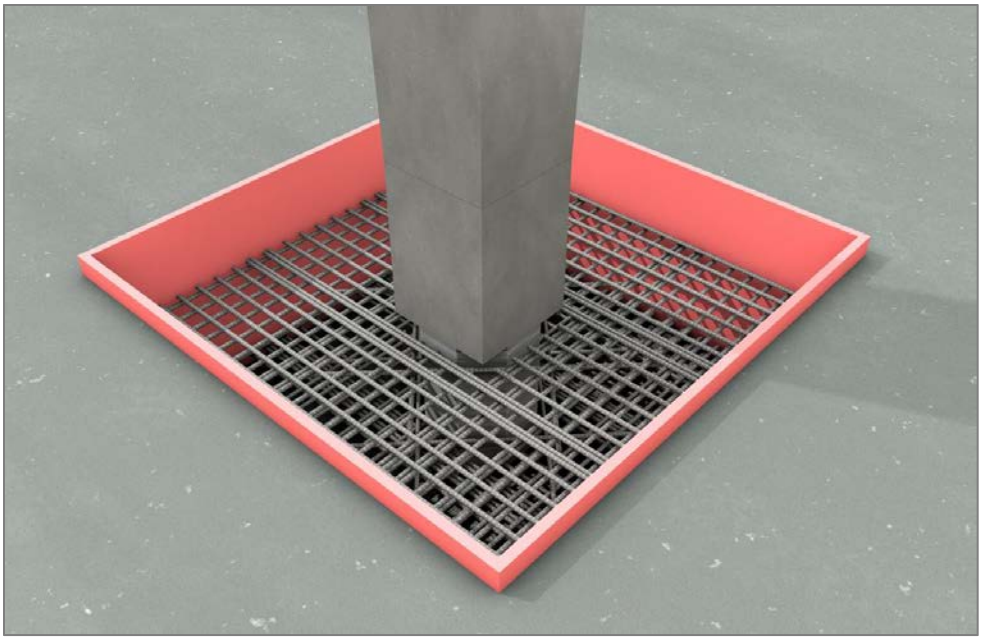


Figure 5.2.3.5-1. Corrugated Column End Connection
(Source: FHWA Highways for LIFE).

An important aspect of this connection is the development of the vertical column reinforcement in the footing. Large diameter bars will require significant development lengths. This connection does not allow for hooking of the vertical column bars into the footing; therefore. other means of anchoring the bars is required. The proposed method is to use reinforcing bars that are fabricated with integral heads. Headed reinforcing bars can be manufactured using several methods and are covered under ASTM Specification ASTM A970 Standard Specification for Headed Steel Bars for Concrete Reinforcement [45]. The development length of these bars can be calculated using the provisions of the ACI Building Code [46]. It may be feasible to hook the longitudinal bars within the column core for columns with small diameter bars.

5.2.4. Seismic Design

Connections play a key role in seismic design and detailing since they often experience the highest seismic force demand in the bridge. There are two current design specifications in use for seismic design of bridges. The first is the AASHTO LRFD Bridge Design Specifications [5]. This specification is a force-based design procedure, where seismic forces are primarily resisted by the development of plastic hinges in pier columns. This was accounted for in the design through the use of “Response Modification Factors” or “R Factors.” The hinges dissipate energy during the seismic event.

The second seismic design specification that is in use today is the AASHTO Guide Specifications for LRFD Seismic Bridge Design [44]. This guide specification is based on the displacement method of analysis. Using this approach, the designer investigates different earthquake resisting elements and systems (ERS). The systems are used to accommodate the calculated seismic displacement of the structure during the earthquake. Plastic hinging is still a major ERS approach to the design of bridges using the guide specification. The guide specification also includes provisions for “push over analysis,” where the resistance of the bridge “system” is calculated as multiple plastic hinges form in resisting elements.

Regardless of which design specification is used, the need to develop plastic hinges in elements such as columns is an integral part of the seismic design of the bridge. The guide specification design places a greater emphasis on the ductility of the hinge connections; therefore, it is important to study the ductility of the various connections that are in use today.

There is ongoing research into the design of post-tensioned column connections for seismic loading. The goal is to achieve a system that can not only resist seismic forces, but can right the structure after the seismic event. This is done through the use of un-bonded tendons in the column sections. The challenge is to get a system that can resist forces and dissipate energy. Studies have included the use of un-bonded post-tensioning systems combined with minor amounts of reinforcement. This approach may be used in the future after the approach is successfully demonstrated through research. Meanwhile, most designers are still pursuing more traditional designs for prefabricated bridges.

The majority of past research on the seismic performance of bridge structures was based on CIP concrete construction. Fortunately, research into the use of precast concrete elements in high seismic regions has been progressing rapidly as accelerated bridge construction has become more common. Most research is based on emulative behavior, which is behavior that is similar to CIP construction. The following sections contain information on several common details that show the most promise for emulative behavior.

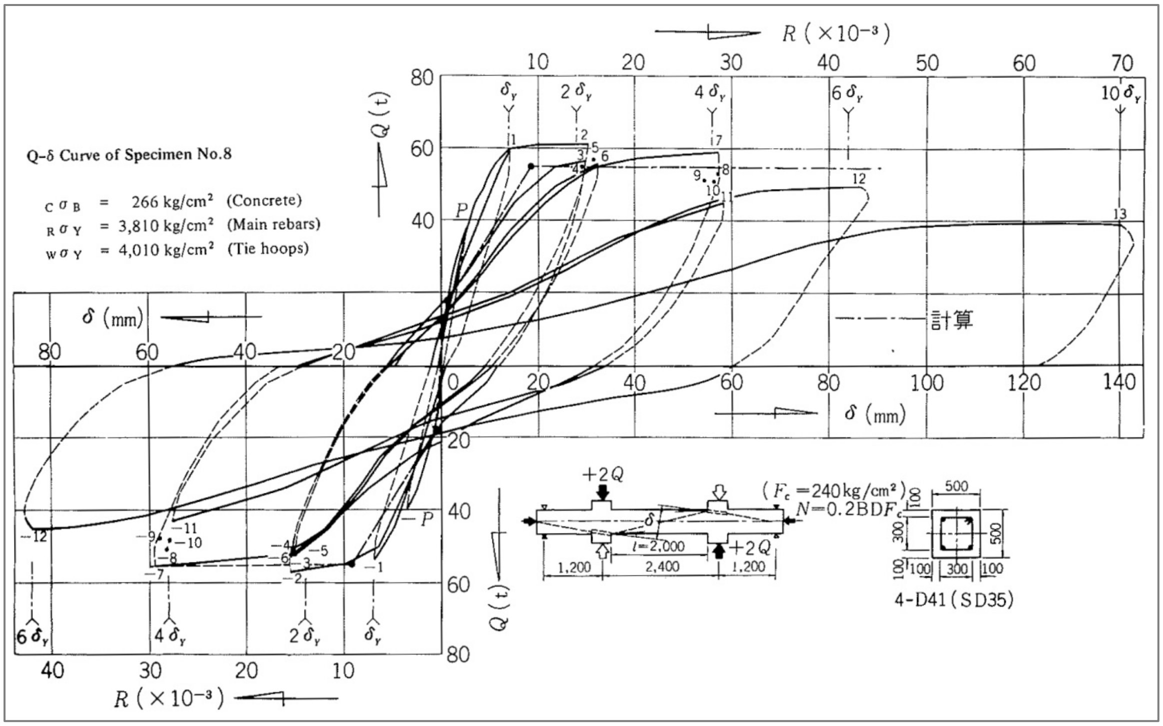
Grouted Reinforcing Splice Couplers: The use of grouted reinforcing splice couplers was covered in Section 5.2.3.1. These versatile couplers can be used in a number of connections throughout a bridge structure, including pier column connections. Grouted reinforcing splice couplers were developed in the U.S. over 30 years ago. The technology migrated to Japan, where it was further developed and used in buildings and bridges in high seismic zones. One of the manufacturers studied the performance of these connectors in seismic connections [84].

Figure 5.2.4-1 shows test results from two tests that were completed as part of this research. The specimens were column elements subjected to combined axial load and bending moments in excess of the column plastic moment. The axial load was set at 20 percent of the ultimate compressive force. The plots show the lateral shearing force (Q) versus the lateral drift ratio (R). The upper plot is the control specimen that was constructed with CIP concrete and no couplers. The lower plot is a specimen with grouted couplers placed within the hinge zone. The specimens were subjected to cyclic bending beyond the yield moment. After close examination of these tests, it is apparent that the connection behaves in a true “emulative” mode, with performance matching that of CIP concrete.

The current AASHTO LRFD Bridge Design Specifications [5] only require that mechanical couplers develop 125 percent of the specified minimum yield strength of the connected bar (Article 5.11.5). The ACI Building Code [46] has this same requirement for normal mechanical splice hardware (Type 1); however, it also includes provisions for a higher level of mechanical splice hardware. The higher level connector, or Type 2 connector, is required to develop

100percent of the specified minimum ultimate strength of the connector. It is recommended that this higher level of mechanical connector be used in bridge structures due to the relatively small number of columns in a typical bridge pier and the improved seismic performance of these connectors. See Section 5.2.4.1 for more information on these connectors.

Current ongoing research in the U.S. is studying the performance of these connectors. These tests are investigating both the strength and ductility of this connection. The results of this research are not yet available, but early test results appear to be favorable.



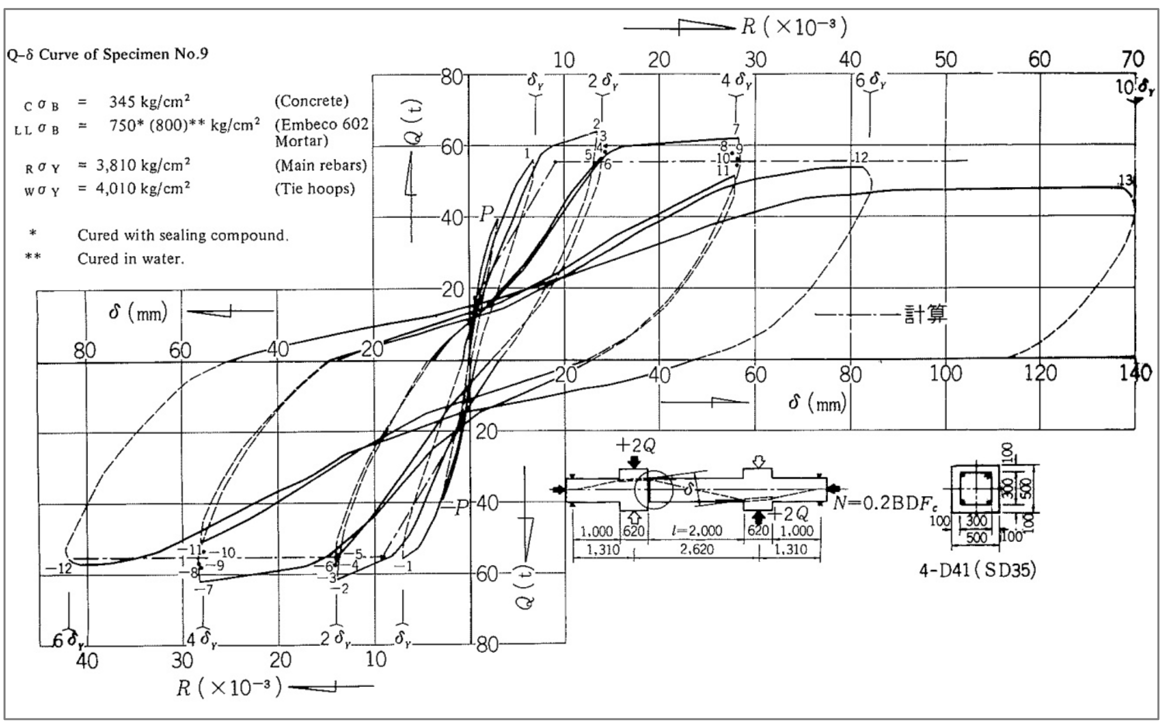


Figure 5.2.4-1. Cyclic Column Ductility Tests.
Top: Test on Monolithic Concrete Column.
Bottom: Test on Precast Column with Grouted Reinforcing Splice Couplers.
(Source: Splice Sleeve North America [84]).

One of the concerns with this connection with regard to the AASHTO Guide Specification is the flexibility or rotational stiffness of this connection. The couplers are significantly more rigid than the connected bars. There is a benefit to this fact, but there is also a downside. The benefit is that the couplers greatly reduce the possibility of main steel buckling as the column experiences loss of concrete cover. The down side is that the rotational stiffness of the hinge zone is increased. Designers should account for this stiffness when calculating the lateral drift of the structure. A conservative approach would be to assume that the hinge starts at the top of the coupler.

Another approach to this issue is to place the couplers away from the hinge zone. The couplers are typically installed in the pier cap; therefore, rotational stiffness of the hinge zone is not an issue. The preferred detail for couplers at the column-to-footing connection is to place the couplers in the column. While this is the preferred method of installation, it is not unreasonable to invert this connection, placing the couplers in the footing. Figure 5.2.4-2 shows the connection of a precast column to a footing using both methods. Designers using the inverted connection should specify that the couplers be capped prior to installation in order to prevent debris from getting into the couplers. Several manufacturers provide caps specifically for this purpose. Figure 5.2.4-3 shows the construction of an inverted footing connection.

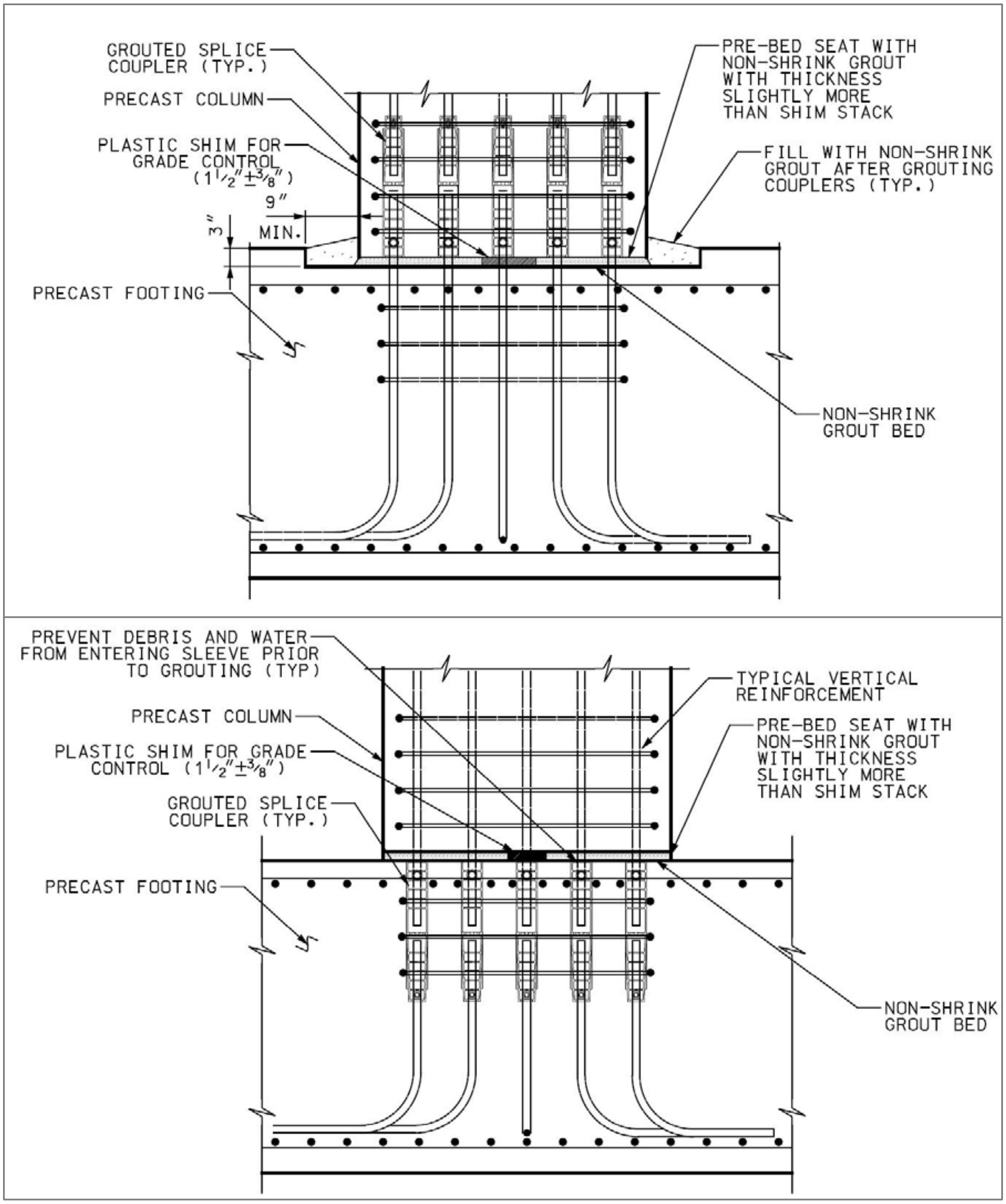


Figure 5.2.4-2. Column to Footing Connections using Grouted Reinforcing Splice Couplers.
Top: Standard Coupler Detail (shown with recessed pocket for added shear capacity).
Bottom: Inverted Connection with Couplers in Footing.
(Source: PCI Northeast).

Figure 5.2.4-3. Inverted Grouted Reinforcing Splice Coupler Connection (Source: Hanson Structural Precast, Salt Lake City).

Grouted Post Tensioning Ducts: The use of grouted post tensioning ducts was covered in Section 5.2.3.2. These connections can be used in column to cap connections. It may also be possible to use them in column-to-footing connections.

This connection was studied under NCHRP Research Project Number 12-74 [35]. Figure 5.2.4-4 shows results from one of the tests. The plot is similar to the plot shown in Figure 5.2.4-1 (lateral force versus displacement). The specimens were column elements subjected to combined axial load and bending moments in excess of the column plastic moment. An examination of this plot also indicated that the connection behaves in a true "emulative" mode, with performance matching that of CIP concrete.

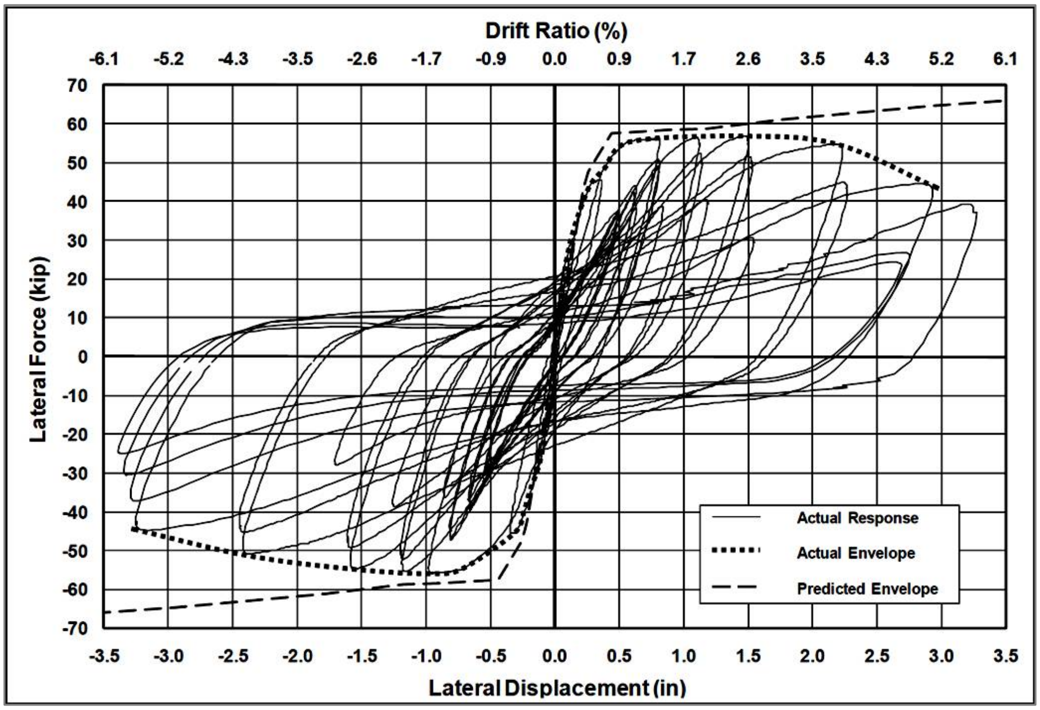


Figure 5.2.4-4. Cyclic Column Ductility Test. Grouted Post Tensioning Duct Column to Cap Connection (Source: NCHRP Project 12-74 [35]).

Concrete Filled Corrugated Metal Pipe Voids: The use of concrete filled corrugated metal pipe voids was covered in Section 5.2.3.3. These connections can also be used in column-to-cap connections. It may also be possible to use them in column-to-footing connections.

This connection was studied under NCHRP Research Project Number 12-74 [35]. Figure 5.2.4-5 shows results from one of the tests. The plot is similar to the previous plot shown in Figure 5.2.4-4 (lateral force versus displacement). The specimens were column elements subjected to combined axial load and bending moments in excess of the column plastic moment. At first glance, this plot looks to have less ductility, when compared to the others; however, it should be noted that the test was limited due to buckling of the vertical reinforcing in the column at the joint location. This test specimen had a thick joint between the two elements (approximately 2.5 inches), which left approximately 5.5 inches of bar unconfined. This can be corrected by either reducing the thickness of the joint, or by placing a hoop stirrup within the grout layer. With these modifications, the connection will behave in a true "emulative" mode, with performance matching that of CIP concrete.

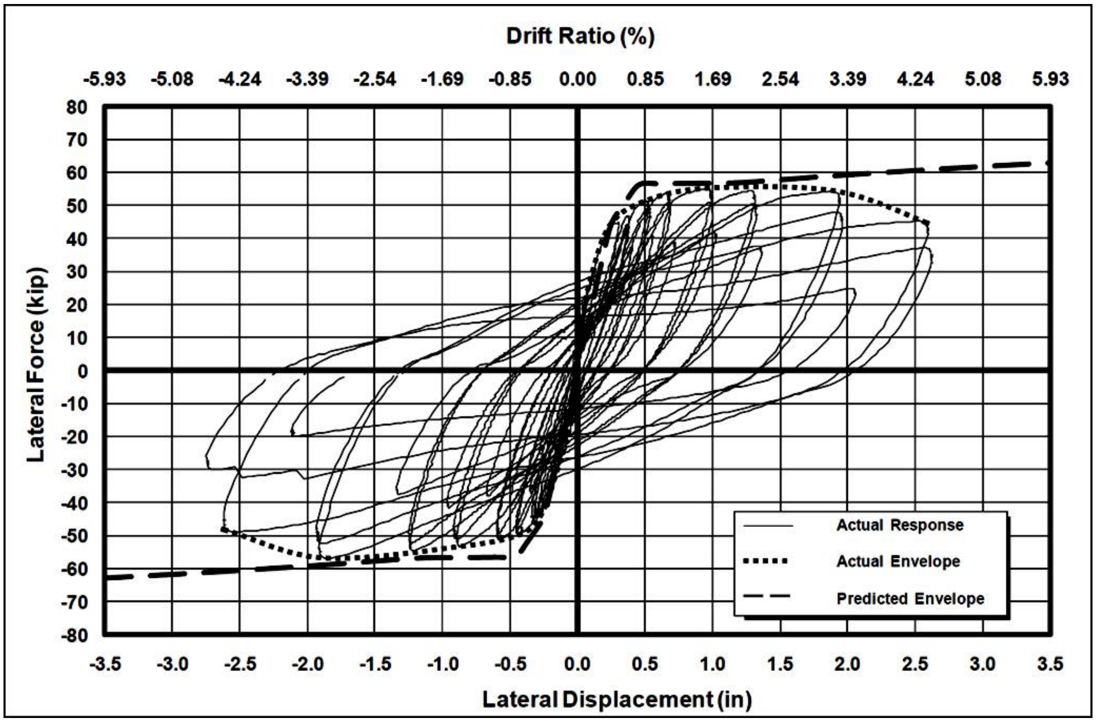


Figure 5.2.4-5. Cyclic Column Ductility Test. Concrete Filled Corrugated Metal Pipe Column to Cap Connection (Source: NCHRP Project 12-74 [35]).

Corrugated Column End Connection: The use of corrugated column ends was covered in Section 5.2.3.5. This connection is ideal for footings with large amounts of piles or drilled shafts. The research that was completed as part of the FHWA Highways for LIFE program demonstrated that this connection can achieve a fully ductile connection that produces little or no damage to the footing with performance that emulates CIP concrete. The research investigated various footing thicknesses. Early results indicate that the minimum footing thickness be kept to the diameter of the supported column. The results of the research, including design examples, should be available through FHWA in 2013.

Effect of Joint Width on Ductility Performance: Typically, the width or thickness of the joints between precast elements is a function of the erection tolerances. Thicker joints allow for more vertical adjustability in the field. This is not an issue for low-seismic regions; however, seismic research has shown that large joints can have an adverse impact on the performance of a ductile connection [35]. A wide joint can result in a relatively large gap between the confinement reinforcement in the column element and the adjacent cap or footing element. The wide gap can lead to premature column bar buckling. If a thick joint is preferred, an additional tie or hoop should be added within the joint to prevent this failure. Figure 5.2.4-6 shows the detail of this hoop.

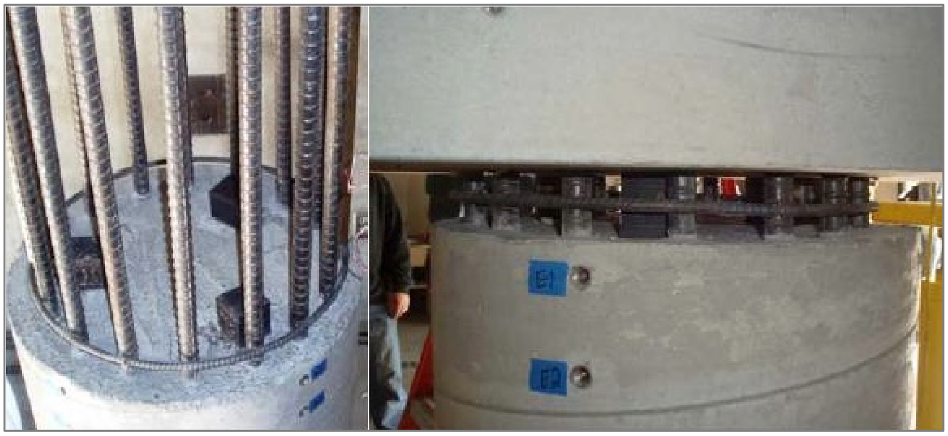


Figure 5.2.4-6. Seismic Hoop Reinforcement in Wide Joint
(Source: NCHRP Project 12-74 [35]).

Armed with these various technologies, designers should be able to design prefabricated pier elements in high seismic zones. The performance of these connections mimics or exceeds the behavior of conventional CIP concrete structures. Designers should refer to the various research project reports for more information on how to design, detail, and specify these connections.

5.2.4.1. AASHTO Limitations on Mechanical Couplers

The current AASHTO LRFD Code [5] does not allow the use of mechanical connectors in the plastic hinge zone of columns. Article 5.10.11.4.1f, which covers detailing of seismic reinforcement in columns, states:

Full-welded or full-mechanical connection splices conforming to Article 5.11.5 may be used, provided that not more than alternate bars in each layer of longitudinal reinforcement are spliced at a section, and the distance between splices of adjacent bars is greater than 24.0 inches. measured along the longitudinal axis of the column.

Article 5.11.5 specifies that mechanical connection splices must develop 125percent of the specified yield strength of the connected bar. A grade 60 reinforcing bar has a yield strength of 60 ksi and an ultimate strength of 90 ksi. The value of 125percent of the specified yield strength only represents a stress of only 75 ksi, which is why the limitation on splicing is included in the AASHTO LRFD Bridge Design Specifications.

In most cases, it is not feasible to accommodate this AASHTO provision in precast column elements. There are several approaches that can be used to address this in the design of columns for seismic zones. The following sections contain advice on how to address this issue:

Place Mechanical Connection in Connected Elements: The AASHTO limitation on mechanical connectors is for the detailing of column reinforcement. It is possible to place the connectors in the elements that are connected to the column (footing and cap) instead of placing them in the actual column. While this technically avoids the specification requirements for column detailing, questions still may arise with respect to the ability of the connection to resist full ductile bending moments. Designers should consider following the requirements of the ACI Building Code [45] for this scenario (see text below).

By placing the mechanical connection in the connected elements (away from the hinge zone), the question of reduced rotational flexibility is avoided. The plastic hinge zone will only have conventional reinforcing; therefore, the rotation of the hinge under plastic loads will be the same as with CIP concrete.

Use Research Data to Justify Details: The AASHTO LRFD Bridge Design Specifications are intended to cover the majority of bridge designs in the country. The basis of the specification is years of research that support the provisions in the specification. The current specification does not have many provisions that cover the use of prefabricated elements and their connections. The provisions for column detailing in seismic regions do not address many of the connections discussed in this chapter. The provisions in the AASHTO specifications can be supplemented by engineering judgment, especially when supported by sound research. The following is an excerpt from Article 1.1 of the AASHTO LRFD Bridge Design Specifications:

These Specifications are not intended to supplant proper training or the exercise of judgment by the Designer, and state only the minimum requirements necessary to provide for public safety. The Owner or the Designer may require the sophistication of design or the quality of materials and construction to be higher than the minimum requirements.

The results of research do not automatically find their way into the design specifications. There is a formal process wherein the research results could potentially be integrated into the AASHTO LRFD Specifications. In the interim, owner agencies have the ability to allow the design of certain bridge features based on currently available research results that have not yet been incorporated into the specifications. This document contains numerous references to recent research regarding seismic detailing of bridges built with prefabricated elements. Much of this research contains recommendations for design and construction specifications. Agencies can choose to allow certain details to be used and designed according to the recommendations of the available research.

Use ACI Building Code Provisions: As with the previous discussion, the owner agency can choose to use engineering judgment by allowing the use of other code provisions for the design of bridge elements.

The ACI 318 Building Code Requirements for Structural Concrete [45] do allow the use of mechanical connectors in the plastic hinge zone of precast moment frames, provided that they can meet certain requirements. There are two levels of mechanical splice devices specified in the ACI 318 Code. Type 1 mechanical splices need to be capable of developing 125percent of the specified minimum yield strength of the bar. Type 2 mechanical splices need to be capable of developing 100percent of the specified minimum tensile strength (ultimate strength) of the bar (equal to 150percent of specified yield for standard grade 60 bars).

The ACI 318 code has restrictions that are similar to the AASHTO Specifications for Type 1 mechanical splices; however, the use of Type 2 mechanical splices are unrestricted for special moment frames constructed with precast concrete (including the plastic hinge zone). The recent seismic bridge research on precast connections supports this provision; therefore, use of this nationally recognized design specification would seem appropriate. Designers should verify with owner agencies that this approach is acceptable for the interim until AASHTO LRFD design specifications are written for precast concrete connections in high seismic zones.

5.2.4.2. Use of Seismic Isolation Design

Seismic isolation design has been in use in the U.S. for many years; however, the volume of bridges designed with isolation has been relatively low. The approach to seismic isolation is to "isolate" the structure from the ground motions. This is done by uncoupling the superstructure from the substructure. This is normally achieved through the use of bridge bearings that are placed between the superstructure and the substructure. In theory, isolation devices can be placed anywhere in the structure. Isolation normally involves two factors. First, the isolation system is used to lengthen the natural frequency of the structure by "softening" the stiffness.

Figure 5.2.4.2-1 is the idealized Design Response Spectrum that is specified in the AASHTO LRFD Bridge Design Specifications [5]. The horizontal axis is the period of the structure. The vertical axis is the elastic seismic coefficient, which is essentially the acceleration that the bridge experiences during the seismic event. Most short- to medium-span bridges have fundamental periods between 0.5 and 1.0 seconds. By using seismic isolation design principles, the period of the structure can be increased significantly. By moving the period well beyond 1.0 seconds, the elastic seismic coefficient can be greatly reduced, thereby reducing the seismic forces in the structure.

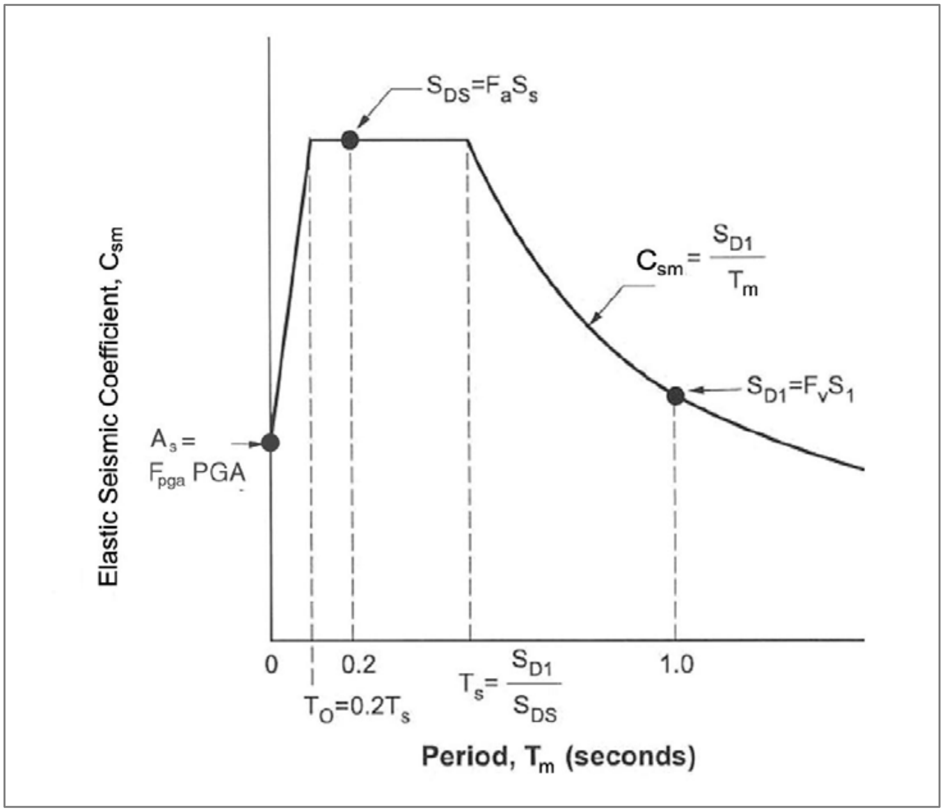


Figure 5.2.4.2-1. AASHTO Seismic Design Response Spectrum [5].

The second factor in seismic isolation design is to increase the damping in the structure. This is done to absorb energy and to reduce displacements during the seismic event.

Seismic isolation design can be a very effective tool in ABC construction. The following are the potential benefits and their effect on ABC:

- Isolation minimizes seismic loads, which reduces the size of substructures and foundations (including piles and drilled shafts). Reduction in the number of piles or drilled shafts will save time during construction. Reduction in the size of substructure elements will facilitate installation of prefabricated elements.

- Isolation normally involves the use of bearings between the superstructure and the substructure. This is contrary to conventional seismic engineering, where the superstructure is often made integral with the superstructure. The use of bearings eliminates complex connections that are difficult to build with prefabricated elements. Bearings also provide significant source of tolerance adjustment.

There are no specific seismic isolation design issues as it relates to ABC. ABC bridges and normal bridges use the same design procedure. The design of bridges for seismic isolation should follow the requirements of the AASHTO Guide Specifications for Seismic Isolation Design [47]. This design specification was most recently updated in 2010.

5.2.4.3. Use of Shear Keys (Keeper Blocks)

Shear keys or keeper blocks are an effective way to restrain a bridge structure from excessive movement during seismic events. Shear keys are normally used in conjunction with bearing systems. The bearings allow the structure to expand and contract with temperature. The shear keys restrain the bridge from movement in the transverse direction.

Shear keys are typically concrete blocks that are cast into the substructure elements. They are installed with small gaps between the key and the superstructure. The force in the blocks is typically transferred to the substructure via interface shear design principles.

The gaps that are specified for shear key are typically small (1/4 inch to ½ inch), therefore it is difficult to build a prefabricated element with precast shear keys due to the tolerances of the elements and the erection methods. The potential of an earthquake striking during construction is remote; therefore, it is reasonable to install the shear keys after the bridge is in service, thereby eliminating this construction from the critical path. This helps to accelerate the construction of the bridge.

5.2.4.4. Use of Hinged Connections

Installing hinges in bridge structures can have a beneficial effect on the seismic performance of the bridge. Pins can relieve bending moments in substructure elements while still transferring seismic shear to the desired substructures.

The California Department of Transportation (CalTrans) sponsored research into the design and detailing of hinged connections in concrete structures [47]. The objectives of the research were to develop and test potential details, and to develop design procedures for the hinge pins. Previous designs were based on the shear capacity of the pin; however, as with anchor bolts, the surrounding concrete plays a significant role in the failure mechanism. The proposed details include a telescopic pipe-pin assembly that can rotate in two directions. The hinges consist of a steel pipe that is anchored in the column with a protruded segment that extends into the bent cap. Previous hinge designs consisted of mild reinforcing bars placed near the center of the elements. These details reduced the bending moment transfer, but still transferred significant moments into the substructure.

There is potential to use hinged connections in accelerated bridge construction applications. Although not developed for ABC construction, the connection is a promising alternative to grouted splice sleeves, grouted post-tensioning ducts, or corrugated pipe void connections. The connection is simple and easy to construct, even with prefabricated elements. The pins and sockets can be cast into the elements and joined together in the field.

CHAPTER 6. DESIGN OF MODULAR CULVERT/ARCH SYSTEMS

A growing number of bridges are being built using modular culvert/arch systems. In the past, arch elements had somewhat limited span capability. These limitations were set based on shipping limitations. New details and connections have been developed allowing for span lengths in excess of 100 ft.

Speed of construction is helpful for culverts, especially when complex water handling methods are required. Prefabricated culverts are an attractive option, since the contractor can complete the site work during periods of low flow. Box shaped culverts with bottoms are good for scour protection; however, soft soil conditions, environmental permitting concern over the unnatural bottom in manufactured culverts, and other factors lead to a desire to eliminate the culvert bottom slabs. Some agencies are moving toward increased use of three-sided culverts and arch to structures with natural bottoms. These systems allow for installation overflowing water, thereby simplifying the water handling needs during construction.

This chapter covers some of the design aspects of culvert/arch structures. Some of these systems are proprietary, which makes designing and bidding these systems difficult due to the sole source restrictions in Federal regulations. In some locations, multiple products can work for a given crossing. In these cases, the design engineer can simply specify the crossing opening limitations and specify allowable manufacturers of prefabricated systems. The manufacturers would then design the elements using proprietary software and submit calculations to the agency for review and approval. The following sections contain more information on the use of culvert and arch systems.

6.1. PRECAST CONCRETE BOX CULVERTS

One of the most common prefabricated culvert elements in use today is precast concrete box culverts. Precast box culverts can be built significantly faster than CIP concrete culverts. Some agencies have expressed concerns over the joints in the culverts. Other agencies have had excellent performance provided that proper detailing and construction practices are followed. The maximum span of a box culvert is about 16 ft and maximum height is about 12 ft, both of which are primarily limited by shipping and handling of the box sections.

Design provisions for buried structures are included in Section 12 of the AASHTO LRFD Bridge Design Specifications [5] with supplementary special considerations under Section 5.14.5. Section 12.11 specifically covers the design requirements for precast concrete box culverts. This section covers:

- Loads and live load distribution.
- Service limit state design.
- Safety against structural failure.
- Construction and installation.

The analysis of a box culvert is similar to that of any substructure element. Soil forces need to be calculated using conventional geotechnical engineering principles. Live loads are calculated based on code provisions. The provisions vary based on the amount of soil cover on the culvert. For instance, culverts with less than 2 ft of cover have different distribution of live loads than culverts with more cover. Using these forces, the designer can model the cross-section of the culvert in a frame analysis program. This will generate the forces acting on the side walls, roof, and floor of the culvert. Then the sections can be designed using conventional reinforced concrete design principles. Considerations for flow conditions and the anticipated water levels throughout the service life should also be addressed during design.

The basis of design for most reinforced concrete box culvert is a sectional analysis. This means that the structural system is simplified to a 2D section with unit loads. As with a concrete deck design, the main reinforcement is run perpendicular to the centerline of the culvert. Distribution reinforcement is placed parallel to the culvert centerline. Based on this, it is acceptable to have unreinforced joints in the culvert and still be consistent with the design assumptions. Therefore, the design of precast box culvert sections can follow the same procedure as with a CIP concrete box culvert.

Aprons and wing walls may be CIP concrete or made with precast elements. Utah DOT has developed typical detail sheets for precast aprons and walls for box culverts (www.udot.utah.gov; search ABC). Figure 6.1-1 contains one of the details for these structures. More information on precast concrete footings and walls can be found in chapter 5 of this manual.

The process described above is straightforward; however, it can be time-consuming. There are several software programs that have been developed to facilitate the design of box culverts. Some are based on the procedures described above, while others use more sophisticated finite element methods to more accurately predict the soil/structure interaction. Some state agencies have developed their own programs that are available to designers. Other agencies maintain standards for CIP box culverts, which can be used as a reasonable starting point for a precast culvert design.

There are several AASHTO specifications that have been developed to aid in the design and construction of precast concrete box culverts. AASHTO Specification M 259 [60] covers standard dimensions of single-cell precast reinforced concrete box sections intended to be used for the construction of culverts. AASHTO M 273 [61] is used for preliminary design of box sections with less than 2 ft of cover subjected to LRFD highway loading.

These specifications contain standard designs and the criteria that are used to develop the designs. If these designs are used, the agency should carefully investigate the soil used for the construction and ensure that the manufacturing of the sections conforms to the specifications. Since it is likely that soil properties will vary from region to region, these specifications may only be applicable for preliminary design. Items such as concrete cover and concrete strength should also be checked to verify that the details are consistent with agency policies. Site-specific designs may be required if these variables differ from the AASHTO specifications.

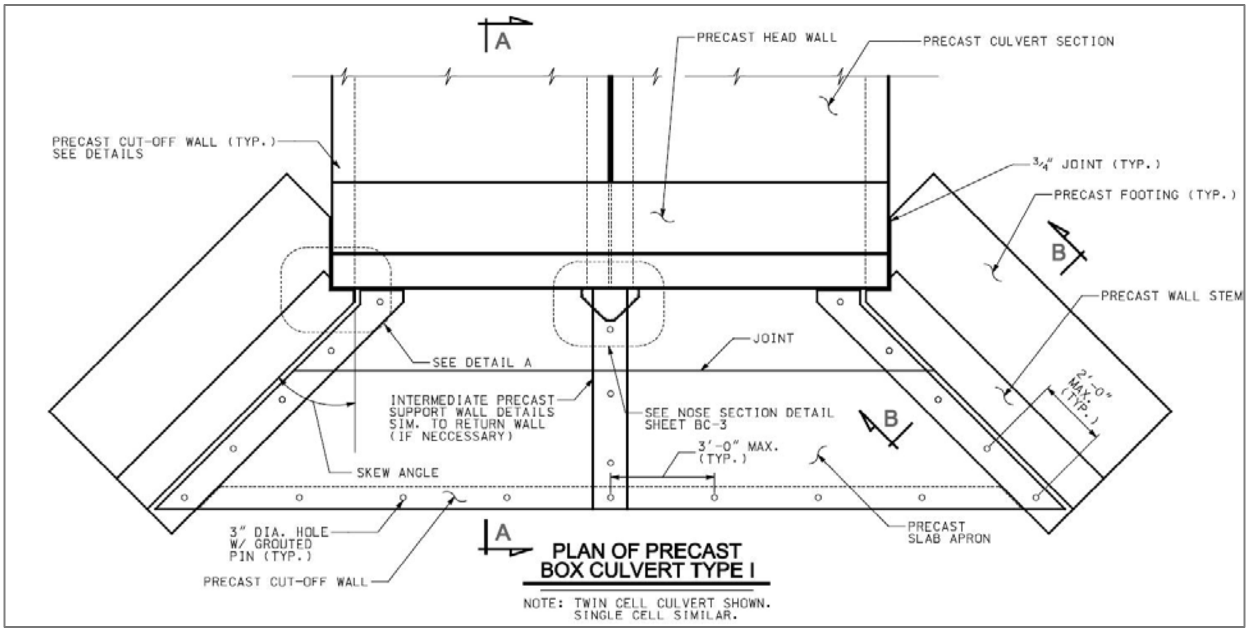


Figure 6.1-1. Plan - Precast Wingwalls and Aprons for Precast Concrete Box Culverts (Source: Utah DOT).

Some state agencies have developed standards for different types of box culverts. Designers from different states should carefully investigate the assumptions used for the development of other state standards including special loading, backfill materials, concrete cover, etc.

6.2. PRECAST CONCRETE ARCHES AND FRAMES

There are several types of precast concrete arches and frames in use in the U.S. The design of precast concrete frame and arch structures is similar to the design of precast reinforced concrete box culverts. Design provisions for buried structures are included in Section 12 of the AASHTO LRFD Bridge Design Specifications [5]. Section 12.11 specifically covers the design requirements for reinforced concrete arch structures. Section 12.14 specifically covers the design requirements for precast concrete three-sided structures.

Soil forces can be calculated using conventional geotechnical engineering principles, and live loads can be calculated based on code provisions. Using these forces, the designer can model the cross-section of the culvert in a frame analysis program. This will generate the forces acting on the arch elements and foundations. The sections can then be designed using conventional reinforced concrete design principles. The design of frames and arches differ from culverts in that the foundations are called upon to resist thrust forces generated by the frame action of the elements. The soil pressure on the frame and arch elements can be used to counteract and reduce the thrust forces. The geometry of the structure will have a significant influence on the thrust forces. The use of prescribed soil backfill construction methods can be effective in controlling geotechnical forces on the culvert structure. Designers should investigate this during the preliminary design phase of a project in order to determine if the geometry for the structure can meet the geometry of the site.

The analysis of an arch or frame system can get complicated due to the geometry and the sloped surfaces of the structure and of the indeterminate nature of the rigid structure itself. Fortunately, manufacturers of these systems routinely provide engineering support for their products. They have developed custom software to design and detail the systems that includes soil-structure interaction that can determine the soil pressure and live load effects on the curved/sloped surfaces. Designers can use these services to size a structure and verify the hydraulic capacity. Design plans can then be drawn without specific reinforcement details. The project specifications can be written to require that the manufacturer produce detailed design calculations for submission and review with the shop drawings.

A frame is typically a three-sided structure supported on footings or wall stem. Figure 6.2-1 shows a sketch of a typical three-sided precast concrete frame structure.

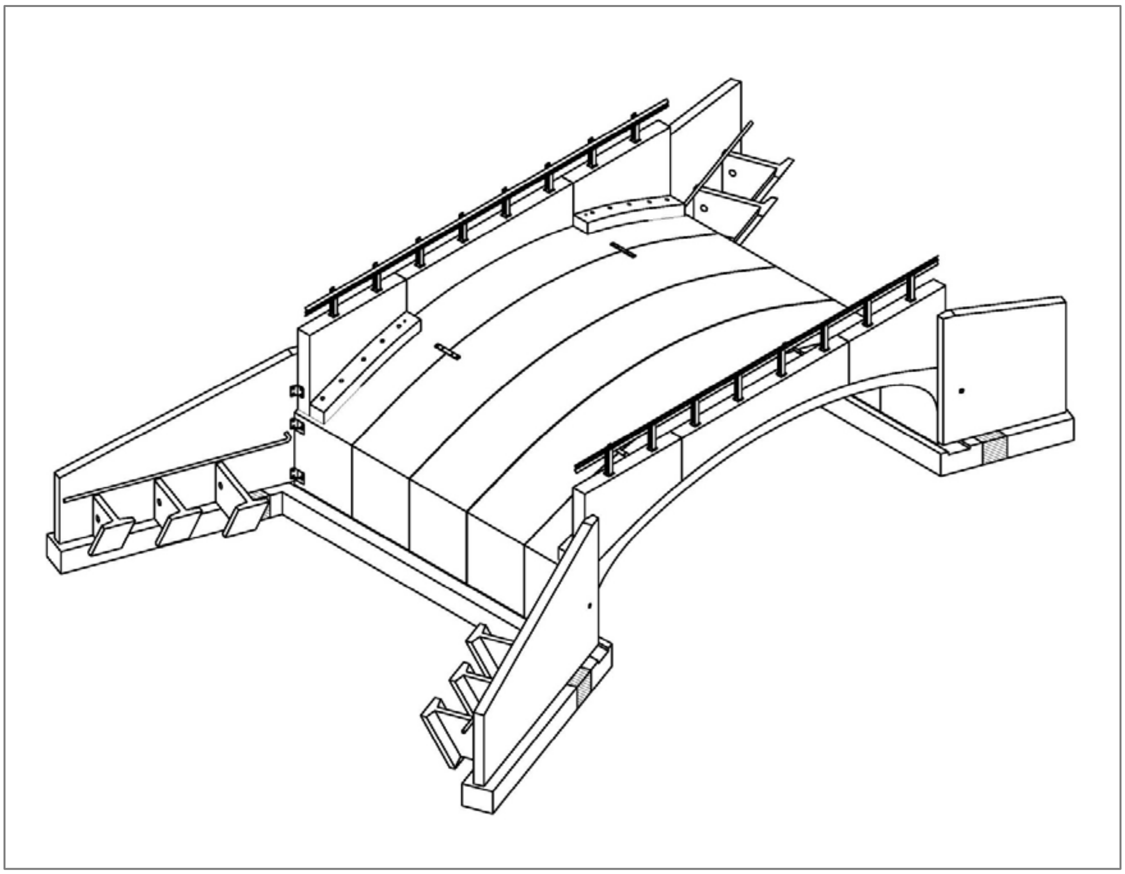


Figure 6.2-1. Three-Sided Precast Concrete Frame Structure
(Source: Contech Engineered Solutions).

The span range of precast concrete three-sided frames is approximately 12 to 65 ft. The design is based on rigid frame action with rigid connections at the top corners. The connection to the footing is typically considered a pinned connection. Resistance to thrust is accommodated by setting the frame legs in grouted recesses. It is important to note that the height of the vertical legs is an integral part of the design. The soil forces acting on the leg counteract the thrust forces that develop in the frame. For longer spans, the height of the leg needs to meet certain minimum criteria. To solve this problem, the footing can be lowered to lengthen the legs. It is also possible to add a CIP floor slab to tie the two footings together; however, this complicates construction and adds cost.

Precast arch structures are similar to frames except that the elements are curved (no corners). Figure 6.2-2 is a photo of precast concrete arch structure under construction. The shape of the arch can either be parabolic or circular. The span range of precast concrete arches is approximately 12 to 102 ft. Spans are typically designed as two hinge arches. The connection to the footing is typically considered a pinned connection. Resistance to thrust is accommodated by setting the arch legs in a grouted recess. There are limits to shipping arch structures due to the geometry of the elements. For longer spans, the arch elements are cast in two pieces. A simple thrust block is detailed to temporarily support the arch. The connection is completed with a simple closure pour that uses the thrust block as a form. This connection is shown in Figure 6.2-2. It should be noted that this type of construction requires the use of two cranes, or shoring of one half while the other half is erected and connected.

Figure 6.2-2. Precast Concrete Arch Structures (Source: Contech Engineered Solutions).

6.3. CORRUGATED METAL PIPE, ARCHES, AND FRAMES

Corrugated metal pipes are common in roadway drainage construction. Corrugations are used to improve the structural resistance of the structure walls and allow for the use of thinner metals. There are a wide variety of shapes and sizes for corrugated metal structures. Pipe sizes can range from 6 inches to 50 ft. Smaller diameter pipes are manufactured using a helical process that produces a complete pipe section. Larger pipes and non-round sections are manufactured with plate sections that are bolted together in the field.

Typical metal plate structures and approximate span ranges include:

- Steel or aluminum arch spans: 5 ft to 26 ft.
- Long spans teel arch spans: 18 ft to 65 ft.
- Steel or aluminum box culvert spans: 9 ft to 45 ft.

The arch spans are typically designed as two hinge arches. The connection to the footing is typically considered a pinned connection. Resistance to thrust is accommodated by setting the arch legs in a grouted recess. The box culvert sections typically have a bottom plate to resist compression and thrust forces. Metal culverts interact with the surrounding backfill soils; therefore, careful control and specification of the backfill soil materials is required to ensure a successful installation.

There are also material options for metal structures. The most common is galvanized or aluminum coated steel and aluminum. In some cases, agencies will specify additional coatings to extend the service life of the structures. There are several AASHTO Materials Specifications that cover corrugated metal pipe structures. Table 6.3-1 lists the common specifications that are in use.

Table 6.3-1. AASHTO Materials Specifications for Metal Pipe Structures.

Specification	**Description**
AASHTO M218 [62]	Standard Specification for Steel Sheet, Zinc-Coated (Galvanized), for Corrugated Steel Pipe
AASHTO M274 [63]	Standard Specification for Steel Sheet, Aluminum-Coated (Type 2), for Corrugated Steel Pipe
AASHTO M197 [64]	Standard Specification for Aluminum Alloy Sheet for Corrugated Aluminum Pipe
AASHTO M246 [65]	Standard Specification for Steel Sheet, Metallic-Coated and Polymer-Precoated, for Corrugated Steel Pipe

Design provisions for buried structures are included in Section 12 of the AASHTO LRFD Bridge Design Specifications [5]. Section 12.9 specifically covers the design requirements for metal box structures. Section 12.7 covers the design of buried corrugated and spiral rib metal pipe and structural plate pipe structures. Section 12 covers the design of buried long-span structural plate corrugated metal structures.

The analysis of a buried metal structure is similar to the design of any substructure element. Soil forces need to be calculated using conventional geotechnical engineering principles. Live loads are calculated based on code provisions. Using these forces, the designer can model the cross section of the culvert in a frame analysis program. The design of buried metal structures relies on the interaction between the structure and the soil. In order for an arch structure to fail, multiple hinges must form in the system. An inward buckling hinge can only form if a corresponding outward hinge forms simultaneously. If the outward hinge is resisted by soil, the potential for the inward hinge to develop is reduced.

Commercial software is available for soil-structure interaction analysis. In addition, manufacturers of these systems routinely provide engineering support for their products. They have developed custom software to design and detail the systems. Designers can use these services to size a structure and verify the hydraulic capacity. Design plans can then be drawn without specific plate thickness and bolting specifications. The project specifications can be written to require that the manufacturer produce detailed design calculations for submission and review with the shop drawings.

CHAPTER 7. DESIGN OF PREFABRICATED FOUNDATION ELEMENTS

Foundations are defined in the AASHTO LRFD Bridge Design Specifications [5] as footings (spread and pile caps), driven piles, drilled shafts, and micropiles. The construction of foundations can represent some of the most significant time challenges for ABC projects.

This chapter covers the design of foundation systems for ABC projects. Foundations on ABC projects can consist of only precast elements or a combination of precast elements and CIP concrete. Designers can take advantage of the reduced foundation loads during construction by specifying interim concrete strengths to allow for partial construction of the bridge, while the foundation concrete cures. These concepts as well as others are explored in this chapter.

7.1. SPREAD FOOTINGS

The use of spread footings has diminished over the history of bridge construction in the U.S. Many older bridges built on spread footings have been performing well for many decades. Recent research suggests that spread footings are underutilized. Although spread footings are not recommended when subject to deep scour or excessive settlement, there are many locations where shallow foundations are a viable cost-effective solution.

Design engineers should coordinate with the owner, the geotechnical engineer, and the hydraulic engineer to determine if shallow foundations are an acceptable option. Spread footings set below predicted riverbed scour or socketed into sound rock have been used in locations subject to scour. Design engineers should consider the use of shallow foundations in order to expedite the construction of the foundations and reduce the overall time for construction of the bridge.

7.1.1. Precast Concrete Spread Footings

Precast concrete spread footings have been used successfully on a number of ABC projects. There are several options for the connections of the spread footing to the substructure elements. The most common connection is grouted reinforcing splice couplers. This connection is covered section 5.2.3.1 of this manual.

Figure 7.1.1-1 shows a detail for a precast spread footing to precast wall or stem connection. The detail was developed by the PCI Northeast Bridge Technical Committee. The main features of this detail are as follows:

- The reinforcement in the footing is the same as a CIP footing. The design of the transverse reinforcement is the same as a CIP concrete footing.
- Leveling devices are used to set the grade of the footing. A mechanical device is shown; however, it is recommended that the contract be allowed to substitute other devices. Stacked shims are an acceptable device.
- Ports are shown for the placement of grout or flowable fill under the footing. The number of ports and the size and spacing of the ports are typically left up to the contractor to determine (means and methods).

- The detail shows grouted couplers placed in the substructure element. It is possible to place the couplers in the footing (inverted). See Figure 5.2.4-2 for details of this option.
- The detail shows the substructure element grouted into a key in the footing. In many cases, this is not necessary. Reasons for using this detail include lateral shear resistance and protection from ground water. If a key is used, the designer will need to lower the top reinforcing bars in the footing. It is recommended that the width of the key be set by the contractor based on the grouting methods for both the key grout and the ports for the grouted splice couplers. See Figure 5.2.4-2 for more information on this option.

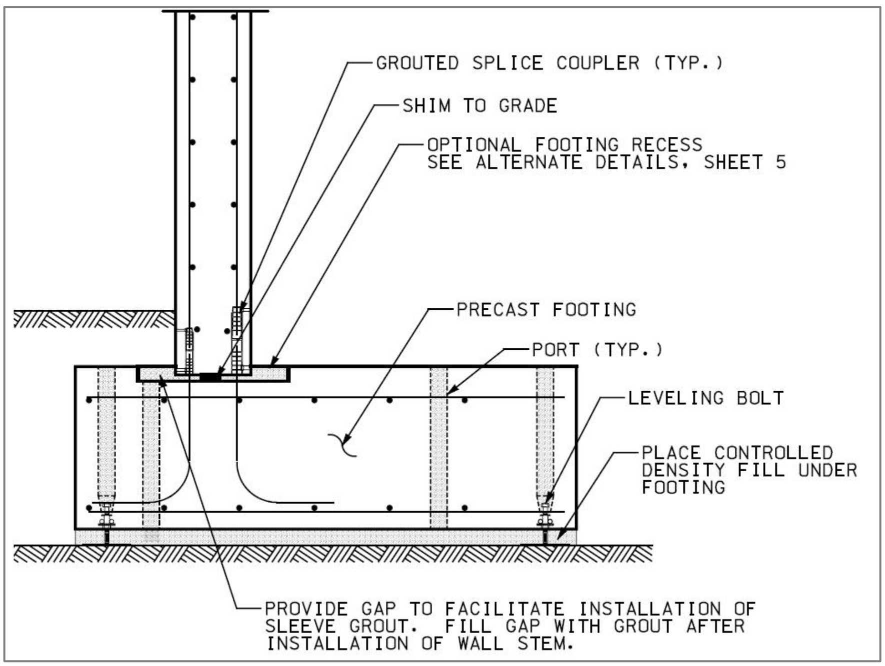


Figure 7.1.1-1. Precast Concrete Spread Footing
(Source: PCI Northeast).

The detail in Figure 7.1.1-1 shows the use of flowable fill to seat the spread footing on the substrate. This is done to provide uniform bearing. If precast spread footings are placed directly on soil, the lateral sliding resistance may be significantly reduced. The use of flowable fills for this void should alleviate this issue, since flowable fill has a significant amount of bond to concrete.

Connections between adjacent precast footing elements can be detailed in a number of ways.Reinforced concrete closure pours can be used (see Section 7.1.3). The elements can also be set adjacent to each other with no physical connection. When using this concept, design the supported elements to distribute the load to the adjacent foundation elements. Grouted keyways are also effective. The use of adjacent elements with no connection or adjacent elements with grouted keyways is based on design of continuous spread footings using sectional design principles. The reinforcement that runs longitudinally in the footing is either temperature and shrinkage reinforcement or distribution reinforcement. It is feasible to discontinue this reinforcement without compromising the integrity of the footing. This principle is only applicable to wall footings. Pier footings are designed with two-way slab action. Section 7.1.3 has more information on this subject.

Figure 7.1.1-2 shows a detail for a precast concrete spread footing with a grouted keyway. The detail was developed by the PCI Northeast Bridge Technical Committee. The main features of this detail are as follows:

- The width of the joint is set to allow for the use of normal concrete as a fill material. This is more cost-effective than non-shrink grout.
- The width of the key is a function of element casting tolerances and elements setting tolerances. See Chapter 10 of this manual for more information tolerances.

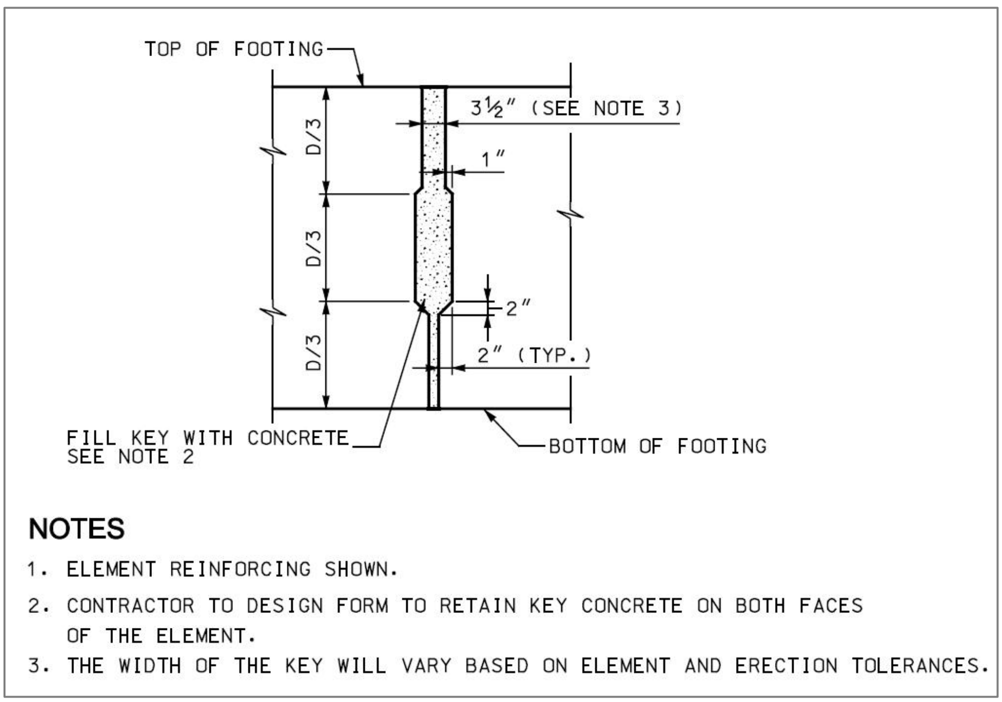


Figure 7.1.1-2. Precast Spread Footing Key (Source: PCI Northeast).

The elimination of reinforcement across joints in wall footings may lead to concerns over shifting and differential settlement in the precast foundation elements. Offsetting the joints in the wall elements above the footing elements and/or providing continuous cap elements mitigates this concern. Figure 7.1.1-3 is a detail that is based on those included in the PCI Northeast Bridge Technical Committee typical guide details. The detail is for a wall pier; however, it would be applicable to a cantilever abutment and a cantilever retaining wall. The Committee chose to recommend a minimum offset of 3 ft between the footing joints and the wall joints. There is no scientific or empirical basis for the value, and the minimum offset can be adjusted based on the geometric constraints of the project under consideration.

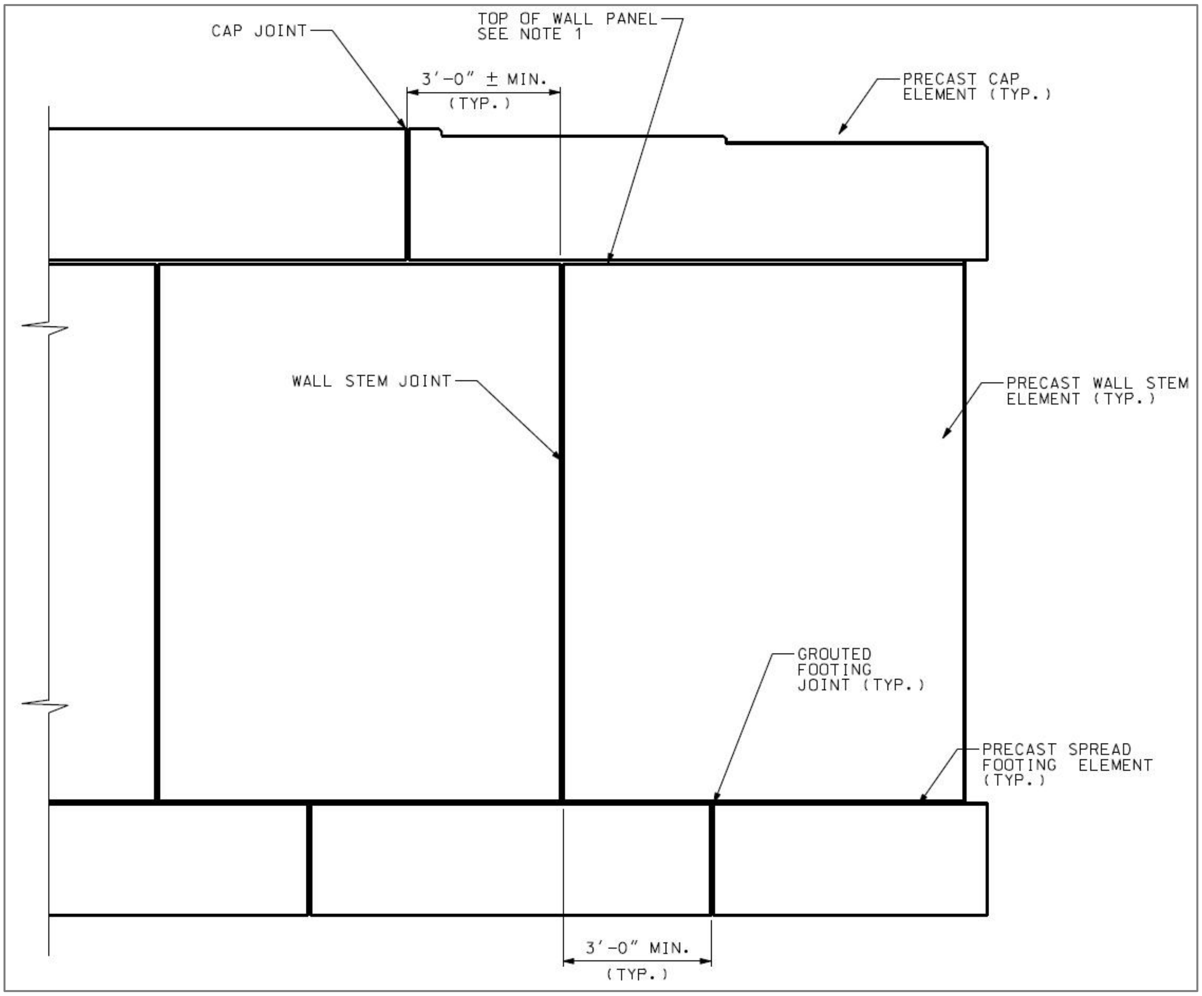


Figure 7.1.1-3. Precast Wall Footing Layout (Source: PCI Northeast).

7.1.2. Cast-In-Place Concrete Spread Footings

The use of CIP concrete footings on an ABC project may appear to be incongruent with the purpose of ABC. In reality, the casting of a spread footing is one of the easiest concrete pours on a project. The concrete is placed at grade; therefore, fall protection is not required for workers. The ground makes up a substantial portion of the forming, which results in a relatively fast construction process. In addition, the reinforcement bar cage can be pre-tied off-site and placed as a single unit.

One benefit to the use of CIP concrete footings is that very large footings can be used. Attempting to precast very large footings is impractical in most cases due to the shipping and handling size and weight.

There are several options for CIP concrete footings that accelerate construction. One concept is to use the footing as the “closure pour” between the substructure and the ground. The wall stems or pier columns are cast with reinforcing bars projecting into the footing. The precast elements are temporarily braced to support them until the footing concrete is cast. Connection elements for the braces can be cast into the element when required. Once in place and braced, the footing reinforcing can be placed and the footing poured. This concept was developed byWashington State DOT (WSDOT) and has been used on several projects.

Figure 7.1.2-1 is a sketch of this method of construction. Figure 7.1.2-2 is a sketch of a similar detail. The difference is that headed reinforcing bars are projected from the wall stem in place of hooked bars. This allows the stems to be installed after the placement of the footing reinforcement (which could be pre-tied). The main features of this detail are as follows:

- The design of the reinforcement in the footing is the same as conventional wall design.
- The design of the connection of the wall stem into the footing is the same as conventional wall design. If headed bars are used, a strut and tie design approach would be appropriate.
- The construction of the remainder of the wall and bridge could continue within a few days after casting the footing.

The details shown in Figure 7.1.2-1 are for a wall structure for a cantilever abutment. These details have also been used for pier columns. Another pier column option has been developed through research entitled “Precast Bent System for Use in High Seismic Regions,” which is sponsored by the FHWA Highways for LIFE program. The connection of the pier column to the footing is made by casting the column end with external corrugations. The column is simply set on the ground, the footing reinforcing is placed around the precast column, and the footing is then cast. The vertical forces are resisted via interface shear between the column corrugations and the footing concrete. No dowels or shear reinforcement is used between the precast column and the footing. Bending moments are resisted through dowel action of the entire column within the footing.

Testing has proven that this connection has excellent performance for strength and for ductility. The testing included a study of various footing thicknesses. In general, the footing should be thicker than the diameter of the column. WSDOT has built one bridge using this concept. Figure 7.1.2-2 shows construction of this connection. The column shown in the photographs is square, but the base is octagonal. The square shape was selected for aesthetic reasons, with the column core being circular.

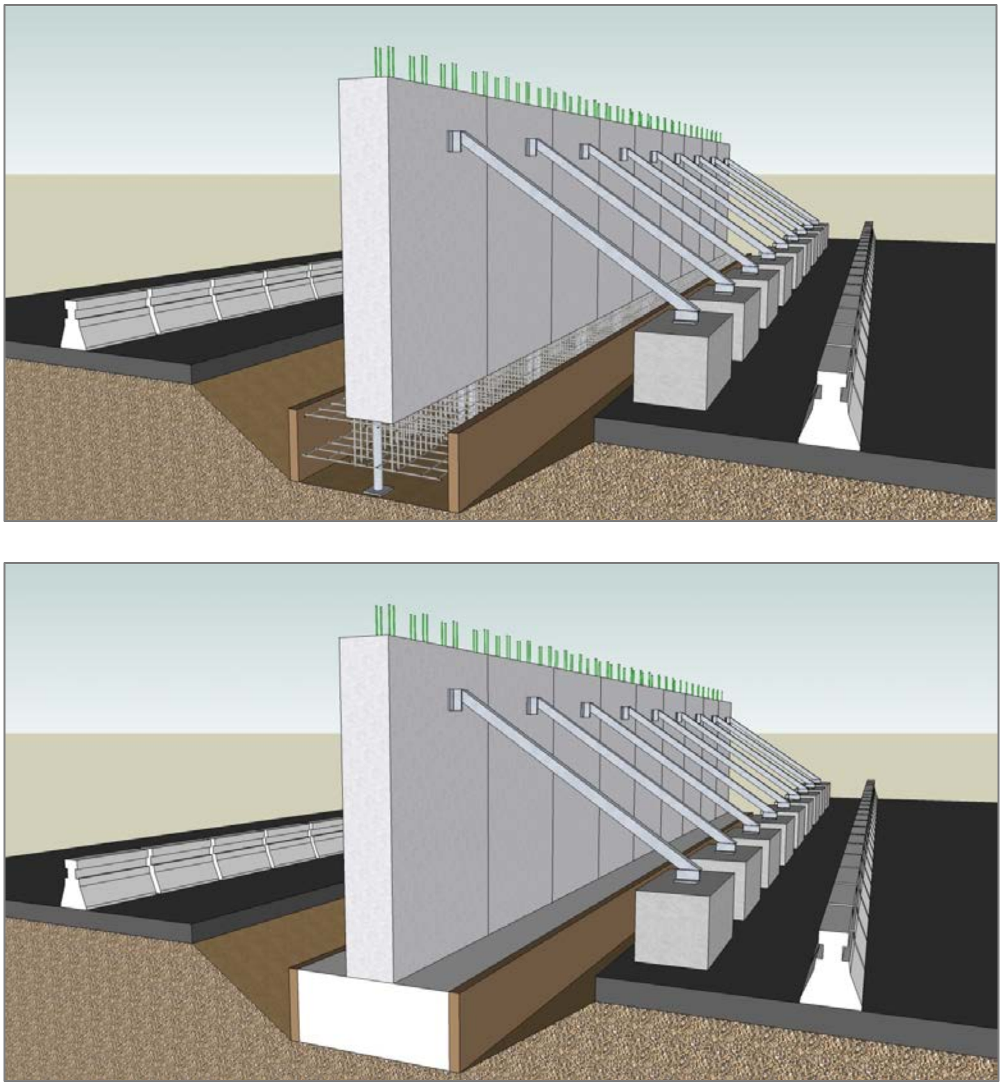


Figure 7.1.2-1. CIP Concrete Spread Footing with Precast Wall Stems.
Top: Precast wall stems supported on temporary struts.
Bottom: After casting of footing.

The design of this connection is based on strut and tie modeling techniques. The development of the column reinforcing is an integral part of the design. The vertical column reinforcement is the tie that resists the tension forces brought on by flexure in the column. In order to use strut and tie methods, it is important to anchor the tie properly. The researchers used headed reinforcing bars to quickly anchor the column bars. Headed reinforcing bars can be manufactured using several methods and are covered under ASTM Specification ASTM A970 Standard Specification for Headed Steel Bars for Concrete Reinforcement [45]. This allows for a thinner footing based on the geometry of the strut and tie model. The research results, including design examples, should be available in 2013.

Figure 7.1.2-2 Corrugated Column End with Cast-in-place Concrete Footing.
Left: Installation of column on temporary pad.
Right: Placement of concrete after installation of top reinforcing mat.
(Source: FHWA Highways for LIFE).

7.1.3. Combined Precast and Cast-in-Place Concrete Footings

Another option for large footings is to combine precast concrete with CIP concrete. The concept is to use precast concrete footings under the column or wall elements, and use CIP concrete to extend or connect the footings. The CIP portion is a simple closure pour. Figure 7.1.3-1 shows several examples of this method of construction. The top sketch is a pier bent with individual footings. The CIP portion is used to enlarge the footings. The middle sketch is another pier bent, where the CIP portion is used to connect the individual footings. The lower sketch is a wall pier where the CIP portion is used to create a continuous footing (a cantilever abutment or cantilever wall would be similar).

The approach for the construction of these options would be to install the precast footings, then install the columns or stems, and then cast the closure pours. The casting and curing of the closure pour concrete does not necessarily need to be completed prior to subsequent construction activities. The precast footings can be designed to support the total dead weight of the structure and the combined precast/CIP portion designed to support the total structure loads. In theory, the precast portion would support more load than the CIP portion; however, the minor long-term settlement of the spread footing would tend to equalize the pressure under all portions of the footing. If this assumption is used, it should be verified with the geotechnical engineer during design.

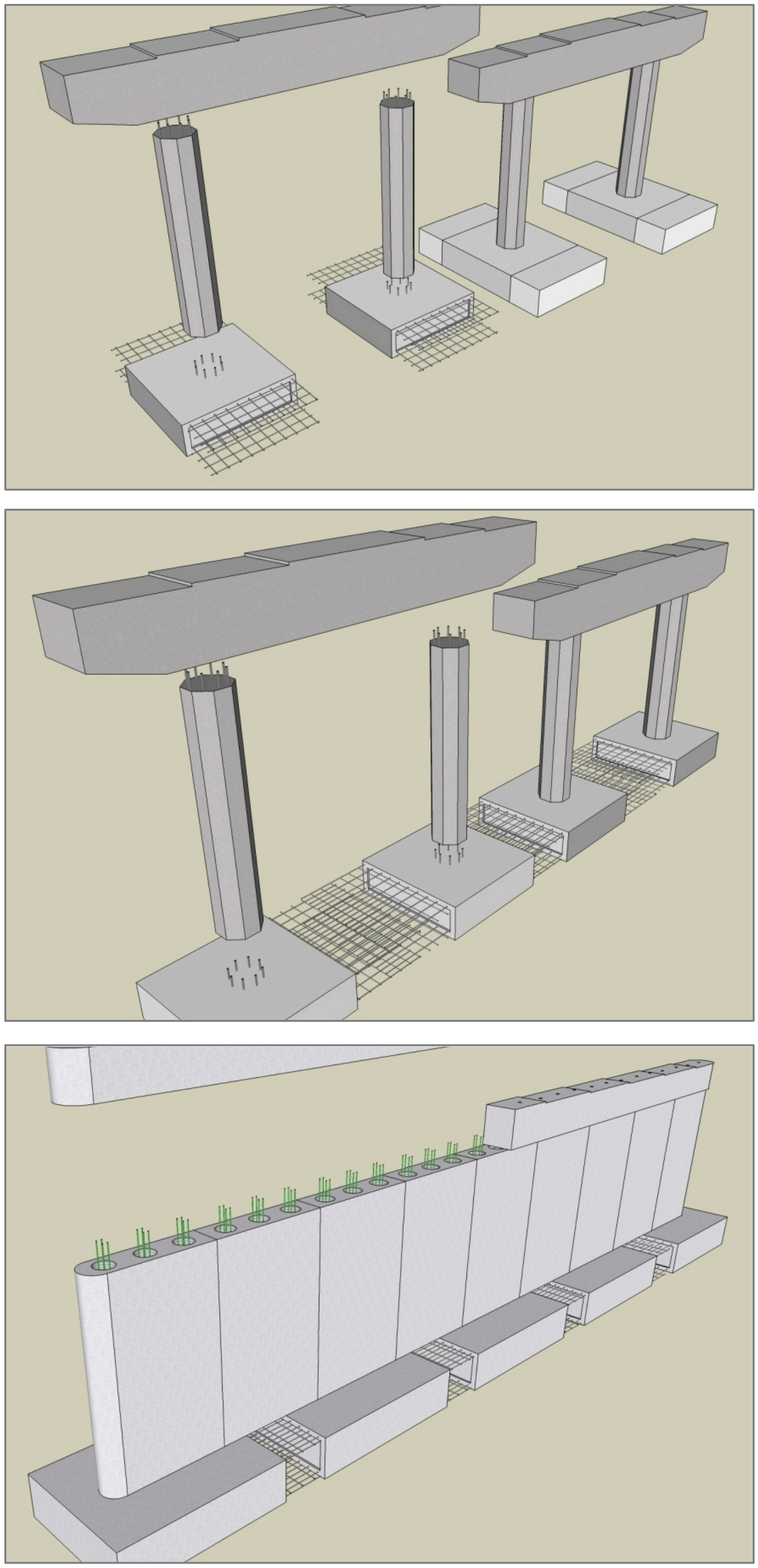


Figure 7.1.3-1. Partial Precast Footing Examples.

7.1.4. Bedding Under Footings

The goal of a precast concrete spread footing is to connect the footing to the substrate in a uniform manner. In theory, it should be possible to accurately compact and grade the soil under the footing and simply set the precast footing on the soil. In reality, it is very difficult to compact soil to a high degree of accuracy. Inevitably, there will be minor gaps between the footing and the soil, which will lead to increased long-term settlement as the contact areas settle and the pressures re-distribute. Placing the footing directly on the soil will also make it difficult to meet the desired vertical installation elevations.

To date, most precast footings have been installed slightly above the substrate and the minor void filled with a cementitious material. Early PBES projects with precast footings used non-shrink grout to fill a small void under the footings, which worked very well. The major issue with the use of non-shrink grout is the cost of the material. In general, non-shrink grouts cost approximately 5 to 10 times more than concrete. The strength of non-shrink grouts is also more than what is normally required for a spread footing. Typical allowable soil pressures under footings are in the range of 4,000 to 10,000 lbs per sq ft. This equates to 28 psi to 69 psi. The control is the strength of the substrate. A typical non-shrink grout has strength in excess of 6,000 psi, which is a magnitude greater than what is required. The cost-effective solution is to use materials that have lower strength and good flowability.

One option is to use a lean self-consolidating concrete (SCC). Lean concrete is a low-strength mix that typically has a compressive strength that is in the range of 1,000 psi to 2,000 psi. The SCC properties should provide a mix that can be placed in a thin void and spread throughout the width of the footing. Another option is to use controlled density fill (flowable fill). Section 2.1.4 contains information on this material. “Non-excavate-able” flowable fill is a moderate strength material of 1,000 psi to 1,200 psi, which is adequate for use under shallow foundations such as precast spread footings. This material is ideal for most applications and is far less costly than non-shrink grout and even lean concrete. Flowable fill and will shrink a minor amount during curing; therefore, the thickness of the void should be kept to less than 6 inches.

In order to give the most flexibility and obtain the best value, designers should offer options for fill materials under footings. In most cases, multiple materials can be used with acceptable results.

The placement of fill materials under spread footings is typically made through ports that are cast into the footing. The area around the footing can be dammed up to form the perimeter. The material should be placed through the ports starting from the center of the footing and progressing outward. The movement of the fill material can be inspected through the adjacent ports. The determination of the size and spacing of the ports should be left to the contractor and based on the material chosen. This falls under the category of contractor “means and methods.” The need for the design of the size and spacing of ports should be incorporated into the specification for the structure “assembly plan.” See Section 14.2 for more information on assembly plans.

7.2. PILE CAPS

Many bridges are supported on deep foundations. The common forms of deep foundations are driven piles (precast concrete, steel H, and steel pipe) and drilled shafts. In general, drilled shafts have higher capacity than driven piles; therefore, fewer piles are required in a typical bridge. Pile caps differ from pile bents. In pile bents, the piles are exposed and connected directly to the pier cap. Section 7.3 contains more information on pile bents.

The loads from the substructure are typically transferred to the piles through a reinforced concrete pile cap or footing. An exception to this is when a large diameter drilled shaft is connected directly to a pier column. Precast concrete pile caps can be used in many cases.

7.2.1. Use of Corrugated Metal Pipe Voids

The connection between the pile and the cap is typically made through the use of a grouted void. One option is to use a corrugated metal pipe void that is cast into the footing, which is similar to the connection of a pile to a precast integral abutment (see Section 5.1.2.2). Figure 7.2.1-1 is a sketch of a precast concrete pile cap with the connection made using corrugated metal pipe voids. The voids are cast into the precast footing and the footing reinforcing is placed adjacent to the void. The void is run completely through the footing in order to facilitate the placement of closure pour concrete. It is possible to run the footing reinforcing through the void; however, this adds some complexity and cost to the fabrication of the footing.

The main difficulty with this approach is that the spacing of the footing reinforcing needs to be adjusted by a significant amount in order to get around the void. The sizing of the void is based on the pile size and the specified pile installation tolerance (see Section 5.1.2.2). For a typical 12-inch H pile installation, the void will need to be approximately 30 inches in diameter, which is too large for most situations. Based on this, this detail should only be used for smaller round piles or micropiles, or the reinforcing can be specified to be run through the void.

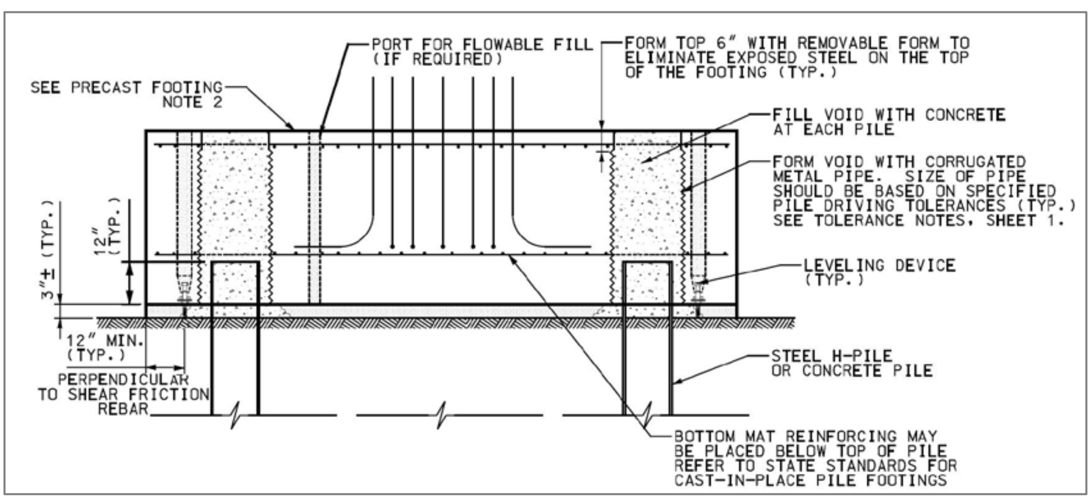


Figure 7.2.1-1. Corrugated Metal Pipe Void for Pile connection (Source: PCI Northeast).

Corrugated metal pipe voids can accommodate a significant amount of vertical force. The force is transferred to the pile cap via interface shear (also known as shear friction). Section 5.1.1.4 contains information on the design of this connection. In general, the AASHTO provisions for interface shear are used to calculate the punching shear capacity of the void. Figure 5.1.1.4-1 contains sample calculations for this approach.

7.2.2. Cast-in-Place Closure Pours

Another approach to connecting deep foundations to pile caps is to use a CIP concrete footing. Figure 7.2.2-1 is a detail that has been developed by the PCI Northeast Bridge Technical Committee, showing a CIUP footing connecting piles to a precast concrete column or stem element. The detail shows hooked bars projecting from the stem into the footing. The precast element is supported on a temporary strut that is cast into the elements. This detail is based on details developed by WSDOT, and has been used successfully on several projects. The bottom mat of reinforcing could be placed prior to installation of the column or stem; however, the top mat of reinforcement would need to be placed after erection.

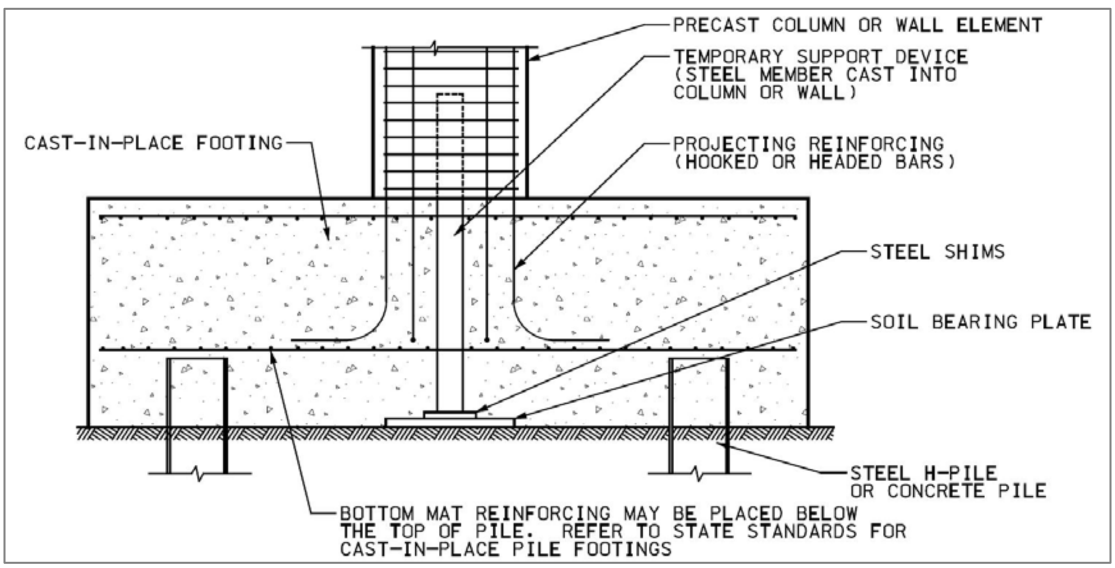


Figure 7.2.2-1. Cast-In-Place Concrete Pile Cap (Source: PCI Northeast).

Another option for a CIP pile cap that supports a precast concrete column is the detail shown in Section 7.1.2-2. Section 7.1.2 contains information on the design of this connection. The benefit for this detail is that both mats of reinforcement can be placed prior to erection of the column.

7.2.3. Precast Columns Connected to Drilled Shafts

A detail that is gaining popularity in many states is the direct connection of pier columns to large diameter drilled shafts. Figure 7.2.2-2 shows a detail taken from the WSDOT Bridge Design

Manual [71]. The approach to this connection is to oversize the drilled shaft so that the column reinforcing can be placed within the shaft reinforcing. The size of the shaft is set to accommodate the specified pile installation tolerance. The concrete in the drilled shaft is cast to a specified elevation. The column reinforcing is then placed within the shaft reinforcing. The connection is then completed using a CIP closure pour.

This detail has been used with CIP concrete columns, but it can also be used with precast concrete columns. The precast portion would start at the upper construction joint shown in the detail. The precast column can be supported on a temporary strut that is cast into the column end (similar to the detail shown in Figure 7.2.2-1).

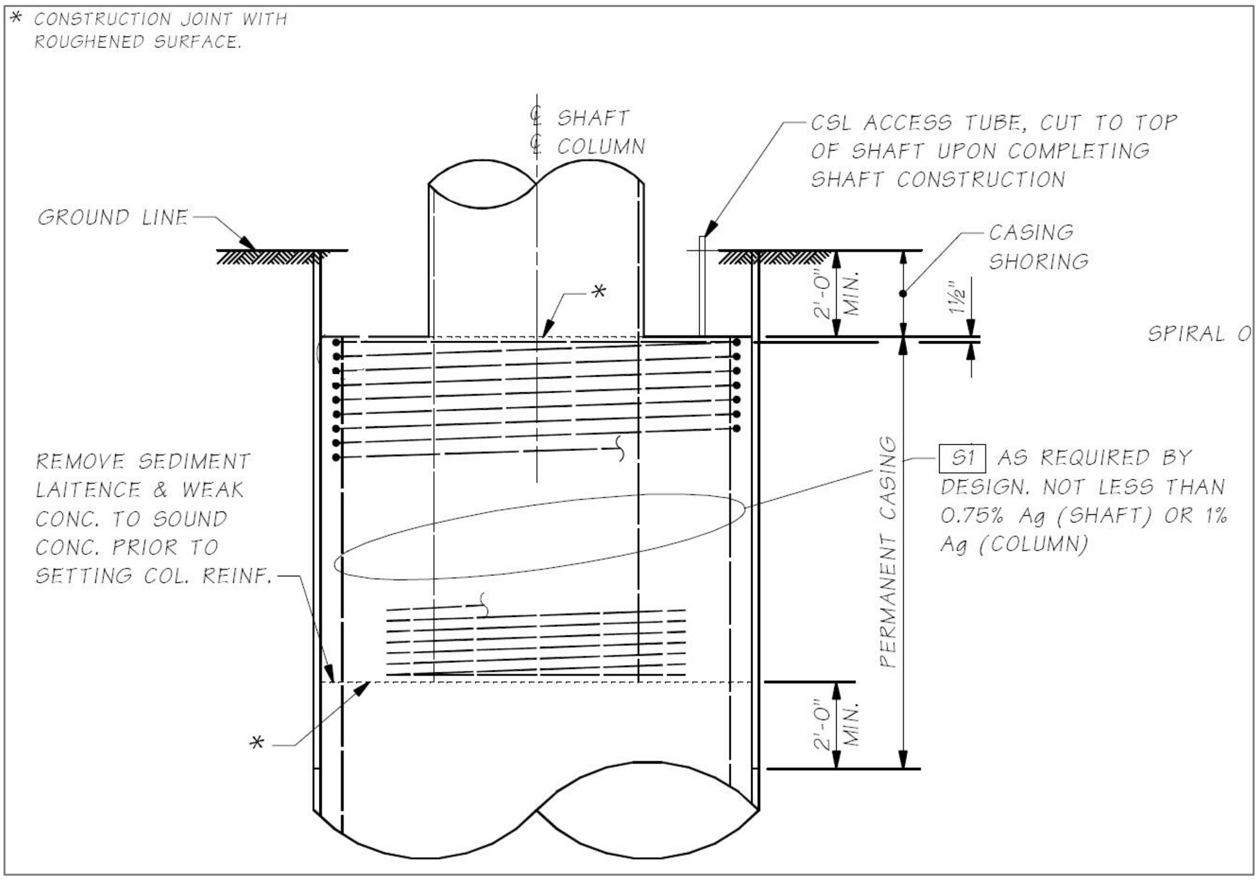


Figure 7.2.2-2. Connection of Pier Column to Large Diameter Drilled Shaft (Source: Washington State DOT Bridge Design Manual [71]).

The design of this connection must account for the fact that the longitudinal reinforcing bars within the connection are not a “contact lap splice” as defined by the AASHTO LRFD Bridge Design Specifications [5]. AASHTO Article 5.11.5.2.1 states:

Bars spliced by non-contact lap splices in flexural members shall not be spaced farther apart transversely than one-fifth the required lap splice length or 6.0 inches.

Therefore, bars spaced farther than this requirement are not considered lap splices and the equations for lap splices in the AASHTO specifications are not applicable. Because of this, WSDOT completed research in order to determine the requirements for this connection. The results of the research are included in the publication entitled “Noncontact Lap Splices in Bridge Column-Shaft Connections” [70]. This information has also been incorporated into the Washington DOT Bridge Design Manual [71].

The approach for the design of this connection is based on the general principles of the modified compression field theory that is used for shear design in concrete beams. During testing, inclined cracks developed in the concrete between the offset lapped reinforcing bars, which defined compression struts. Transverse reinforcement running between the bars was required to provide equilibrium to the forces in the compression struts.

Figure 7.2.2-3 is a portion of the Bridge Design Manual that addresses this design. The spacing of the shaft transverse reinforcement is set based on the parameters of the column reinforcement. Figure 7.2.2-4 shows the typical layout of the reinforcing in the lap zone. The bridge manual contains more information on the allowable shaft installation tolerances for various shaft sizes.

In single column/single shaft configurations, the spacing of the shaft transverse reinforcement in the splice zone shall meet the requirements of the following equation, which comes from the TRAC Report titled, “NONCONTACT LAP SPLICES IN BRIDGE COLUMN-SHAFT CONNECTIONS”:

$$S_{max} = \frac{2\pi A_{sh} f_{ytr} l_s}{k A_l f_{ul}} \qquad (7.8.2\text{-}1)$$

Where:

- S_{max} = Spacing of transverse shaft reinforcement
- A_{sh} = Area of shaft spiral or transverse reinforcement
- f_{ytr} = Yield strength of shaft transverse reinforcement
- l_s = Standard splice length of the column reinforcement
- A_l = Area of longitudinal column reinforcement
- f_{ul} = Specified minimum tensile strength of column longitudinal reinforcement (ksi), 90 ksi for A615 and 80 ksi for A706
- k = Factor representing the ratio of column tensile reinforcement to total column reinforcement at the nominal resistance. This ratio could be determined from the column moment-curvature analysis using computer programs Xtract or SAP 2000. To simplify this process, $k = 0.5$ could safely be used in most applications

Figure 7.2.2-3. Washington State DOT Design Requirements for Transverse Reinforcement In Non-Contact Lap Splices Between Column Reinforcement and Drilled Shaft Reinforcement (Source: Washington State DOT Bridge Design Manual [70]).

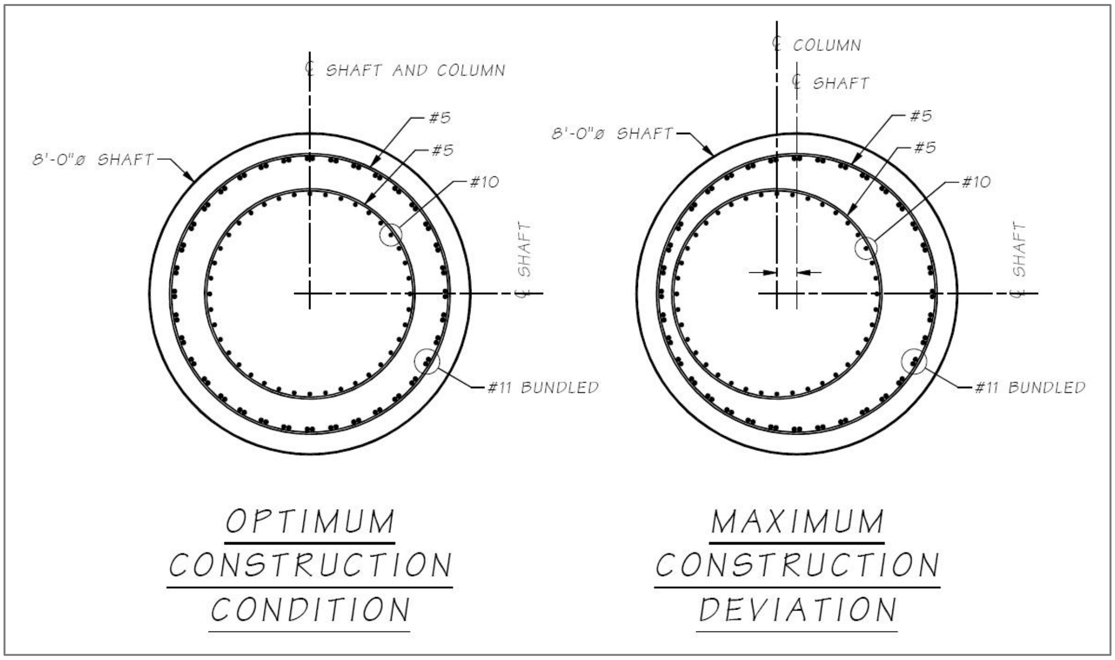


Figure 7.2.2-4. Washington State DOT Shaft Installation Tolerance Details
Typical Section of Drilled Shaft in Column Splice Zone
(Source: Washington State DOT Bridge Design Manual [70]).

7.2.4. Pier Box Cofferdams

Historically, the design and detailing of piers in deep water have involved the use of cofferdams. The intent of cofferdams is to temporarily displace water so that the bridge foundation can be constructed in the dry. Shallow cofferdams are constructed by driving sheeting into the substrate around the intended foundation. Piles can then be driven within the cofferdam, followed by the casting of a concrete pile cap. In some cases, it is necessary to seal the bottom of the cofferdam to prevent water from percolating up through the bottom and flooding the excavation. This is typically done through the use of an underwater concrete pour called a tremie pour. The tremie pour is made after the installation of the piles. If the cofferdam is deep, layers of transverse framework need to be installed in order to maintain the structural integrity of the cofferdam. All of this leads to a slow and expensive construction process.

The advent of the use of large diameter drilled shafts has changed the way that deep water foundations are constructed. The drilled shafts can be constructed within steel casings that can be left in place. The steel casings create isolated mini-cofferdams at each shaft.

The use of large diameter drilled shafts opens up the opportunity to construct the pile cap at an elevation that is above the bottom of the waterway and just below the surface of the low-water elevation. This can dramatically reduce the cost of the substructure since there is less material required. The difficulty with the construction of a perched pile cap is that there is a need to support the formwork for the pile cap.

One way to construct a perched pile cap is to drive additional piling and construct an underwater forming system. A tremie seal will most likely be required in order to prevent water from entering the joints in the forms, especially around the piling. Once the concrete has been placed for the pile cap, the formwork can be removed. This is a difficult operation since it is underwater and under the pile cap. Supporting the formwork on floating systems may be feasible for lakes that do not see fluctuation in the water surface elevation.

Several projects have been built that make use of precast concrete pier box cofferdams. The intent is to replace the cofferdam with a precast structure that will remain in place after the casting of the pile cap. The use of precast concrete allows the incorporation of an integrally cast granite façade that is commonly used to protect the cap from impacts and ice.

If the substrate is adequate, the precast cofferdam can be supported on temporary piling. The design of the pier box needs to accommodate several stages of construction. First, it needs to be able to restrain the hydrostatic pressures caused by the adjacent water. Second, it needs to be designed to support its own dead weight and the weight of the wet concrete. A reduction in the dead load can be made by accounting for the hydrostatic uplift on the system. In tidal situations, this force should be calculated at extreme low tide. The dead weight of the wet concrete can also be reduced by accounting for the fluid forces acting directly over the pile. This can be a significant force when large diameter piles are used.

If the substrate soils are not favorable, it is possible to hang the precast pier box from the drilled shafts. The design of this system needs to be carefully thought out and detailed. Figures 7.2.4-1 and 7.2.4-2 show a precast concrete pier box cofferdam that was used on a project in Rhode Island. A steel framework was installed under the pier box and hung from the steel casing on the drilled shaft. This system was used on two-shaft piers and single-shaft piers. Figure 7.2.4-1 shows details of the system. Large fabricated clamps were installed on the drilled shafts supported by hanger rods attached to the casing top. Once the clamps were fastened in place, the pier box support framing was installed and the precast sections placed. A tremie pour was used to seal the bottom of the cofferdam, followed by placement of reinforcing and concrete. Figure 7.2.4-2 shows the interior of the cofferdam on the single shaft pier caps. The equipment shown is removing the laitance and weak concrete at the top of the shaft. This system was used on a number of pier caps, resulting in savings of months of construction time that would have been associated with the installation and subsequent removal of temporary piles.

The support framework and precast cofferdam needs to be designed for all of the forces previously discussed in this section. In most cases, the precast cofferdam is simply a stay-in-place form that is not designed to support bridge loads. Reinforcing placed within the cofferdam is used to support these forces. If buoyancy is used to counteract the effects of dead loads, the designer should account for the potential fluctuations in water surface. If frame support clamps are used, as shown in Figure 7.2.4-1, the designers should not count on the friction that may or may not develop between the clamp and the shaft. The friction between two steel elements underwater that may have gaps due to tolerances is at best variable. The vertical resistance of this type of system should be supplemented by other means. The detail shown in Figure 7.2.4-1 made use of vertical post-tensioning rods that were attached to the top of the shaft. Other support methods can also be used.

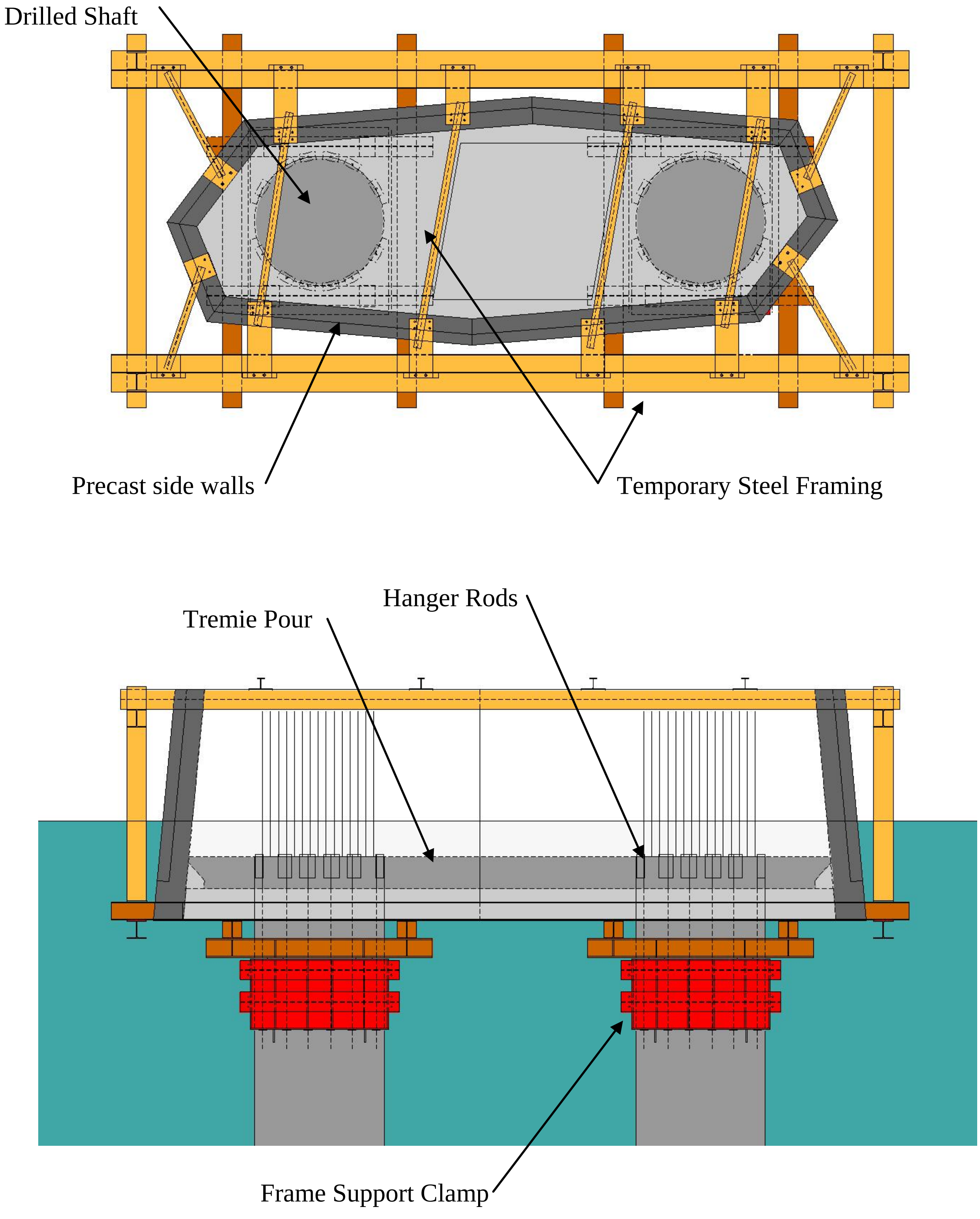


Figure 7.2.4-1. Precast Concrete Pier Box Cofferdam Installation.
Top view: Plan.
Bottom view: Section.

Figure 7.2.4-2.
Top and Bottom photos: Precast Concrete Pier Box Cofferdam Installation in Rhode Island.

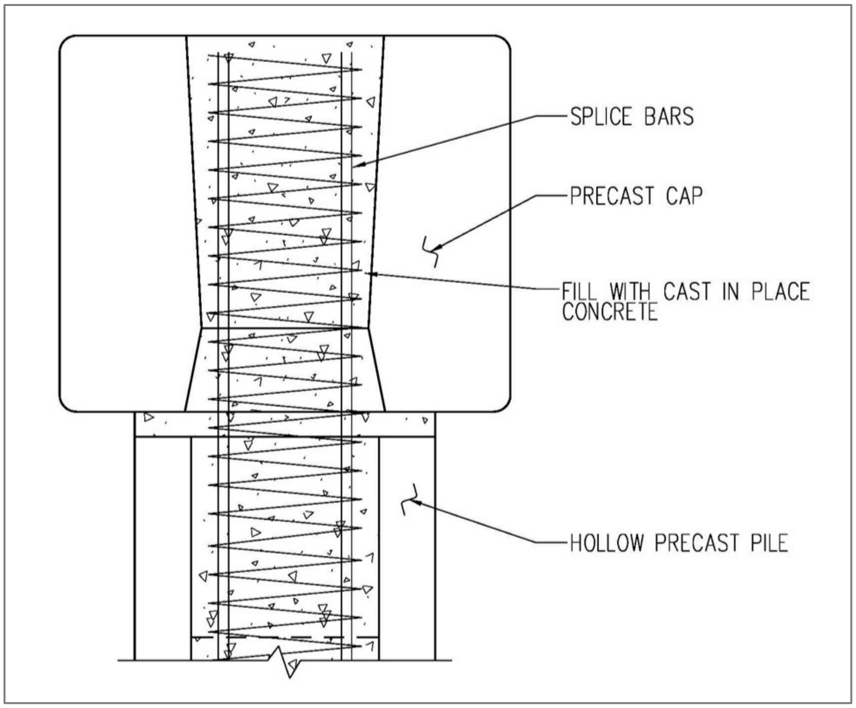
7.3. PILE BENTS

A cost-effective way to design and detail short-span piers is to use pile bents. Pile bents differ from open frame pier bents in that the pier cap is directly connected to the foundation piles. The resulting structure is very efficient since there is no need to excavate and cast a footing on top of the piles.

The speed of construction of pile bents can be further improved through the use of precast bent caps. The key to the design of these caps is the connection with the pile. There are several options for the design of the pile cap connections that are based on research and general engineering principles.

The first option is to project reinforcing steel from the pile into a void in the cap. This approach is ideal for CIP concrete piles. It is more difficult for precast piles because the reinforcing steel needs to be drilled and grouted into the pile after it is cut off. Figure 7.3-1 includes two details that have been used. The reinforcing is projected into a cap pocket that is filled with closure pour concrete. The reinforcing is designed to resist the forces at the connection. Vertical forces are resisted by either seating the cap in a grout bed, or through interface shear (CMP void). See Chapter 5 of this manual for more information on the design of connections using CMP voids.

The second option for this connection is to embed the pile into a pocket in the pile cap. It is common for designers to embed piles into CIP concrete caps. The moment capacity of this connection is obtained by the key action of the embedded pile. Resistance to vertical forces is accommodated through bearing at the top of the pile. Figure 7.3-2 includes two details that have been used. The depth of the embedment should follow the same agency requirements as for CIP construction.



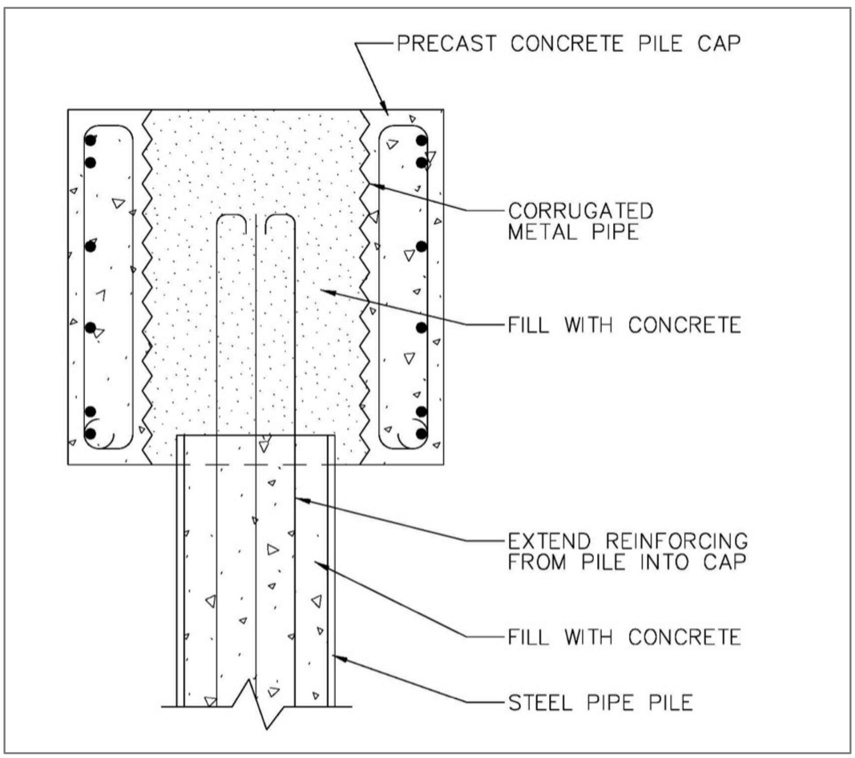
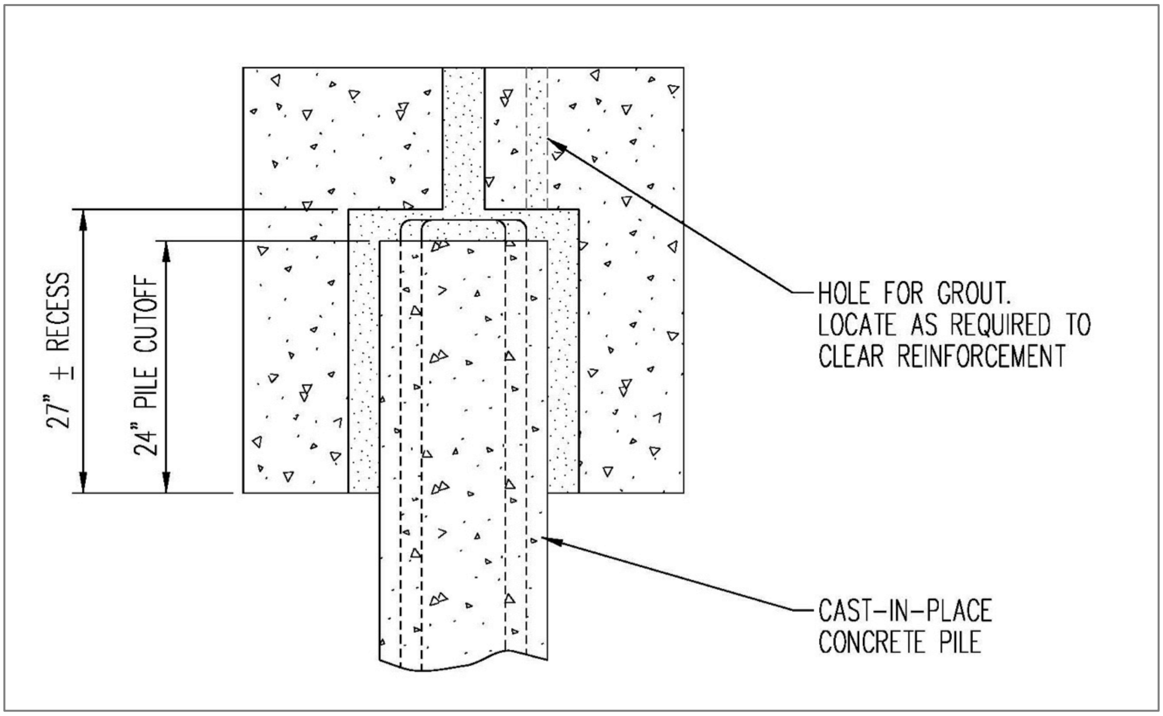


Figure 7.3-1. Pile Cap Connection using Extended Reinforcing Steel.
Top: Detail used in Florida.
Bottom: Detail used in Iowa.
(Source: FHWA Connection Details for PBES [3]).



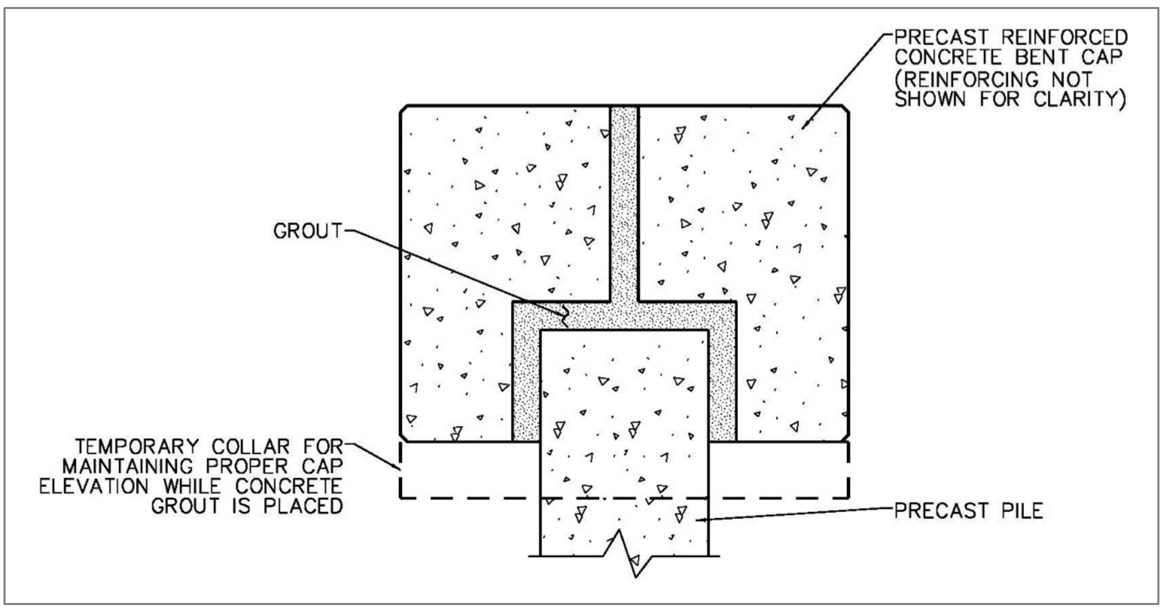


Figure 7.3-2. Pile Cap Connection using Embedded Pile.
Top: Detail used in Minnesota.
Bottom: Detail used in South Carolina.
(Source: FHWA Connection Details for PBES [3]).

CHAPTER 8. DESIGN OF MISCELLANEOUS BRIDGE ELEMENTS

The preceding chapters covered the design of major elements in a typical prefabricated bridge. These elements make up the majority of the bridge. Most bridges also contain miscellaneous minor elements that are not structurally significant, but that can have a dramatic influence on the performance of the bridge (short-term and long-term). This chapter will cover these miscellaneous elements and how they apply to ABC projects built with PBES.

8.1 ACCOMMODATION OF MISCELLANEOUS ELEMENTS IN ABC/PBES DESIGNS

In most cases, miscellaneous elements have little impact on the design and detailing of prefabricated elements. The one exception is the railing and barrier systems. Elements such as deck joints and bearings are sensitive to horizontal and vertical installation tolerances; therefore, a level of adjustability needs to be incorporated into the design and detailing.

In most cases, the design of the miscellaneous elements is the same as with non-prefabricated construction. Detailing is where the variation is found.

8.1.1. Bridge Deck Expansion Joints

Deck expansion joints are used to accommodate thermal movement in bridge superstructures. In older bridges, expansion joints were used to separate adjacent spans in multi-span bridges that were designed as simple spans. Continuity of girders became more prevalent in the 1960s, thereby eliminating many joints in bridge decks; however, typical conventional continuous girder bridges still may require deck expansion joints at abutments and at intervals that are congruent with the expansion capabilities of the joint system. For instance, a strip seal joint can accommodate approximately 4 inches of total thermal movement. This equates to deck expansion joints that are spaced approximately 350 to 400 ft apart. There are expansion joint systems that can accommodate much larger thermal movement; however, these joints can be expensive to build and maintain.

Most owner agencies encourage designers to minimize the use of expansion joints due to their historically poor performance. Leaking deck joints lead to deterioration of the deck concrete, deterioration of concrete beam ends, corrosion of steel elements, and deterioration of bearings, and deterioration of substructure elements. The performance of beam and substructure elements near supports in continuous structures without expansion joints is measurably better than elements below expansion joints.

Based on these facts, most owner agencies prefer to build bridges with as few deck expansion joints as possible. This has led to the development of “jointless” bridges that include integral abutments and semi-integral abutments combined with continuity across piers.

Fortunately, the development of prefabricated deck elements has taken this fact into account. Typical details for prefabricated decks include accommodation of beam continuity. This can be achieved through the use of longitudinal post-tensioning. The post-tensioning can be designed to resist the tension forces that develop in the deck due to composite dead loads and live loads. Chapter 4 contains more information on the design of prefabricated decks for continuity. Integral abutment designs also can be used with prefabricated deck elements. This is normally accomplished through the use of a CIP concrete closure pour at the interface of the superstructure and the abutment stem. This closure pour provides a continuous reinforced concrete structure from the deck down into the integral abutment stem, which results in a jointless connection. Section 5.1.2 of this manual contains more information on integral abutment design using prefabricated elements. Figure 5.1.2.1-1 shows a typical section of an integral abutment bridge built with precast abutment element and a precast deck system.

It also is possible to build jointless decks without beam continuity. Link slab technology has been used on prefabricated bridges that are designed with modular beam/deck elements. The use of link slabs allow for span-by-span construction. Span-by-span construction has been used for many years in prestressed concrete beam bridges. The deck at the piers is typically cast continuous over the piers. The joint below the deck also can be designed for live load continuity.

Most multi-span steel bridges are designed for full continuity (dead load and live load). This requires the use of field splices that are typically placed near dead load inflection points. Recently, bridge engineers have developed steel girder designs that make use of span-by-span techniques combined with link slab technology. The beams are not continuous, but the link slabs make the bridge deck continuous, which achieves the goal of a “jointless” deck. There is a loss of efficiency in the steel framing; however, the elimination of girder field splices can be used to offset the additional steel. This technique works very well for modular prefabricated deck/beam elements. The elements can be set quickly without shoring towers. The link slabs complete the connection between the elements. Section 4.3 of this manual contains more information on the design of link slabs.

Even with continuous bridges, integral abutment bridges, and bridges with link slabs, it is inevitable that deck expansion joints will be required for some bridges. This is especially true for long bridges that are not appropriate for integral abutment design. In most cases, the joint system is not integrated into the prefabricated deck elements. This is done for several reasons:

- The layout and erection of deck elements often requires adjustments to account for tolerances in the elements and the framing below the deck. This adjustability can lead to a deck joint geometry that may be beyond the tolerance of the joint system.

- Some deck elements are not built for the full width of the bridge (wide bridges or bridges with crowns). These bridges typically contain a longitudinal closure pour strip. Integration of a deck joint system in the prefabricated deck panel may result in loss of continuity of the joint system across this closure strip.

- Some precast concrete deck panel systems include the use of longitudinal post-tensioning. Incorporation of a joint system directly over the anchorage assembly would make stressing of the deck virtually impossible. Potential leakage of the joint could also lead to deterioration of the anchorage assembly if the anchorage pour back concrete was compromised.

- Article 9.7.1.4 of the AASHTO LRFD Bridge Design Specifications [5] requires that the edge of a concrete deck at a joint be strengthened or supported by an edge beam. Most agencies integrate the deck edge into an end diaphragm or cross frame. Incorporating this design into a prefabricated deck element would add significant complexity to the detailing of the panel edge.

Based on these reasons, most designers choose to incorporate the bridge deck joint into the closure pour at the end of the deck. This allows for substantial installation tolerance adjustment, accommodation of longitudinal deck closure pours, long-term protection of the anchorage assembly, and the ability to support the free edge of the deck at the joint. Figure 8.1.1-1 shows a detail for an in-span expansion joint for a precast concrete deck panel that was developed by the PCI Northeast Bridge Technical Committee. The detail shows a joint system that is incorporated into the overlay. In the case of a bare deck system, similar details could be used for a joint system that is incorporated into the deck pour.

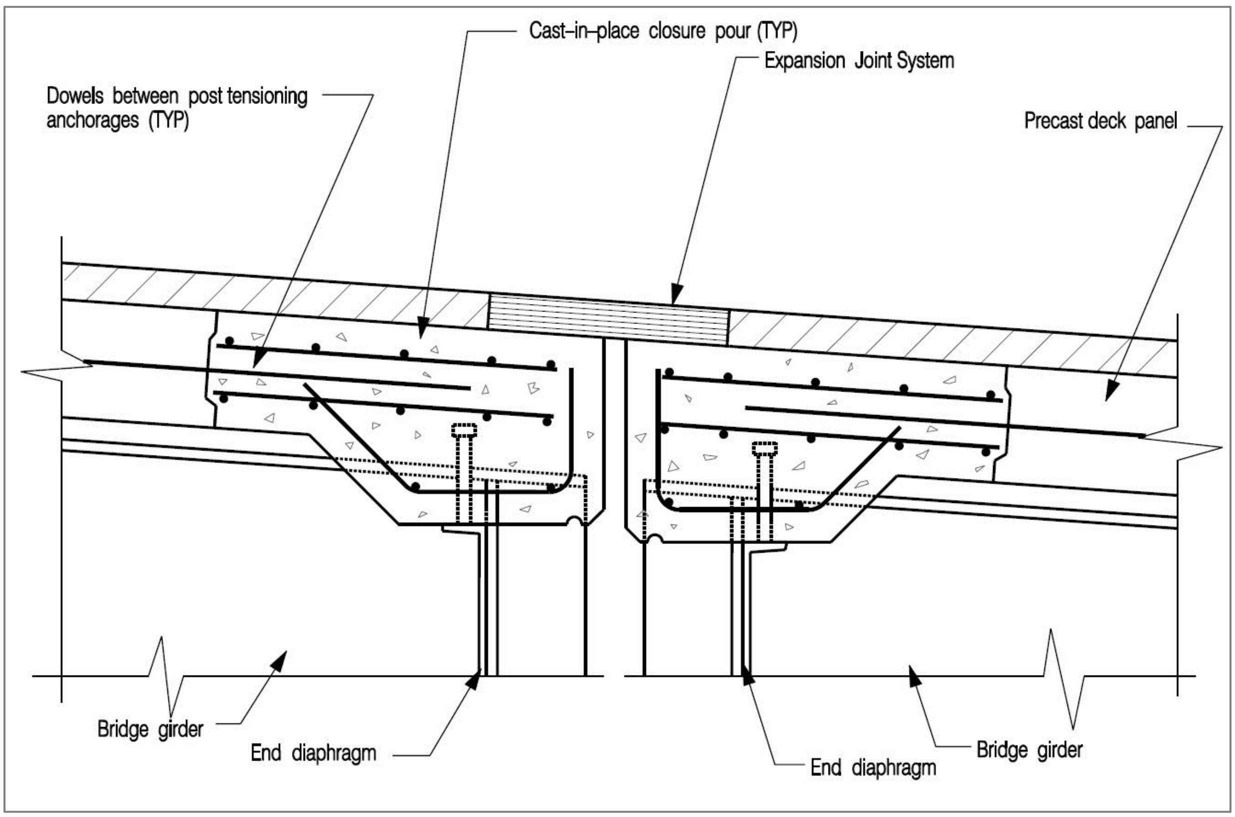


Figure 8.1.1-1. Bridge Deck Expansion Joint in Deck Closure Pour (Source: PCI Northeast).

8.1.2. Bearings

The design of bearings for ABC projects built with PBES is similar to conventional construction. If anchor rods are required for a design, the embedment of the anchor rods in the substructure elements should account for horizontal erection tolerances. This can be accomplished by casting an oversized hole in the substructure element. The size of the hole should account for the following tolerances:

- Dimensional fabrication tolerance for the substructure element.
- Dimensional fabrication tolerance of the superstructure element.
- Erection tolerance of the substructure element.
- Erection tolerance of the superstructure element.

Note: See Chapter 10 for more information on tolerances.

Once the substructure element is installed in the field, the rods can be grouted into the positions that are set by a survey team. If there is room, the bolts can be grouted after placement of the superstructure elements. It may be desirable to set the bolts before erection, since they can be used to ensure the proper location and alignment of the superstructure element. To allow for minor adjustments on steel bridges, the bearing top plate can be oversized and left unattached to the girder, and then field welded in place after the girder is in place.

Bearing assemblies should account for vertical tolerances. The amount of vertical adjustment is a function of:

- Vertical beam seat construction tolerance.
- Fabrication tolerance of the substructure element (thickness measured from bottom of bearing to top of element).

Vertical tolerance adjustment at the bearings is critical for modular deck/beam elements. This adjustment is the last opportunity to produce a quality riding surface on the bridge deck. This is not as critical is there are subsequent opportunities to adjust final grade, such as a prefabricated deck, deck overlay, or deck grinding.

The easiest way to accommodate vertical adjustment is to detail a shim in the bearing assembly. Figure 8.1.2-1 is a sketch of an elastomeric bearing assembly on a modular deck/beam element. Three plates are shown between the beam and the bearing. The top plate is the beam sole plate that is welded to the beam. The bottom plate is a bearing top plate that may be vulcanized to the bearing. The middle plate is the vertical adjustment plate. The approach for installation would be to detail the thickness of the middle plate to equal the anticipated vertical “downward” adjustment. Additional variable thickness plates could be fabricated and kept on site to provide adjustability. The following are examples of how this adjustment could take place:

- Adjustment of -3/8 inch: The middle plate could be removed and replaced with a plate that is 3/8 inch thinner than the specified plate.
- Adjustment of +1/4 inch: The middle plate could be removed and replaced with a plate that is ¼ inch thicker than the specified plate, or a ¼ inch thick plate could be added to the top of the middle plate.

The bearing shown in Figure 8.1.2-1 does not have a masonry plate. If a masonry plate is used, the variable shim plate could be located under the masonry plate. This approach may be easier to install since the adjustment will not require the loosening and re-torqueing of the bearing bolts.

Figure 8.1.2-1. Adjustable Bearing Assembly for Deck/Beam Elements.

8.1.3. Drainage Assemblies

As with bearings, the design of drainage assemblies is no different than with conventional construction. Drainage assemblies are typically cast into the deck and piped to an outlet beneath the bridge. The piping of the drainage is typically mounted on the face of the substructure elements. The attachment brackets can be attached with drilled anchors after erection of the bridge.

Drainage assemblies in the bridge deck can be accommodated in several ways. They can be installed integrally within the prefabricated deck elements in the fabrication shop. The outlet piping of the assembly will most likely penetrate the underside of the deck, which can create difficulties in the construction of the deck element if it is a precast element. The penetration of the piping will require a hole to be placed in the bottom of the deck form. While this is not impossible, it adds another layer of difficulty to the fabrication process. The second way that drainage assemblies can be accommodated is to detail the deck element with a reinforced blockout. After fabrication, the drainage assembly can be cast into the blockout. This can be done either at the fabrication plant as a secondary pour, or in the field after installation of the deck element.

It may be possible to detail and built specialized custom deck drain that can fit within the precast deck section. These types of drains may add cost to the project due to the need for custom frames.

8.1.4. Railings

The AASHTO LRFD Bridge Design Specifications [5] define railings as both post and beam systems and concrete barriers. For the simplicity of this section, the term “railings” will refer to post and beam systems, and “barriers” will refer to concrete parapets.

8.1.4.1. Crash Testing Requirements

FHWA regulations require that all roadside safety devices (including barriers and railings) used on the National Highway System meet the guidelines contained in the NCHRP Report 350 or the American Association of State Highway and Transportation Officials’ Manual for Assessing Safety Hardware (MASH). The FHWA Memorandum “Identifying Acceptable Highway Safety Features” of July 25, 1997, provides further guidance on crash testing requirements of longitudinal barriers.

These requirements add complexity and cost to the development of prefabricated barriers and railings. FHWA maintains a listing of barriers and railings that have been successfully crash tested. They can be found at the FHWA website at: http://safety.fhwa.dot.gov/roadway_dept/policy_guide/road_hardware/barriers/bridgerailings/index.cfm.

8.1.4.2. Concrete Barriers and Slip Forming

The use of precast barriers or parapets has been very limited. This is due to the fact that there are relatively few precast barrier systems that have been crash tested. Attachment of a sloped or vertical faced barrier is very difficult because the bolts need to be installed in a plane that is parallel or nearly parallel to the face of the barrier. Systems that have been developed require fairly large pockets to install the anchors. One of the crash tested precast barriers in the market is a “New Jersey Shape” design with a sloping curb. The anchors are placed in the curb portion of the barrier in large blockouts. These blockouts need to be grouted in order to provide a smooth surface to the barrier face.

Many states are moving toward “F-Shape” and “Single Slope” barriers. These barriers are even more difficult to bolt down. The F-Shape has a smaller curb reveal and the Single Slope has no curb reveal, making the installation with bolts very difficult. The complexity of the attachment of precast barriers to bridge decks has led most designers to use CIP barriers on ABC projects. In some cases, the barrier can be cast after the bridge is opened to traffic if there is an adequate shoulder to provide a work zone. Temporary crash barriers can be placed in the shoulder while the barrier casting and curing is completed.

Utah DOT has developed details for integrally casting the barrier into a precast full depth deck panel. Figure 8.1.4.2-1 shows the installation of a deck panel with an integral barrier. The use of this detail allows for the use of standard "crash tested" barrier reinforcement. Designers should verify that the reinforcement used is the same as in the end of the parapet. Section 4.2.5 contains more information on this detail and the design of deck overhangs.

Figure 8.1.4.2-1. Integral Barrier on Full Depth Deck Panel (Source: Utah DOT).

If casting of barriers is required, there are ways to accelerate the production of the barrier casting operation. Slip forming is a system that uses very low slump concrete that is self-supporting and requires no formwork. A specialty extrusion machine is used to place the concrete and form the desired shape. The reinforcing steel for the barrier needs to be placed and properly secured to prevent lateral movement during the extrusion process. The production rate of slip forming operations is impressive. The rate of installation is up to 3 ft per minute. With this installation rate, it is possible to place over 1,400 linear ft of barrier in an 8-hour work day.

The use of slip forming techniques requires the use of special low slump concrete mixtures that can remain vertical after placement. Admixtures may be required to produce a smooth finish on the barrier. If not properly executed, slip forming can lead to several problems, including:

- Sagging of the concrete.
- Uneven surface finish.
- Shifting of the rebar during concrete placement.
- Ghosting of rebar on the surface (reflective bumps on the surface at each bar).
- Lack of consolidation of the concrete.

All of these problems can be prevented with concrete mixture changes and modifications to construction methods. Most agencies require that trial barrier placements be completed and inspected prior to the installation of the final bridge barrier. The trial installation is inspected for finish and consolidation. Destructive testing is normally done that includes coring and partial demolition in order to determine the presence of voids and segregation.

8.1.4.3. Bolt Down Railing Systems

Many crash tested railing systems are “bolt down” systems. There are crash tested railings that are made with various materials, the most common being steel. Other materials include aluminum and timber.

Bolt down systems are readily adaptable to ABC projects with prefabricated elements, since the major components of the rail are fabricated off site. The anchor bolts can be cast into the deck elements in the fabrication shop. Installation of a rail system on a typical single-span bridge can be accomplished in one day.

8.1.5. Utility Supports

Many bridges are required to accommodate utilities that are present within the highway right of way. In many cases, the utilities are hung under the bridge. Common utilities that are present on bridges include:

- Water.
- Natural gas.
- Sewer.
- Electric.
- Telephone.

In most cases, the utilities are hung between the beams. The attachment of the utilities to the bridge is done in several ways. They can be attached to cross frames that run between the girders. Another method is to attach the utility supports to the underside of the slab.

Accommodation of utilities on prefabricated bridges is the same as with conventional construction. If the utility is to be supported by cross frames, the attachment would be identical to conventional construction. If the attachment is to a prefabricated deck, the connection can be made using standard threaded inserts that can be installed in the fabrication shop.

The accommodation of utilities on modular superstructure elements and superstructure systems can be further simplified. It may be possible to pre-install the new utilities on the elements prior to installation. This will facilitate the construction and limit the amount of work under the bridge.

CHAPTER 9. DESIGN FOR DURABILITY

The goal of building bridges using ABC and PBES is to "Get in, Get out, and Stay out." The first two parts of this statement refer to accelerating construction. The third part refers to building bridges with a high degree of durability to eliminate the need to go back to perform significant maintenance, rehabilitation, or replacement.

The AASHTO LRFD Bridge Design Specifications [5] defines the "design life" of a bridge as:

> *The period of time on which the statistical derivation of transient loads is based: 75 years for these specifications.*

The specifications define the "service life" as:

> *The period of time that the bridge is expected to be in operation.*

The design life value is used to determine specification requirements such as extreme events and fatigue. There are no specific design requirements that are linked to the service life of the bridge in the AASHTO LRFD Bridge Design Specifications [5]. Many agencies use the 75-year design life as the expected service life, since the design is linked to this value based on statistical data. Some agencies have stated that the goal is to reach a 100-year service life for most bridge structures.

Bridges built with ABC and PBES can be used to improve the service life of bridges. Prefabrication of elements in a controlled environment can lead to better quality when compared to traditional construction in exposed environments. Even with outdoor facilities, the fabrication of elements can be scheduled to coincide with good weather days. This is possible because the fabrication facility is not subject to the time constraints of traffic management. Even if fabrication is done in an outdoor facility, curing, heating, or cooling is easier to set up and control when compared to traditional construction.

In recent years, the durability of bridges has been improved through the advent of high performance materials and jointless technology. The use of prefabrication does not preclude the use of high performance materials. The following sections contain information on how to incorporate jointless technologies into prefabricated bridges.

9.1. JOINTLESS DECK DESIGN

The incorporation of deck joints into ABC bridges built with PBES was discussed in the previous chapter. It was noted that deck joints are a major source of long-term deterioration of bridges. Agencies are constantly in search of the perfect deck joint that can be waterproof, durable, maintenance free, and cost-effective. This is an elusive search. It is debatable that there is a solution in the current market.

Deck joints are inevitable on long bridges exposed to temperature extremes. In the past, designers used deck joints on all bridges, based on the approach of isolating the superstructure from the substructure. Lack of continuous girder designs also led to numerous deck joints on multi-span bridges. Modern technology and detailing of bridges is changing these approaches. Integral designs are becoming common, and continuity of multi-span bridges is standard.

The use of prefabricated elements involves connecting deck elements in the field. Some agencies have expressed concern over the use of joints in prefabricate decks. The thought is that these joints result in bridge decks that are not durable. It is important to note that the joints in prefabricated decks are not the same as deck expansion joints. Prefabricated joints are developed and designed to perform as good as or better than construction joints. Most prefabricated joints are reinforced with either post-tensioning or mild reinforcement. High performance materials are used in the joints. The goal is to have a joint with long-term leak-free performance. To date, several details have been developed that meet the goal.

The following sections contain information on how to design prefabricated bridges without deck expansion joints. These technologies are also applicable to all bridges; however, the nuances of these designs on prefabricated bridges will be discussed.

9.1.1. Continuity / Post-Tensioning

Article 9.7.5 of the AASHTO LRFD Bridge Design Specifications [5] contains provisions for full depth precast concrete bridge deck panels. These provisions contain requirements for the incorporation of longitudinal post-tensioning (parallel to the girders) in order to provide adequate distribution of load between panels. The specified level of net post-tensioning stress is 250 psi. This approach has been used for over 20 years and has yielded bridge decks that have had excellent performance.

The design of bridge decks on continuous bridges can result in longitudinal tension stresses in the deck brought on by composite dead loads and live loads. In CIP concrete decks, this stress is counteracted by the addition of reinforcing steel that is run parallel to the girders. Precast concrete deck panels have joints; therefore, it is not normally possible to have continuous reinforcement in negative moment regions. Longitudinal post-tensioning can be used to resist these forces. The amount of post-tensioning can be designed to provide a deck that will be in compression under all loading conditions. Bridges built to these recommendations have performed very well for over 20 years.

9.1.2. Span-by-Span Construction and Link Slabs

The prestressed concrete industry has been building bridges for many years based on span-by-span techniques. This approach differs from full continuous designs where the girders are designed and installed as continuous units. In span-by-span construction, the girders are designed to act as simple-span units to resist dead loads, and continuous units to resist composite dead load and live load (this method is often referred to as simple for DL, continuous for LL). This is done by connecting the beam ends after application of the deck concrete. The resulting structure is jointless, which provides a durable, long-lasting structure. The method of superposition is used to track the stresses in the girders as the structure is changed from a simple-span structure to a continuous structure. Internal stresses brought on by long-term camber changes need to be addressed with this design, but these stresses are not a detriment to the use of this system.

The method of “Simple for DL, continuous for LL” was developed by the precast concrete industry in order to build bridges fast with the need for girder splices and temporary falsework. This may not be the most structurally efficient design; however, it has proven to be a cost-effective solution. The steel bridge industry has historically provided girders that are fully continuous for dead load and live load. This is due to the fact that it is relatively easy to make girder splices in the field through the use of bolted and welded field splices. Recently, the industry has investigated the concept of “Simple for DL, continuous for LL” for short-span steel girder bridges. Some girder efficiencies are lost with this approach, but the need for field splices is eliminated, which can offset the loss of girder efficiency. This method also accelerates construction since erection of elements can proceed rapidly without field splices and potential falsework. A variety of continuity connections have been used at the piers, including bolted flange splices, concrete diaphragms pours, and the use of deck reinforcement to resist negative moments.

The process of making simple-span bridges continuous for live load can be complex. Prestressed concrete girders develop internal stresses brought on by camber changes. This needs to be addressed in the design and detailing. The methods for making steel girders continuous for live load also come with a level of complexity in design and detailing.

An alternative to the simple span made continuous for live load is to design the spans as simple spans but make the deck continuous. It is referred to as “link slab” technology. A link slab is a portion of the bridge deck that is run across the joint between the girders at the piers. Note that this method provides a jointless deck, not a continuous girder.

The theory with link slabs is that the slab is designed to flex when subjected to girder end rotation. Figure 9.1.2-1 shows a schematic detail of a beam end elevation with a link slab. The deck is intentionally de-bonded over the pier region, which creates a flexible deck section that can accommodate rotation without excessive cracking. Research has shown that a de-bond zone equal to approximately 5 percent of the adjacent span length is adequate for this purpose. In the de-bond zone, there are no shear connectors, and the interface between the beam and the girder is intentionally isolated with a de-bonding material.

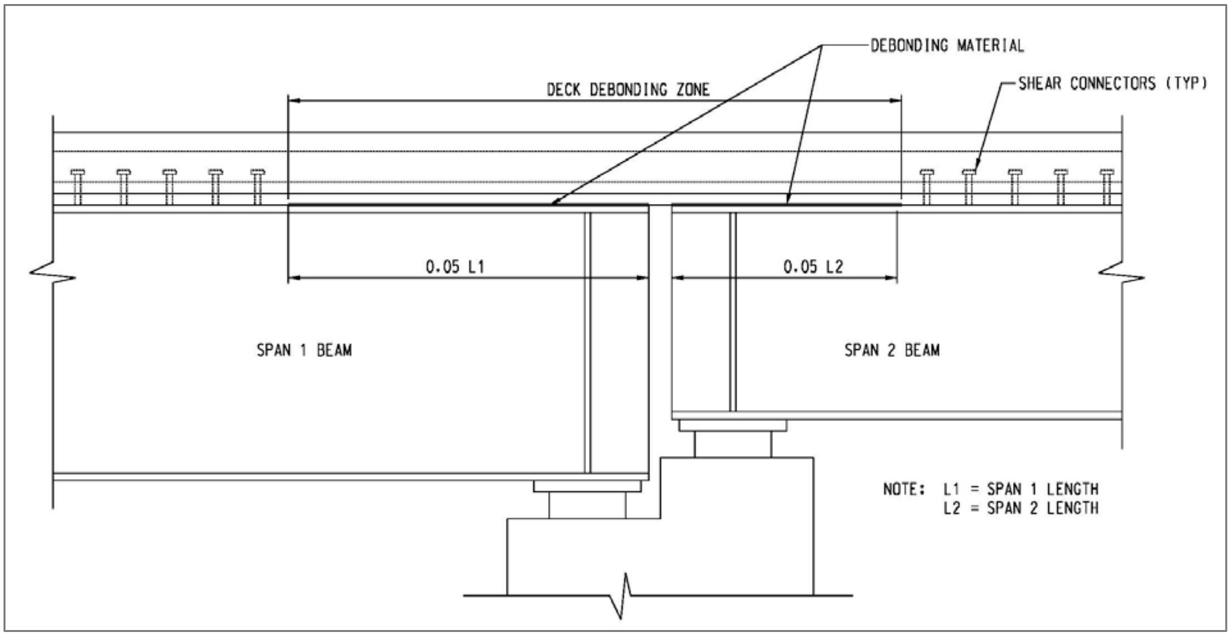
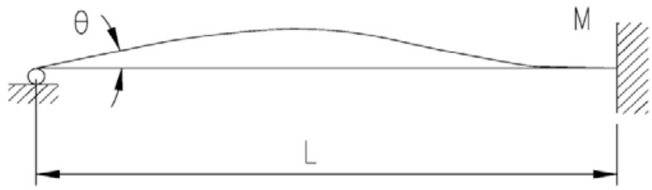


Figure 9.1.2-1. Schematic Link Slab Detail.

Figure 9.1.2-2 is a free body diagram of the link slab theoretical behavior. The bending moment generated in the link slab is driven by the girder end rotation. An important note to consider is that the moment of inertia of the link slab affects the bending moment. One might think that thickening a slab in this region might help with the design. In reality, the thickened slab would generate much higher bending moments, since the moment of inertia is a function of t^3. For example, increasing a slab thickness from 8 inches thick to 12 inches thick would increase the bending moment by a factor of 3.375. Designers should also note that the modulus of elasticity of the concrete will also affect the moment generated in the link slab. Higher strength concretes will lead to higher moments; however, the effect is not significant due to the fact that the modulus of elasticity is a function of the square root of the concrete strength.

Once the bending moment is calculated, the designer can check the section for cracking. If the stress in the concrete exceeds 80percent of its modulus of rupture, supplemental longitudinal reinforcement should be added, according to Article 5.7.3.4 of the AASHTO LRFD Bridge Design Specifications.

Link slabs can be used on both steel and concrete girders. More information on link slab design can be found in the paper entitled “Behavior and Design of Link Slabs for Jointless Bridge Decks” [30].



M = 2 E I Θ / L

Where:

Θ = Girder End Rotation (rad)
L = Total de-bond length (inches)
E = Modulus of Elasticity of link slab concrete
I = Gross moment of inertia of link slab

Figure 9.1.2-1. Schematic Link Slab Detail.

9.1.3. Integral and Semi-Integral Abutments

The previous sections discussed ways to eliminate deck joint within a multi-span bridge. Deck joints at abutments can also be eliminated through the use of integral abutments or semi-integral abutments. The terms "integral abutment" or "semi-integral abutment" refer to abutments that are made integral with the superstructure. The connection between the two can either be a moment or pinned connection (integral), or a roller connection (semi-integral). Even when the connection is detailed as a moment connection, the bridge can often be designed as a pinned connection due to the relative flexibility of the piles when compared to the superstructure. Integral abutments have no joints between the superstructure and substructure. Semi-integral abutments have a joint that is typically located below the beam seat.

An added benefit of integral and semi-integral abutments is that the soil forces acting on the rear face of the abutment are resisted by the superstructure, which acts as a compression strut. Integral abutments are normally pile supported and are designed to accommodate thermal movements and girder end rotations. These movements lead to bending stresses in the piles. In order to minimize these effects, a single row of piles are typically specified. This is possible due to the large resistance of the superstructure to compression forces from the lateral soil loads.

Integral and semi-integral abutments are common in the U.S. Details have been developed for prefabricated integral and semi-integral abutments. Sections 5.1.2 and 5.1.3 contain more information on prefabricated integral abutments.

9.2. SUBSTRUCTURE CONNECTIONS

The substructures located near water or adjacent to roadways with de-icing salts can be exposed to severe corrosive environments. Prefabricated substructure elements can be used to provide better protection through the use of high-quality plant produced elements. Even with the best elements, some designers are concerned with the potential for deterioration of the joints between the elements.

9.2.1. Protection of Critical Connections

The joints between substructure elements are typically grouted in place. These grout joints do not necessarily need to be replaced in the future. Long-term maintenance could potentially include re-pointing of the joints if deterioration were to occur in the future.

The one joint that has led to increased concern is the joint at the top of a footing in a typical substructure element. Figure 9.2.1-1 shows the connection of a precast column to a precast footing that was developed by the PCI Northeast Bridge Technical Committee. The joint is located on the top of the footing. The concern is that water might settle in this area and lead to increased potential for long-term deterioration.

There are several ways to achieve a higher level of protection at this joint:

- The grout can be recessed slightly and a flexible sealer could be installed along the joint.
- The joint can be placed in a recess and grouted into place (see Figure 9.2.1-2).

If the recessed joint is used in conjunction with grouted reinforcing bar couplers, the designer should specify adequate room to allow for grouting of the couplers. The detail shown in Figure 9.2.1-2 shows a 9-inch-wide offset between the column and the recess. This should provide adequate room to grout the couplers. It should be noted that conventional construction would provide a construction joint at this interface. A construction joint could be a location where moisture could penetrate; however, experience has shown that long-term deterioration of these regions has not been a problem.

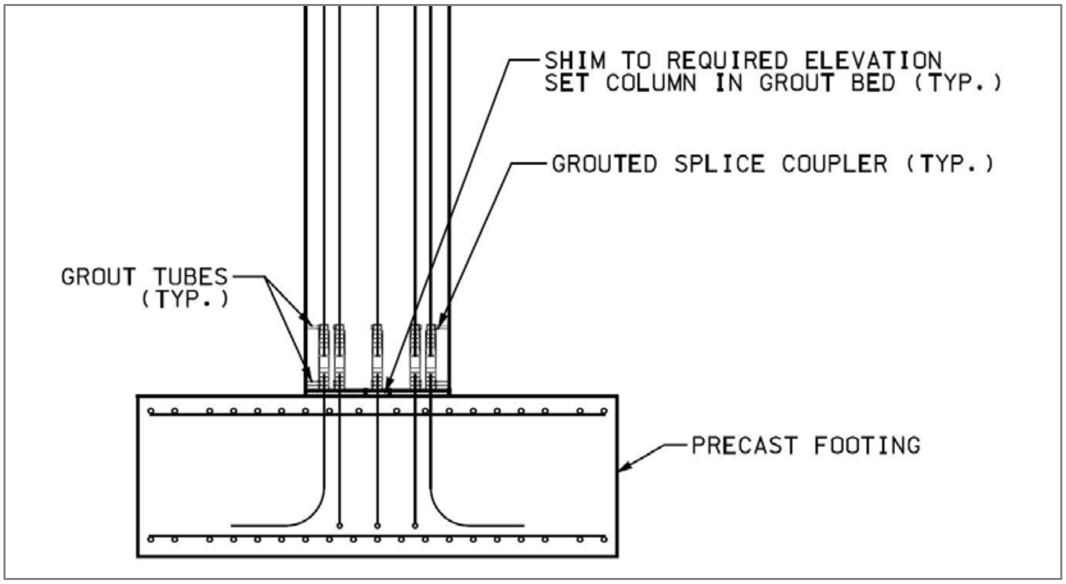


Figure 9.2.1-1. Precast Column to Precast Footing Connection
(Source: PCI Northeast).

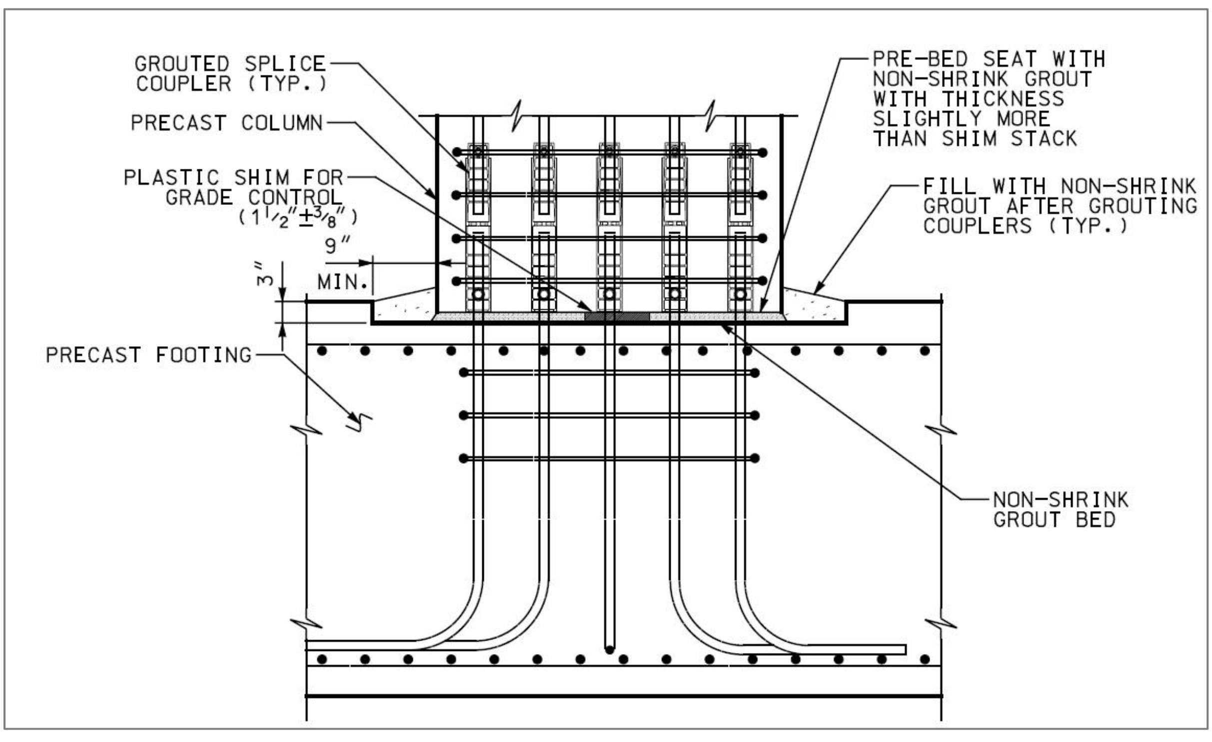


Figure 9.2.1-2. Recessed Precast Column to Precast Footing Connection
(Source: PCI Northeast).

CHAPTER 10. TOLERANCES AND FABRICATION

One of the major differences between building a bridge with prefabricated elements in place of CIP concrete is the need to account for tolerances. The process of CIP construction creates opportunities to make adjustments to dimensions as construction progresses. Prefabricated elements require a certain amount of forethought to ensure that the elements fit together and the bridge is built to the desired dimensions.

It is important to note that all elements are built to a tolerance. There is no such thing as a perfect element. Likewise, there is no such thing as a perfect field built structure. It is normally the designer's role to specify acceptable tolerances. Designers should note that specifying very small tolerances may result in very high fabrication costs due to the high potential for rejection of elements. Specifying reasonable and attainable tolerances will result in more economical structures that can be built without problems. The fabrication industries (both steel and precast) have developed guidance on reasonable fabrication tolerances. Designers should consult with industry partners to learn more about this topic.

10.1. TOLERANCE REQUIREMENTS FOR DESIGN PLANS AND SPECIFICATIONS

Designers should be aware of various types of tolerances when designing and detailing plans. Tolerance details should be included on the plans or in the project specifications. The following is a description of the basic tolerances that should be accounted for in the design and detailing of a prefabricated bridge:

Element Size Tolerance

This is the tolerance associated with the dimensional size of the element. Size tolerances can include length, thickness, height, sweep, and the angles between adjacent planes. Figure 10.1-1 shows a typical element size tolerance detail that was developed by the PCI Northeast Bridge Technical Committee.

Element Insert, Void, and Projecting Hardware Tolerances

These are the tolerances associated with the location of various appurtenances associated with connecting elements. It is typically expressed in two axes (longitudinal (X) and transverse (Y)), measured from a common working line.

Element Horizontal Erection Tolerance

This is the tolerance of the accuracy of placement of an element in the horizontal plane. It is typically expressed in two axes (longitudinal (X) and transverse (Y)), measured from a common working line. Figure 10.1-2 shows a typical element size tolerance detail that was developed by the PCI Northeast Bridge Technical Committee.

Element Vertical Erection Tolerance

This is the tolerance of the accuracy of placement of an element with respect to the vertical datum. It is typically expressed in a vertical axis (Z) elevation measured from a benchmark elevation or sea level elevation.

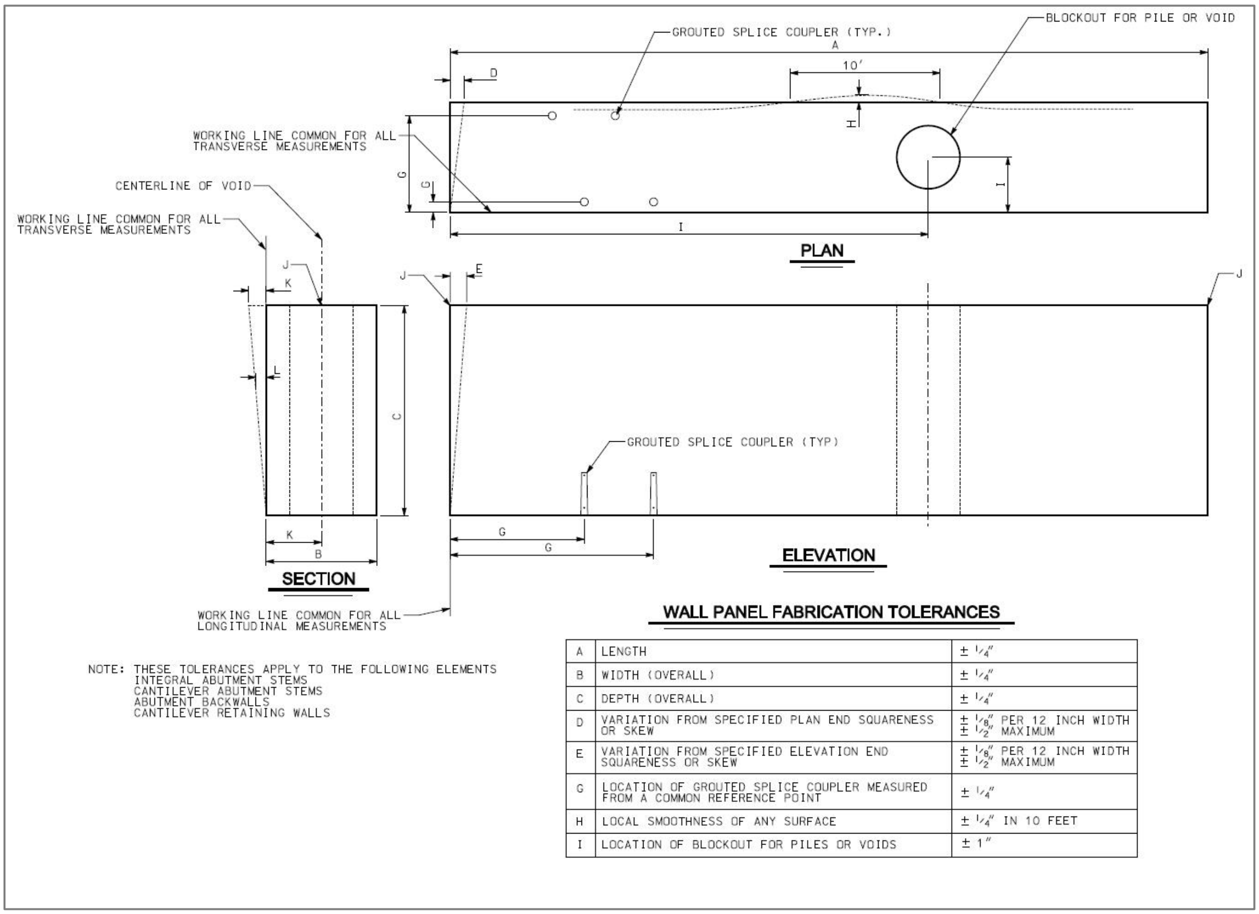


Figure 10.1-1. Example Element Tolerance Detail (Source: PCI Northeast).

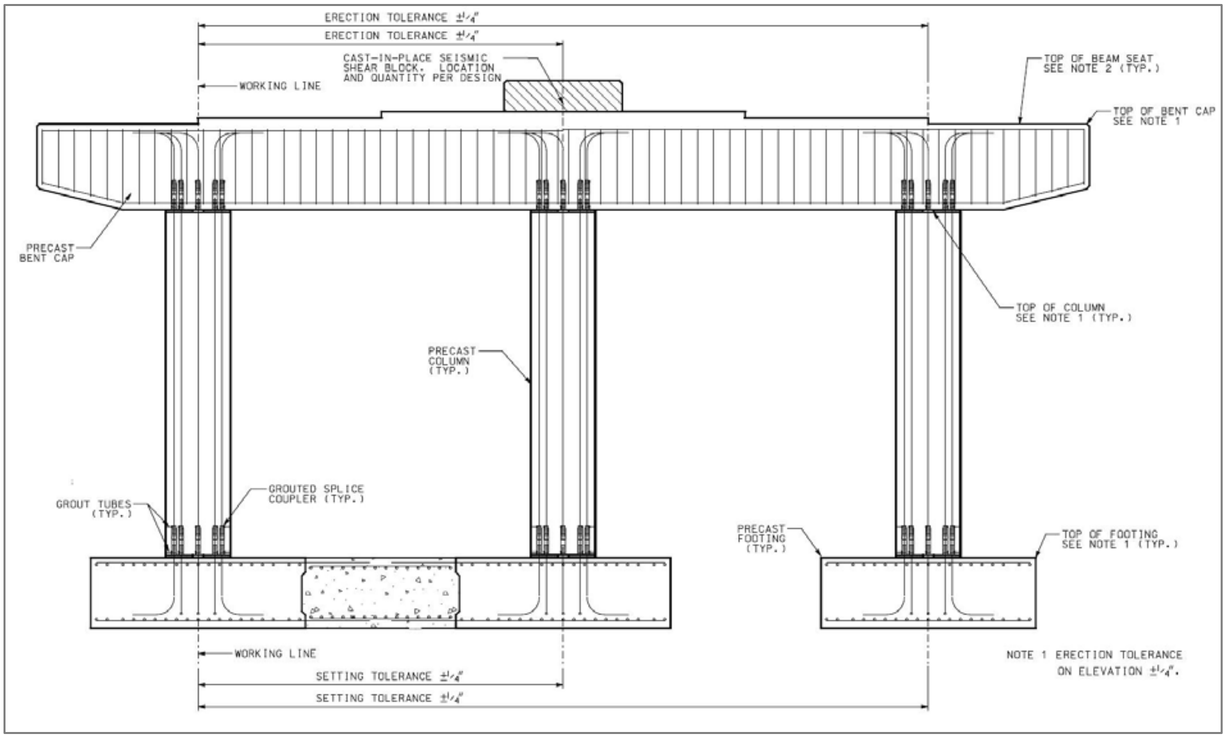


Figure 10.1-2. Example of Element Erection Tolerance Details (Source: PCI Northeast).

One of the most important aspects of measuring tolerances is to always measure from a common working point, line, or plane. Measurements taken from centerline to centerline will result in a build-up of errors, which can lead to a phenomenon called "dimensional growth."

Figure 10.1-3 shows a hypothetical connection between two precast elements. The top element has pockets that are properly laid out with a projecting hardware tolerance of plus/minus 3/8 inches measured from a common working line. The bottom element has projecting reinforcing bars that are laid out with the same tolerance, but measured from center to center of bar. Both approaches have the same tolerance, but the approach to the lower element led to a build-up of errors that resulted in the growth of the overall dimensions. This led to conflicts between several of the bar/pocket connections. Had the lower element been laid out using a working line approach, the maximum offset of the pocket would be 3/4 inches (if the dowel was off 3/8 inches to the left and the pocket was off 3/8 inches to the right). This same approach is applicable to the Element Horizontal Erection Tolerance.

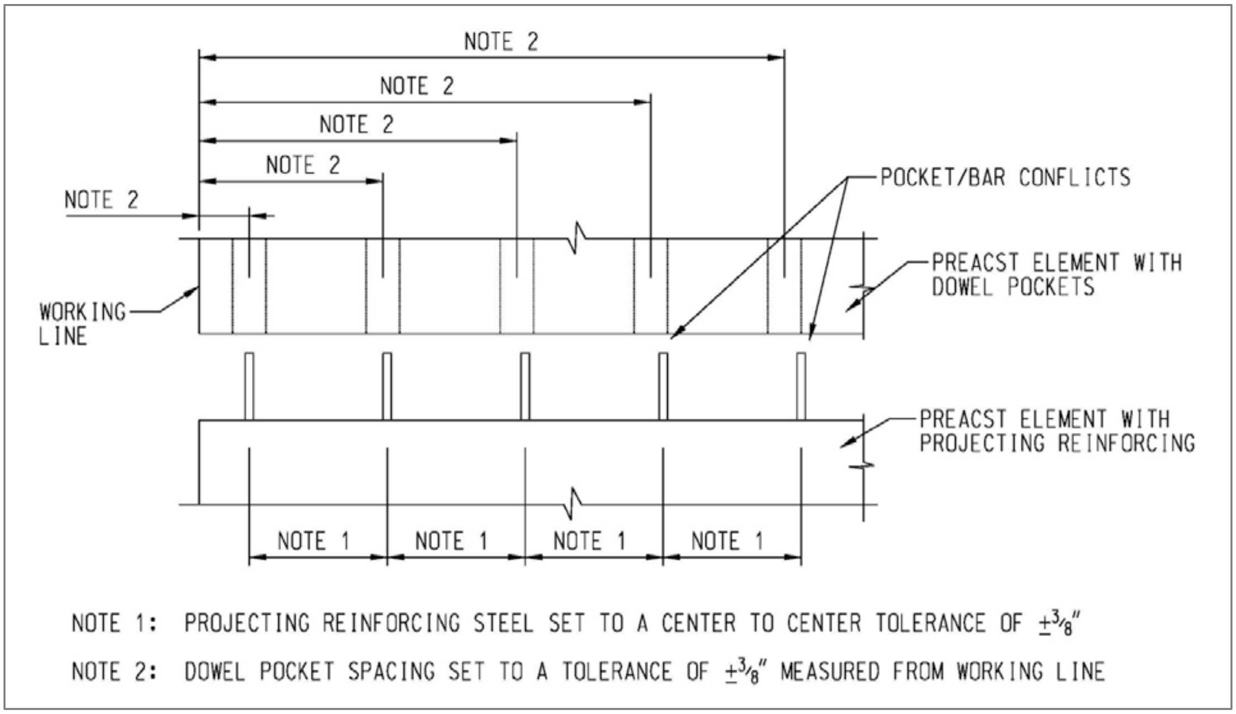


Figure 10.1-3. Example of Dimensional Growth.

10.2. EFFECT OF TOLERANCES ON THE DESIGN AND DETAILING OF JOINTS AND CONNECTIONS

Tolerances do not have significant effect on the design of elements. The major effect is on the detailing of the elements and specifically the joints between elements. The width of joints should be directly related to the element erection tolerance and the element size tolerance.

The width of vertical joints needs to account for the following:

- The possibility that one or both of the adjacent elements will be fabricated wider or longer than detailed, but within the specified element tolerance.
- The possibility that one or both of the adjacent elements will be erected closer together than detailed but within the specified erection tolerance.
- The minimum width of the joint must accommodate the joint filler material after all tolerances are accounted for.

Based on this, the minimum required joint width specified on the plans would be the absolute minimum tolerable joint width, plus the half the width tolerance of the two adjacent elements, plus the erection tolerance of the two adjoining elements. Half of the width tolerance is used for each element, assuming that the over-width would be taken up, half on each side of the element. The equation for the specified vertical joint width would be:

$$W_j = W_{min} + ½ *(w_{tl} + w_{tr}) + (e_{htl} + e_{htr})$$

Where:

W_j = Specified Joint width
W_{min} = minimum tolerable joint width
w_{tl} = width tolerance of left element
w_{tr} = width tolerance of right element
e_{htl} = horizontal erection tolerance of left element
e_{htr} = horizontal erection tolerance of right element

Using the same logic, the joint width tolerance would be half the width tolerance of the two adjacent elements, plus the erection tolerance of the two adjoining elements.

T_{jw} = Joint width tolerance = $\pm$ (½*(w_{tl} + w_{tr}) + (e_{htl} + e_{htr}))

The width of horizontal joints is slightly different than vertical joints. Unlike horizontal layout measurements, vertical layout is not normally based on offsets from working lines. This is due to the fact that most bridge structures are made up of only a few stacked elements. The more common way to specify vertical layout is to specify the top elevation of the element within a specified elevation tolerance. When the element is set, the top of the upper element will be checked for elevation. The thickness (width) of the lower connection is adjusted to accommodate the elevation tolerance of the lower element and the height of the upper element.

Elevation check should be specified and measured at the center of the connection, not necessarily at all areas of the top surface. For instance, if a footing is to support a column, the elevation tolerance only needs to be checked at the center of the column, not the corners of the footing.

The thickness (width) of horizontal joints needs to account for the following:

- The possibility that upper elements will be fabricated taller than detailed, but within the specified element tolerance.
- The possibility that the lower element will be erected too high but within the specified elevation tolerance.
- The possibility that the upper element will be erected too high but within the specified elevation tolerance.
- The minimum thickness (width) of the joint must accommodate the joint filler material after all tolerances are accounted for.

Based on this, the minimum required joint width specified on the plans would be the absolute minimum tolerable joint width, plus the total height tolerance of the upper element, plus the vertical erection tolerance of the upper element. The total height tolerance is used since the lower joint will be required to accommodate the entire over-height tolerance of the upper element. The equation for the specified horizontal joint thickness would be:

$T_j = T_{min} + h_{tu} + e_{vtl} + e_{vtu}$

Where:

T_j = Specified Joint thickness

T_{min} = minimum tolerable joint thickness
h_{tu} = height tolerance of upper element
e_{vtl} = vertical erection tolerance of lower element
e_{vtu} = vertical erection tolerance of upper element

Using the same logic, the joint thickness tolerance would be the height tolerance of the upper elements, plus the vertical erection tolerance of the two adjoining elements.

T_{jt} = Joint thickness tolerance = $\pm (h_{tu} + e_{vtl} + e_{vtu})$

The following are examples of joint width determination:

Example 10.2-1. Width of Joint Between Two Precast Concrete Wall Stem Elements.

Givens:

Specified element width tolerance of left element w_{tl} = $\pm$ 0.25 inches.
Specified element width tolerance of right element w_{tr} = $\pm$ 0.25 inches.
Specified element horiz. erection tolerance of left element = e_{htl} = $\pm$ 0.25 inches.
Specified element horiz. erection tolerance of right element = e_{htl} = $\pm$ 0.25 inches.
Minimum tolerable joint width for placement of grout = W_{min} = 1.0 inches (based on materials to be used).

Joint width to specify on plans = $W_j = W_{min} + ½*(w_{tl} + w_{tr}) + (e_{htl} + e_{htr})$
W_j = 1.0 inch + ½*(0.25+0.25) + (0.25 + 0.25)
= 1.75 inches

Maximum anticipated joint width
= Specified joint width + 0.5($w_{tl} + w_{tr}$) + ($e_{htl} + e_{htr}$)
= 1.75 inches + ½ (0.25 inches + 0.25 inches) + (0.25 + 0.25)
= 2.5 inches

Note: *The maximum joint width is based on the assumption that both elements are at the maximum width tolerance and both are set closer to each other than detailed but within the maximum setting tolerance.*

The joint width tolerance would be:
T_{jw} = Joint width tolerance = $\pm$ (½*($w_{tl} + w_{tr}$) + ($e_{htl} + e_{htr}$))
= $\pm$ (½ *(0.25 inches + 0.25 inches) + (0.25 inches + 0.25 inches)
= $\pm$ 0.75 inches

The joint width specification note would then read: 1.75 inches $\pm$ 0.75 inches.

Example 10.2-2. Thickness of the Joint Between a Precast Footing and a Precast Column.

Givens:

Specified element height tolerance of upper element $h_{tu} = \pm$ 0.25 inches.
Specified element vert. erection tolerance of lower element = $e_{vtl} = \pm$ 0.25 inches.
Specified element vert. erection tolerance of upper element = $e_{vtu} = \pm$ 0.25 inches.
Minimum tolerable joint thickness for placement of grout = T_{min} = 1.0 inch (based on materials to be used).

Joint width to specify on plans = $T_j = T_{min} + h_{tu} + e_{vtl} + e_{vtu}$

T_j = 1.0 inch + 0.25 + 0.25 + 0.25
= 1.75 inches

Maximum anticipated joint width

= Specified joint width + $h_{tu} + e_{vtl} + e_{vtu}$
= 1.75 inches + 0.25 inches + 0.25 inches + 0.25 inches
= 2.5 inches

Note: *The maximum joint width is based on the assumption that the upper element is at the maximum height tolerance, the lower element is set higher than detailed (but within the vertical elevation tolerance), and the upper element is set lower than detailed (but within the specified vertical elevation tolerance).*

The joint thickness tolerance would be:

T_{jt} = Joint thickness tolerance = $\pm (h_{tu} + e_{vtl} + e_{vtu})$
= $\pm$ (0.25 inches + 0.25 inches + 0.25 inches)
= $\pm$ 0.75”

The joint width specification note would then read: 1.75” $\pm$ 0.75”

These examples show the relationship of element tolerances, erection tolerances, and joint widths. The same holds true for inserts, voids, and projecting hardware. The erection tolerance and hardware tolerances are interconnected.

If a connection involves the insertion of a reinforcing bar into a device (coupler or duct), the specification for tolerances would be based on the assumption that the bar is installed to one side (say, to the left) and the coupler installed to the opposite side (say, to the right). The combination of these two potential installation tolerances needs to be kept within the tolerance of the insertion of the bar in the device.

The equation for the specified projecting bar location tolerance would be:

$$T_b = ½ * T_{id}$$

Where:

T_b = Specified bar location tolerance
T_{id} = Insertion tolerance of the bar on the device

Based on the requirements of the manufacturer of the device, the equation for the specified device location tolerance would be:

$$T_d = ½ * T_{id}$$

Where:

T_d = Specified device location tolerance
T_{id} = Insertion tolerance of the bar on the device

Based on the requirements of the manufacturer of the device, the following is an example of a connection tolerance determination:

Example 10.2-3. Determine Element Hardware Tolerance on Projecting Reinforcing and Grouted Splice Coupler.

Given:
Manufacturer specified coupler-to-bar installation tolerance = $T_{id} = \pm\ 0.50$ inches

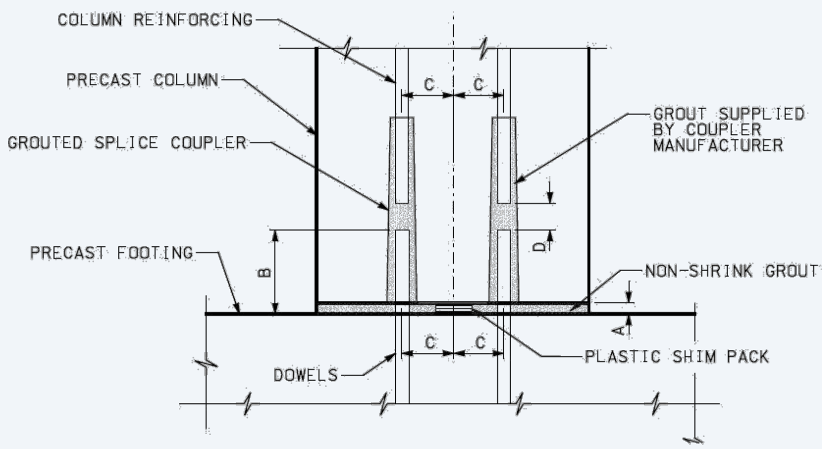
The equation for the specified projecting bar location tolerance would be:

$$T_b = ½ * T_{id}$$
$$= ½ * 0.5 \text{ inches}$$
$$= 0.25 \text{ inches}$$

The equation for the specified coupler location tolerance would be:

$$T_d = .5 * T_{id}$$
$$= .5 * 0.5 \text{ inches}$$
$$= 0.25 \text{ inches}$$

Figure 10.2-3 shows a coupler installation tolerance detail that was developed by the PCI Northeast Bridge Technical Committee.



GROUTED SPLICE COUPLER DETAILS

NOTES:
1. USE MATCHING TEMPLATES AND JIGS FOR THE LOCATION OF REINFORCEMENT AND GROUTED SPLICE COUPLER PLACEMENT WITHIN THE ELEMENTS TO CONTROL CRITICAL DIMENSION "C".
2. CONSULT MANUFACTURER OF THE GROUTED SPLICE COUPLER FOR PROPER DIMENSIONS "B" AND "D" 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.

GROUTED SPLICE COUPLER TOLERANCES

A	SHIM PACK HEIGHT	1 1/2" ± 3/8"
B	DOWEL HEIGHT	CONSULT MANUFACTURER
C	LOCATION OF COLUMN REINFORCING, GROUTED SPLICE COUPLER, AND FOOTING DOWELS MEASURED FROM A COMMON REFERENCE POINT	± 1/4"
D	GAP BETWEEN DOWELS AND COLUMN REINFORCING	CONSULT MANUFACTURER

Figure 10.2-2. Sample Coupler Installation Tolerance Detail.
(Source: PCI Northeast)

10.3. QUALITY CONTROL

In order to meet the tolerance requirements discussed above, the fabrication process for the elements needs to be carried out to a certain degree of quality. The key benefit of prefabrication is that the elements are built in a controlled environment where more precise fabrication practices can be followed. This is not to say that on-site construction is not precise, but that plant fabrication is typically completed with multiple layers of quality control. Fabrication plants have in-house QA/QC personnel that check every step of the process. In addition, agencies will sometimes oversee the QA/QC process with their own staff.

Tolerances can be checked at several steps in the fabrication process in order to ensure that the elements are built within the specified tolerances. Portions of steel elements can be checked prior to shop welding or bolting. Precast elements can be checked during production by measuring forms before casting. Steel elements should be checked after welding since the heat of welding can lead to element distortion. Precast and prestressed elements should be checked after casting, since forms can flex, and prestress can change the dimensions and camber of the element. A good quality control program should include multiple checks on dimensions during the fabrication process.

10.3.1. Fabricator Certifications

Plant certification for element fabrication has been an issue in some parts of the country. Industry organizations have put significant effort into developing quality control programs that will produce elements that are built to a high degree of accuracy.

10.3.1.1. Precast and Prestressed Concrete Elements

Most states require that prestressed and precast concrete elements be fabricated in plants that are certified according to the Precast/Prestressed Concrete Institute (PCI). PCI certification carries the knowledge base, experience base, and research base that a national institute can provide.

PCI specifies certain levels of bridge product certification in the "Manual for Quality Control for Plants and Production of Structural Precast Concrete Products" [73]. The following levels of certification are used by PCI for bridges:

- B1 – Precast Bridge Products (No Prestressed Reinforcement). Mild-steel-reinforced precast concrete elements, including some types of bridge beams or slabs, as well as products such as piling, sheet piling, pile caps, retaining wall elements, parapet walls, and sound barriers.

- B2 – Prestressed Miscellaneous Bridge Products (Non-Superstructure). Any precast, prestressed elements except for superstructure beams. This includes piling, sheet piling, retaining-wall elements, stay-in-place bridge deck panels, full depth deck panels, and all products covered in B1.

- B3 – Prestressed Straight-Strand Bridge Beams (Superstructure). All precast, prestressed superstructure elements using straight, pre-tensioning, or post-tensioning strands such as box beams, I-girders, bulb-tee beams, stemmed members, solid slabs, segmental box beams, and all products covered in B1 and B2.

- B4 – Prestressed Deflected-Strand Bridge Beams (Superstructure). Precast concrete bridge members that are reinforced with deflected pre-tensioning or post-tensioning strand. Included are box beams, I-girders, bulb-tee beams, stemmed members, solid slabs, and all products in B1, B2, and B3.

- BA – Bridge Products with an Architectural Finish. These products are the same as those in Group B, but they are produced with an architectural finish. They will have a form, machine, or special finish. Certification for Group BA production supersedes Group B in the same category.

Designers should specify the appropriate level of plant certification based on these descriptions. It is acceptable to specify different certification levels for different elements. For instance, the designer could specify fabrication of prestressed girders in a B4 fabrication plant, and substructure elements by a B1 fabrication plant. In many cases, the B4 plant will also maintain a B1 certification; therefore, this type of specification will not necessarily result in fabrication at multiple plants.

10.3.1.2. Steel Elements

The American Institute of Steel Construction (AISC) certifies steel fabrication facilities. This institute also maintains fabrication certification programs that ensure high-quality fabrication. The issue of self-performance of fabrication by contractors is not as significant an issue since most contractors do not make steel elements in the field, as is the case with some concrete elements.

AISC maintains the following levels of fabrication plant certification categories:

- Steel Bridge Structures: Consist of un-spliced rolled sections.

- Major Steel Bridge: Typical bridges that do not require extraordinary measures.Typical examples might include:
    1. Rolled beam bridge with field or shop splices, either straight or with a radius over 500 ft.
    2. Built-up I-shaped plate girder bridge with constant web depth (except for dapped ends), with or without splices, either straight or with a radius over 500 ft.
    3. Built-up I-shaped plate girder with variable web depth (e.g., haunched), either straight or with a radius over 1,000 ft.
    4. Truss with a length of 200 ft or less that is entirely or substantially pre-assembled at the certified plant.

- Advanced Bridges: Those requiring an additional standard of care in fabrication and erection, particularly with regard to geometric tolerances. Examples include tub or trapezoidal box girders, closed box girders, large or non-preassembled trusses, arches, bascule bridges, cable-supported bridges, moveable bridges, and bridges with a particularly tight curve radius.

- Standard for Bridge and Highway Metal Component Manufacturers (CPT): This category is modeled on the Building Category but describes certification requirements for facilities that manufacture and supply specific components, composed primarily of metal to bridge and highway construction projects. Certification is appropriate for manufacturers of components that include bracing not designed for primary loads (diaphragms, cross frames and lateral bracing); camera, light, sign, and signal support structures; bridge rails; stairs; walkways; grid decks; drains; scuppers; expansion joints; bearings; ballast plates; and mechanical movable bridge equipment.

Designers should specify the appropriate level of plant certification based on these descriptions. It is acceptable to specify different certification levels for different elements. For instance, the designer could specify fabrication of plate girders in a Major Steel Bridge fabrication plant, and bridge railing elements in a CPT plant.

10.3.1.3. Elements Built with Steel and Precast Concrete

In some instances, designers have details elements that contain both fabricated steel and precast concrete. This is the case for modular deck/beam elements made with steel beams. This type of element brings about a certain complexity of fabrication and certification.

The steel elements should be fabricated in an appropriate steel fabrication plant. The precast decking should be fabricated in an appropriate concrete fabrication plant. The issues arise in the certification of the complete element. In this case the contractor should coordinate between the two plants to certify that the product meets both standards.

10.3.2. Near-Site Fabrication

Issues have arisen in rural regions that do not have nearby fabrication facilities. It is not uncommon for bridges to be located several hundred miles from the nearest precast fabrication facility. In this situation, the shipping of precast elements can lead to higher construction costs. This has led some agencies to consider the use of site-cast concrete and “near-site” fabrication.

In some regions, the location of concrete batch plants can also be problematic, leading to the need for temporary batch plants that are expensive to establish. This can also lead to increased costs. In these cases, the cost of shipping the precast concrete elements may offset the cost of temporary batch plants.

Based on these issues, some agencies have explored the possibility of allowing contactors to “self-perform” precast fabrication of basic elements (not prestressed fabrication). The idea is that most contractors have experience with casting of basic elements. Utah DOT requires that contractors who wish to self-perform precast fabrication obtain PCI certification for precast products.

If PCI certification is not required, it is important to require that the contractor develop and put in place a detailed quality control program. This program should be reviewed and approved by the agency prior to fabrication of any elements. The agency should work with the contracting industry on the development of programs prior to advertising projects. The requirements should be clearly understood by all parties so that the cost of the certifications can be included in the bid. Failure to complete this effort up front may result in a potential for extra work claims during construction.

CHAPTER 11. DESIGN FOR SHIPPING, HANDLING, AND ASSEMBLY

This chapter will explore the effect of shipping, handling, and assembly on the design process. In general, these processes do not change the design process. This is due to the fact that shipping and handling of prefabricated elements falls under the purview of "contractor means and methods." In most cases, design engineers should not be specifying element lift points or lifting methods. The design engineer, however, should be familiar with the capabilities of potential suppliers and contractors for any given project. The engineer should collect the information on prefabricated element handling techniques and equipment, and on the rigging and hauling equipment that could be employed. A long prefabricated element can be lifted at two points, four points, or even eight points depending on the type of element and the ability to resist the lifting and handling forces.

There are exceptions to this approach. For example, handling and shipping of 150 to 220-ft-long precast prestressed concrete I-girders require insert points that may be 15 to 25 ft away for the girder ends. Since the girders are typically designed to be lifted near their ends, the design engineer may need to include either top prestressing steel, reinforcing steel, or both. In some cases, conflict between the demands of handling and bottom fiber tension limits due to full loading require that top strands be de-bonded in the middle zone and later de-tensioned before the diaphragms and deck are installed.

Another example is full width full depth precast deck panels. These are generally cast, stripped, and shipped flat. Coordination between the design engineer and fabricators could be undertaken in the design stages to discuss possible conflicting demands of the handling, shipping, erection supports, and final supports at girder lines.

A third example is the movement of bridge systems. In this case, the bridge superstructure may very well be supported on temporary supports that are not coincident with the final bridge supports. The design engineer needs to verify the preliminary locations of the temporary supports and verify that the bridge can resist the temporary stresses. Even in this case, the contractor should be allowed to modify the temporary support locations within the specified design parameters. The key to successful PBES/ABC projects is to provide as much flexibility as possible to the contractor. This will allow innovation and the use of assembly methods that are tailored to the contractor's equipment.

The major change in bridge construction using PBES is the use of assembly plans. These plans are similar to erection plans, except they are more detailed and they contain a timeline for mechanical connections, post-tensioning, placement of concretes, and installation of grouts. The recommended specifications for assembly plans are included in this chapter.

11.1. ELEMENT DIMENSIONS AND WEIGHT

A common question regarding PBES that is posed by design engineers is: "What are the maximum element weights and dimensions?" Unfortunately, there is no easy answer to this question. Modern roadway transport equipment can haul elements in excess of 250,000 lbs. SPMTs routinely transport power plant equipment in excess of 600,000 lbs.

The maximum weight of elements will vary from region to region and even site to site. The design engineer needs to make a rational decision on the maximum element size and weight based on the location of the bridge and the anticipated fabrication locations. The design engineer should coordinate this with the owner. In general, the designer should consider a maximum element weight of between 50,000 and 70,000 lbs.

Using a larger number of smaller elements is not necessarily a cost-effective approach. In fact, it is quite commonly the opposite. In precast concrete building construction, it is quite common to use the largest element that can be produced, shipped, and erected. Thus, it is an optimization exercise the engineer should engage in before “panelization” of the structure is determined.

The design engineer can also consider site accessibility, crane size, and potential crane locations when evaluating member sizes. Large reach requirements significantly reduce the lifting capacity of cranes; however, short reaches can increase the capacity of cranes. The design engineer can determine the most likely location of a crane that will be used to erect girders. The designer can then pick a reasonable crane for the girder lift, and determine the capacity of the same crane for the other element lifts. In many cases, the substructure lifts will be at a shorter radius, resulting in the ability to lift a larger element. By using this approach, a designer may be able to increase the design element weight without incurring significant expense to the project.

The best way to handle element sizes is to detail and specify smaller elements with optional construction joints. The contractor can then be allowed to eliminate the construction joints and use larger elements, thereby maximizing the use of available equipment and minimizing the construction cost.

One way to reduce element weight is through the use of lightweight precast concrete. Many states are using lightweight concrete for bridge decks. The reduced weight of a deck may have a significant effect on the size of the beams. The effect on substructure and foundation costs is not as dramatic, but it is measurable. Lightweight concrete has not been used for many bridge substructures, since these are typically built with CIP concrete. Prefabrication of substructure elements makes consideration of fabrication of elements with lightweight concrete more attractive. Precast substructure elements can be the heaviest elements in a typical prefabricated bridge. It is not unusual for elements to exceed 200,000 lbs. If a 200,000-lb element were to be constructed with lightweight concrete, the weight could be reduced to under 160,000 lbs. This reduction would dramatically reduce the shipping and handling costs, which could offset the additional cost for lightweight aggregates. There have been concerns regarding the durability of lightweight concrete. Recent unpublished studies have shown that lightweight fine aggregate can be used to improve the durability of concrete [89]. These studies have shown that the lightweight fine aggregate has the ability to provide long-term hydration of the cement during curing, which leads to higher quality concrete.

11.1.1. Shipping of Elements

The use of prefabrication brings about the need to move elements from one location to another. The elements will most likely need to be hauled over roads, and set into place using cranes or other lifting equipment.

The cost of shipping an element increases as the weight/size of the element increases. Unfortunately, the relationship between cost and weight/size it is not linear. Shipping and handling of very large or very heavy elements can dramatically increase the cost. Handling and assembly of the element can also increase with the size and weight of the element as the required crane sizes increase. The location of the cranes during handling can also greatly affect the cost.

There is no simple way to calculate the cost of shipping and handling. Each state has specific rules regarding over-size and over-weight trucks. There are also different rules for escort vehicles and police escorts.

On DBB projects, the design team typically does not know the fabrication location, so specific shipping routes will be unknown. In these cases, the design team should look to use smaller elements with optional construction joints. Designers on DBB projects should work with the state truck permitting agencies to determine reasonable element size and weight limits.

DB projects offer the opportunity for the design team to work with the fabricator and erector on the hauling routes and crane sizes. The DB team can customize the element size and weight to match the available equipment and routes.

11.1.2. Handling of Elements

Prefabrication of element will inevitably bring about the need to handle the elements after fabrication. The most common form of handling is the use of conventional cranes. As with hauling routes, the design engineer on a DBB project does not know what crane will be used on the site. Designers on DB projects can work with the contractor on element weight to maximize the use of available cranes.

On certain projects, elements can be installed using gantry crane systems. Gantries are very flexible in that they can lift and maneuver the elements without the need to reach. Gantry cranes are quite common in fabrication facilities, but not common on project sites, due to the complexities of work zones. In most cases, the gantry will need to be customized in order to fit the work site. The cost of the customization needs to be weighed against the cost of using larger conventional cranes. Designers can contact gantry crane suppliers during design to investigate what type of gantry will work on the site. If gantries are feasible, the maximum size and weight of the elements could potentially be increased (assuming that the larger element can be hauled to the site).

One way to set the maximum element handling weight is to investigate the crane required to set the bridge girders, and check the same crane for the setting of the other bridge elements. In some cases, designers can take advantage of the reduced pick radii for substructure elements. This is because it is common to have the cranes placed near substructures to avoid conflicts with existing travel lanes and rails.

11.2. SPECIFICATION REQUIREMENTS FOR BRIDGE ASSEMBLY

In general, the specifications for prefabricated bridges are similar to specifications for precast concrete or steel beams. The same materials and fabrication requirements often apply to most prefabricated elements.

On most conventional bridge projects, the contractor is required to submit an erection plan for bridge girders. The erection plan, which is normally developed by a registered professional engineer, is used to establish the size and location of cranes, the location of delivery trucks, and the sequence of erection. PBES/ABC projects have similar requirements, except the erection is expanded to other prefabricated elements.

The New Hampshire DOT (NHDOT) developed the concept of an assembly plan for an early prefabricated bridge project in Epping, New Hampshire. Assembly plans are similar to erection plans, except they are more detailed and they include time components for element installation and placements of concretes or grouts. It is recommended that project specifications require the submission of an assembly plan. As a minimum, the assembly plan should include the following:

- Drawings depicting the assembly procedures for the prefabricated elements. The plan should be prepared by a registered Professional engineer.
- A work area plan, depicting items such as utilities overhead and below the work area, drainage inlet structures, and protective measures.
- Details of all equipment that will be employed for the assembly of the structure.
- Details of all equipment to be used to lift elements, including cranes, excavators, lifting slings, sling hooks, and jacks. Include crane locations, operation radii, and lifting calculations.
- Calculations for all aspects of the assembly plan including, but not limited to, erection calculations, crane capacities, rigging calculations, temporary falsework and framing, and element handling stresses. The calculations should be prepared by a registered Professional engineer.
- Detailed sequence of construction and a timeline for all operations. Account for setting and cure time for grouts, mechanical connections, and concrete closure pours.
- Methods of providing temporary support of the elements. Include methods of adjusting and securing the element after placement.
- Procedures for controlling tolerance limits—both horizontal and vertical. Include details of any alignment jigs, including templates for projecting anchors and dowels.
- A detailed installation procedure for connecting mechanical connectors, including grouting procedures.
- Include methods for curing grout and closure pour concrete.
- Proposed methods for installing non-shrink grout and the sequence and equipment for the grouting operation.
- Methods for placement of fill materials.
- Methods for forming and sealing closure pour.
- Methods and equipment for installation of post-tensioning systems, including stressing and grouting procedures.

11.2.1. Handling Forces and Stresses

The calculation of handling forces and stresses is common in the bridge erection industry. Common prefabricated elements in conventional construction are beam and girder elements. These elements are designed to accommodate larger vertical bending and shear forces in the final condition; however, they can be quite unstable prior to placement of the composite concrete deck. Erection engineers typically check the beams for these temporary conditions. For complex structures, the erection engineer needs to check the partially completed structure for forces such as wind. The role of the design engineer is to specify the submission of handling and erection calculations and review them upon receipt. Most states now require that erection calculations be prepared by a registered professional engineer.

The handling of other prefabricated elements follows the same approach. The erection engineer should be checking the forces and internal stresses on each element caused by handling, shipping, and erection. The handling of precast concrete elements needs to be checked at various phases of construction. Often, the controlling condition is when the element is stripped from the formwork. At this time, the concrete may not be cured to final design strength, and the removal of the concrete from the forms will induce forces caused by suction against the forms.

The PCI Design Handbook (Seventh Edition) [33] is an excellent source for information on handling, erection, and bracing of precast concrete products. The manual includes examples of handling calculations and information on form suction forces. The manual also includes recommendations for handling stresses, including recommended maximum stresses for handling of elements to have no discernible cracking. The recommended stress level for this is the modulus of rupture of the concrete divided by a 1.5 safety factor.

The handling of thin concrete panels is not normal in bridge construction projects; however, it is very common in building construction. The PCI Design Handbook contains recommendations for checking thin panels. Design engineers should specify that the lifting and handling stresses be checked by the contractor's erection engineer using the PCI Design Handbook. The previously published FHWA manual entitled "Accelerated Bridge Construction – Experience in Design, Fabrication and Erection of Prefabricated Bridge Elements and Systems" [4] contains an example of lifting calculations for a precast full depth bridge deck panel. Design engineers should become familiar with these calculations in order to properly review handling calculations submitted by the contractor's erection engineer.

11.3. STRENGTH REQUIREMENTS DURING ASSEMBLY OF PREFABRICATED ELEMENTS

It is common to connect prefabricated elements through the use of grouts and concrete pours. The design engineer should specify materials that can resist all forces imposed on the structure in its final configuration. Often, the materials are specified to be compatible with the adjacent precast elements. The final design strength of the grouts or concretes is not necessarily required during assembly. Typically, only a fraction of the strength is required before a contractor can move on to the next step of the assembly. This approach is not specific to PBES projects. It is common for construction to proceed prior to gaining the required design strength. The project specifications should include language allowing the assembly to continue prior to final strength gain. It should be clear that the contractor would be working at the risk of not attaining the design strength, which would require removal or repair. Typically, contractors will choose grouts that are significantly stronger than design strength to ensure that this does not happen.

One way to accelerate the construction of connections between elements is to design the connection for a strength that is lower than the typical design strength of the adjacent elements. The material used in the connection should be specified to match the adjacent elements, but should also allow for opening of the bridge with a lower strength. For example, the Massachusetts 93 Fast 14 project used closure pours to connect the prefabricated beam/deck superstructure elements. The deck closure pour was designed for a concrete compressive strength of 2,000 psi. The concrete specification compressive strength was set at 4,000 psi; however, the contractor was allowed to reopen the bridge when the concrete reached the 2,000 psi strength. This approach greatly reduced the risk to the contractor. The nature of concrete strength gain is that it is relatively easy to get concrete to gain early strength quickly. The final portion of the strength gain can take significantly more time.

11.4. DESIGN OF BRIDGES FOR LARGE SCALE BRIDGE MOVES

The design of bridge systems for large-scale bridge moves is different than the design of lifting and handling of other prefabricated elements. This form of ABC requires that the design engineer establish a viable installation plan based on the bridge geometry and the geometry and topography of the bridge site. The design engineer needs to check the design of the superstructure for the anticipated temporary conditions during casting and moving of the bridge. The majority of calculations required for a large-scale bridge move are done by a heavy lift engineer. This engineer, who is employed by the contractor, should be experienced with large-scale bridge moves and the equipment used for the operations. The project specifications should require the use of an experienced heavy lift engineer or consultant. Specifications should require the submission of the resume (personal or corporate) of the heavy lift engineer for review and approval.

The major change to the detailing of bridges built with large-scale bridge moves is the details at the abutments and piers. The use of traditional detailing on these structures will inevitably lead to problems during installation and long-term performance problems. Several very good details have been developed that not only facilitate the installation, but can provide better long-term performance. Figure 11.4-1 shows a semi-integral abutment detail that was developed by the PCI Northeast Bridge Technical Committee. Figure 11.4-2 shows this detail in use on an SPMT bridge installation in Utah. The detail, which has been used in Washington State for some time, has been used on many SPMT bridge moves. The backwall is made integral with the beam ends and deck. The backwall can also be wrapped around the corner of the superstructure to form a sidewall or cheekwall. The integral backwall seals the beam ends and bridge seat from any potential deck leakage. This detail has many of the positive attributes of an integral abutment bridge, but it also greatly facilitates installation using large-scale bridge moves. The gap behind the abutment stem can be set to accommodate the anticipated construction tolerances.

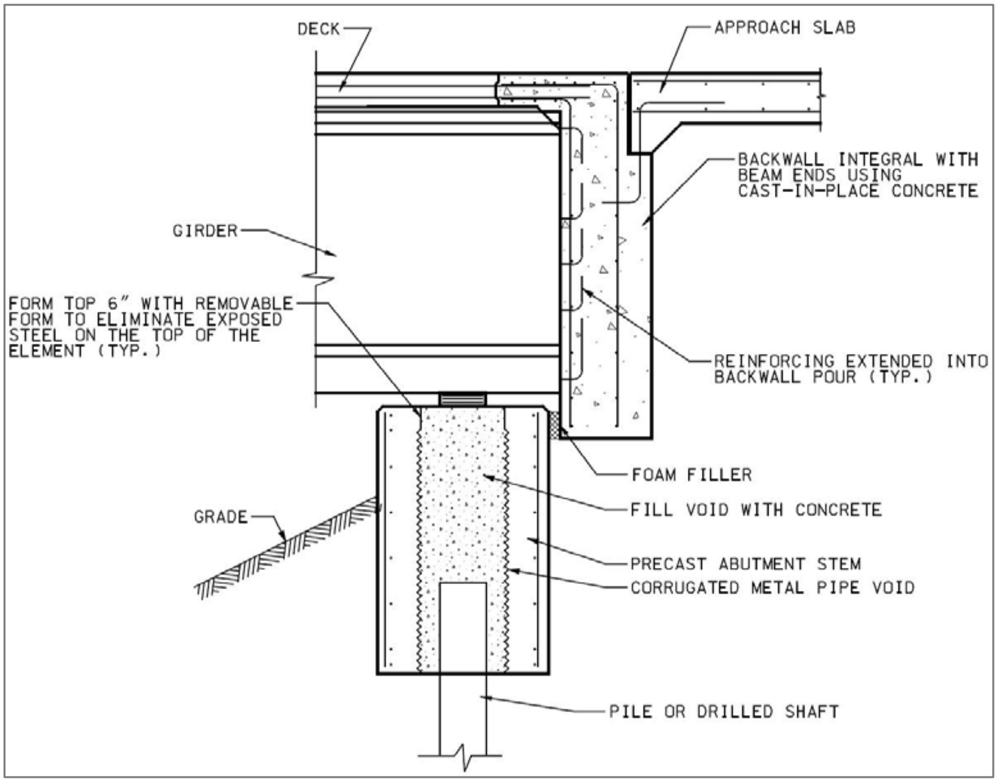


Figure 11.4-1. Semi-Integral Abutment Section
(Source: PCI Northeast Bridge Technical Committee).

Figure 11.4-2. Semi-Integral Abutment Detail used on a SPMT Bridge Move in Utah.

The following sections contain more guidance on the design and construction of an ABC project built using large-scale bridge moves.

11.4.1. Sources of Secondary Stresses During Large-Scale Bridge Moves

In some cases, the design engineer should investigate stresses within the bridge system if the proposed installation method will induce significant stresses in the system. This would be the case with a bridge that is moved with large cantilevers at the beam ends.

As with any bridge built using CIP construction, shrinkage stresses are the major source of deck tension stress, which can cause cracking. Secondary stresses are induced from dynamic effect of the moving structure and changes in support conditions from the time of deck casting until the bridge is moved and set into its final position. These shrinkage stresses in the deck are not calculated in the design process because they are greatly affected by the mix design and the curing methods. The way that this is accommodated in the design process is to add temperature and shrinkage reinforcement in the deck. If the shrinkage stresses in the deck exceed the modulus of rupture, the deck will crack. The width of the crack is controlled by the temperature and shrinkage reinforcing steel.

During a large-scale bridge move, additional secondary stresses can be induced in the bridge. The causes of these stresses can be:

1. Stresses induced from changes in support conditions from deck casting to moving the bridge.
2. Variation in the slope of the support elevations between adjacent spans that can twist the structure, including:
    a. Variation in the slope of the support elevations between adjacent spans on SPMT is caused by variation in the hydraulic pressure between hydraulic zones (see Section 11.4.3 and 11.4.4).
    b. Variation in pressure is caused by unbalanced loads, lateral loads from wind, lateral loads from grades, stopping, starting, and turning.
        i. Variation in the slope of the support elevations between adjacent spans on lateral slide moves is caused by construction inaccuracy in the sliding surfaces.
        ii. Variation in the slope of the support elevations between adjacent spans on lateral slide moves using hydraulic supports (used to compensate for defections in slide beams) can be caused by construction inaccuracy and hydraulic pressure variation from lateral loads.
        iii. Variation in elevations of adjacent supports while maintaining the relative support slopes can induce bending in the structure. Designers can use this effect to their advantage with proper planning and coordination with the bridge mover. Multi-span bridges have been moved with the bent supports lowered relative to abutment supports to induce compression in the deck over the bent during the move.
    c. Variation in support elevations between adjacent girder supports can cause bending in the abutment or bent diaphragms or cross frames. Typically, this is caused by construction inaccuracy.

Most steel or concrete girder bridges are flexible with respect to torsion; therefore, they can accommodate significant twist. The design engineer (on DBB projects) or the heavy lift engineer (on DB projects) should calculate the maximum allowable twist in the structure (see Section 11.4.6). It should be noted that even with a detailed structural analysis, there is still potential for cracks to develop in the deck. This is due to the fact that the induced stresses are additive to the internal stresses brought on by temperature changes and deck shrinkage.

Source of dynamic stresses during bridge moves is the vertical inertial effect on the bridge as it is moved across the travel path. The AASHTO Bridge Construction Specifications [29] do not currently address this potential source of stress. Utah DOT has studied the effects of SPMT bridge moves on superstructures. During the initial group of SPMT bridge installations, the department installed numerous strain gauges on the major structural elements. The goal was to monitor the internal stresses during the entire installation process. Utah State University was asked to review the data and report back on their findings. Figure 11.4.1-1 shows the results of one of the bridge moves. It was a single-span steel bridge that was cast while end-supported, and lifted at a significant distance from the beam ends.

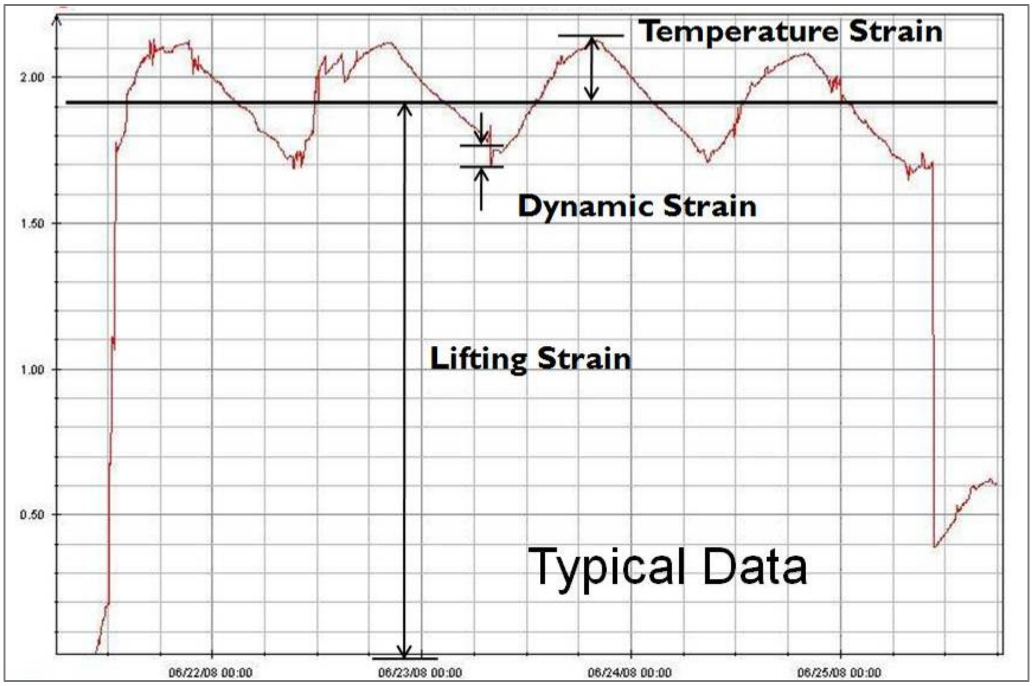


Figure 11.4.1-1. Strain Measurements During an SPMT Bridge Move
(Source: Utah DOT and Utah State University).

The gauge shown was located on the steel girder near the lifting point. The gauges were set to zero after the deck was cast and while the bridge was supported at the ends. The initial strain is due to the transfer of the bridge from the end-supported condition to the SPMT-supported condition. This represents a very large strain that equates to a steel stress of approximately 55 ksi. At first glance, this may seem excessive, but it is important to note that the strain represents a reversal. The area in question was stressed in compression when the gauge was attached. The large strain represents the change from positive bending to negative bending.

The bridge was supported on SPMTs for four days. During this time, two things happened: The structure was moved several times, and the temperature changed daily. The thermal strain, which is represented as semi-sinusoidal curves, is not related to an internal stress change since there was no restraint in the system. This simply represents the length change of the girder when exposed to temperature variations. The jagged portions of the curve represent the strain due to dynamic effects. The Utah State University study showed that the dynamic strains were in the range of 5 percent to just over 10 percent of the dead load strain at the lift points (points of maximum stress). Based on this study, Utah DOT now specifies a dynamic stress allowance by specifying that the dead load of the structure be increased by 15 percent. It is recommended that this dynamic effect also be applied to SPMT bridge placement methods.

Vertical dynamic effects of bridges installed using lateral sliding techniques should be less than SPMT bridge moves; however, this has not yet been studied. The same 15 percent impact force can be used as a conservative estimate.

Transverse dynamic forces also exist during bridge moves; however, they typically do not result in significant stress in a typical bridge superstructure. This is due to the large stiffness of the deck system. Transverse dynamic forces can have an effect on the design of temporary falsework that is used to support the bridge. The starting, stopping, and turning of SPMTs can induce significant lateral forces. Unfortunately, this effect has not yet been studied. In lieu of a detailed study, a lateral dynamic factor can be calculated by looking at the anticipated rate of deceleration of the SPMT. Full speed for an SPMT is approximately three to five miles per hour. Experience has shown that it takes approximately two seconds to abruptly stop an SPMT. This results in a deceleration of 2.5 ft/ sec^2. This corresponds to seven and seven-tenths percent of the acceleration due to gravity. Based on this, a safe recommended value of lateral force can be obtained by applying a lateral force equal to 10 percent to 15 percent of the structure weight. Designers should specify that the heavy lift engineer design the falsework for this level of force in all directions in order to ensure the stability of the shoring and SPMT lift system.

Variation in the slope of the support elevations between adjacent spans can twist the structure. Most steel or concrete girder bridges are relatively flexible and able to accommodate torsion and significant twist. The following are common causes of twist in a structure during bridge moves:

- Variation in the slope of the support elevations between adjacent spans on SPMT is caused by variation in the hydraulic pressure between hydraulic zones. Variation in pressure is caused by unbalanced loads, lateral loads from wind, lateral loads from grades, stopping, starting, and turning.
- Variation in the slope of the support elevations between adjacent spans on lateral slide moves is caused by construction inaccuracy in the sliding surface or deflections of the supports under the sliding surface.
- Variation in the slope of the support elevations between adjacent spans on lateral slide moves using hydraulic supports (used to compensate for defections in supports) can be caused by construction inaccuracy and hydraulic pressure variation from lateral loads.
- Variation in elevations of adjacent supports while maintaining the relative support slopes can induce bending in the structure. Designers can use this effect to their advantage with proper planning and coordination with the bridge mover. Multi-span bridges have been moved with the bent supports lowered relative to abutment supports to induce compression in the deck over the bent during the move.
- Variation in support elevations between adjacent girder supports can cause bending in the abutment or bent diaphragms or cross frames. Typically, this is caused by construction inaccuracy.

On DBB projects, the design engineer does not know the specifics on the bridge move equipment. On DB and CM/GC projects, the design team can work with the heavy lift firm to identify potential sources of secondary stresses.

11.4.2. Installations Using Self-Propelled Modular Transporters

The previously published FHWA manual entitled “Manual on the Use of Self-Propelled Modular Transporters to Remove and Replace Bridges” [2] contains significant information on the design of bridges.

The nature of building a bridge for an SPMT bridge move is that the superstructure can be built using conventional construction methods, including the use of a CIP concrete deck. The superstructure is supported on temporary falsework or abutments at a location away from the final position and moved into place during a short-term roadway shutdown.

The major requirements for the design engineer's role are to:

- Detail the bridge to facilitate the installation (see Section 11.4).
- Specify the need for a detailed analysis of the bridge (in most cases) based on the selected installation methods and equipment.
- Determine and show an appropriate staging area and travel path on the plans. It is important to determine and specify the allowable soil and pavement pressures under these areas.
- Check the calculations that are supplied by the contractor. The installation of a bridge superstructure is essentially the same as the erection of a beam or element. The installation method and the required calculations are in the hands of the contractor based on the anticipated equipment and falsework. This falls under the category of "contractor means and methods;" therefore, it is not under the purview of the design engineer.

11.4.3. Installation Using Lateral Sliding

Lateral sliding is different than SPMT installations in that the bridge is not rotated or moved significant distances. In some cases, the bridge is not even lifted a significant amount. Lateral sliding is becoming more prevalent in the U.S. The reason for its increased use is that lateral sliding is often less expensive than an SPMT move due to the reduction in equipment needed. The trade-off for this reduced cost is that the bridge has to be constructed in parallel to the final position; therefore, the construction of the new superstructure will be completed over the feature crossed (river or roadway). This results in an increase in impacts during construction. The ideal location for lateral slides is for bridges over low-volume roadways and waterways.

In the case of most lateral slide projects, the design engineer can design the bridge the same as a conventionally built bridge. The major role of the design engineer is to detail the abutment and pier details to accommodate the lateral slide. In most cases, the slide should take place on the level. This means that the bridge seats should be detailed as level as possible. A common detail that is finding favor among lateral slide projects is to cast a semi-integral diaphragm at the beam ends. This diaphragm can accommodate cross slopes and superstructure variations due to roadway profile and cross slope. The bottom of the diaphragm is cast level along with the top of the abutment seat. Figure 11.4.4-1 shows a semi-integral abutment diaphragm. A similar approach can be used at piers. Another approach to the accommodation of cross slopes is to detail the bearings with variable height shims or bolsters. This approach may be attractive for narrower bridges with minor cross slopes.

On DBB projects, the project specifications should require that the contractor's heavy lift engineer check the ability of the structure to resists forces during the slide. The scenarios listed in Section 11.4.1 should be checked.

11.4.4. Effect of Support Points During Deck Casting

During deck placement the bridge girders are non-composite. Once the deck cures the girders become composite. At this time, the girder is stressed and the deck only has shrinkage stresses. Any change in support condition from the deck casting condition will induce additional stresses in the deck. The magnitude of the stresses induced in the deck is a function of the change in moment on the system. It is important to note the change in moment on the system is not equal to the absolute moment on the structure.

There are several ways to support a bridge superstructure on temporary falsework during deck casting. The two primary methods are listed below. Engineers/contractors have successfully moved bridges using these methods.

The most common approach is to support the beams at the final bearing locations during deck casting. This method allows the contractor to carefully control grades during casting so that the bridge will fit properly into place and follow the intended profile. Figure 11.4.4-1 shows the construction of a typical Utah bridge *prior to* the SPMT bridge move and one *during* a bridge move. The issue with this approach is that often, the bridge needs to be lifted at locations that differ from the final bearing locations. This is due to the fact that the SPMT travel path requires a certain amount of room for locating and maneuvering the SPMTs.

Figure 11.4.4-1. Construction and Installation of a Bridge System
Top Photo: Bridge in staging area.
Bottom Photo: Bridge during SPMT installation.

This approach can have an effect on the design of the bridge beams and deck, since the lift point regions are designed for positive flexure, but experience significant negative flexure during the move. Section 11.4.6 contains more information on the design of superstructures for this scenario.

It is possible to avoid these cantilevers and the associated stresses. The bridge can be supported at the SPMT pick points during deck casting. One benefit of this method is that once the deck is cast, the static stress in the deck is essentially zero during the bridge move. Another benefit is that once the bridge is set in its final position, the deck is pre-compressed due to the reversal of bending moments near the beam ends. This approach will have an effect on the deflections, cambers, and beam haunches; therefore, this should be carefully checked. The trade-off with this method is that the temporary support framing can get more complex and expensive. Figure 11.4.4-2 shows a temporary support frame for a bridge in Utah that was built with this method. During the bridge move, the SPMTs were driven under this frame, the bolts on the spreader beams were removed, and the entire assembly was moved on the SPMTs.

Figure 11.4.4-2. Temporary support frame on an SPMT move where the deck is cast in the “as lifted” position.

11.4.5. Forces and Stresses During Bridge Moves

The sources of force and stress during bridge moves are discussed in 11.4.1. Designers must evaluate both deck and girder stresses during bridge moves. Coordination with the owner is critical in determining allowable stresses during the bridge move. The designers should evaluate girder stresses in the lifted condition per the AASHTO LRFD Bridge Design Specifications using the service limit state.

Designers should supply and consider the following:

- Permanent and temporary support elevations.
- Permanent bearing size.
- Minimum size of the temporary bearing.
- Any temporary bracing required by design.
- List of assumptions used in design.

- Consider sizing the web of steel girders to accommodate the temporary support location without stiffeners. Refer to AASHTO LRFD Chapter 6 Appendix D6.5 [5].

SPMT Installations: The installation of a bridge superstructure using SPMTs can impart significant forces and stresses into the system during the bridge move. This is due to the need to support the system away from final support points in order to make room for the SPMTs. The previous section contained information on various methods that can be used to support the bridge during deck casting. These different methods produce different forces and stresses in the bridge system either during deck casting or during the bridge move. Subsequent sections will contain more information on how to design bridges for these scenarios.

Parapet cracking is difficult to control when the structure is lifted at a location other than the casting location. Successful strategies for avoiding parapet cracking include the use of discontinuous parapets with full joints at less than 16 ft spacing, compression of parapets with post-tensioning, and addition of longitudinal rebar to control crack widths.

A conservative strategy to design for deck stresses on steel girder bridges is to provide one percent steel in the entire deck per AASHTO Article 6.10.1.7 [5]. This strategy assumes deck stresses due to shrinkage and moving loads will exceed 90 percent of the modulus of rupture.

Alternatively, cracking can be controlled per Article 5.7.3.4 [5]. Designers should coordinate with the owner agency to determine the required exposure condition. Owners may consider using the Class 1 exposure condition given the transitory nature of the move loading and the fact that the stresses that induce the cracks will reverse once the bridge is set in its final position.

Systems using prestressed or post-tensioned decks can be designed for the requirements of Article 5.9.4.2.2 [5]. Designers should coordinate with the owner to determine the required stress limit. Owners may consider using the higher concrete stress limit of $0.19(f'c)^{0.5}$ given the transitory nature of the move loading and the fact that the stresses that induce the cracks will reverse once the bridge is set in its final position.

Lateral Slide Installations: There are two approaches that can be used for lateral sliding. The first approach is to support the structure at or very near the final bearing locations during deck casting. This method will not induce any significant stress reversals into the bridge; therefore, a detailed stress analysis of the superstructure is not necessary. The drawback is that the support locations need to have special detailing so that the structure can be slid into place with no interference from the substructure seats. If steel tracks are used, the structure will need to be lifted in its final position to allow for the removal of the track. Some engineers have replaced the track with an integral diaphragm that spreads the beam reactions to several discrete points. The slide shoe is cast integrally into the bottom of the cap and the whole assembly is moved on top of the abutment on a series of closely spaced PTFE pads. Once in place, the structure can easily be jacked a small amount and the final bearings installed. Figure 11.4.5-1 shows a bridge in Utah that used this approach.

It is important to have the heavy lift engineer check the stiffness of the support frame and the substructures. There have been issues with lateral slides where the support frame flexed, which led to binding of the bridge during the slide. There also have been problems where the substructure flexed as the bridge was slid on top of it. This could be an issue with substructures that are very flexible combined with track loads that are off center. In these cases, an analysis of the substructure by the heavy lift engineer may be necessary.

Figure 11.4.5-1. Lateral Slide Bridge in Utah
Top Photo: Sliding pads on abutment seat with skid shoe to the right.
Bottom Photo: Bridge in final position supported on integral skid shoes.
(Source: HB Engineering)

The second approach is to support the bridge away from the final bearing locations. This simplifies the detailing of the bearings and substructures. The trade-off is that a larger skid frame is required and the superstructure needs to be checked for stress reversals in the temporary condition. Figure 11.4.5-2 shows a bridge replacement project in Oregon. In this scenario, both the existing bridge and the new bridge were slid during the same short closure period.

Figure 11.4.5-2 Lateral slide project in Oregon using independent skid frames (Source: Oregon DOT).

11.4.6. Modeling and Analysis of Systems

On a DBB project, the design engineer does not know the makeup of the temporary falsework that will be used to support the bridge during deck casting or the falsework that will be used between the bridge and the top of the SPMTs. The design engineer should not design or detail these falsework systems. This should be left up to the contractor's heavy lift engineer. Contractors can get quite resourceful in the design of falsework by using materials on hand and rented equipment such as cargo shipping containers. The design engineer can assume that the superstructure is supported on rigid supports that do not deflect vertically when loaded.

Modeling of systems requires a thorough understanding of the move system, the modeling program, and modeling techniques. Parametric studies of varying boundary conditions and structure properties can be critical in identifying design requirements and modeling errors. Consider running the model with fixed and pinned boundary conditions to understand how the variation affects the response of the structure. Run the model with and without cross frames to understand how the cross frames contribute to the response. There are many variations that can be evaluated. The increase in understanding of these effects greatly improves the designer's ability to set up an accurate model to predict the response of the structure.

11.4.6.1. Modeling of the Superstructure and Twist Analysis

The method of analysis of the superstructure is dependent on many factors. Many designers and owners have been concerned with twisting of the superstructure during bridge moves. Experience has shown that a typical bridge superstructure built with I-shaped beams (steel or concrete) is quite flexible. Torsional cracking in bridge decks also has not been prevalent.

The design team need not perform a twisting analysis for most bridges. This effort should be left up to the heavy lift engineer employed by the contractor. The heavy lift engineer can determine what level of twist is reasonable to expect with the equipment proposed. Once this is known, the heavy lift engineer can analyze the bridge to determine if the structure can accommodate the twisting movement.

If deck stresses are limiting the allowable twist, additional reinforcing steel can be added to the deck. If the twist does not significantly increase the deck stresses above the normal live load stresses it may not be necessary to try to torsionally stiffen the superstructure to limit the deflections.

If cross frame stresses are limiting the allowable twist the cross frame size can be increased or decreased. A smaller, more flexible cross frame will attract less force. An increased cross frame size will attract more force but will also reduce the magnitude of the deflection due to unbalanced loading. Both options are viable solutions.

For simple girder bridges moved with lateral sliding or SPMTs without complex travel paths (variable terrain and turns), a 2D grid analysis should suffice for twist analysis. Special structure such as through girder bridges, trusses, and arches may require a more sophisticated 3D finite element analysis for twist calculations. This is due to the fact that 2D gird analysis does not properly capture warping torsion effects. This can have a dramatic effect on the results.

A 3D finite element analysis should be used for structures supported on flexible falsework. Figure 11.4.3-1 shows a bridge being transported on SPMTs with a large transverse cross beam. If a flexible support system is used, the entire system, including the falsework, should be modeled using a 3D model and checked for the inevitable redistribution of forces as the falsework flexes.

Two approaches can be used to modify the falsework to produce equal load at each beam support:

- Shims can be placed under each beam to counteract the flexing of the falsework. The height of the shims is equal to the anticipated deflection of the falsework under load. The tallest shims will be under the beam that is above the point with the largest deflection. When the bridge is lifted, the tall shim pack will engage the superstructure. As the falsework flexes, the subsequent shim packs will engage until the entire structure is supported on the falsework.

- A hydraulic leveling system can be installed under each beam. The system typically consists of a series of hydraulic jacks, one under each support point, that are linked together with a hydraulic manifold. This keeps constant and equal pressure on each jack, which keeps the reaction at each jack the same.

By using these methods, the modeling of the superstructure may be simplified to a 2D grid.

Designers should use full composite properties in the design model. Continuous parapets combined with narrow structures can significantly alter the response of the system; therefore, the parapet should be included in the model. It is appropriate to model the bridge with and without the parapets to evaluate the potential structure response if the parapets crack.

11.4.6.2. Modeling of Falsework and Transporters

One reason to have the heavy lift engineer check the structure has to do with the setup of the SPMT system. The common understanding of how SPMTs work is that there is equal load on all wheels. While this is essentially true, there is some variation in the system.

SPMT moves have the additional complexity of the support system and the hydraulic grouping. On simple-span well-balanced bridges with limited skew the hydraulic grouping has little impact on the structure response. On multi-span bridges or bridges with unbalanced load or reactions the hydraulic grouping can redistribute the reactions and cause distortion in the system.

The preferred method is to have three equal zones that form a hydraulic tri-pod. Each zone is interconnected through hydraulic lines and controlled by the operator. It is common knowledge that a tri-pod is quite stable and determinant.

It is possible to have more than three zones; however, the indeterminacy of the system makes control of it more difficult. In these cases, the operators may need to stop and re-adjust the system on a regular basis during the bridge move. The key to keeping all axles load-equal, is to place the centroid of the bridge at the centroid of the system zones. This can be calculated by determining the centroid of each axle group, and the finding drawing a triangle using the centroid of the groups. The centroid of the axle groups should coincide with the centroid of the structure being lifted. Figure 11.4.6.2-1 shows a potential SPMT setup with three zones. Note that the centroid of all of the zones is shifted to account for the fact that the yellow zone has twice as many axles.

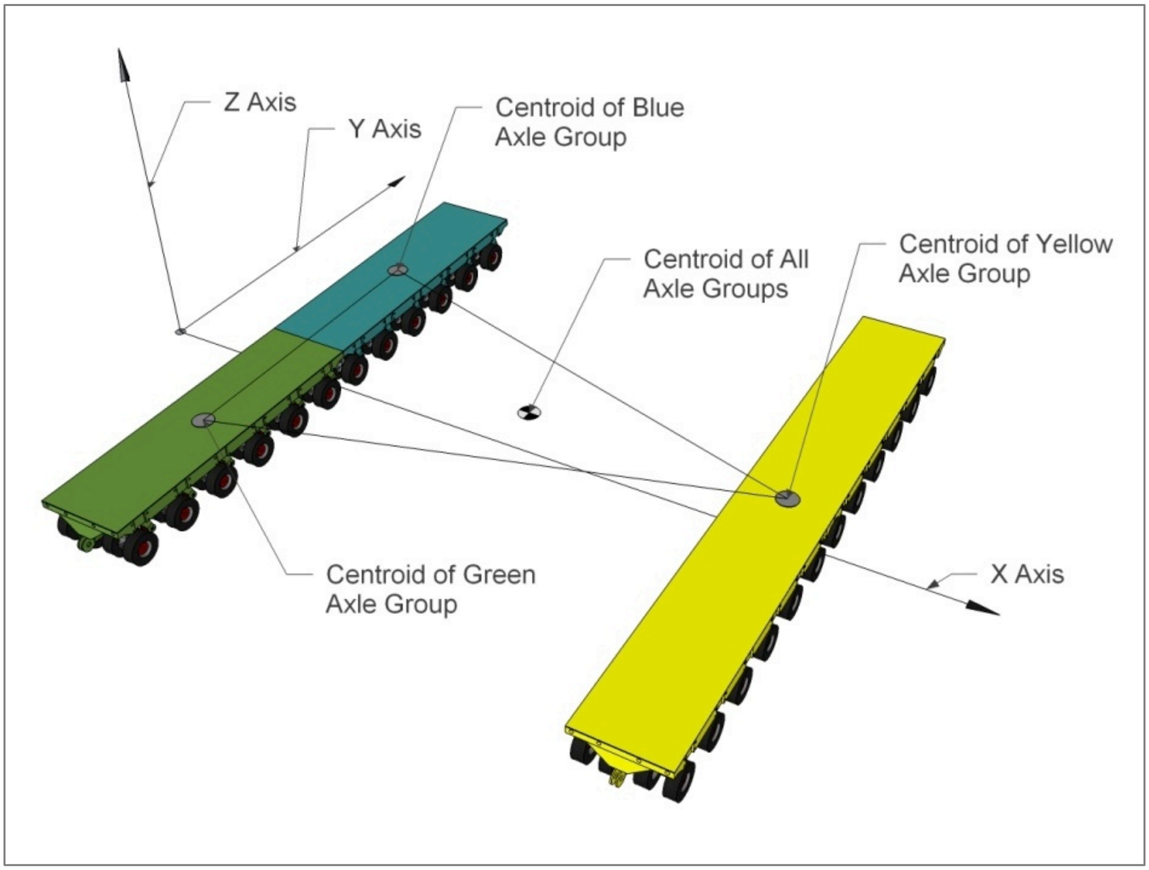


Figure 11.4.6.2-1. Potential SPMT Hydraulic Zone Configuration for Single-Span Bridge Move.

The design engineer does not have knowledge of the final setup of the system in a DBB scenario; therefore, detailed twist and lifting calculations cannot be done during the design phase of the project. The design engineer should be knowledgeable in these systems so a proper review of the lifting procedure can be completed. The following information describes the way that twist calculations are executed with SPMT systems.

To understand the origin of twist in bridge moves we can evaluate Figure 11.4.6.2-2 in response to unbalanced loads or lateral loads. The section drawings indicate pinned support conditions under the SPMT groups. An analogy to the SPMT support is a canoe floating in water. Off-center loads will cause a group of axles to rotate as the center of gravity of the load is shifted.

Another important aspect of SPMT stability is to understand that the individual transporters do not have stability about their longitudinal axis (local Y axis of each transporter shown in Figure 11.4.6.2-1). This inherent lack of stability will mandate that significant bracing be used between the transporters to keep them from rolling over (again, like a canoe in the water). The bracing can be installed using several methods. A still beam can be run transversely on top of the transporters, X bracing can be run between the towers, and Y bracing can be run up to the superstructure. All of these options are acceptable and can provide a safe and stable structure.

Heavy lift engineers should evaluate both the global stability of the lift system as well as the local stability of each transporter as part of their heavy lift calculation process. Project specifications should require these calculations as part of the heavy lift submission.

For the example detail in Figure 11.4.6.2-2, consider the system response to an off-center vertical load (Y direction) at the deck level. This load will cause a moment on the system that will be resisted by a change in reaction of the hydraulic groups. The axial load in the blue group will increase, the axial loads in the green group will decrease, and the axial load in the yellow group will remain unchanged but the entire support under the yellow group will rotate about the pin under the yellow group. The rotation is required to redistribute the vertical loads in the blue and green groups.

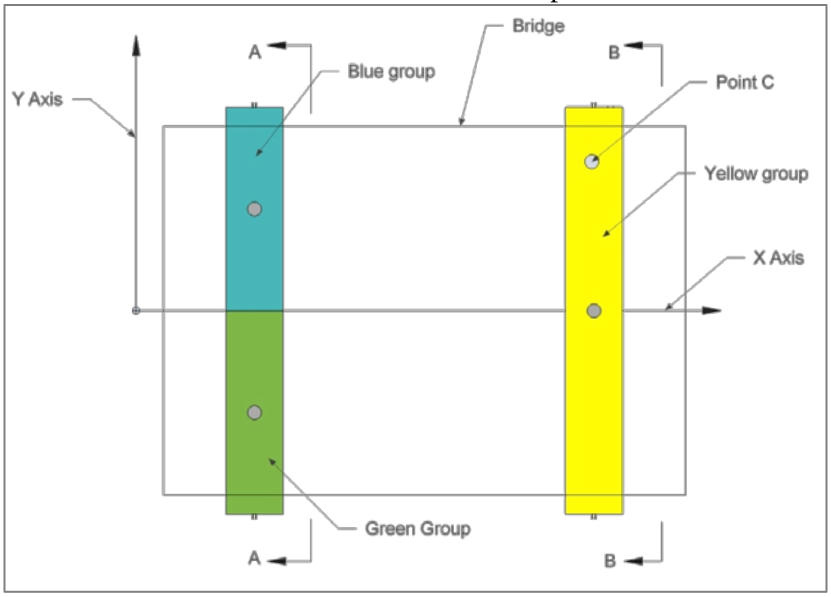
Now consider an unbalanced point load at Point C. Since the yellow group is a single hydraulic group, the bridge will rotate about the pinned support until the system reaches equilibrium. The system reaches equilibrium by increasing the load transferred through the girders above the yellow group on the opposite side of the Point C load. This is accomplished by a twist in the bridge. The magnitude of the twist is a function of the magnitude of the unbalanced load and the torsional stiffness of the structure.

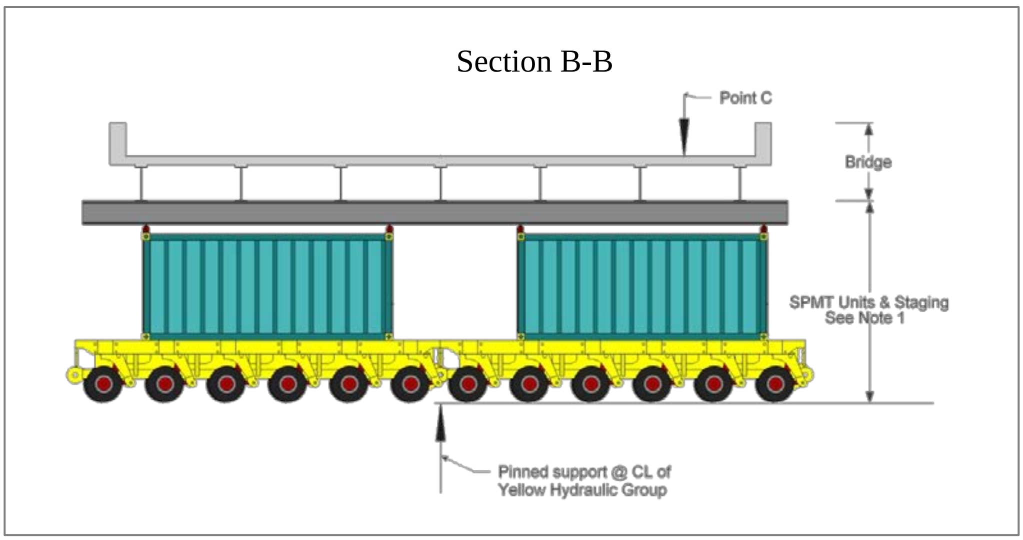
Only one hydraulic grouping scenario is shown. There are other possible configurations that can also be used. This example demonstrates how the analysis of the bridge for twist can only be done once the hydraulic systems are established by the heavy lift contractor.

The designer must supply the anticipated center of gravity (CG) of the structure so the SPMT supplier can plan the hydraulic groupings and layout of the SPMTs. By adjusting the number and location of the wheels in the hydraulic groups, the SPMT supplier can compensate for any unbalanced loads.

It is important to understand that the pinned support located at the center of the hydraulic grouping is critical in determining the response of the structure during the move.

Plan: SPMT Groups





Note: SPMT units and supports can be considered rigid for bending about the X axis.

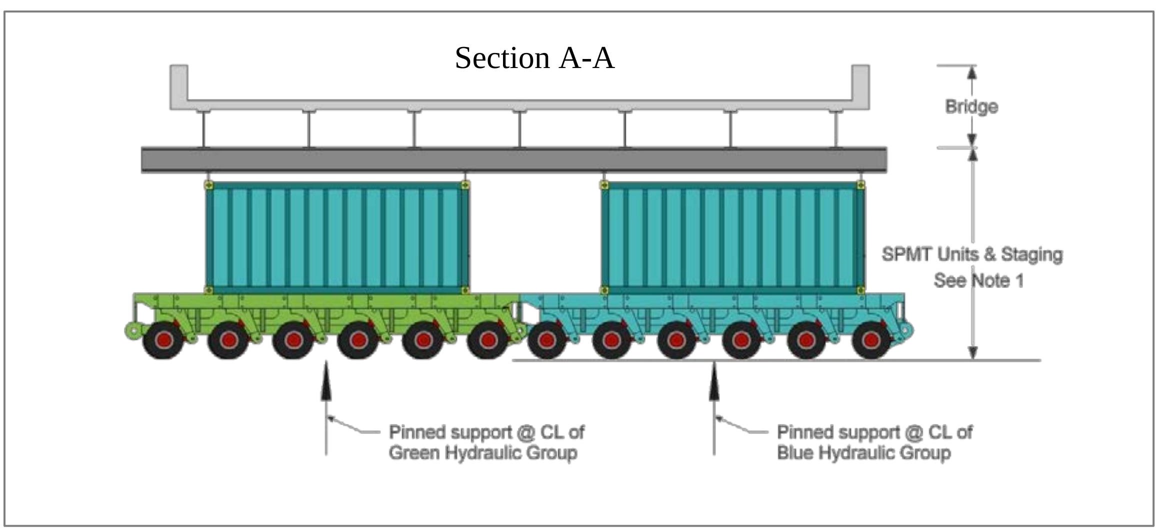


Figure 11.4.6.2-2. Example SPMT Layout for a Single Span Bridge.

As with the SPMT setup, the design of the falsework is not completed by the design engineer. Typically, the SPMT supplier or mover will design all the supports between the bridge and the SPMT. The designer should specify the minimum required bearing areas. Designers reviewing the falsework should consider the potential for differential deflection within the falsework framing.

If the falsework is flexible, as described in Section 11.4.6.1, the modeling and analysis of the falsework should also include a model of the entire bridge. The flexibility of the falsework will impact variable forces in the superstructure and falsework as the system flexes. It is important to capture this behavior and the inevitable redistribution of forces through the entire system.

Specifications should include language that requires these types of analysis. Specifications should also require that the contractor develop a twist monitoring system. These systems can get quite elaborate, or they can be simple. String lines crossing diagonally across the bridge can be a useful simple measure of differential corner movement.

11.4.7. Design of Decks and Barriers for Temporary Effects During Moves

When the bridge is lifted away from the final supports, negative bending moments can develop in the superstructure, especially at the ends of the bridge. The negative bending moments may crack the deck and barriers. The design engineer needs to determine a reasonable location for the SPMTs and check the superstructure for these negative moments. The design of the beam and deck can be checked according to AASHTO Article 6.10.1.7. There is potential for the deck to crack under this condition; however, experience has shown that once the superstructure is set, the width of the cracks close to almost zero. The same approach can be used for the concrete barrier. Designers may choose to place more closely spaced contraction joints in the concrete barriers in the regions that will experience temporary negative bending moments.

Bridges built with pre-tensioned prestressed concrete beams are somewhat vulnerable to large negative bending moments near the beam ends. This is due to the fact that the beams typically have internal tension in the top of the beam brought on by the release of the prestress force. The design of these cantilevers is similar to the design of prestressed beams that are made continuous for live load. The beams should be checked and detailed to resist these temporary negative moments. The composite deck also can be used to help resist the moments.

If the temporary negative bending moments in the superstructure become excessive, the designer can use several different methods to reduce the negative bending moments during SPMT bridge moves:

- Move the SPMT pick points as close to the ends of the bridge deck as possible. Experience has shown that placement of the SPMT within 15 percent of the beam ends will keep the negative moments within a reasonable level.

- Support the bridge at the SPMT pick points during deck casting. See Section 11.4.3 for more information on this method of construction.

- Use longitudinal post-tensioning in the concrete deck to counteract the negative bending moments. There is no reason why precast concrete full-depth deck panels cannot be used even if the bridge is built off-line. This will provide a highly durable crack-free deck that is pre-compressed.

- Use lightweight concrete in the bridge deck. This will reduce the negative moments in the cantilever regions without sacrificing strength. Utah DOT had used this approach on one SPMT bridge move. The resulting deck had virtually no cracking caused by the move.

- Stop the casting of the deck short of the beam ends in order to reduce the negative bending moments in the beams. The trade-off with this option is that a large closure pour will be required once the bridge is set. This may not be a significant problem since the pour will most likely not be over live traffic. This option is also attractive for bridges that do not require immediate opening of the roadway on the bridge.

11.4.8. Launching of Bridge Superstructures

Longitudinal launching of superstructures is not common, but can be a form of accelerated bridge construction for bridges over busy waterways or roadways. It is possible to launch very large structures, including multi-span bridges. There are a variety of methods in use, but the one commonality is that the bridge superstructure is built behind one of the abutments. It is typically built in a shallow pit that places the girders close in elevation to the final position. The bridge can be built at a higher elevation, but this will make the launch and the subsequent lowering more difficult.

In most cases, the launching is done on top of the final substructure elements, using them as support for the launched bridge. In some cases, temporary bents can be used to control the launching stresses. Another method of launching is to use a barge (for waterways) or SPMTs (for roadways) to support the leading edge of the launched structure.

Launching of completed or partially completed structures requires significant engineering and planning. Designers should coordinate with the owner to set stress limits during the launch operation (see Section 11.4.5 for a discussion on stresses during bridge moves). There are two general types of bridge launches:

1. The structure is progressed longitudinally in a cantilevered fashion with sliding surfaces or rollers placed at stationary supports. Geometry control is critical in longitudinal launches. The system supports the need to have the ability to adjust vertically and horizontally in order to accommodate features such as vertical curves and potential out-of-alignment tracking.

2. The support system for the launch can be designed to be rigidly attached to the bridge at a single location and the support moves with the bridge. The support can be a supplemental sliding beam where the bridge support slides along the supplemental beam, or the support could be an SPMT-style trailer that carries the bridge across the gap. For water crossings, the support can be a barge.

The process of designing a bridge for a longitudinal launch can be different than the design of a typical bridge. The forces induced in the structure during the launch can be significant and may even control the design. The most significant forces in a launch operation are typically in the leading span in the front of the launch. This span will experience significant negative bending moments as it is cantilevered across the spans.

If the bridge is a multiple-span structure, the subsequent spans will typically see lower bending moments; however, there most likely will be moment reversals during the launch. Many launches of steel bridges have been done without the deck in place, so as not to crack the deck during the launch. Post-tensioned concrete structures have been launched as a whole, with the post-tensioning used to resist the launching stresses.

There are several ways to launch a bridge without causing damage:

- Steel Bridge: Launch the bridge without a concrete deck in the first span. Design the steel to accommodate the launching forces, including web crippling along the bottom flange and web buckling at the push locations.
- Add a launching nose to the end of the bridge. This is usually a lightweight steel frame that is bolted to the end of the bridge superstructure. The length of the launching nose is designed to reduce the bending moments in the first span during the launch.
- Add a strong-back system to the first span. This consists of a frame built over the first span. The frame provides added strength and stiffness to the front span during the launch.
- Install temporary bents between the piers to reduce the forces in the leading span during the launch.

The design of a bridge superstructure for a longitudinal launch needs to be carefully coordinated with the launching methods. For this reason, launching is probably best suited for DB and CM/GC projects. The interaction between the designers and the contractor is key to the success of this process.

Potential design changes that may need to be considered include:

- Design the web of steel girders to resist the rolling stresses and web crippling at the piers. This may require a thicker web plate, or the addition of more web stiffeners.

- Care should be taken with the detailing of the bottom flange plate thickness changes and the design of bolted splices. The accommodation of flange thickness changes and the potential interference of bolts need to be coordinated with the heavy lift contractor.

- The webs of concrete girders should be checked of the rolling stresses. The webs of large box girders may need to be thickened to accommodate these stresses.

- The profile geometry of the bridge needs to be accounted for in the design. Significant vertical curves can lead to geometry problems with the launching systems. This may require the use of adjustable rolling devices.

CHAPTER 12. DESIGN EXAMPLES

The approach for most PBES designs is to emulate CIP concrete with prefabricated elements. In a true emulative design, the element designs should essentially be the same as with a non-prefabricated design. The variation in design is normally limited to the connections between the elements.

Figure 12-1 shows a precast concrete hammerhead pier detail that was developed by the PCI Northeast Bridge Technical Committee. This is for a pier supported on a spread footing. The major connections make use of grouted reinforcing couplers that provide an emulative design. The following items are the major design aspects:

- The cap design is exactly the same as CIP construction.
- The reinforcing cage for columns needs to be embedded deeper into the column in order to provide cover over the couplers (see Figure 5.2.3-1 for an example of this design change).
- The footing design shows partial precast. The connection between the precast and the CIP concrete is simply a construction joint; therefore, no design change is required.

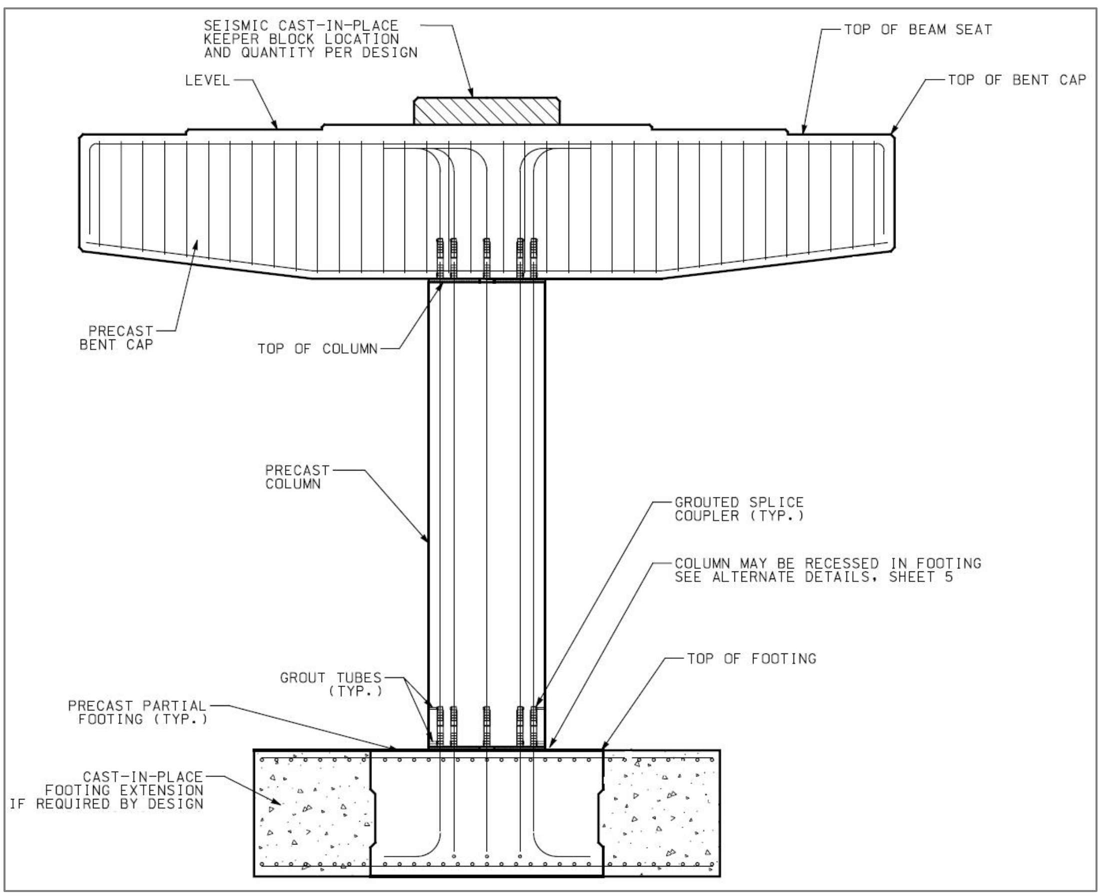


Figure 12-1. Precast Concrete Hammerhead Pier (Source: PCI Northeast).

This is one example of how emulative design can be used to simplify the design of prefabricated elements. The following sections will contain more information on specific structure types and how the design needs to be modified in order to account for prefabrication.

12.1. REDESIGN OF EXISTING LRFD DESIGN EXAMPLES USING PBES

In an effort not to duplicate previous work, this manual will make use of previously published LRFD design examples that are posted on the FHWA website at:
http://www.fhwa.dot.gov/bridge/lrfd/examples.htm.

This website contains two complete bridge designs that were developed in 2003. The AASHTO LRFD Bridge Design Specifications [5] have changed since 2003, but the general design principles have remained the same. The major features of the two examples include:

- Comprehensive Design Example for Prestressed Concrete (PSC) Girder Superstructure Bridge with Commentary (in U.S. customary units) [74].
- CIP concrete deck.
- Concrete parapets.
- Two-span prestressed concrete girders (AASHTO Type VI).
- Integral abutments with integral wingwalls.
- Four-column concrete open frame pier bent.

- LRFD Design Example for Steel Girder Superstructure Bridge with Commentary [75].
- CIP concrete deck.
- Concrete parapets.
- Two-span continuous steel plate girders.
- Cantilever abutments and wingwalls.
- Single-column pier bent (hammerhead).

The variation from conventional design that is brought about by the incorporation of prefabrication will be covered in this chapter.

12.1.1. Bridge Deck

Section 4.2.6 contains information on the design requirements for precast bridge decks. The PCI manual entitled "State of the Art Report on Full-Depth Precast Concrete Bridge Deck Panels" [27] contains a fully worked design example for a precast deck.

12.1.2. Integral Abutment

The AASHTO-LRFD Bridge Design Specifications [5] does not contain specific detailed design criteria for integral abutments. The one section in the specifications that does cover integral abutment design (Section 11.6.1.3) states:
"Maximum span lengths, design considerations, details should comply with recommendations outlined in FHWA Technical Advisory T 5140.13 (1980), except where substantial local experience indicates otherwise."

Unfortunately, this technical advisory no longer exists. Recently, FHWA has published a document entitled “Steel Bridge Design Handbook” [76]. This document has a section that contains some information on the design of integral abutments. A number of states have also developed specific design criteria and guidelines based on general engineering principles and experience.

In general, the design of integral abutments is based on several principles:

- The superstructure is assumed to act as a simple span. Frame analysis of the system is not normally done.
- Thermal movement is accommodated by flexure of the piles. This movement can induce passive soil pressure on the faces of the abutment stems.
- Longitudinal forces in the superstructure are resisted by passive soil pressure behind the abutment.
- Transverse superstructure forces are resisted by bending in the piles.

There are several details that are being used throughout the U.S. for precast concrete integral abutments. Aside from two features, the design of the abutments and stems is the same as with a CIP abutment. The features that vary are included in the following calculations.

The following pages contain revisions to the design of the integral abutment in the FHWA LRFD Design Example: Comprehensive Design Example for Prestressed Concrete (PSC) Girder Superstructure Bridge with Commentary (in US Customary Units) [74].

Example 12.1.2. Concrete Integral Abutment Redesign.

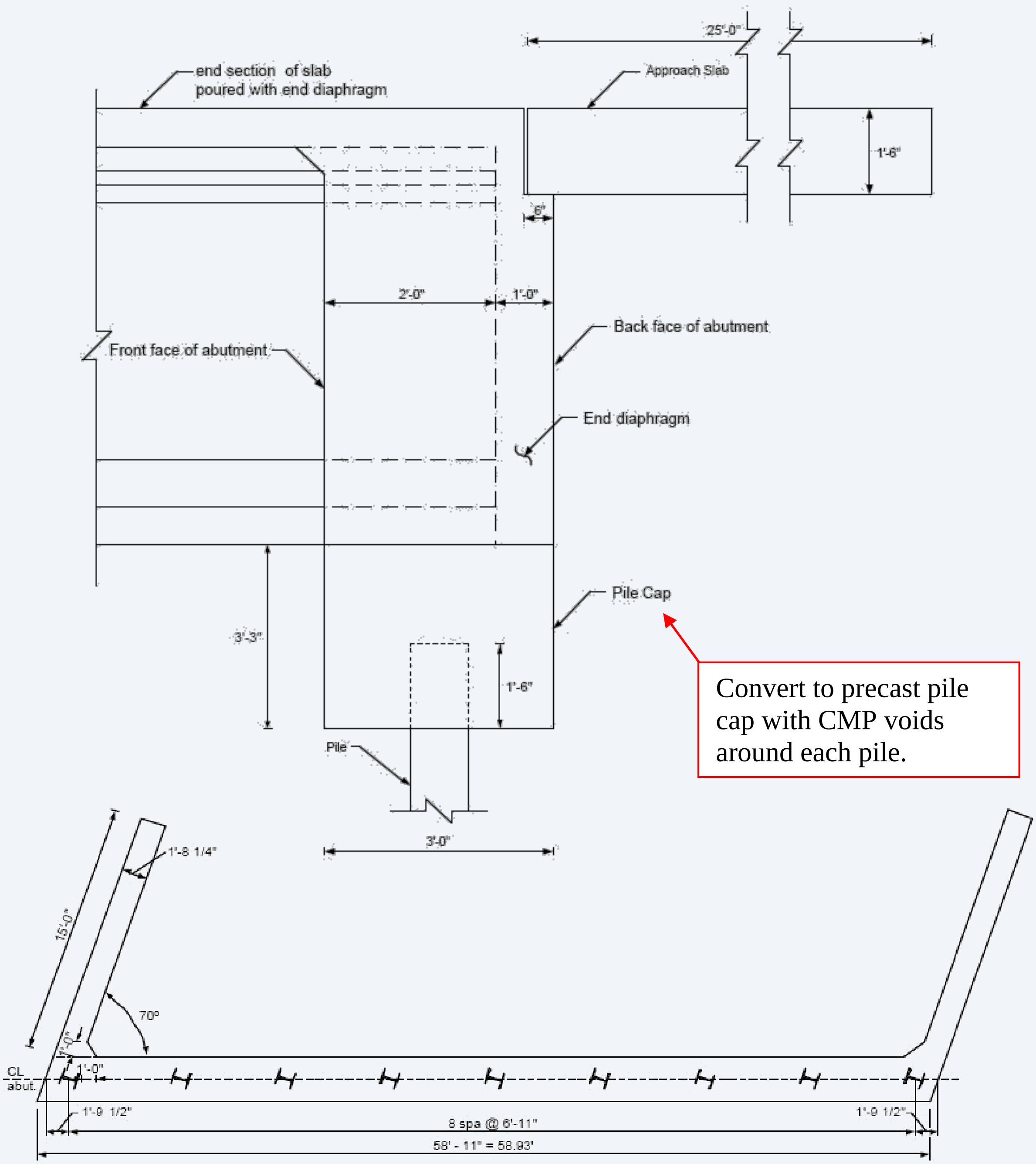
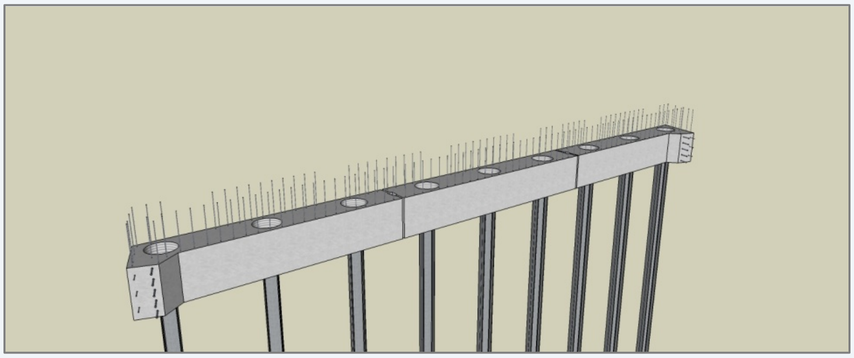


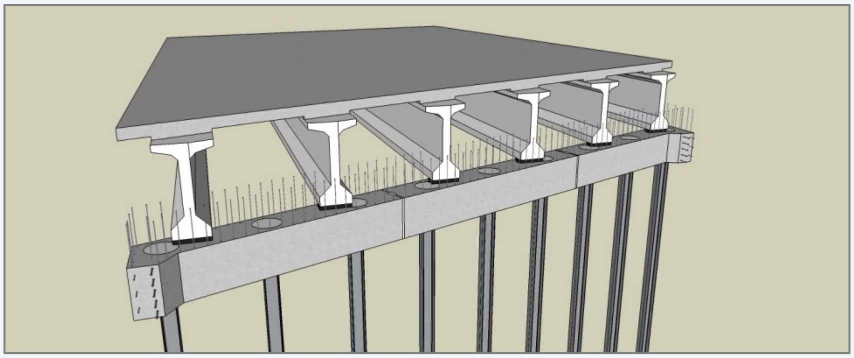
Figure 12.1.2-1. Details of Example Integral Abutment.
Top: Typical Section.
Bottom: Plan.

The following figures show the proposed construction sequence:

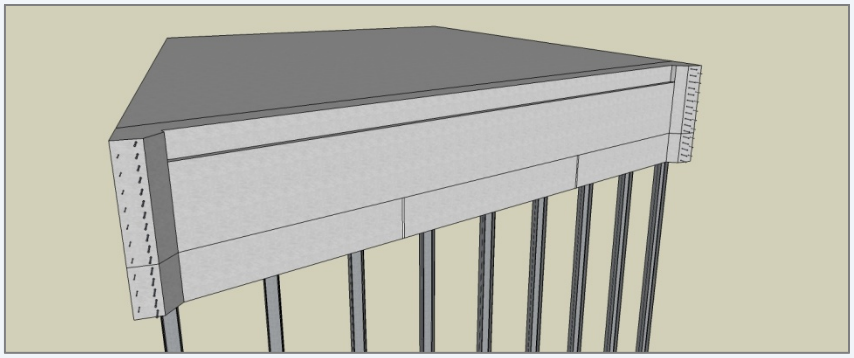
Step 1:
Install piles
Set precast cap elements



Step 2:
Set Girders
Cast Deck
Note: Barrier Reinforcement not shown



Step 3:
Cast integral diaphragm



Step 4:
Install precast wingwalls

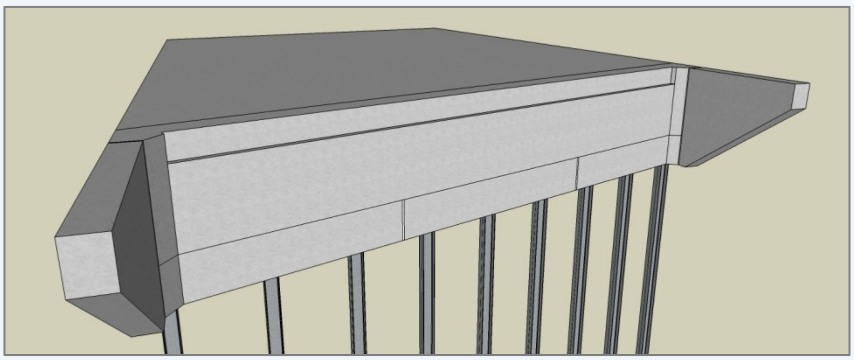


Figure 12.1.2-2. Proposed Precast Integral Abutment Details.

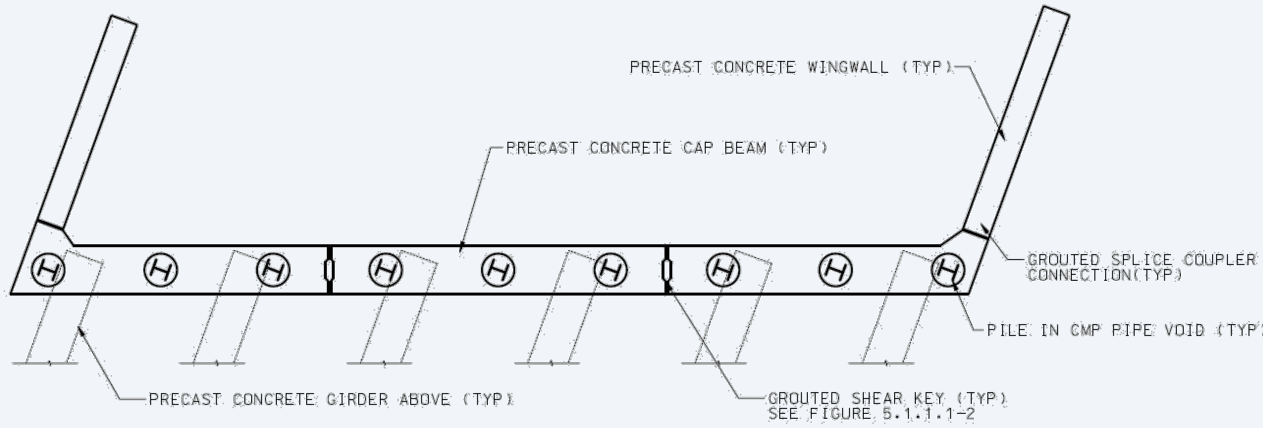


Figure 12.1.2-3. Proposed Precast Integral Abutment Pile Cap (stem).

Step 1: Calculate the minimum CMP pipe size.

The development of the use of corrugated metal pipe (CMP) voids was based on research for precast integral abutments [36]. Section 5.1.1.4 contains more information on the results of this research. That research concluded that the CMP pipe had no effect on the design and detailing of the cap reinforcement. Therefore, the only change to the design of a precast integral abutment cap would be the interface shear (shear friction) between the CMP and the adjacent concrete.

Given: Refer to Design Step 7.1.3 of the reference document [74]

Width of cap = 36 inches

Size of pile = HP12x53

Depth = 11.78 inches

Width = 12.05 inches

Assumptions:

Pile Installation Tolerance = $\pm$ 3 inches total in any direction

(assume driving template is used)

Minimum Clear Distance between pile corner and void = 0.5 inches

Minimum distance from face of stem to edge of pile = 5.125 inches

(3-inch cover + #5 + #6 + 0.75 inches gap = 5.125 inches)

CMP Corrugation amplitude = 0.75 inches

Effective pile diameter = Dpile $= \sqrt{11.78^2 + 12.05^2} = 16.85\ inches$

Pile driving tolerance = $Tpile = 3\ inches$

Clear cover from corner of pile to side of CMP = *Dclear = .5 inches*

Therefore: CMP ID Pipe size = 16.85 inches +2(3 inches) + 2*.5 = 23.85 inches
Round to next largest 3-inch increment (pipe standard), use D = 24 inches

Distance from outside edge of CMP to face of cap = CMP_{COV}

CMP_{COV} = (36-inch stem – (24-inch CMP + 2*0.75 inches amplitude) / 2
= 5.25 inches $\geq$ 5.125 inches **OK**

Note: *The assumptions for clear distances were chosen as small values in order to fit the CMP within the 36-inch wide stem section. If larger tolerances were desired, the CMP would need to be larger and the stem width would need to be increased in order to accommodate the void. This would necessitate a redesign of the stem and piles.*

Figure 12.1.2-4 shows the details of the final CMP void.

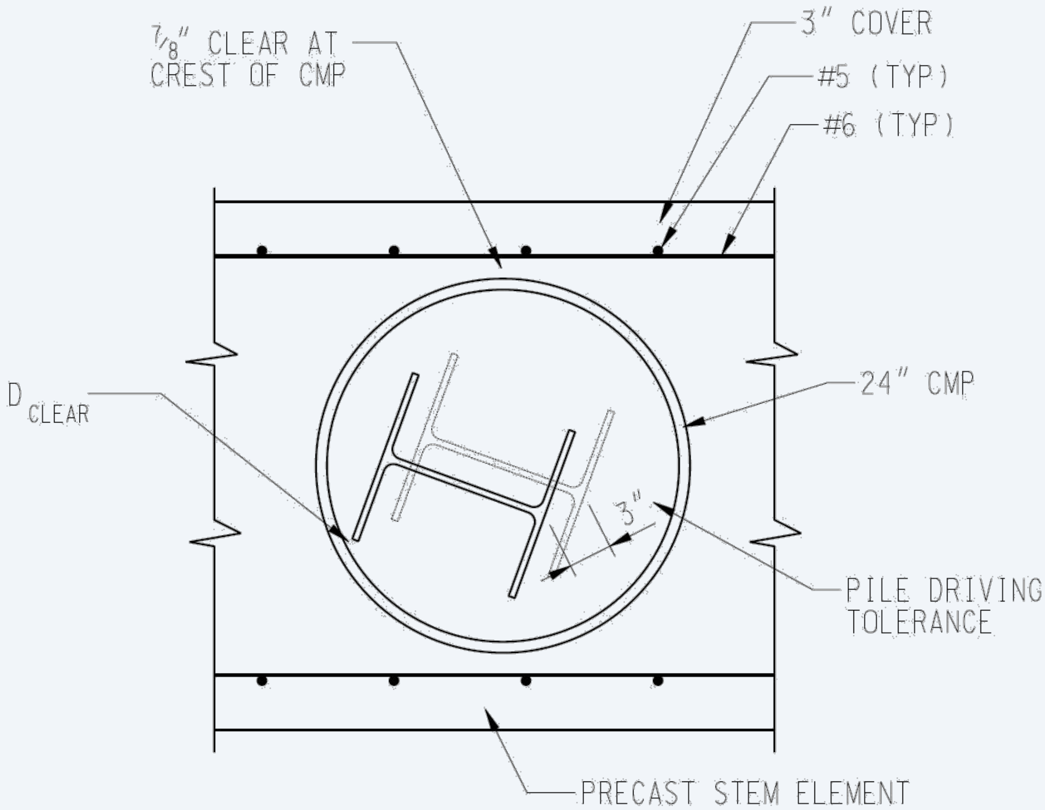


Figure 12.1.2-4. Detail of CMP Void and Pile Tolerance.

Step 2: Calculate the punching shear capacity of CMP void.

The pile cap is designed to support the girders, deck, haunches, and abutment diaphragm. In the FHWA design example, this is referred to as "Stage 1." The abutment end diaphragm is designed to support all other loads and is referred to as "Stage 2."

Punching shear design is based on AASHTO Article 5.13.2.5.4

Given:

$f'c = 4$ ksi (assumed strength of void concrete)

Circumference of pipe = 24 inches * π = 75.4 inches

Length of pipe above pile top = 39 inches – 18 inches = 21 inches
(Figure 7.1-1 [74])

Punching Shear area per linear foot of pipe
Acv = 75.4 inches(12 in./ft) = 905 inches^2

Nominal Punching Shear Resistance

$Vn = 0.125(f'c)^{1/2}$ Acv
$= 0.125(4)^{1/2}(905)$
= 226 kips per linear ft of pipe

Nominal Resistance of CMP void for this pile cap
= 226k/ft * 21 inches/(12 inches/ft) = 396 kips

The actual load on the CMP void is the dead load of the beams, deck, and abutment diaphragm. The diaphragm can help to resist other loads that are applied to the structure after the diaphragm is cast.

The factored maximum total girder reactions for Stage 1

PSI (TOTAL) = 2(147.4) + 4(158) = 926.8 kips
(Design step 7.1.3.2 [74])

Stage 1 load per pile = 926.8/9 piles = 103 kips per pile

Check capacity of CMP Void

Vri = ΦVni

And the design shall satisfy:
Vri ≥ Vui
Vui = factored shear force = 103 kips
Vri = ΦVni = 0.90*396 kips = 356 kips
356 kips ≥ 103 kips

Therefore, the CMP void shear resistance is adequate.

Step 3: Design of Stem Reinforcement.

The intent is to use a precast concrete pile cap combined with precast concrete wingwalls. Figures 12.1.2-2 and 12.1.2-3 show the intended change from CIP concrete to precast concrete for the abutment pile cap. The only significant change from the CIP option is the use of two grouted shear keys. The CIP pile cap was designed as a continuous beam. The precast cap is similar to a continuous beam; however, the shear keys create a hinge in the cap. Figure 12.1.2-5 shows two free body diagrams for the cap element. The roller supports represent the piles and the beam represents the cap.

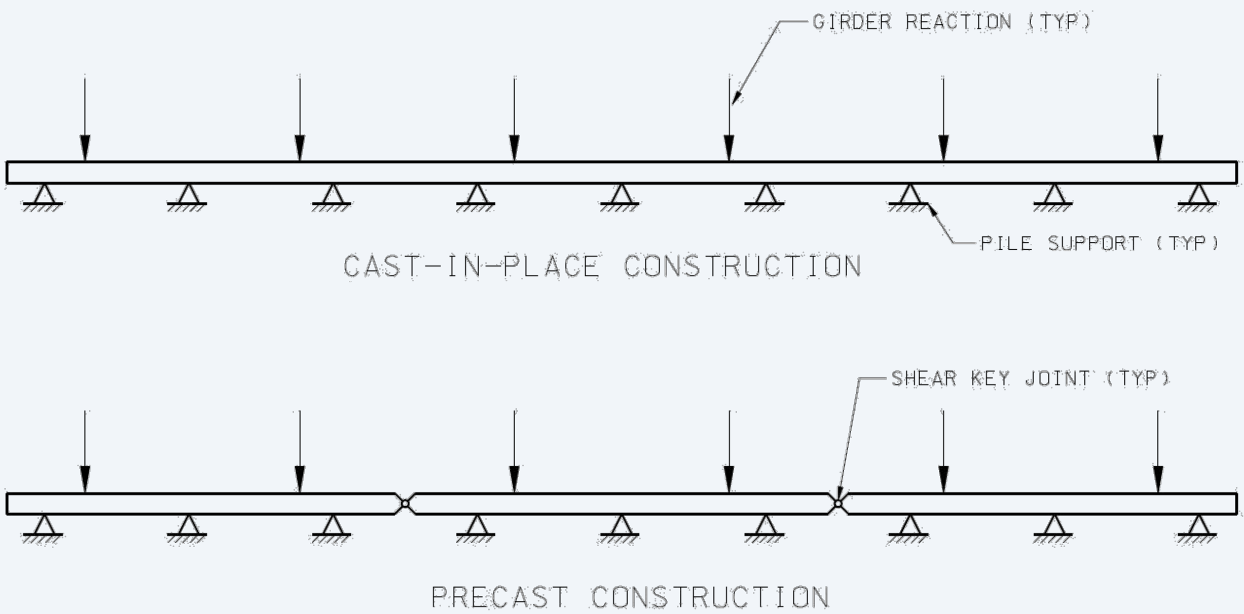


Figure 12.1.2-5. Free Body Diagrams for Pile Cap.
Top: CIP construction.
Bottom: Precast construction based on the details shown in Figures 12.1.2-2 and 12.1.2-3.

The FHWA design example treated the cap as simple spans. The resulting answer for positive moment bending caused by vertical forces was multiplied by 0.8 to account for continuity. The same moment was applied to the negative moment section. A continuous beam design would give more accurate results.

In this redesign, the positive moment will be calculated assuming simple span action, which is conservative. The following is a recalculation of the cap beam positive moment due to vertical loads. The redesign of negative flexure can be recalculated based on the hinge located mid-way between the piles. AASHTO Section 5.6.3 recommends that a strut and tie model be used for the design of a pile cap. This can be accomplished using the free body diagram shown above.

Positive Flexural design due to vertical forces
(Refer to page 7-17 of the FHWA design example [74].)

Assume the beam is located mid-way between the piles.
Do not use 0.8 factor to adjust for continuity due to the presence of the shear keys.

$$Mu = Pul/4 + wul^2/8$$

l = spacing of piles = 6.917 ft

$$Mu = 189.6(6.917)/4 + 6.96(6.917)^2/8$$

$$= 369.5 \text{ k-ft}$$

Resisting moment of the cap section

Mr = 490 k-ft > Mu = 369.5 k-ft **OK**

Negative Flexural design due to vertical forces near shear key
(Refer to page 7-17 of the FHWA design example [74].)

$$Mu = wu\,(l/2)^2/2$$

Cantilever beam with span of l/2
There are no beam reactions in this region

l = spacing of piles

l/2 = 6.917/2 = 3.46 ft

$$Mu = 6.96(3.46)^2/2$$

= 41.67 k-ft **OK by inspection**

Shear design check:
The shear design equation for the FHWA LRFD design example is based on the beam being located directly adjacent to a pile. This is still the case with the precast option chosen; therefore, the cap design for shear is adequate for the precast option.

Step 4: Design of Integral Abutment Diaphragm.

The FHWA LRFD design uses a built-up composite concrete beam that is made up of the pile cap and the integral diaphragm. The use of the precast cap with the shear key leads to a discontinuity of the longitudinal cap reinforcement at the key. Therefore, the design of the diaphragm will not account for the composite action of the pile cap.

Design of diaphragm for flexure:
The maximum positive moment is calculated assuming the girder reaction is applied at mid-span between piles taking 80 percent of the simple span moment.

Note: *This approach is conservative since it does not take into account the dead loads transferred into the cap beam below or the composite action between the cap beam and the diaphragm.*

Mu = 498.2 k-ft (See page 7-21 of the FHWA LRFD Design Example.)

Mn = As fy (ds – a/2)

Where:

As = use four #8 bars
= 4(0.79) = 3.16 inches^2

fy = 60 ksi

ds = depth of dia. – cover – stirrup – ½ bar dia.
= 84.75 inches – 3 inches – 0.625 – ½(1.0)
= 80.625 inches

a = As fy / 0.85 f'c b
= 3.16(60)/[0.85(3)(36)]
= 2.07 inches

Mn = 3.16(60)(80.625 – 2.07/2)/12
= 1257 k-ft

Therefore,
Mr = 0.9(1257)
= 1132 k-ft > Mu = 498.2 k-ft **OK**

By inspection:
Mr > 4/3(Mu). This means that the minimum reinforcement requirements of S5.7.3.3.2 are satisfied.

Check compression control resistance factor based on concrete strain (S5.5.4.2).

$\varepsilon = 0.003(d\text{-}c)/c$

d = 80.625 inches
c = a/0.85 = 2.44 inches

$\varepsilon = 0.003(80.625-2.44)/2.44$

= 0.0961 > 0.005 **OK** Tension Controlled Section $\Phi = 0.90$

Add four #8 bottom longitudinal reinforcing bars at the bottom of the diaphragm between the girders.

Shear design of diaphragm:
The shear area at the face of the pile is unchanged with the precast cap design. In the FHWA example, Vc was adequate for the shear design. The shear mid-way between the pile is reduced by 50percent due to beam action. By inspection of the FHWA design calculations, the diaphragm section is adequate to resist the shear at the section over the precast cap connection.

Step 5: Design of Backwall as a Horizontal Beam Resisting Passive Earth Pressure.

The resistance of the backwall can be conservatively checked assuming that the pile cap has no resistance to passive earth pressure. The pile cap will transmit the passive pressures into the diaphragm; therefore, the backwall moments can be applied to the section that is above the pile cap.

Check reinforcing in diaphragm for total backwall moments:
Mu = 300 k-ft/ft
(See page 7-28 of the FHWA LRFD Design Example.)

Mn = As fy (ds – a/2)

Where:
As = area of longitudinal reinforcement bars at front face (tension side) of the abutment diaphragm (seven #6 bars)
= 7(0.44)
= 3.08 inches^2

fy = 60 ksi

ds = width of backwall – cover – vert. bar dia. – ½ bar dia.
= 36 inches – 3 inches – 0.625 inches – ½ (0.75)
= 32.0 inches

a = As fy / 0.85 f'c b (b = height of diaphragm = 80.75 inches)
= 3.08(60)/[0.85(3)(80.75)]
= 0.897 inches

Mn = 3.08(60)(32 inches – 0.897 inches/2)/12
= 486 k-ft

Mr = 0.9(486)
= 437 k-ft > Mu = 300 k-ft **OK**

By inspection:

Mr > 4/3(Mu). This means that the minimum reinforcement requirements of S5.7.3.3.2 are satisfied.

Check compression control resistance factor based on concrete strain (S5.5.4.2):

$\varepsilon = 0.003(d-c)/c$

d = 32.0"
c = a/0.85 = 1.06 inches

$\varepsilon = 0.003(32.0-1.06)/1.06$

= 0.0876 > 0.005 **OK** Tension Controlled Section $\Phi = 0.90$

Therefore, the CIP diaphragm can resist passive earth pressure with existing reinforcement.

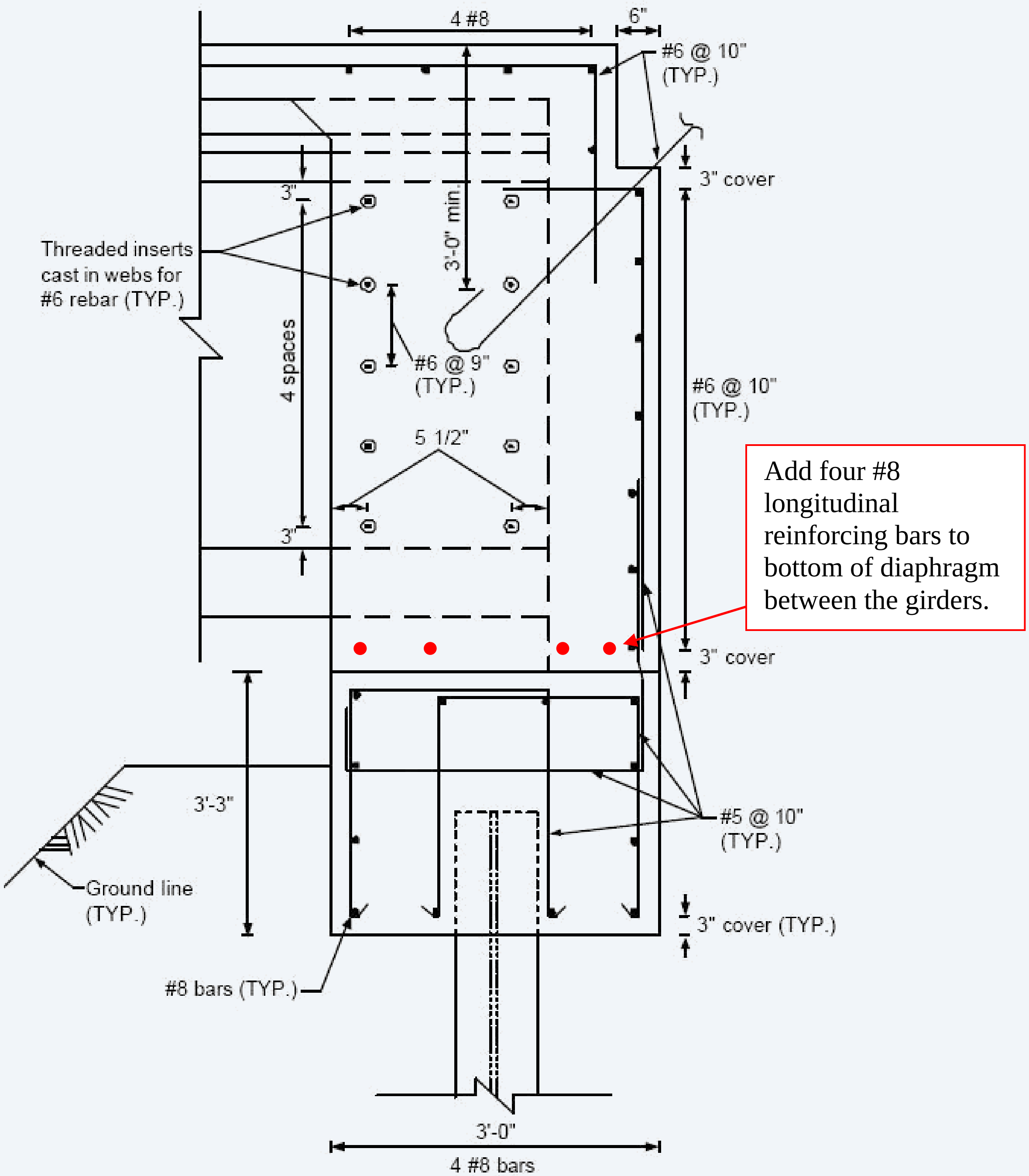


Figure 12.1.2-6. Changes to Integral Abutment
Based on Changing Pile Cap to Precast Concrete.

Step 6: Design of Precast Flying Wingwall.

The design of the wingwall is based on a cantilever section. The main reinforcing is run horizontal. One method of converting this wall to precast concrete is to detail a small closure pour. Reinforcing bars can be projected from the integral abutment backwall into a closure pour area. Reinforcing bars also can be projected from the precast wingwall into the same area. The design of connection is a simple lap splice. The width of the closure pour will be equal to the required lap splice plus an additional width for tolerance adjustment.

In this example, an attempt will be made to design the connection using grouted reinforcing splice couplers. See Section 5.2.3.1 of this manual for more information on grouted reinforcing splice couplers. Figure 12.1.2-6 shows the layout of the wingwall.

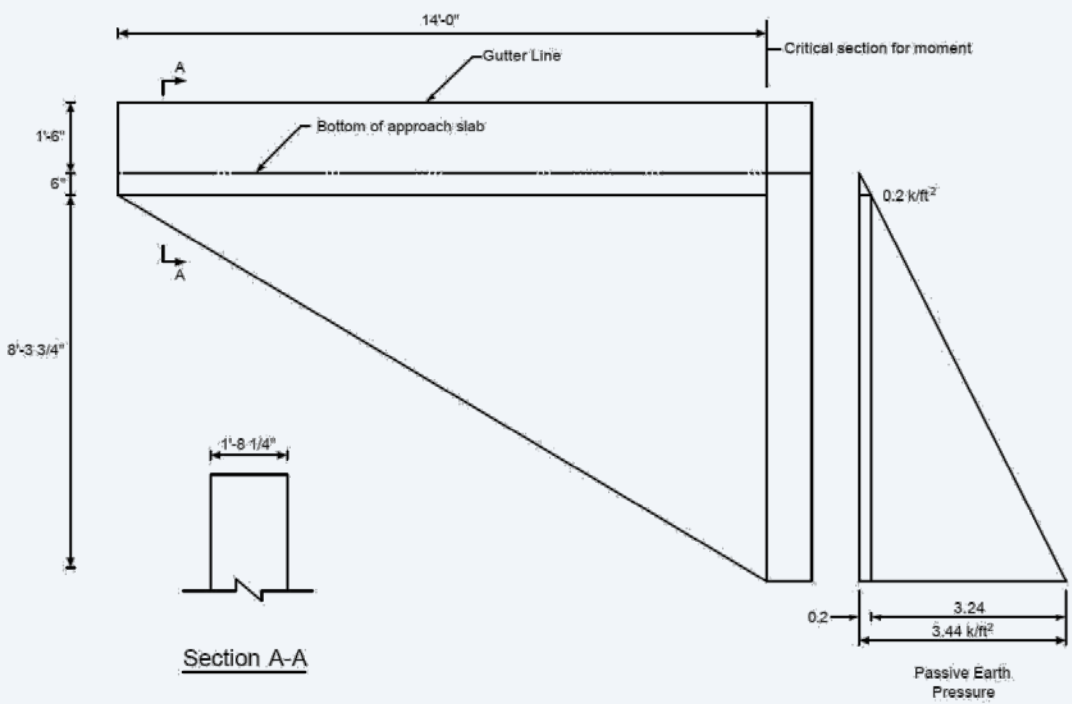


Figure 12.1.2-6. Flying Wingwall Layout (as designed CIP option).

Figure 12.1.2-7 shows the flying wingwall reinforcing as designed CIP option.

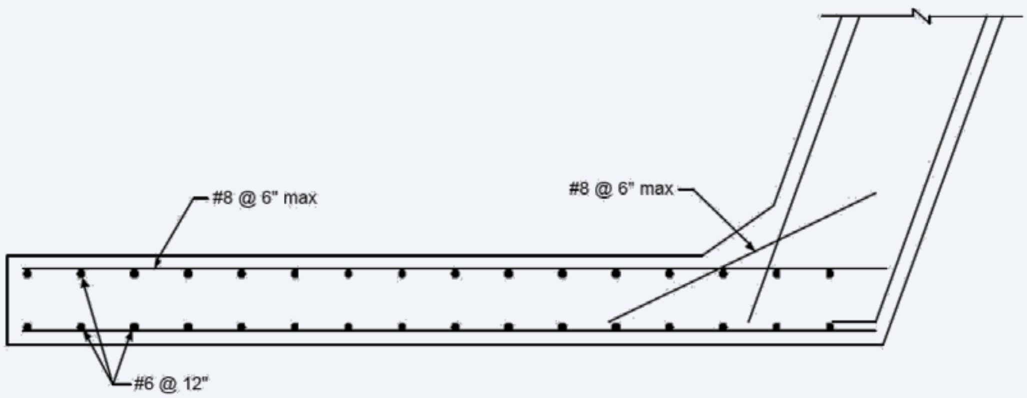


Figure 12.1.2-7. Flying Wingwall Reinforcing (as designed CIP option).

The controlling load case for the design of this wingwall is impact from a vehicle on the barrier.

Mn required = 1271.5 k-ft
(See page 7-32 of the FHWA LRFD Design Example [74].)

Redesign with grouted reinforcing splice couplers:
The couplers can fully develop the strength of the bars; therefore, no design is required for the coupler. The only design change is to account for the size of the coupler. The couplers need to be moved inward to provide adequate concrete cover over the coupler (see example calculation 5.2.3-1 in this manual).

The PCI Northeast Bridge Technical Committee has developed a coupler size chart that accounts for the size of commonly available couplers. The outside diameter of #8 and larger couplers is as follows:

#8	3.5 inches
#9	3.5 inches
#10	3.5 inches
#11	4.0 inches

Since the cost of couplers is a concern, it is recommended that larger couplers be used at larger spacing in lieu of smaller couplers at close spacing. Based on this, the redesign will make use of #10 couplers. The required spacing will be calculated for these bars based on the adjusted bar depth that accounts for the coupler diameter.

For a #10 coupler, the location of the adjusted ds would be:

ds = overall depth – cover – 1/2 coupler dia.
= 20.25 inches – 3 inches – ½(3.5 inches)
= 15.5 inches

Redesign of wingwall connection using grouted couplers:

Mu = 1271.5 k-ft/ft (Φ = 1.0 for Extreme Event)

Try 16 #10 @ 8 inches

Mn = As fy (ds – a/2)

Where:

As = area of longitudinal reinforcement bars at rear face (tension side) of the flying wingwall (16 #10 bars)
= 16(1.27)
= 20.32 inches^2

fy = 60 ksi

ds = 15.5 inches (see above)

a = As fy / 0.85 f'c b (b = height of wall = 123.75 inches)
= 20.32(60)/[0.85(3)(123.75)]
= 3.86 inches

Mn = 20.32(60)(15.5 inches – 3.86 inches/2)/12
= 1378 k-ft

Mr = 1.0(1378)
= 1378 k-ft > Mu = 1271.5 k-ft **OK**

Check Minimum Reinforcement (S5.7.3.3.2):

Mr $\geq$ lesser of 1.2(Mcr) or 4/3(Mu)

By inspection, Mr > 4/3(Mu), therefore check 1.2(Mcr):

Mcr = Sc fr (for non-prestressed, non-composite section)

Where:

Sc = Section modulus = 1/6(bh^2)
= 1/6[(123.75 inches)(20.25 inches)^2]
= 8458 inches^3

fr = 0.37(f'c)$^{0.5}$
= 0.641 ksi

$M_{cr} = 8458(0.641)/12$
$= 452$ k-ft

$1.2(M_{cr}) = 1.2(452)$
$= 542$ k-ft

$M_r = 1464 > 1.2(M_{cr}) = 542$ **OK**

Check compression control resistance factor based on concrete strain (S5.5.4.2):
$\varepsilon = 0.003(d-c)/c$

$d = 15.5$ inches
$c = a/0.85 = 4.54$ inches

$\varepsilon = 0.003(15.5-4.54)/4.54$

$= 0.007 > 0.005$ **OK** Tension Controlled Section

Therefore, use 16 #10 Bars spaced at 8 inches.

Outside face reinforcing is not by design (it is only meant for shrinkage). Use #6 spaced at 12 inches. Adjust bar location to provide cover over the coupler. The coupler diameter for a #6 bar is 3.0 inches; therefore, set the centerline of the bar at:

Location of outside face bar = cover + ½ coupler dia.
= 3 inches +1/2 (3.0 inches)
= 4.5 inches

12.1.3. Cantilever Abutment

In general, the change of a CIP cantilever abutment to precast concrete involves the connection of the wall stem to the footing. The use of a precast cap beam is also proposed. This is done to make the abutment stem elements similar and to provide a continuous beam seat.

The abutment is a standard cantilever abutment supported on spread footings. The proposed redesign involves the redesign of the following:

Element	Proposed lower connection type
Wall Stem	Connection to footing by placing the wall stem on temporary supports with projecting reinforcing dowels and casting the footing under the stem.
Bridge-seat/backwall element	Corrugated Metal Pipe void with reinforcing dowels

The following pages contain revisions to the design of the cantilever abutment pile connection in the FHWA LRFD Design Example: LRFD Design Example for Steel Girder Superstructure Bridge with Commentary [75].

Example 12.1.3. Concrete Cantilever Abutment Redesign.

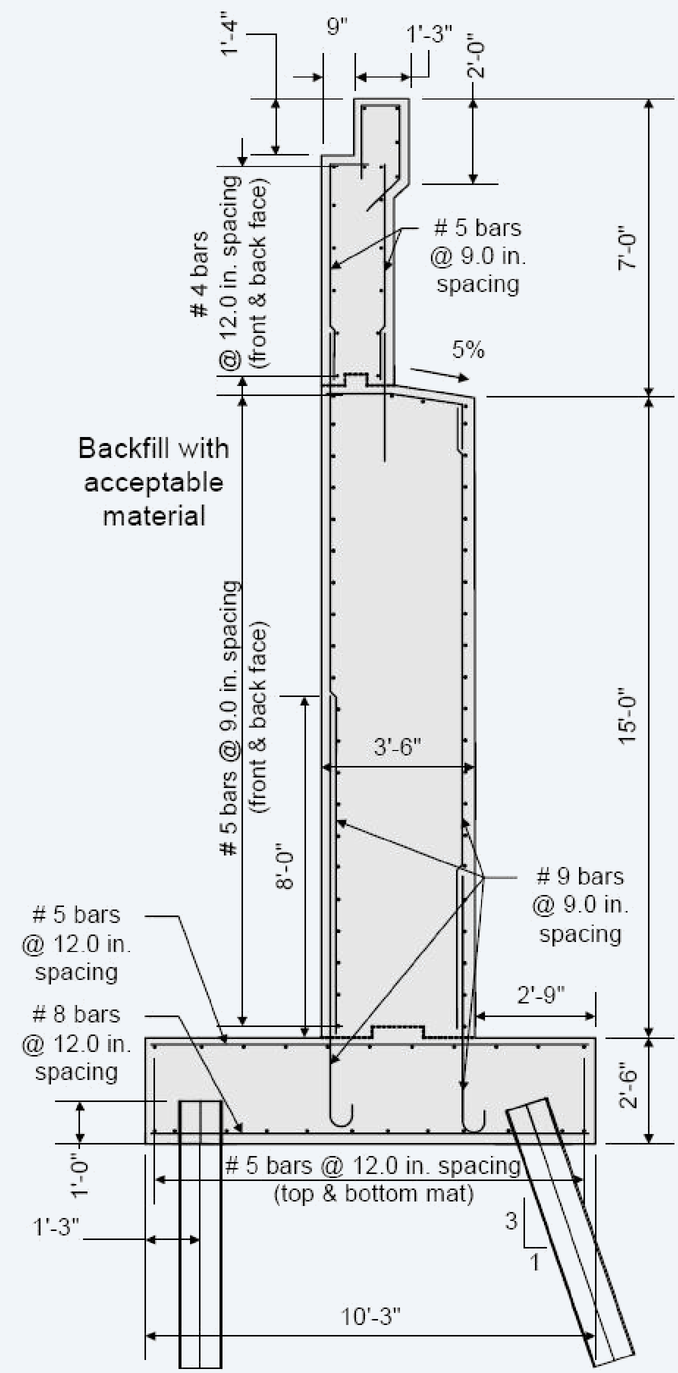


Figure 12.1.3-1. Example Cantilever Abutment (as designed CIP option).

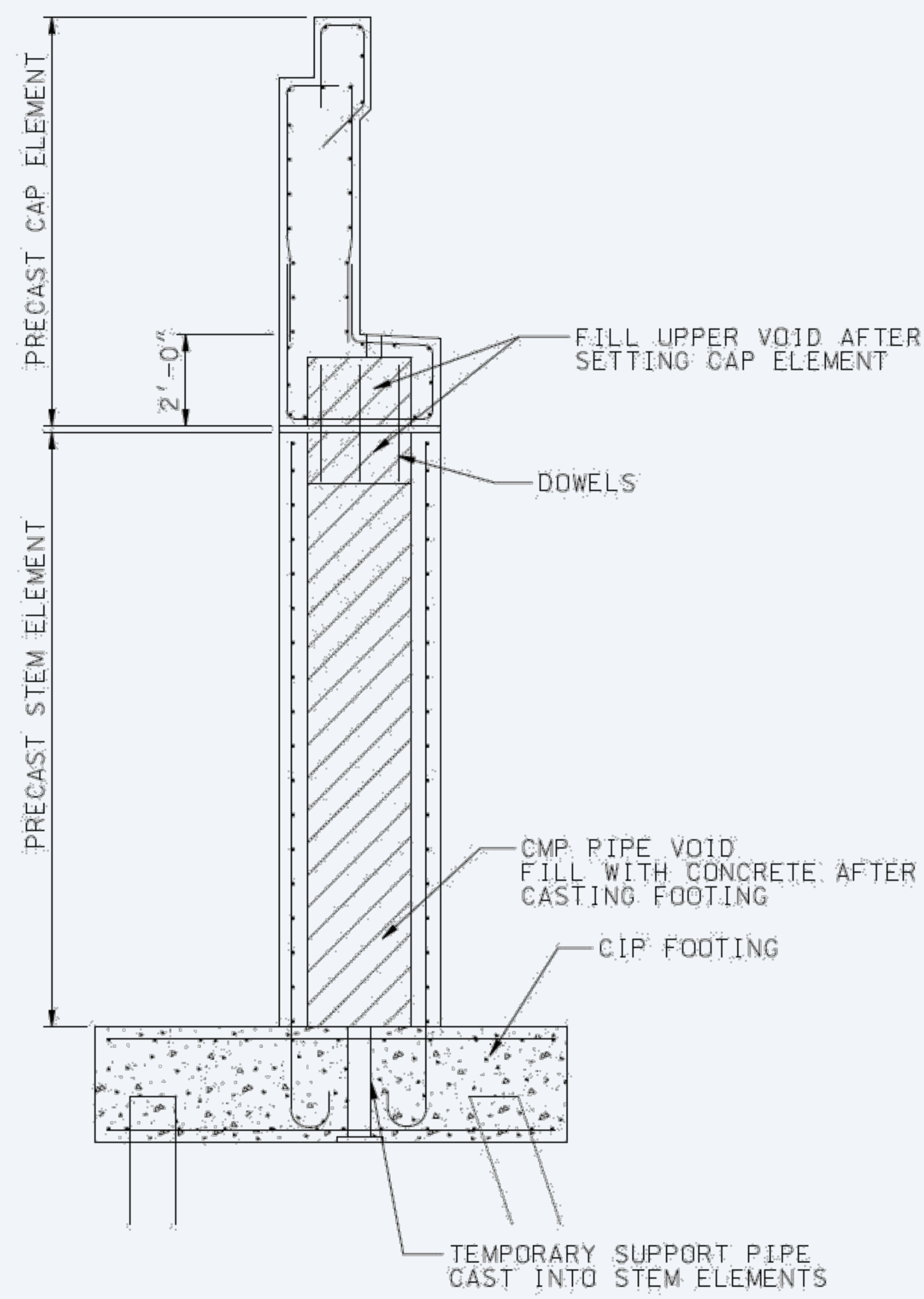


Figure 12.1.3-2. Proposed Precast Concrete Abutment Element Details.

Step 1: Design of Bridge Seat Element and Connection to Stem.

The approach for the design of the upper portion of the abutment is to detail the section with the same reinforcement as the CIP option. The connection between the cap and the stem is located approximately 2 ft below the bridge seat. There are several reasons for this approach:

- The complexity of the casting of the bridge is limited to a few cap elements as opposed to every stem element.
- The cap can be used to tie the stem elements together.
- The moment demand on the stem section at this elevation is very low. In most cases, the stresses across the connection will not be in tension. This greatly simplifies the connection design.

The connection of the cap to the stems is made by placing a series of reinforcing bars within a void located on the underside of the cap. The void is formed with corrugated metal pipe (CMP). The void in the cap is aligned with voids in the stem elements. Once placed, the void is filled with concrete to complete the connection.

The stem reinforcing need not be changed in this scenario. The backwall reinforcing can be developed into the cap portion of the element. The following calculations will demonstrate the adequacy of the cap-to-stem connection.

Figure 12.1.3-3 is a sketch of a precast cantilever abutment. The sketch shows precast footings. For this example, the precast footing will be changed to a CIP footing.

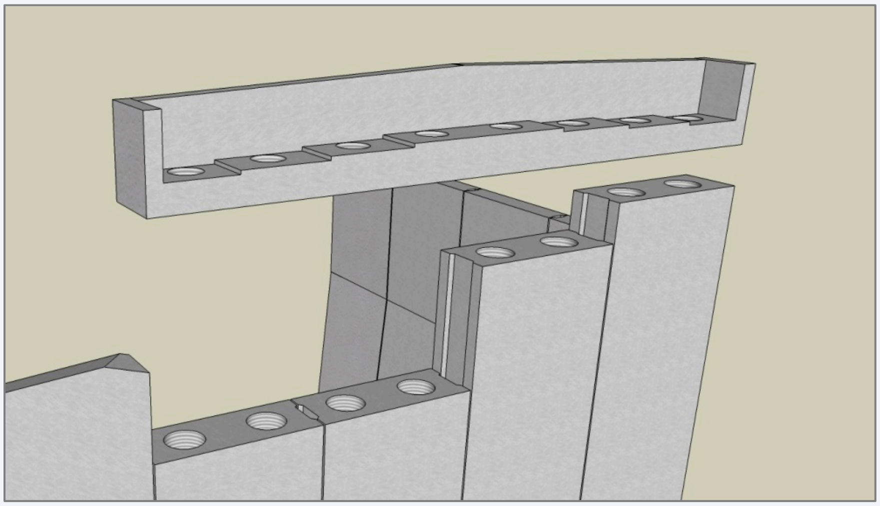


Figure 12.1.3-3. Sketch of Cantilever Abutment Cap Concept.

Check connection of cap element to stem element:

The approach for this design is to verify whether or not there is tension across the connection. If there is no tension, then the connection will not need a significant amount of reinforcement. This approach is not new to the bridge construction industry. Stone masonry gravity abutments with unreinforced joints have been in service for hundreds of years. The calculation of overturning for this example will be at the Service I limit state (un-factored loads).

Figure 12.1.3-4 is a detail of the cap showing the overturning forces and the resisting forces. Girder live load and wheel loads on the approach slab are not used, since they will help to resist the overturning.

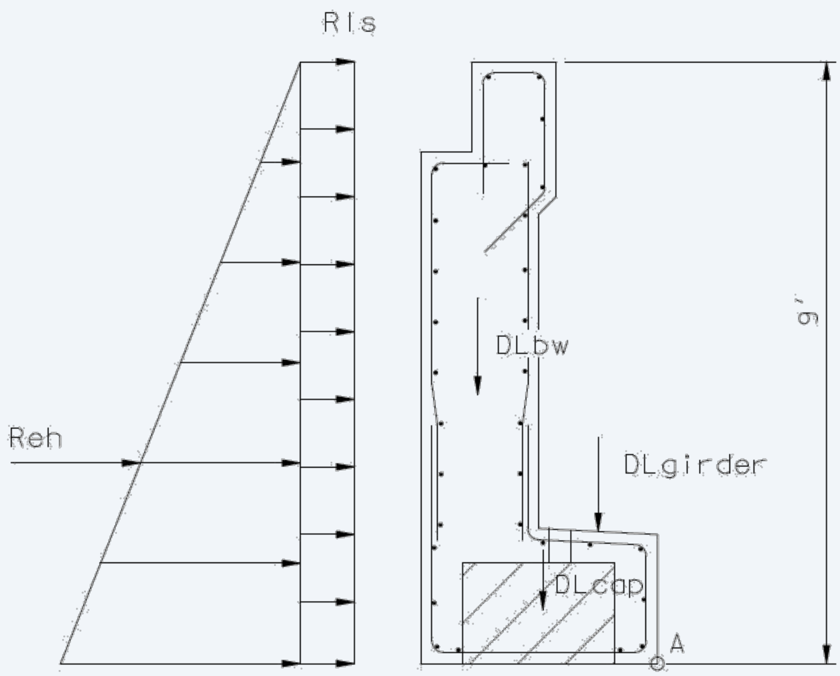


Figure 12.1.3-4. Forces on cap that maximize overturning.

Calculation of forces: All forces are calculated on a “per foot of wall” basis.

Girder Reactions
(See page 7-34 of the FHWA LRFD Design Example [75].)

$R_{DC\ TOTAL}$ = 7.66 k/ft
$R_{DW\ TOTAL}$ = 1.20 k/ft

R_{DL} = 7.66 + 1.20 = 8.86 k/ft

Active Soil Pressure on rear face of the cap element.
(See page 7-19 of the FHWA LRFD Design Example [75].)

$p = k_a \gamma_s z$

Where:

$k_a = 0.3$
$\gamma_s = 0.120$ kcf
z = height of cap = 9 ft

$p = 0.3(0.120)(9) = 0.324 \text{ k/ft}^2$

Reh = Horizontal soil force acting on wall
= ½ p z
= ½ (0.324)(9)
= 1.46 k/ft

Live Load Surcharge:
H = height of wall = 9 ft

Use AASHTO Table 3.11.6.4.1 to obtain equivalent height of soil.

Interpolate:
H = 5 ft, → heq = 2.0 ft
H = 10 ft, → heq = 3.0 ft

Therefore, for H = 9 ft → heq = 2.8 ft.

Change in earth pressure due to live load surcharge:
$\Delta_p = k_a \gamma_s$ heq
= 0.3(0.120)(2.8)
$= 0.101 \text{ k/ft}^2$

Rls = Force due to live load surcharge
$= \Delta_p H$
= 0.101(9)
= 0.909 k/ft

Dead load of precast cap:
DLbw = Backwall Dead Load
(See page 7-34 of the FHWA LRFD Design Example [75].)
= 1.71 k/ft

DLcap = Seat Dead Load
= 2 ft(3.5 ft)(0.15kcf)
= 1.05 k/ft

Calculate Overturning Moment:

Sum moments about point A

$$\Sigma M_{OT\ A} = Reh(Yeh) + Rls(Yls)$$

Where:

$Yeh = H/3 = 3ft$

$Yls = H/2 = 4.5ft$

$\Sigma M_{OT\ A} = 1.46(3) + 0.909(4.5) = 8.47$ k-ft

Calculate Resisting Moment:

Sum moments about point A

$\Sigma M_{RESIST\ A} = DLbw(Xbw) + DLcap(Xcap) + R_{DL}(X_{Girder})$

Where:

$Xbw = 3.5\text{-}0.833 = 2.67ft$

$Xcap = 3.5/2 = 1.75ft$

$X_{Girder} = 7.5$ inches$/12 = 0.625ft$

$\Sigma M_{RESIST\ A} = 1.71(2.67) + 1.05(1.75) + 8.86(0.625)$
$= 11.81$ k-ft

$\Sigma M_{RESIST} > \Sigma M_{OT}$ Therefore the connection is in compression
(see construction sequence note below)

The result of this calculation is that the connection does not need reinforcement. Dowel bars will be used in the connection to provide positive reinforcement for shear in addition to the sear capacity of the concrete within the CMP void. By inspection, the void concrete and dowel bars will be sufficient to resist the lateral forces acting on the cap.

Construction sequence: This calculation assumes the girder and deck are placed prior to backfilling. Alternatively the dowel bars could be designed to take the moment if the section is backfilled prior to placing girders. Designers should note the assumed construction sequence on the plans.

Step 2: Design of Stem Element and Connection to Footing.

The approach for the abutment stem design is to make all of the stem sections a uniform size. By separating the bridge seat from the stem sections, it is possible to keep the detailing of the stem simple and consistent. CMP voids are used for this element for two reasons:

- They can be used to reduce the weight of the element for shipping and handling.
- The void can be used to make the connection to the cap element.

The width of the stem elements can be set based on the shipping and handling limitations for the project. Vertical connections between stem elements are akin to contraction joints in CIP abutments. The AASHTO LRFD Bridge Design Specifications place no limitation on the minimum spacing of contraction joints; therefore the connections can be relatively closely spaced.

The proposed footing (pile cap) is made with CIP concrete. The reason for this is the presence of piles (including battered piles). It is possible to detail a precast footing in this situation with CMP voids used to transfer the pile forces into the footing. The proximity of the piles to the edge of the footing would require increasing the width of the footing significantly (both front and back) in order to make room for the CMP voids and to provide adequate cover over the voids. The CIP option avoids increasing the size of the footing and does not necessarily result in a slow construction process. The switch to CIP concrete for the footing will add approximately one day to the construction schedule.

The stem elements will be cast with a protruding temporary support that is cast into the stem element. The stems will be supported on the temporary support, plumbed, and braced. The footing reinforcement can then be placed and the footing cast. See Section 7.1.2 for more information on this process. Figure 12.1.3-5 is a sketch of a wall pier built with this concept.

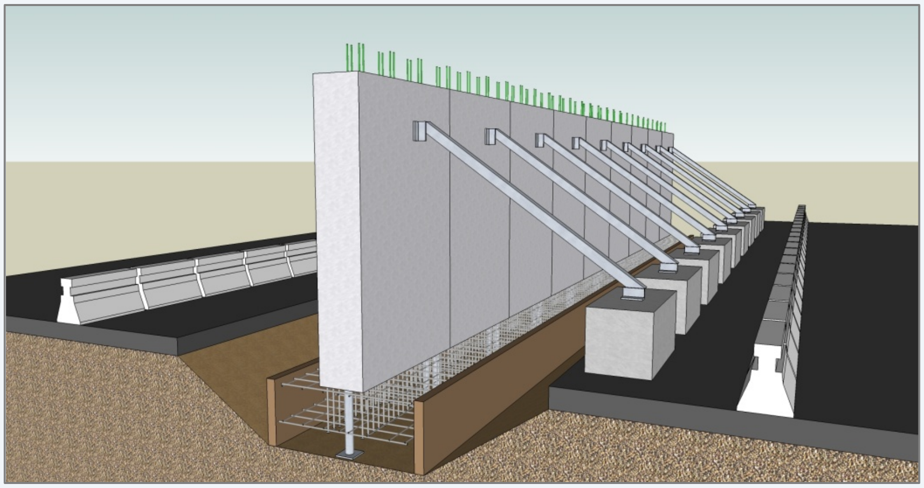


Figure 12.1.3-5. Precast Stem to CIP Footing Concept.

The construction sequence for this concept is as follows:

- Install the footing piles or drilled shafts.
- Set the stem elements to line and grade, supporting the stem elements on the temporary integral posts. Shim under the posts to provide the proper grade.
- Install the footing reinforcing and form the footing. Note that most or all of the footing reinforcing and forms may be placed prior to setting the stem elements.
- Cast the footing and cure the concrete until it attains adequate strength to support the stems.
- Fill the CMP void with concrete. Fill the void to the elevation of the bottom of the cap dowels.
- Grout or concrete the vertical connections between the stem elements.

The reinforcing in the stem element can be kept the same as the CIP option. The filling of the CMP void will ensure that the stem will emulate the CIP option. The splicing of the vertical reinforcement at the base of the stem can be eliminated. The vertical stem bars can be projected into the footing pour. The hooked bars can be replaced with headed reinforcing bars in order to facilitate the installation of the stem after the placement of the footing bars.

12.1.4. Pier Bent

Several details are used throughout the U.S. for precast pier bents. Many are discussed in Chapter 5. The details chosen for this example are ones that most closely emulate CIP concrete. The details presented are based on details developed by the PCI Northeast Bridge Technical Committee. This committee has chosen the use of grouted reinforcing splice couplers for connections between precast elements. The general approach is to replace construction joints with precast connections. Lap slices are replaced with grouted couplers.

One of the convenient features with this approach is that the design of the precast pier bent is the same as the design of the CIP pier bent. The change of the design to precast concrete is more of a detailing problem than a structural problem.

The details chosen for this example make use of grouted splice couplers located in the top of the footing and the underside of the cap. This is done for several reasons:

- The couplers are located outside of a potential plastic hinge zone in the column. Note that this is not required in this example because the bridge is located in Seismic Zone 1.
- AASHTO Article 5.10.11.4.1f [5] does not allow the use of mechanical connectors in one plane in high seismic zones. Note again that this is not required in this example because the bridge is located in Seismic Zone 1.
- If the couplers were located in the columns, the longitudinal column reinforcing would need to be moved farther into the column core in order to achieve adequate cover over the couplers. This would affect the design of the column, requiring more longitudinal reinforcing.

An important aspect of detailing with mechanical couplers is to ensure that the couplers do not interfere with the reinforcing in the elements. This will be checked in this example.

The following pages contain revisions to the design of the concrete pier bent in the FHWA LRFD Design Example: Comprehensive Design Example for Prestressed Concrete (PSC) Girder Superstructure Bridge with Commentary (in US Customary Units) [74].

Example 12.1.4. Concrete Pier Bent Redesign.

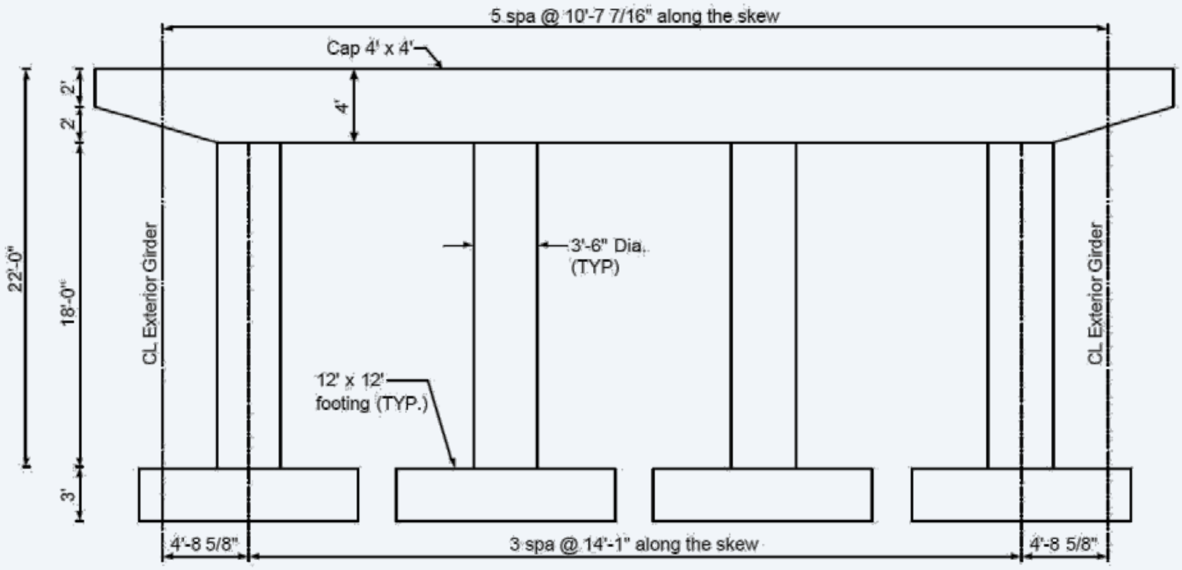


Figure 12.1.4-1. Example Pier Bent (as designed CIP option).

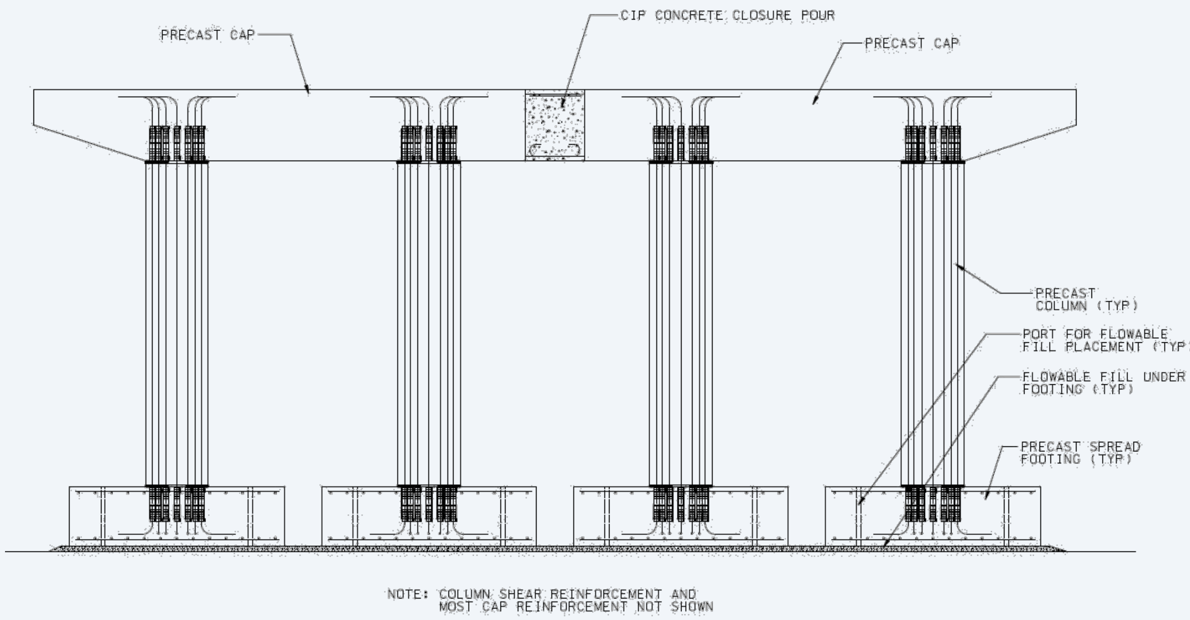


Figure 12.1.4-2. Proposed Precast Concrete Abutment Element Details.

Step 1: Design and Detailing of the Pier column and Cap.

The design of the precast cap is the same as the design of the CIP cap. The internal longitudinal bars are the same, and the shear stirrups are the same. Minor adjustments to the bar spacing are typically required in order to accommodate the couplers and associated reinforcing bars.

The pier column is included in this design step because the size and spacing of the column bars have a direct impact on the detailing of the pier cap.

The use of grouted reinforcing splice couplers provides a true emulative design. The couplers replace the conventional lap splices that would be located near the base of the column. Typically, the longitudinal column reinforcing is extended into the pier cap to form the cap-to-column connection. In this example, the connection of these two elements will make use of the couplers placed within the cap element.

The CIP pier example included a column with 16 #8 bars. Figure 12.1.4-3 shows the layout of the column reinforcement in the CIP pier columns.

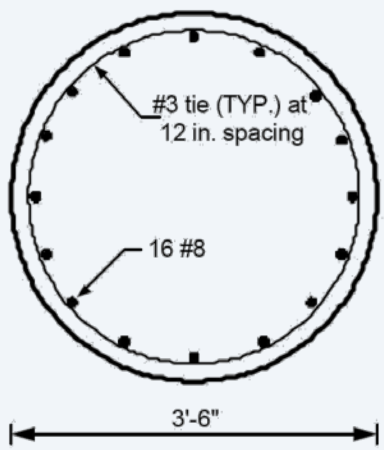


Figure 12.1.4-3. CIP Column Reinforcement.

The use of grouted reinforcing splice couplers with this reinforcing bar layout will inevitably result in detailing problems in the cap. This is due to the large number of longitudinal cap bars in typical concrete caps. For this example, the cap contains nine #8 bars. The diameter of a #8 bar coupler is approximately 3.5 inches. With the pattern shown, it is not possible to fit every cap reinforcing bar through the couplers. Figure 12.1.4-4 shows this situation.

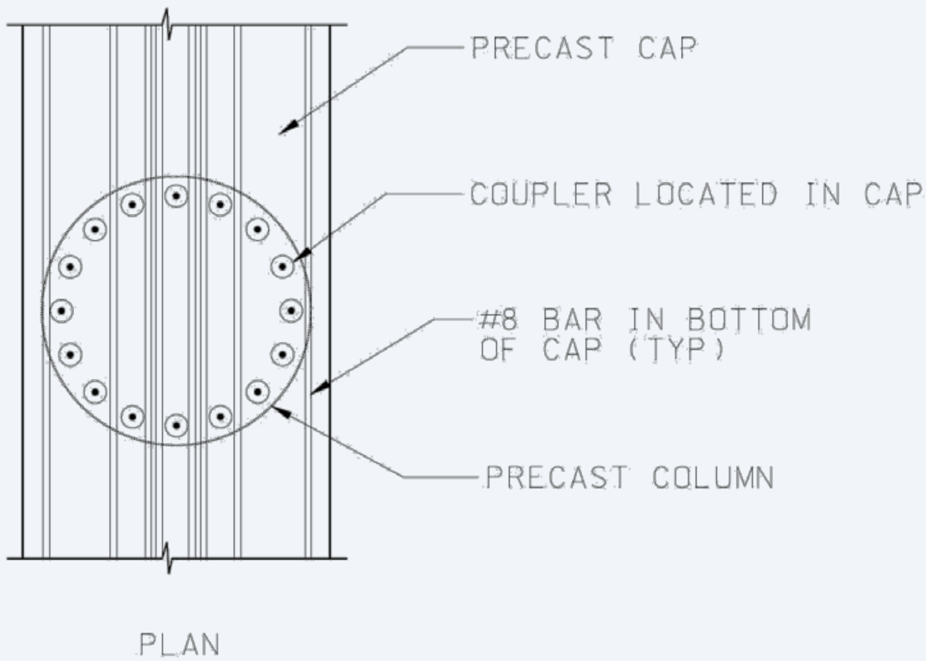


Figure 12.1.4-4. Conflicts With Grouted Couplers and Cap Reinforcement.

With this configuration of column reinforcement, it is only possible to fit eight out of the nine required bars between the couplers. There are two options to resolve this issue:

- Redesign the cap with multiple layers of bottom bars.
- Use fewer larger bars in the column.

The second option is typically more desirable. Multiple layers of flexure reinforcement are not as efficient as a single layer. Another reason for this option is the cost of the couplers. In general, it is less expensive to use fewer larger couplers than more, smaller couplers. This is due to the fact that the manufacturing and installation costs make up the majority of the cost of the coupler, not the material.

The goal is to replace the column reinforcement with an equivalent or slightly greater amount of reinforcement. Figure 12.1.4-5 is the proposed column section combined with the re-layout of the cap reinforcement. The 16 #8 bars are replaced with 9 #11 bars. This results in slightly more reinforcement. With this arrangement, the bottom cap reinforcement needs to be shifted in order to accommodate the couplers, which can be seen in the figure.

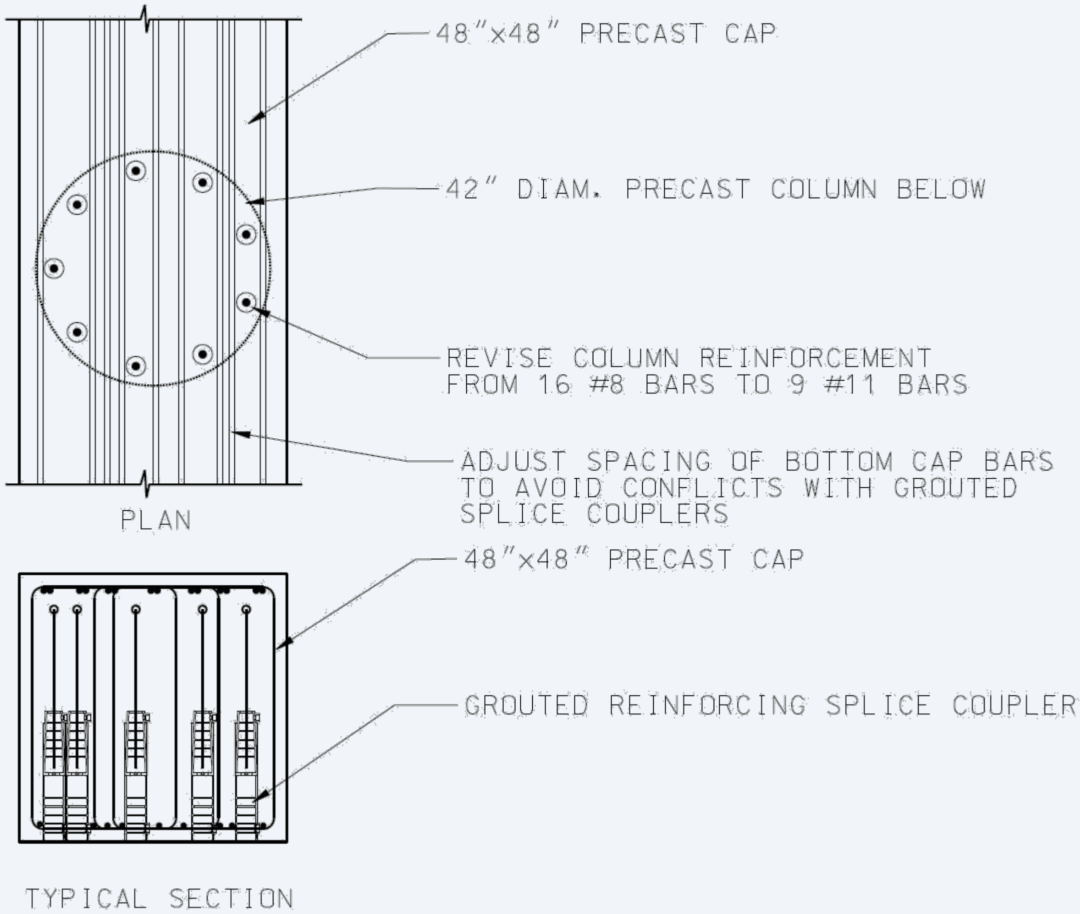


Figure 12.1.4-5. Redesign of Column and Cap Reinforcement.

The centroid of the larger diameter longitudinal bars in the column is deeper within the column core. Therefore, a strength check of the column should be undertaken. The calculations for this analysis are significant. They are normally completed using software, which was done in this case (as well as the FHWA LRFD Design Example [74]). Figure 12.1.4-6 is a plot of the column moment/axial interaction diagram for the revised reinforcement. The figure also includes data points for the Maximum Factored Load Effects and the Concurrent Load Effects for the strength limit states (see Table 7.2-2 on page 7-76 of the FHWA LRFD Design Example). Based on this plot, it can be concluded that the column is adequate to resist the forces in the design example. In fact, it may be possible to reduce the number of longitudinal column reinforcing bars.

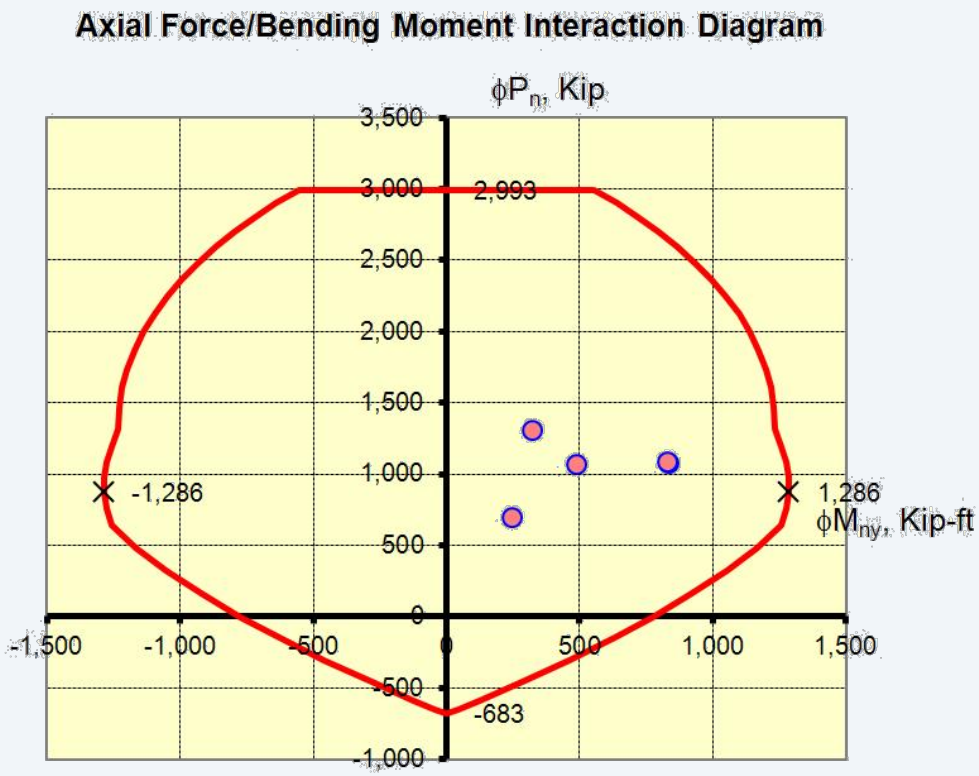


Figure 12.1.4-6. Revised Column Interaction Diagram.

The proposed cap is a reinforced concrete element. It is possible to design a prestressed precast concrete cap; however, in most cases, a non-prestressed precast cap is feasible. In this example, the cap is quite large, spanning over four columns. If the cap was to be fabricated as one piece, it would weigh more than 130 kips. It is possible to transport an element of this size and weight. Depending on the work area, it may be difficult to position a crane that is large enough to erect the cap. In this case, designers can detail an optional cap splice.

Figure 12.1.4-7 is a free body diagram of the pier. A review of the layout shows that the beam reactions in the center of the pier are located very close to the interior columns. Splitting the cap into two pieces is feasible since there are no beams located in the center of the cap. It is possible to locate a concrete closure pour in this area. The closure pour does not necessarily need to slow construction, since the closure pour concrete can be placed and cured while the superstructure is being constructed.

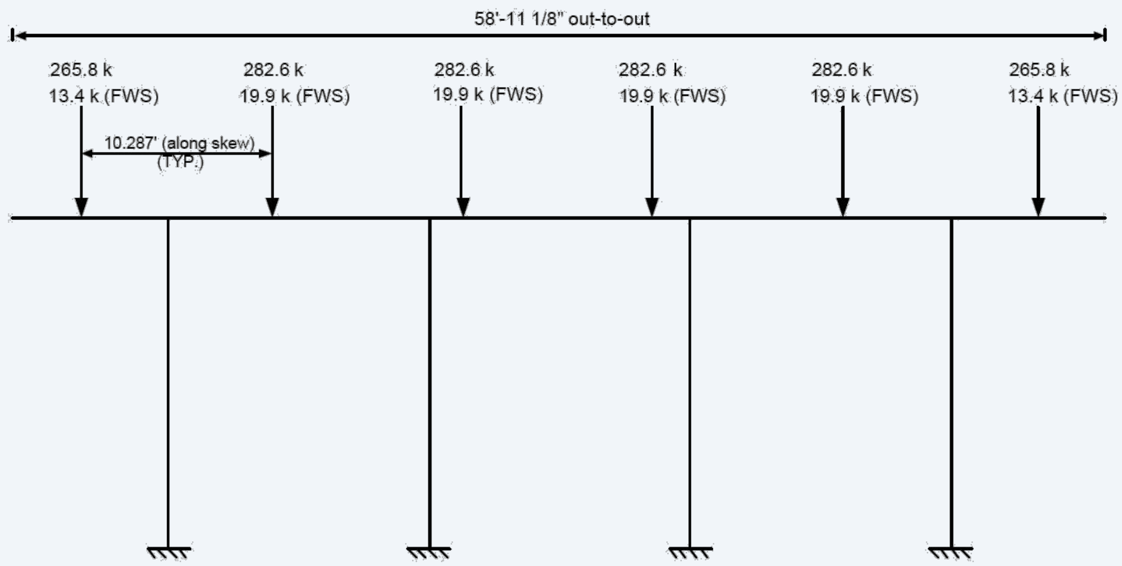


Figure 12.1.4-7. Free Body Diagram of the Design Example Pier Bent.

Another option that could be considered would be to detail a two-piece pier cap. An open joint could be used in the center of the cap. This would require a complete redesign of the pier, since the structural frame would be different than the original design example.

Figure 12.1.4-2 is a detail of the precast pier showing the closure pour (minus the shear reinforcement). The exact moments and shear acting on this section are not available in the FHWA Design Example; therefore, a detailed design of this connection cannot be presented in this redesign example. The reinforcement in the closure needs to be developed. Hooked bars are shown for the bottom tension reinforcement based on the assumption that this is a positive moment region and the bars are relatively highly stressed. The top bars are shown as a simple lap splice based on the assumption that the bars are with low stressed, or are in a compression zone. These assumptions would need to be verified and the design adjusted if necessary.

Step 2: Detailing of the Footing.

The detailing of the column-to-footing connection is similar to the cap connection. The spacing of the reinforcement in the footing is wider; therefore, it is easy to make adjustments to the spacing of the footing bars to avoid conflicts with the couplers.

The example footing is a spread footing that is supported on soil. If a large precast footing were to be set directly on a prepared substrate, it would be inevitable that there would be gaps under portions of the footing, resulting in high bearing stresses under the points of contact. Over time, the high stress bearing areas would most likely settle, leading to a redistribution of soil bearing that would eventually provide even bearing over the entire footing. A potential problem with this is that the amount of settlement would be unknown and most likely variable.

To avoid this, one approach for the use of precast footings is to set the footing slightly above the substrate, and fill the void with a flowable material to provide even bearing. Early precast footing designs made use of flowable non-shrink grout. This is a very good material, but it comes at a relatively high cost. The strength of the non-shrink grout is magnitudes larger than the actual stress under the footing.

In this design example, the maximum factored bearing stress under the footings is:

$\sigma = 11.71$ ksf
(See page 7-79 of the example problem [74]).

Expressed in pounds per square inch:

$\sigma = 11.71 \text{ ksf } (1000)/144 = 81.3 \text{ psi}$

This demonstrates the lack of need for a high strength grout material. Several agencies are not using flowable fill to span the gap between the footing and the substrate. Most flowable fills can easily attain this strength level; therefore, they would be acceptable for this situation.

12.1.5. Wall Pier

The FHWA LRFD Design Examples do not contain a wall pier. The steel girder design example has a large hammerhead pier. In theory, the CIP concrete hammerhead pier could be redesigned as a precast structure. Details similar to the pier bent example in section 12.1.4 could be used, as well as a number of other details that shown in Chapter 5. Figure 12.1.5-1 shows the dimensions of the pier in the FHWA Design Example. The reality is that the hammerhead portion of this pier weighs approximately 350,000 lbs. Prefabrication and lifting of this cap is feasible with very large equipment, but not practical.

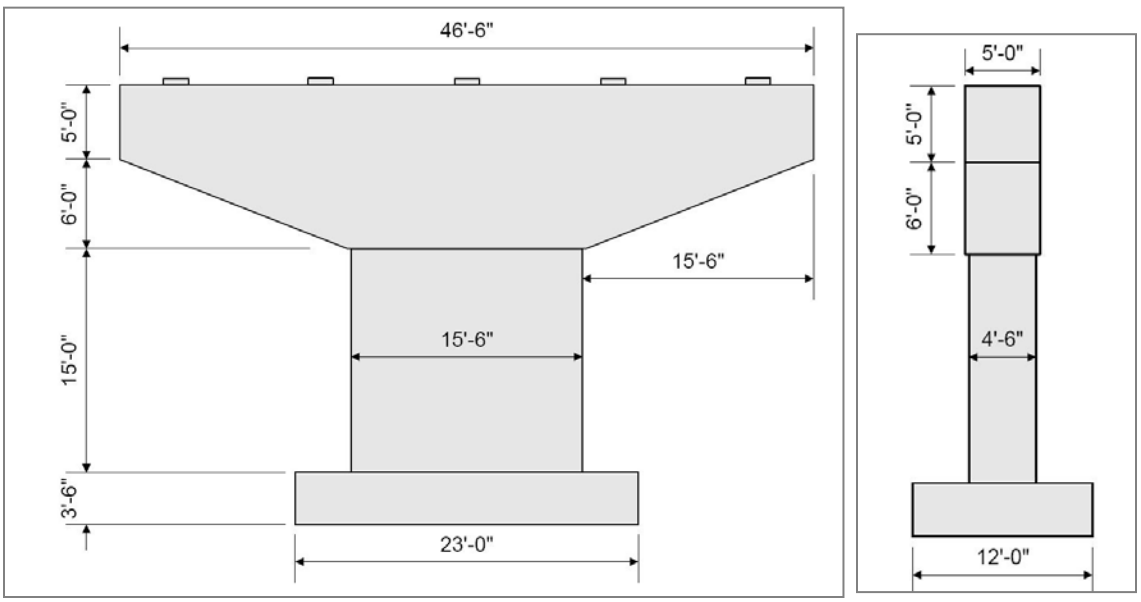


Figure 12.1.5-1. Hammerhead Pier in the FHWA LRFD Design Example [75].

If this design was changed to a wall pier, the design for prefabrication could follow the same procedure as the cantilever abutment that is detailed in Example 12.1.3. Figure 12.1.5-2 shows a detail of a wall pier that was developed by the PCI Northeast Bridge Technical Committee.

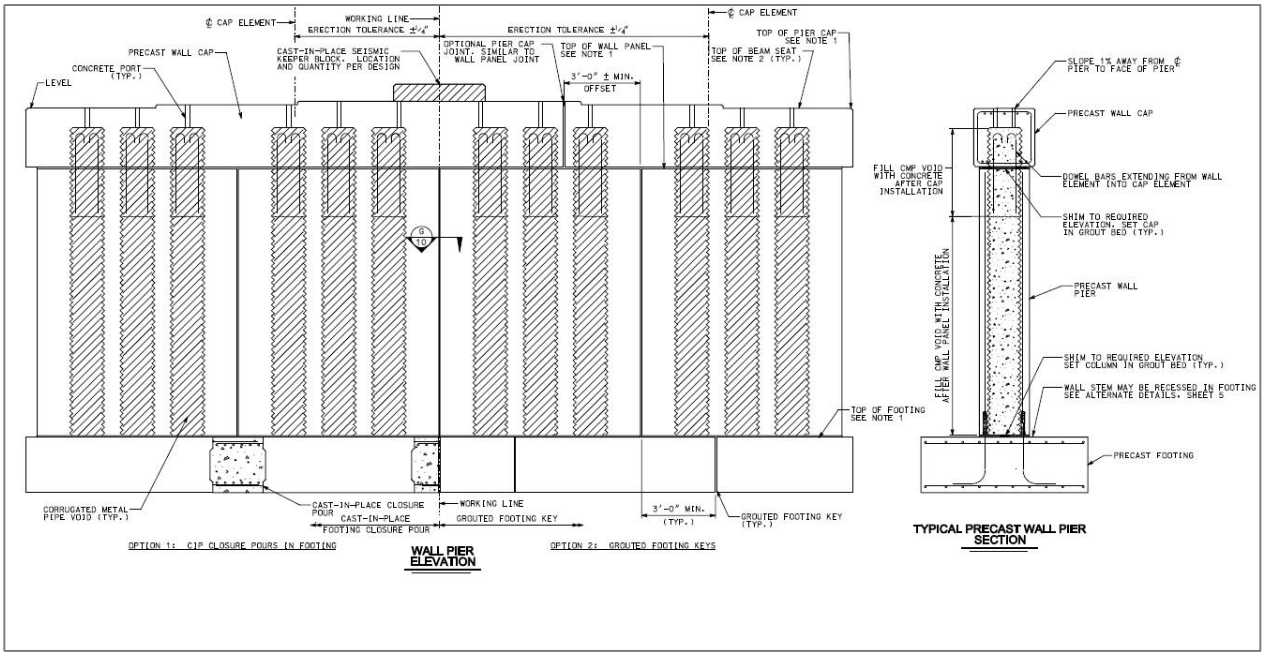


Figure 12.1.5-2. Precast Concrete Wall Pier Details (Source: PCI Northeast).

The design could be undertaken following portions of the previous examples as follows:

1. Wall elements could be detailed with corrugated metal pipe voids as shown in Figure 12.1.5-2 to reduce weight. The cap could be precast and attached to the wall elements. The design of this connection would be similar to the cap designed in Example 12.1.3, Step 1.

2. If a precast spread footing supported on soil was used, the connection of the wall stem to the footing could make use of grouted reinforcing splice couplers. The design would be similar to the footing connection outlined in Example 12.1.4, Step 2. The interface between the footing and the soil could be made by placing flowable fill in the void as outlined in Example 12.1.4, Step 2.

3. If the footing was supported in piles or drilled shafts, a CIP concrete footing could be used. The design of the footing would be similar to the design in Example 12.1.3, Step 2.

12.2. MODULAR DECK/BEAM ELEMENT SUPERSTRUCTURES

There have been several very successful projects in the U.S. that made use of modular deck/beam elements. The approach for this type of construction is to place a deck on either steel or prestressed concrete beams in a fabrication facility. One reason that this approach is attractive is that the complexity of making a composite deck connection to a girder in the field is eliminated. Prefabrication of the deck eliminates issues with spacing of shear connectors, the need for post-tensioning, and the significant grouting that is often necessary for prefabricated deck installations. Figure 12.2-1 is a portion of the cross section of a modular deck/beam bridge that was built in Massachusetts as part of the 93 Fast 14 project.

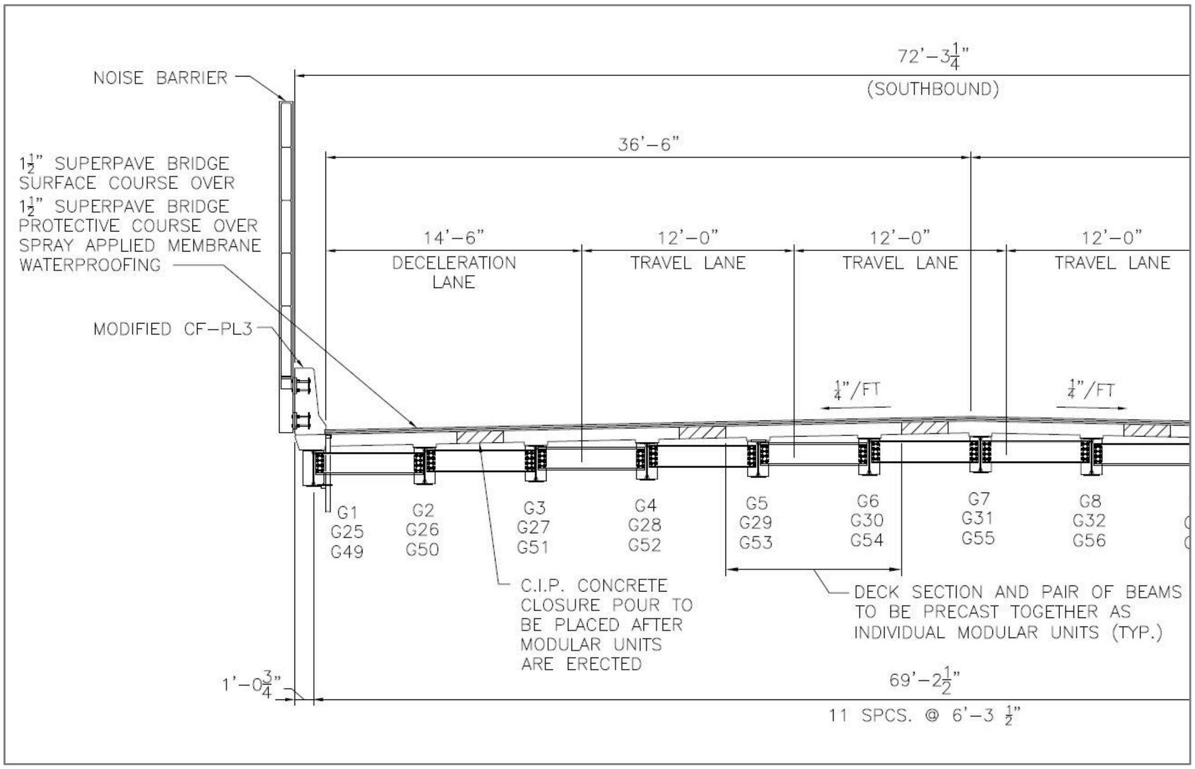


Figure 12.2-1. Typical Bridge Section with Modular Deck/Beam Elements (Source: MassDOT 93 Fast 14 Project).

The design of these structures is similar to the design of superstructures for bridges installed with SPMTs or lateral sliding techniques. In general, the superstructure is designed as if it was built in place. As with some SPMT installations, there are minor changes to the design process.

Modular deck/beam elements have been studied as part of SHRP 2, which was administered by the Transportation Research Board (TRB). The SHRP2 program is described on the TRB website as follows:

The second Strategic Highway Research Program (SHRP 2) is program was authorized by Congress to address some of the most pressing needs related to the nation's highway system: the high toll taken by highway deaths and injuries, aging infrastructure that must be rehabilitated with minimum disruption to users, and congestion stemming both from inadequate physical capacity and from events that reduce the effective capacity of a highway facility. These needs define the four research focus areas in SHRP 2:

*One of the four focus areas is rapid renewal. This portion of the research was entitled "*Innovative Bridge Designs for Rapid Renewal.*" As part of this work, the research team* *developed a document called the "*ABC Toolkit.*" In this toolkit, a detailed design example for a modular deck/beam bridge was developed. This document is available in draft form. It is anticipated that it will be available in its final form in the near future. Designers are encouraged to obtain a copy of this document for detailed information on the design of modular deck/beam elements.*

12.2.1. Generation of Input for Computer Programs

Most girders are designed using software. Designers are typically required to calculate values for input into the software programs. The following sections contain information on modifications to typical software input variables.

Dead Load Distribution: The 93 Fast 14 project made use of CIP concrete closure pours to connect the elements. Normally, deck concrete would be accounted for as non-composite dead load in the design. In this case, the closure pour concrete would be treated as a composite dead load. Some software programs may not allow this approach. If so, designers may need to perform post-processing calculations in order to verify the structural integrity of the sections based on this approach.

Barrier Load Distribution: There is no consistent application of the distribution of bridge barrier loads. Previous versions of the AASHTO Bridge Design Specifications allowed uniform distribution of barrier loads to all girders; however, later versions of the specifications have different requirements. Section C4.6.1.2-4b of the AASHTO LRFD Bridge Design Specifications [5] states:

However, heavier concentrated line loads such as parapets, sidewalks, barriers, or sound walls should not be distributed equally to the girders. Engineering judgment must be used in determining the distribution of these loads.

Many agencies have developed design requirements for the distribution of barrier loads on conventional bridges. Designers should follow the requirements for the local agency in these cases.

Several projects that make use of modular deck/beam elements have included modules with the bridge barrier cast in the fabrication plant prior to installation. In this case, designers need to exercise engineering judgment as outlined above. Unless special measures are undertaken (such as jacking the fascia module prior to connecting it to the adjacent module), the total load of the barrier will be carried by the fascia module. This load can be distributed equally to each beam in the module or distributed to the beams using a simplified method such as the lever rule.

Live Load Distribution: In most cases, standard live load distribution factors as specified in the AASHTO LRFD Bridge Design Specifications [5] are appropriate. In some cases, the layout of the modules and the widths of the closure pours result in irregular beams spacing. In this case, the use of average beam spacing would be appropriate for use in the AASHTO Live Load Distribution Equations.

Composite Design: The design of the beams for composite action and the associated shear connectors would be the same as for conventional CIP construction.

12.2.2. Deck Design and Connections

The design of the deck for modular deck/beam elements is typically the same as with a conventional deck design. The provisions of Chapter 4 (analysis) and Chapter 9 (deck design) of the AASHTO LRFD Bridge Design Specifications [5] apply to pre-decked sections. The strip method of analysis specified in AASHTO Section 4.6.2.1 is typically used for the analysis of concrete decks. More refined analysis methods (such as finite element analysis) can also be used, since they are specifically allowed in AASHTO Article 4.6.3.2.

The empirical design method that is specified in Section 9.7.2 may not be applicable to concrete topped elements. The design conditions in Article 9.7.2.4 that must be met to use this method of analysis include the following note: *The deck is fully cast-in-place and water cured.*

One could contend that a pre-topped module with a CIP closure pour is akin to a CIP deck with stage construction joints, and this would satisfy this provision. In lieu of clarity in the specifications, it is recommended that this design method not be used for modular pre-topped elements.

The connection of the deck edges is an important aspect of the design process for modular deck/beam elements. Two connections will be covered in this section—the longitudinal deck connection and the connection of the decks at piers.

12.2.2.1. Longitudinal Deck Edge Connections

The text in this section is based on the assumption that the girders are running parallel to the connection. The longitudinal deck edge connection needs to be designed to resist the flexure in the “main direction” of the deck. It is common to locate this connection mid-way between the girders, as this connection is required in most cases to resist the maximum positive moments the deck.

On very rapid ABC projects, the placement of the closure pour material (concrete, UHPC, grout, etc.) is often one of the last steps in the construction process. This means that the strength gain of this connection is critical for the completion of the bridge. It is possible to design this connection based on a material strength that is less than the specified final design strength. By using this approach, project specifications can allow the contractor to open the bridge at the lower interim strength. The specifications should note that this provision does not relieve the contractor from the responsibility of providing a material that can attain the final specified strength. Section 4.2.7 of this manual contains more information on this approach to design. The following design example is for a longitudinal deck connection.

Example 12.2.2.1. Longitudinal Closure Pour Connection Design.

The design example used for this is based on the FHWA LRFD Design Example for Steel Girder Superstructure Bridge with Commentary [75]. This bridge is a multi-span steel girder bridge. Typically, modular deck/beam element bridges are either designed as simple spans (with link slabs), or designed as simple span for dead load and continuous for live load. The intent of this design example is that the bridge would be modified for one of these two methods. The deck design would not change under these conditions.

Figure 12.2.2.1-1 is a typical cross section of the bridge showing the equations for the width of the equivalent strips as specified in AASHTO Article 4.6.2.1.

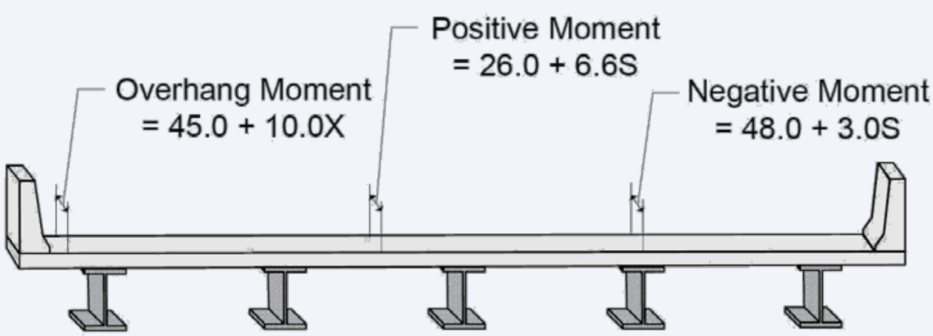


Figure 12.2.2.1-1. Typical Bridge Section as shown in the FHWA LRFD Design Example [75].

Assumptions:

- The location of the connection will be in the second bay from the left. The modular deck/beam units will be a two-beam unit and a three-beam unit, separated by the longitudinal closure pour.
- The length of roadway closure is such that the time for placement and curing of the closure pour material is sufficient to use typical deck concrete.
- f'c = 4000 psi

Step 1: Check Lap Splice Length of Top Reinforcement.

Assume: Compression Lap Splice (positive moment region).

Given:
Required reinforcement for positive flexure
#5 @ 6-inch spacing
(See page 2-22 of FHWA Design Example [75].)
Bottom cover = 1.0 inch
Epoxy coated bars
fy = 60 ksi

Basic Lap Length = lc = 0.5 m fy db, but no less than 12 inches.

Where:
m = adjustment factor
fy = 60 ksi
db = 0.625

Adjustment factor = m
f'c > 3 ksi
No ties
No spirals
Therefore m = 1.0

Therefore, lc = 0.5(1.0)(60)(0.625) = 18.75 inches, say 19 inches

Step 2: Calculate Development Length of Bottom Reinforcement.
(S5.11.2.1.1)

Given:
Required reinforcement for positive flexure
#5 @ 8-inch spacing
(See page 2-16 of FHWA Design Example [75].)
Bottom cover = 1.0 inch
Epoxy coated bars

Basic Development length = ld = 1.25 Ab fy/f'c$^{0.5}$ but not less than 0.4 db fy.

Where:
Ab = 0.31 in^2
db = 0.625 inches
fy = 60 ksi
f'c = 4 ksi

$ld = 1.25(0.31)(60)/(4)^{0.5} = 11.63$ inches

but not less than 0.4(0.625)(60) = 15.00 inches **CONTROL**

Adjustment Factors

Top Bar (S5.11.2.1.2): No

Lightweight Aggregate (S5.11.2.1.2) No

Epoxy coating (S5.11.2.1.2):
Is cover less than 3db? = 3(0.625) = 1.875 inches **YES**
Therefore increase development length by a factor of 1.5

Lateral spacing (S5.11.2.1.3):
Spacing less than 6.0 inches center to center? No
Clear cover less than 3 inches measured in the direction of the spacing? No
Therefore, adjustment for spacing = 0.8

Adjustment for excess reinforcement:
As required/As provided = 0.58 inches^2/0.61 inches^2 = 0.95
Therefore adjust development length by a factor of 0.95

Total adjustment factor:
= 1.5(0.8)(0.95) = 1.14

Therefore the total development length = 15.00(1.14) = 17.1 inches

Step 3: Calculate Splice Length of Bottom Reinforcement.
(S5.11.5.3.1)

Determine Class of Splice (Table 5.11.5.3.1-1).
As provided/As required = 0.95 (see previous calculation).
Percent of As spliced = 100 percent.
Therefore, this is a Class C splice.

Therefore, the lap splice length = 1.7 ld, but not less than 12 inches
= 1.7(17.1 inches)
= 29.1 inches, say 30 inches

Step 4: Calculate the Minimum Width of the Closure Pour.

Basis: Bottom lap splice length plus tolerances.
(Bottom lap splice length controls over top lap splice length.)

Givens:
Specified element width tolerance: $\pm$ 0.50 inches.
Specified element erection tolerance: $\pm$ 0.25 inches.
Minimum connection width based on development length: 30 inches.
Extra width to facilitate element installation = 1 inch (to ensure that the bar ends do not hit the adjacent element).

Minimum required connection width for plans:
= Min. connection width + size tolerance + erection tolerance + clearance at end
= 30 inches + 2(0.50 inches) + 2(0.25 inches) + 2(1 inch) = 33.50 inches

Note: *Tolerances are doubled to account for each element.*

Conclusions:
Detail a 34-inch-wide connection.
Detail the lap splice to be 30 inches + tolerances = 30.75 inches min., say 31 inches.

Figure 12.2.2.1-2 depicts the final closure pour connection made with straight lapped bars.

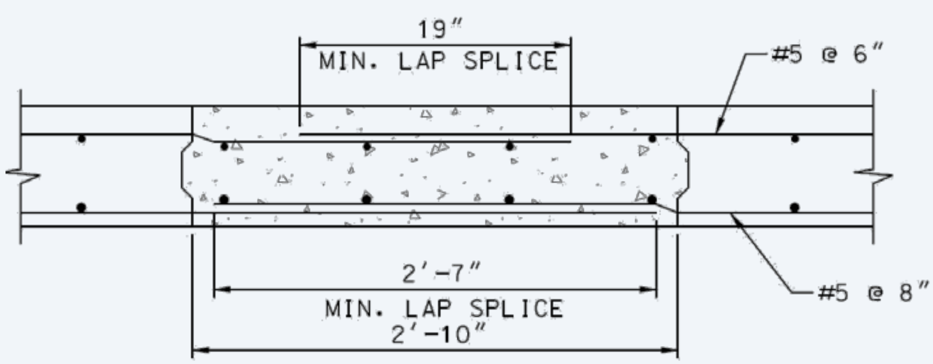


Figure 12.2.2.1-2. Deck Closure Pour with Lapped Bars.

The designer may not wish to detail such a wide connection based on concern about concrete shrinkage cracks and the volume of concrete required to be placed. One option is to change the bottom lap splice to lapped hooked bars. The following calculation details this procedure.

Step 1: Calculate Size of Bottom Bars with a 180-Degree Hook Splice.
(S5.11.2.4)

Basic Hook Length = lhb = 38.0db/(f'c)$^{0.5}$ but not less than 8 db or 6.0 inches.

Where:
db = 0.625 inches
f'c = 4 ksi

lhb = 38.0(0.625)/4$^{0.5}$ = 11.9 inches
but not less than 8(0.625) = 5 inches
but not less than 6 inches

lhb = 11.9 inches, say 12 inches

Modification factors (S5.11.2.4.2)

Yield strength = 60 ksi: No modification required.

Side cover measured perpendicular to the plane of the hook.
Not applicable since the hook is a 180-degree hook.
No modification required.

No ties or stirrups: No modification required.

As required/As provided = 0.58inches^2/0.61inches^2 = 0.95 mod. factor

No lightweight concrete: No modification required.

Epoxy Coated? Yes, use 1.2 modification factor.

Total Modification Factor = 0.95(1.2) = 1.14

Therefore, the total Length of Hook = 12 inches(1.14) = 13.7 inches

The required lap length of the hook will be 14 inches, assuming that the development of the hook is linear.

Check if hook can fit within core of slab concrete
Outside diameter of hook = 2(4db) = 2(4(0.625)) = 5 inches

Height available = 8.5 inches – bottom cover – top cover
= 8.5 inches – 1 inch = 2 inches = 5.5 inches **OK**, hook will fit in deck

Step 2: Calculate the Minimum Width of the Closure Pour.

Basis: Use loop bars (hook for both top and bottom).
Use hook length plus tolerances.

Givens:

Specified element width tolerance: ± 0.50 inches.
Specified element erection tolerance: ± 0.25 inches.
Minimum connection width based on hook length: 14 inches.
Extra width to facilitate element installation = 1 inch (to ensure that the bar ends do not hit the adjacent element).

Minimum required connection width for plans:
= Min. connection width + size tolerance + erection tolerance
= 14 inches + 2(0.50 inches) + 2(0.25 inches) + 2(1 inch) = 17.5 inches

***Note:** Tolerances are doubled to account for each element.*

Conclusions:
Detail an 18-inch-wide connection.
Detail the hook lap to be 14 inches + 2*tolerances = 15.5 inches min., say 16 inches.

Figure 12.2.2.1-3 depicts the final closure pour connection made with loop bars. Note that in order for the loop bars to be one contiguous bar, the spacing of the bottom bars would need to match the top bars (#5 @ 6 inches). In theory, separate non-contiguous hooks could be detailed,

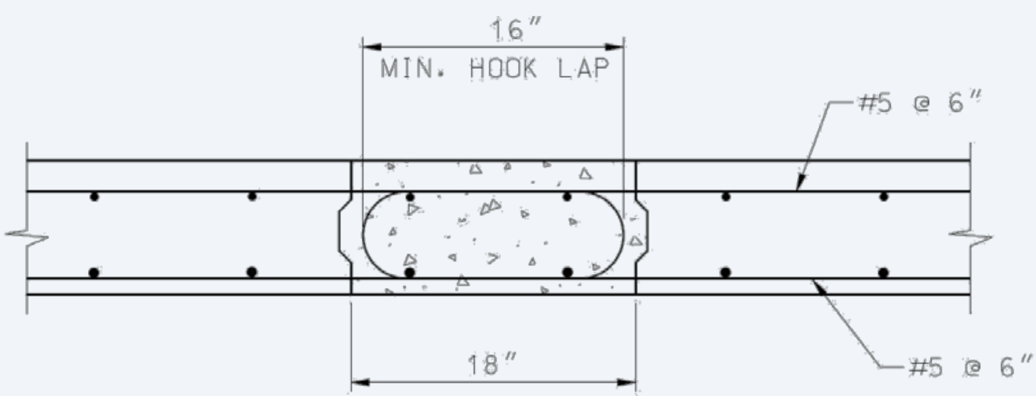


Figure 12.2.2.1-3. Deck Closure Pour with Lapped Loop Bars.

12.2.2.2. Link Slab Connection

As stated in Section 4.3 of this manual, link slab techniques can be used to provide a jointless deck at the piers without live load continuity. This is a cost-effective way to obtain a jointless bridge.

The concept of link slabs is to design the connection of the deck across the pier to accommodate the rotation of the beams without significant cracking. This is done by de-bonding a small portion of the deck near the pier to allow for a wider spread of the rotation strain.

Link slab technology has been in use for many years. The theory of link slab design is best described in the 1998 Journal of the Precast Prestressed Concrete Institute (PCI) Journal Paper entitled “Behavior and Design of Link Slabs for Jointless Bridge Decks,” by Caner and Zia [30].

The FHWA LRFD Design Examples do not have continuous-span bridges; therefore, adding a link slab to the design example is not possible. In lieu of this, a design example is used from the 93 Fast 14 project in Medford, Massachusetts. This project made use of modular deck/beam elements and link slabs.

Example 12.2.2.2. Link Slab Design.

This design example is taken from one of the bridges on the 93 Fast 14 project in Medford, Massachusetts. This ABC project made use of both modular deck/beam elements and link slabs. The project consisted of the replacement of 14 bridges. All but one of the bridges were multi-span (mostly three- and four-span bridges). Links slabs were used to simplify the construction in order to complete the replacement of each bridge during a 55-hour weekend closure.

The basis of the design procedure is a paper that was published in the *Precast Prestressed Concrete Institute (PCI) Journal* entitled “Behavior and Design of Link Slabs for Jointless Bridge Decks” by Caner and Zia [30].

Givens:

Span 1 Bridge Properties
Span Length = L1 = 62.15 ft
End Rotation = Θ_1 = 0.00275 radians

Span 2 Bridge Properties
Span Length = L2 = 74.66 ft
End Rotation = Θ_2 = 0.00250 radians

Note: *Rotations include composite dead load and live load (service limit state).*

Beam Spacing = 6.25 ft

Deck Thickness = 8 inches

f’c =4ksi

Step 1: Calculate Negative Bending Moments in Link Slab.

The theory in link slab design is that a portion of the slab is de-bonded on each side of the pier, leaving a portion of the deck to freely flex under beam end rotation. The recommended length of de-bonding is 5 percent of the corresponding slab length on each side of the pier. Figure 12.2.2.2-1 shows a schematic side view of a typical link slab. The link slab is the only connection between the two girders, which simplifies the construction of the bridge. In modular construction, this area would be a CIP concrete closure pour. Note that the width of the pour need not encompass then entire de-bonded zone of the link slab. Another important feature of this detail is that the beam depths do not need to match. This also simplifies connections.

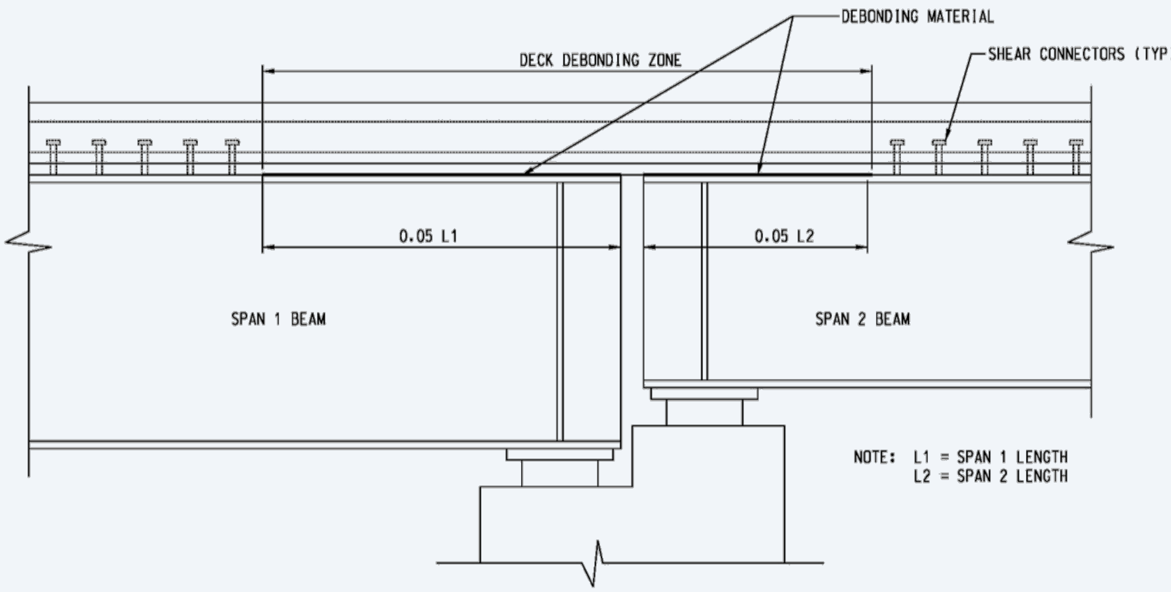


Figure 12.2.2.2-1. Schematic Detail of a Link Slab.

The moment brought on by girder end rotation is converted into a bending moment. The theory in the design of the link slab is to calculate the mending moment brought on by rotation of the girder end. This theory is based on the assumption that the slab is fixed at each end of the link slab. A rotation is applied to the slab separately at each end of the link slab. Then a bending moment can be calculated using a simple bending formula. Figure 12.2.2.2-2 is a free body diagram that demonstrates this approach.

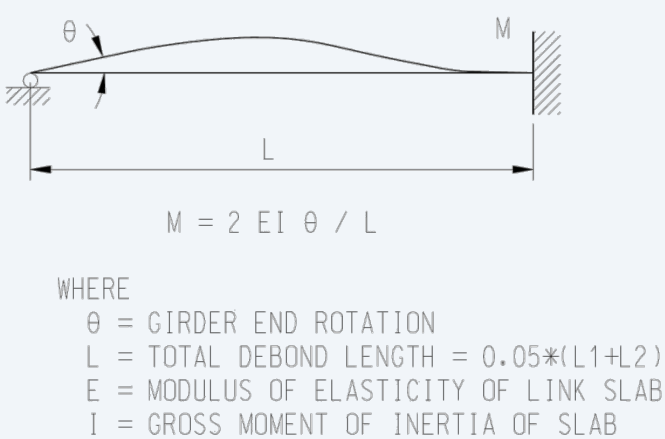


Figure 12.2.2.2-2. Free Body Diagram of Link Slab Behavior [30].

With this formula, moments are calculated on each side of the pier. The maximum moment is used for the design of the slab.

Concrete Properties:
$E_c = 1{,}820(f'c)^{0.5}$ (S5.4.2.4)
$= 1{,}820(4^{0.5})$
$= 3640$ ksi

Calculate the Length of Each Link Slab
Span 1 = 0.05(Span Length)
= 0.05(62.15)(12inches/ft)
= 37.29 inches

Span 2 = 0.05(Span Length)
= 0.05(74.66)(12inches/ft)
= 44.80 inches

Calculate the Moment of Inertia of the Link Slab:
$I_{LS} = bd^3/12$

Where:
b = beam spacing = 6.25*12 = 75 inches
d = deck thickness = 8 inches

$I_{LS} = 75(8^3)12 = 3200$ inches^4

Calculate Negative Moment Caused by Girder Rotation:
$M = 2\ E_c\ I_{LS}\ \Theta\ /\ L$

Where: L = total length of the link slab = 37.29 inches + 44.80 inches = 82.09 inches

Span 1 Side:
M_1 = [2(3640ksi)(3200 inches^4)(0.00275)/82.09 inches]/(12 inches/ft)
= 65.03 k-ft

Span 2 Side:
M_1 = [2(3640ksi)(3200 inches^4)(0.00250)/82.09 inches]/(12 inches/ft)
= 59.12 k-ft

Controlling Moment = Larger of M1 or M2 = 65.03 k-ft

Step 2: Check Cracking in the Link Slab.

Modulus of Rupture for the Link Slab
$fr = 0.24(f'c)^{0.5}$
$= 0.24(4)^{0.5}$
$= 0.48$ ksi

Cracking Moment for Link Slab:
$Mcr = fr\ I_{LS}/\ y_t$

Where: y_t = half depth of slab = 4 inches

Mcr = [0.48(3200)/4/](12 inches/ft) = 32 k-ft < 65.03 k-ft

Negative moment exceeds the cracking moment of the link slab. Additional flexure reinforcement required.

Step 3: Design of Flexure Reinforcement

The approach recommended in the reference [30] for the design of the longitudinal link slab reinforcement is to limit the tensile stress in the bars to 40 percent of the yield stress of the bars. Following this check, the Crack Control Serviceability Check should be made to ensure that the cracks caused by girder end rotation are within acceptable parameters. The reference used a previous AASHTO equation that has since been superseded by a different equation. For this example, the new AASHTO LRFD Section 5.7.3.4 [5] will be used.

Figure 12.2.2.2-3 is a sketch depicting the theoretical stress and strain diagrams of the concrete deck when exposed to girder end rotations.

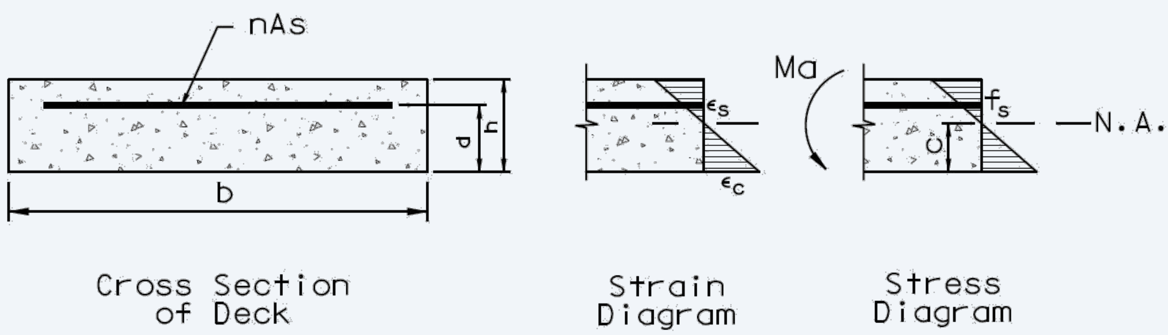


Figure 12.2.2.2-3. Link Slab Stress and Strain Diagrams.

Calculation of Steel Stress

Ratio of tension reinforcement:

$\rho = As / (bd)$

From working stress moment equilibrium theory (found in most concrete text books):

$fs = Ma / As(jd)$

Where:
Ma = moment acting on section

$jd = d\ (1- k/3)$

$k = [(\rho n)^2 + 2\rho n]^{0.5} - \rho n$

n = modular ratio

Calculate fs for this example:

Givens:
fy = 60 ksi
Longitudinal bar size = #5 (As = 0.31 inches^2)
Assume transverse bars are #5 (used to calculate γ)
Top cover = 2 inches
Slab thickness = h = 8 inches
Beam Spacing = b = 75 inches
$f_{allow} = 0.4fy = 0.4(60) = 24$ ksi

Try #5 spaced at 3 inches
As = 75/3 * 0.31 inches^2 = 7.75 inches^2

d = (8 inches – 2 inches – 0.625 inches – 0.625 inches/2) = 5.06 inches

$\rho = As / (bd) = 7.75/ (75(5.06)) = 0.0204$

n = 8 (f'c = 4 ksi)

Ma = 65.03 k-ft = 780.4 k-in (see previous calculations)

$k = [(0.0204(8))^2 + 2(0.0204)(8)]^{0.5} - 0.0204(8)$
$= 0.431$

jd = 5.06 inches(1- 0.431/3)
= 4.33 inches

$f_s = 780.4 / (7.75)(4.33)$
$= 23.24$ ksi $< f_{allow} = 24$ ksi **OK**

Check AASHTO LRFD Crack Control Criteria (S5.7.3.4):

Bar spacing $= s \leq 700\gamma_e/(\beta_s f_{ss}) - 2d_c$

Where:
$\beta_s = 1 + d_c / (0.7(h - d_c))$

d_c = concrete cover measured from extreme tension fiber
to center of flexural reinforcement
= 2 inches + 0.625 inches + 0.625 inches/2
= 2.94 inches

γ_e = exposure factor
= 0.75 for extreme exposure (bridge decks exposed to water)

f_{ss} = tensile stress in steel reinforcement at the service limit state
= 23.22 ksi

h = overall thickness or depth of element = 8 inches

$\beta_s = 1 + d_c / (0.7(h - d_c))$
$= 1 + 2.94 / (0.7(8 - 2.94))$
$= 1.83$

$s \leq 700\gamma_e/(\beta_s f_{ss}) - 2d_c$

3 inches $\leq 700(0.75)/((1.83)(23.22)) - 2(2.94) = 12.36$ inches $- 5.88$ inches $= 6.48$ inches

3 inches $\leq$ 6.48 inches **OK**

Use #5 spaced at 3 inches for the top longitudinal reinforcement.

CHAPTER 13. MATERIAL SPECIFICATIONS

Materials specifications vary from agency to agency; therefore, it is not possible to present precise specification language in this manual. Instead, this chapter will present information that should be included in specifications for materials used on ABC projects with prefabricated elements.

13.1. HIGH EARLY STRENGTH CONCRETES

Many details used throughout the country involve the use of CIP concretes. The timeline for construction will drive the need for concretes that can attain strength quickly. There are several approaches for materials specifications that can be used to allow the construction of the bridge to be expedited.

13.1.1. Use of Higher Final Strength Mixes

The nature of concrete strength gain is that a majority of the strength gain occurs in the early hours after final set. If a higher strength concrete is used, the design strength can be achieved quickly during the early strength gain of the mix. Figure 13.1.1-1 shows a generic concrete strength gain curve for two high early strength concrete mixes. The blue line might represent a 5,000 psi mix, and the red line might represent a 10,000 psi mix.

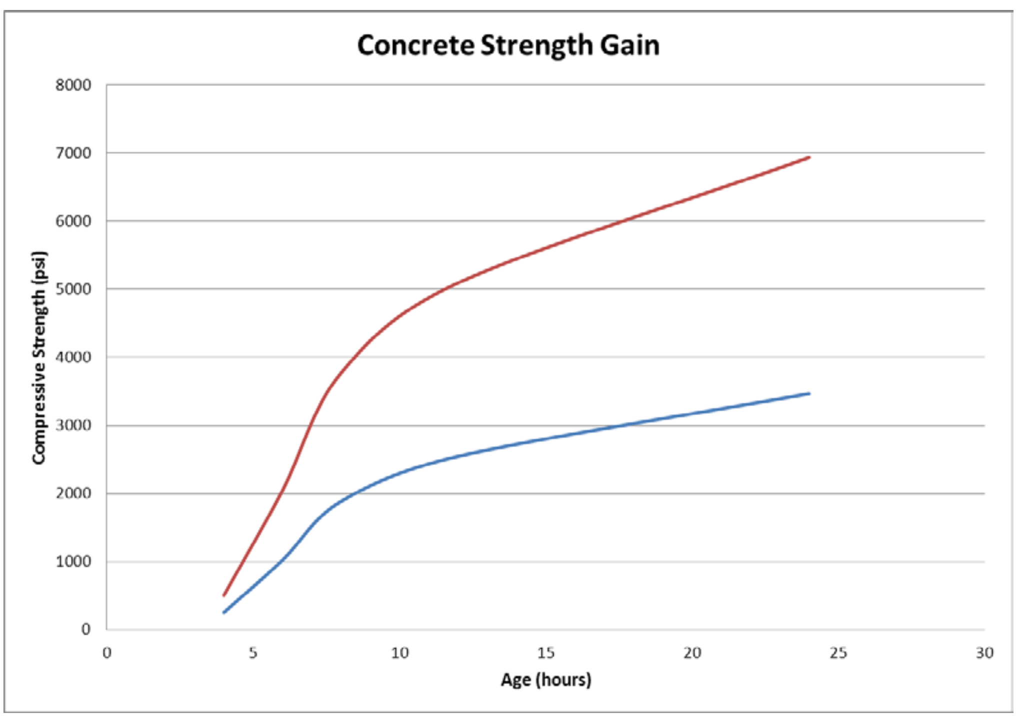


Figure 13.1.1-1. Generic Early Age Concrete Strength Gain Plot.

If a design required a compressive strength of 4 ksi, it would take approximately two days for the 5 ksi mix to attain the required strength, while the 10 ksi mix might attain the same strength in approximately eight hours.

13.1.2. Design of the Connection for a Lower Initial Strength

Section 4.2.7 contains an example of how to design a closure pour for a concrete strength that is lower than the specified concrete strength. For example, a CIP concrete closure pour can be designed for a compressive strength of 2.5 ksi, even though the specified final strength is 4 ksi.

This approach takes advantage of the steeper slope of the strength gain curve during early age curing. For example, if a closure pour design could be redesigned from a compressive strength of 4 ksi to a compressive strength of 2.5 ksi, the time to reach this strength can change from approximately 2 days to 11 hours using the same concrete mix.

13.1.3. Material Specifications for High Early Strength Concretes

Regardless of the approach used to accommodate construction schedules, there will be a need to specify rapid strength gain concretes on some ABC projects. Chapter 2 of this manual includes information on concrete materials used for typical projects.

The type of concrete specification used is a function of the timeline for construction. If a project is accelerated from one year to six weeks, it may be appropriate to use standard prescriptive concrete specifications. This is especially true for concrete pours made during the early construction steps (footings, and substructures). The designer should consult with materials engineers to determine the strength gain characteristics for these standard mix designs, and then investigate whether or not they can be used in the anticipated construction schedule.

If a standard mix design cannot fit within the construction timeline, a high early strength concrete mix would be required. Most agencies do not have standard mixes that can attain strength rapidly. The reason for this is that there are a number of different ways to design a concrete mix to attain high early strength. Some of the methods and materials that can achieve high early strength include the use of (Source: Design and Control of Concrete Admixtures [77]):

- Type III High Early Strength Cement.
- High Cement Content (600 to 1,000 lb/cy).
- Reduced water/cement ratio (0.2 to 0.45 by weight).
- Higher freshly mixed concrete temperature.
- Higher curing temperature.
- Chemical admixtures.
- Silica fume.
- Steam curing.
- Insulation to retain heat of hydration.
- Special cements.

These methods and materials can be used alone or in combination in order to achieve the desired results. The problem with this is that there potentially could be hundreds of different ways to develop a particular mix design. For this reason, most ABC projects make use of performance based specifications. These are specifications that do not contain actual mix design proportions. Instead, they contain certain performance parameters that are required for the mix to be acceptable. Designers may consider specifying the following items in a performance specification:

- Final concrete compressive strength.
- Interim concrete compressive at a certain age.
- Maximum aggregate size for placement.
- Durability requirements (air content, permeability).
- Materials and admixtures that are allowed and not allowed.
- Special curing methods.
- Special testing requirements.

In order for this method of specification to be practical, the owner agency should identify what materials and admixtures are acceptable for use. Many agencies have approved products lists for this purpose; however, some of the admixtures that may be used in high early concrete mixes may not be commonly used in the agency. For this reason, the design engineers need to coordinate closely with the agency materials engineers to ensure that the appropriate materials are acceptable.

If performance specifications are used, the agency should be aware of the need to review and approve mix designs. Requirements for mix design testing and submissions should be clearly stated in the specifications. Timelines for review and approval of mix designs should also be identified in the specifications.

Some agencies are looking to develop long-term ABC programs. These agencies may consider the development of standard high early strength concrete mixes for use. The agency may look to work with local concrete suppliers to develop these mixes. This can be problematic for an agency without a defined program that requires approval of standard mix designs on a regular basis. Without a program, the producers may be resistant to making the capital investment in mix design approvals unless there is a potential to recoup these development costs. If standard mix designs are developed, the agency should share the strength gain characteristics with the design community so that appropriate project schedules can be developed.

13.2. GROUTS

Most engineers refer to grouts as “non-shrink grouts.” This is based on the fact that many grouts were initially developed for placement under column baseplates and machine bases. Grouts used in bridge applications are often called upon to serve different purposes. Chapter 2 contains information on different grouts that can be used in bridges built with prefabricated elements. The grouts used for these purposes often require:

- High early strength gain.
- Low shrinkage.
- Low viscosity (high flow-ability).
- Bond strength.
- Durability.

Some agencies have standard prescriptive grout mixes. These may be acceptable for some applications; however, they may not be acceptable in many ABC projects due to the high performance requirements. Most high performance grouts are sold in pre-packaged bags. The bags can vary in size, but they all are pre-proportioned to meet the requirements of the particular mix. This approach is desirable since workers are not required to carefully proportion materials and admixtures. In many cases, the construction crews simply need to add water.

The use of pre-packaged grouts has led agencies to pre-qualify acceptable grouts. Project specifications simply refer the contractor to the agency-approved products list of acceptable grouts. The development of approved products lists needs to be carefully coordinated with the intended construction methods and schedule. There are many different grouts in the market, andthey all perform differently. Agencies should develop separate lists of approved products for the anticipated grout uses. For instance, one list of approved grouts may be acceptable for grouting substructure connections, but not appropriate for grouting deck voids that require high flowability. Agencies should evaluate each grout based on a predetermined set of parameters for each use. Once a list is established, contractors will be able to make use of the grouts without an extensive approval process.

The methods used to mix and place the grouts can vary from one grout type to another. Surface preparation and weather limitations also vary. Project specifications should clearly indicate that the contractor needs to follow the manufacturer’s written specifications for these processes. Any limitations and special handling requirements should be identified in the contractor’s assembly plan submission. Chapter 11 has more information on specification requirements for bridge assembly plans.

13.2.1. Non-shrink Grout for Connections

The term non-shrink grout is a misnomer. There is no such thing as a “non-shrink” grout. Grouts that fall within this category exhibit “negligible” shrinkage. The term “non-shrink” implies zero length or height change over time. In reality, the dimensions of a volume of a grout will change during the curing process. The goal is to have a grout that is as close as possible to dimensionally stable from casting through final condition. Problems could occur if a grout expands while plastic then shrinks back to its original volume after set. Typically, the expansion occurs in only in the unrestrained directions; however, contraction occurs in all directions. Most cementitious materials will shrink during curing. Non-shrink grouts are designed to overcome this by several different means. In some cases, the grouts will actually expand during early curing and then contract during final curing, resulting in a net volume change that is negligible.

In some cases, connections between prefabricated elements will be exposed to severe environments. This is especially true for connections in bridge decks. In these cases, it is desirable to have a grout material that has negligible shrinkage.

ASTM C 827—Standard Test Method for Change in Height at Early Ages of Cylindrical Specimens of Cementitious Mixtures [78] is commonly used for the measurement of grout height change. The description of this test that is supplied by the American Society for Testing and Materials is as follows:

This test method provides a means for comparing the relative shrinkage or expansion of cementitious mixtures. It is particularly applicable to grouting, patching, and form-filling operations where the objective is to completely fill a cavity or other defined space with a freshly mixed cementitious mixture that will continue to fill the same space at time of hardening. It would be appropriate to use this test method as a basis for prescribing mixtures having restricted or specified volume change before the mixture becomes hard.

This test method can be used for research purposes to provide information on volume changes taking place in cementitious mixtures between the time just after mixing and the time of hardening. However, the specimen used in this test method is not completely unrestrained so that the measurements are primarily useful for comparative purposes rather than as absolute values. Further, the degree of restraint to which the specimen is subjected varies with the viscosity and degree of hardening of the mixture.

This test method is described for use with cementitious mixtures. There are also epoxy grouts in the market. Many agencies accept this test for epoxy grouts as well. This testing specification is used in conjunction with ASTM C 1107—Standard Specification for Packaged Dry, Hydraulic-Cement Grout (Non-shrink) [68].

It is important to note that designers and agencies need to identify the minimum and maximum height change that is acceptable for a grout material. One grout manufacturer recommends that limits of one percent minimum and four percent maximum be used for this purpose.

Specifying grouts using these specifications does not necessarily guarantee acceptable performance. The reason for this is that the test specimens are cured in an unrestrained condition; therefore, the test may not be indicative of grout placed in a confined space. ASTM C 827 is an effective means of measuring the relative shrinkage of various grouts. Agencies may wish to further test grouts in actual voids that are indicative of anticipated field conditions. Once these tests are evaluated, an appropriate approved product list can be finalized and referenced in project specifications.

In some cases, tensile strength is more important than grout change in length. For example, dovetail shear pockets in precast bridge decks would require relatively high pull-out capacity that is better represented by tensile strength of the grout material. It would be advisable in some cases to state that the tensile strength at seven days may not be less that 2,000 psi.

Special care should be taken in considering the volume of the void that will be filled with the grout. Pre-bagged grout manufacturers normally recommend the largest width of joints or volumes of voids that may be filled with each particular grout. Provisions for adding aggregates for larger volumes are typically recommended for larger voids.

Grouts specifications should include testing requirements. The grouts should be sampled on site and tested as appropriate. The following testing should be considered in project specifications:

1. Strength testing: ASTM C109—12 Standard Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-inch or [50-mm] Cube Specimens) [89].
2. Flow testing for grouts requiring flowability: ASTM C1437—07 Standard Test Method for Flow of Hydraulic Cement Mortar 87 [90].
3. Fluidity testing for grouts requiring fluidity: ASTM C939—10 Standard Test Method for Flow of Grout for Preplaced-Aggregate Concrete (Flow Cone Method) [91].

The designers should prescribe the testing requirements for each grout in the project specifications.

13.2.2. Grout for Post-Tensioning Systems

The FHWA has previously published a manual entitled “Post-Tensioning Tendon Installation and Grouting Manual” [80]. This manual contains significant information on post-tensioning and grouting of ducts.

The manual covers the following topics related to grout for post-tensioning applications:

- Post-Tensioning System Materials and Components
- Grouting of Post-Tensioning Tendons
- Personnel Qualifications

The document provides significant guidance on the subject of grouting, including information on materials specifications. The following introductory text is taken from the FHWA Manual [80]:

Grouts made of cementitious materials, water and admixtures batched on site do not always have uniform properties. This arises from variations in materials, day to day mixing differences, crew changes, weather conditions and so forth. Grouts made of only cement and water often exhibit segregation and voids due to excessive bleed water. In an endeavor to eliminate problems related to grout variations and voids, several agencies have obtained greater quality control by requiring “pre-bagged” grouts. In a pre-bagged grout, all the constituent (cementitious) materials have been thoroughly mixed and blended at the factory in the dry condition. This ensures proper blending and requires only that a measured amount of water be added for mixing on site.

A manufacturer of a pre-bagged grout may already have had the material pre-qualified by a State DOT or other agency. In this case, it is appropriate to accept it on the basis of a written certification; providing that the manufacturer has on-going quality control tests that can be confirmed by submitting test reports to the Engineer. The certification should show the mixed grout will meet the pre-qualified standard. On site, daily grout production must be monitored by various field tests in order to maintain quality control and performance.

Acceptance of a grout is usually based upon the results of laboratory tests. Laboratory tests on trial batches of the proposed grout using the same materials and equipment to be used on site are used to qualify a grout. Trial grout should be prepared by personnel experienced in preparing and testing grout mixes. This should be done at an approved material testing laboratory. All tests should be performed at temperature and humidity conditions expected on site. Trials should precede construction by at least eight weeks in order to allow time for testing and resolution of any concerns.

A proposed grout is normally accepted on the basis of the laboratory tests listed in the following sections, performed before construction, or on the basis of certification from the manufacturer that the (pre-bagged) grout materials meet the pre-qualification requirements of the owner or project specifications. Details of the tests to be performed are provided in summary fashion. This is a summary of the key aspects only. For further details refer to the "Specification for Grouting of Post-Tensioned Structures" [81], latest edition, by the Post-Tensioning Institute (PTI) and/or the specific project contract documents.

The manufacturer should have a continuing quality control program to ensure that production continues to meet the specified requirements. Copies of certificates should be checked and a record kept by the contractor and the inspector. Use of a particular grout on site may continue, providing that certification and documentation is kept up to date, that materials in storage remain usable, and that daily grout mix production tests meet specified limits. Approval to use a grout should be withdrawn if these quality control standards are not maintained.

The "FHWA Post-Tensioning Tendon Installation and Grouting Manual" [80] contains recommendations for grout pre-qualification testing including:

- Setting time.
- Grout strength.
- Permeability.
- Volume change.
- Pumpability and fluidity.
- Simulated field high temperature fluidity testing.
- Bleed.
- Corrosion.
- Wet density.

Other documents are available that can be used to develop materials specifications and testing of post-tensioning grouts. The specification entitled "Specification for Grouting of PT Structures"

[81] is a resource that can be used to develop project specifications or included as a reference in a project specification.

13.3. POST-TENSIONING SYSTEMS

The AASHTO LRFD Bridge Design Specifications contains information on the design of post-tensioning anchorages. The basis of these specifications is that the anchorage assembly is made up of steel plates that are embedded in the elements. In practice, most post-tensioning systems are made with steel castings that are specifically designed for post-tensioning applications. These castings do not necessarily apply to the AASHTO specification, since they are not made with flat plate.

The specification of post-tensioning systems normally follows a pre-qualification process where the systems are evaluated prior to or during construction. The evaluation can be done using previous verification testing of the systems. The PTI has published a specification entitled "Acceptance Standards for Post-Tensioning Systems" [83]. The following text is taken from the PTI website description of this specification:

This publication provides specific technical requirements for the approval and acceptance of post-tensioning systems. Standards and performance requirements for prestressing materials, bearing plates, wedge plates, connections, and sheathing are discussed in detail. Qualification tests and acceptance criteria are presented for each of the individual components as well as for the complete system. A system approval summary outlines the test requirements and number of successful tests necessary for approval of a post-tensioning system. This document is not intended to cover unbounded, monostrand post-tensioning systems, which have their own separate specification.

Corrosion protection of post-tensioning tendons is of paramount importance. The "FHWA Post-Tensioning Tendon Installation and Grouting Manual" [80] contains recommendations for corrosion protection. The design engineer should specify the required level of post-tensioning protection. There are multiple ways to establish a certain level of corrosion protection:

- Exterior concrete surface coatings.
- Concrete cover.
- Corrosion resistant ducts.
- Grout in ducts.
- Corrosion resistant strand or bar.
- Protection of anchorage assemblies.
    - Grout – a fully filled tendon, anchor, and grout cap.
    - Permanent grout cap of inert (plastic) material.
    - Concrete pour-back to encapsulate the grout cap and anchor plate.
    - Full encasement of the end of an I-girder within a reinforced concrete diaphragm.
    - Encasement of an anchorage under a deck slab along with sealing of construction joints with an approved sealant (e.g. methyl-methacrylate or similar).

- Application of an approved seal coat or sealant to an anchor pour-back.
- A surrounding enclosure of a watertight and drained hollow box.
- Appropriate application of wearing surface overlays.
- Appropriate details at expansion joints to prevent leaks and ingress of water.

Bridge designers should choose to specify a certain level of corrosion protection by specifying certain materials and limiting others. Three levels of corrosion protection are common. They typically consist of:

1. Adequate concrete cover.
2. Corrosion resistance duct.
3. Grouted ducts.

An additional level of corrosion protection can be added for elements exposed to severe environments. This may include coated strand, high performance concrete, and concrete coatings.

13.4. FLOWABLE FILLS

Flowable fill or controlled density fill is a material that is used to backfill behind structures and under footings. It is a simple material that replaces standard backfill materials with a cementitious sand mixture. Fly ash is commonly used to improve workability.

Specifications for flowable fills are similar to concrete specifications. Several state agencies have developed standard prescriptive specifications. The following pages contain sample specifications from MassDOT and Utah DOT. Other state agencies have similar specifications for these materials. The MassDOT specification was chosen because the specification contains mixes for flowable fills of various strengths and flowability. Flowable fills normally exhibit significant shrinkage during curing. For thin pours (bedding for footings or slabs) this is not a problem. The impacts of the shrinkage must be accounted for in the design of thicker pours.

Massachusetts DOT Specification for Controlled Density (Flowable) Fill:

Controlled Density Fill (CDF) material is a flowable, self-consolidating, rigid setting, low-density material that can substitute for compacted gravel in backfills, fills, and structural fills. There are two main categories of CDFs—excavatable and non-excavatable, with subcategories of flowable and very flowable. It shall be a mixture of portland cement, fly ash (if very flowable), sand, and water designed to provide strengths within the range specified.

The categories of CDFs are:

Type 1: Very Flowable (Non Excavatable).
Type 1E: Very Flowable (Excavatable).
Type 2: Flowable (Non Excavatable).
Type 2E: Flowable (Excavatable).

The Very Flowable mixes (Types 1 and 1E) shall contain a minimum of 250 lbs of Class F Fly Ash or high air (25 percent plus) and will be self-leveling.

Excavatable mixes (Types 1E and 2E) shall be hand-tool excavatable.

Type 1 mixes are intended for permanent installations such as structural fills under structures. It has very flowable characteristics needed for distances and small areas. This type of mix should be not be used as a bedding material. It is used to fill small hard-to-reach areas.

Type 1E mixes are excavatable material designed to have very flowable characteristics needed for filling small or far areas that later may need to be removed.

Type 2 mixes are used in areas where size and distance do not need the very flowable characteristic. It is intended for permanent installations such as thick fills under structures.

Type 2E mixes are excavatable mixes where size and distance of the installation do not require the flowable characteristics of a Type 1E mix.

CDF is to be batched at a ready mix plant and is to be used at a high or very high slump of approximately 10 inches to 12 inches (250 mm to 300 mm). It shall be flowable, require no vibration, and after it has been placed can, for Types 1E and 2E, be excavatable by hand tools and/or small machines.

The ingredients shall comply with the following:

Portland Cement: AASHTO M 85.
Fly Ash: AASHTO M 295. Class F
Sand: See Table Below
Air entraining admixtures: AASHTO M-154

The sieve analysis of the sand shall show it to be well graded and conforming to the following:

Sieve Designation	Minimum	Fine Aggregate Maximum
3/8 inches	100	
No. 4	95	100
No. 16	45	80
No. 50	10	30
No. 100	2	10
No.200	0	3

Notes:

- In lieu of the slump test, a 6-inch-long, 3-inch-diameter tube may be filled to the top and then slowly raised. The diameter of the resulting "pancake" may be measured and the range of the diameter shall be 9 inches to 14 inches.
- The maximum for structural flowable fills may be in the 1,000s of psi and will depend on the engineer's requirements.
- High air may be used instead of fly ash with an adjustment in sand content.

The following Type 1 and Type 1E mix designs are for information only; the actual mix designs submitted by the ready mix operator, in accordance with standard Department practice, must be confirmed by trial batches.

Material	Type 1 Mix Design	Type 1E Mix Design
Cement	100 lbs	50 lbs
Fly Ash	250 lbs	250 lbs
Sand	2,650 lbs	2,700 lbs
Water	60 gallons	60 gallons

Various types of controlled density fill must meet the requirements set forth in the table below:

Controlled Density Fill	Type 1 & 2	Type 1E & 2E
Compressive Strength @ 28 days	30-150 psi	30-80 psi*
Compressive Strength @ 90 days	200 psi maximum	100 psi maximum
Slump	10-12 inches	10-12 inches

* May be changed by Design Engineer to fit particular job requirements.

Utah DOT Specification for Flowable Fill:

FLOWABLE FILL

PART 1 GENERAL

1.1 SECTION INCLUDES

A. Materials and procedures for placing flowable fill.

1.2 RELATED SECTIONS

A. Section 03055: Portland Cement Concrete

1.3 REFERENCES

A. AASHTO M 154: Air-Entraining Admixtures for Concrete

B. AASHTO M 194: Chemical Admixture for Concrete

C. ASTM D 4832: Preparation and Testing of Controlled Low Strength Material (CLSM) Test

1.4 DEFINITIONS Not Used

1.5 SUBMITTALS

A. Batch Proportions: Submit to Engineer seven days before placement.

B. Trial Batch:
1. Submit certified test results or conduct laboratory trial batch to verify strength prior to placement.

PART 2 PRODUCTS

2.1 MATERIALS

A. Cement: Refer to Section 03055 Portland Cement Concrete.

B. Pozzolan: Refer to Section 03055 Portland Cement Concrete.

C. Sand.

D. Coarse aggregate: Determine a suitable aggregate size and gradation for the intended application.

E. Admixtures:
1. Water reducers and set accelerators: AASHTO M 194.
2. Air entrainment: AASHTO M 154.

PART 3 EXECUTION

3.1 INSTALLATION

A. Combine materials to meet the requirements for strength and constructability as required. Determine strength from trial batches at 28 days.
1. Minimum strength: 50 psi. ASTM D 4832.
2. Maximum strength: 150 psi. ASTM D 4832.
3. Slump: 5 inches to 10 inches.

B. Determine a suitable aggregate size and gradation for the intended application.

13.5. ULTRA HIGH PERFORMANCE CONCRETE

Section 2.5.1 contains information on Ultra High Performance Concrete (UHPC). UHPC is currently a proprietary material that comes in pre-packaged bags. It requires special mixing equipment to obtain the best results.

There are several items that limit the widespread use of UHPC. One of the materials used to achieve the strength characteristics is high strength steel fibers that are used in the mix. The fibers currently demonstrated to perform appropriately are not currently manufactured in the U.S.; thus, in some cases it is necessary that the designer to limit the volume of UHPC required in order to abide by the “Buy America” requirements in Federal procurement regulations. At this time, there is only one North American company that manufactures the product for use in bridge infrastructure projects in the U.S. There are other companies in Europe; however, the use of these products can be problematic in light of Federal procurement regulations. It is anticipated that some or all of these items will be resolved in the near future. Designers who wish to specify this material should contact FHWA for information on the current status of this product.

When approved for use, UHPC most likely will be treated as a specialty proprietary product. These types of products would normally be approved for use by an agency and listed in an approved product list.

13.6. GROUTED REINFORCING SPLICE COUPLERS

Chapter 5 of this manual contains information on the use of these couplers on the design and detailing of the bridge. Grouted reinforcing splice couplers are no different than other mechanical reinforcing spice devices. Article 5.11.5.2.2 of the AASHTO LRFD Bridge Design Specifications [5] contains information on the strength requirements of the couplers. Figure 13.6-1 contains an excerpt from the AASHTO article.

5.11.5.2.2 Mechanical Connections

The resistance of a full-mechanical connection shall not be less than 125 percent of the specified yield strength of the bar in tension or compression, as required. The total slip of the bar within the splice sleeve of the connector after loading in tension to 30.0 ksi and relaxing to 3.0 ksi shall not exceed the following measured displacements between gage points clear of the splice sleeve:

- For bar sizes up to No. 14…………….. 0.01 in.
- For No. 18 bars……………………….. 0.03 in.

C5.11.5.2.2

The stress versus slip criteria has been developed by the California Department of Transportation.

Types of mechanical connectors in use include the sleeve-threaded type, the sleeve-filler metal type and the sleeve-swaged type, of which many are proprietary, commercially available devices. The contract documents should include a testing and approval procedure wherever a proprietary type of connector is used.

Basic information about the various types of proprietary mechanical connection devices is given in ACI 439.3R (*1991*).

Figure 13.6-1. AASHTO LRFD Provision for Mechanical Reinforcement Connections.

The ACI Building Code [46] contains similar text, except that a second level of mechanical coupler is allowed that can develop 100 percent of the specified minimum tensile strength of the connected bar. This equates to 150 percent of the specified minimum yield strength for a typical grade 60 bar. Some designers and agencies are using this more stringent criterion for major bridge elements.

Mechanical couplers are proprietary products; therefore, agencies normally use a pre-qualification process where the devices are evaluated prior to or during construction. The evaluation can be done using previous verification testing of the systems. Once this is done, an approved products list can be developed and used in project specifications.

The following page includes specification language text that has been used for some MassDOT projects. It should be noted that this language is not standard at this time. It will be modified as projects are constructed and lessons are learned.

Example 13.6-1. Sample Materials Specification Language for Grouted Reinforcing Splice Couplers.

Grouted Splice Couplers:

Use grouted splice couplers to join precast elements as shown on the plans. The coupler shall be specifically designed to join reinforcing steel from within a precast concrete element to projecting bars from an adjacent precast concrete element. The couplers shall use cementitious grout placed inside a steel casting to develop the strength of the connection. Threaded connections may be used for the portions of the coupler that are placed within the precast element if the strength of the coupler meets or exceeds the requirements of this specification.

Couplers shall be pre-qualified based on the requirements of this specification. Pre-qualification shall be based on certified test reports for the specific coupler and grout proposed for use. The following grouted splice couplers may acceptable for use provided that the requirements of this specification are met.

NMB Splice Sleeve
Splice Sleeve North America, Inc.
192 Technology Drive, Suite J,
Irvine, California 92618-2409
www.splicesleeve.com

Dayton Superior DB Grout Sleeve
Dayton Superior
Corporate Headquarters
7777 Washington Village Dr., Ste. 130
Dayton, OH 45459
www.daytonsuperior.com

Erico Lenton Interlok
ERICO United States
34600 Solon Road
Solon, Ohio 44139
www.erico.com

The grouted splice couplers shall be capable of developing 100 percent of the specified minimum tensile strength of the connected bar. This equates to 90,000 lbs per square inch for reinforcing bars conforming to ASTM A615.

The grout used for the inside of the couplers shall be supplied by the manufacturer of the coupler. The grout shall be matched with the coupler and shall be the same grout as the certified test report for the coupler. No other grout shall be substituted in the couplers unless additional certified test reports are submitted for the grout/coupler system.

CHAPTER 14. CONSTRUCTION SPECIFICATIONS

This chapter covers the requirements for construction specifications for certain products and materials that are common to ABC projects built with PBES. One of the most common products used in these projects is prefabricated elements made with either steel or precast concrete. Most agencies already have very detailed standard specifications for these elements; therefore, they will not be covered in detail here.

14.1. PROPRIETARY PRODUCTS

Several proprietary products are in regular use on ABC projects with PBES. In most cases, multiple manufacturers can provide equal products; therefore, Federal procurement regulations can be met. The following sections contain information on the use of these products.

14.1.1. Mechanical Couplers

Mechanical couplers connect separate pieces of reinforcing steel in concrete. They consist of various threaded and wedge systems that mechanically transfer the force from one bar to another. Most agencies have approved product lists for these devices.

Grouted couplers are a type of mechanical coupler and can be used within an element without significant change to the design and detailing. Chapter 5 of this manual contains information on the impact of these couplers on the design and detailing of the bridge. Design Example 12.1.4 in Chapter 12 contains an example of a design using these couplers.

Specifications can include simple requirements to “follow manufacturer’s installation procedures.” Another approach could be to have the contractor’s erection engineer specify the installation procedure as part of the “Assembly Plan” submission (see Section 14.2). This is commonly done with proprietary products like bagged grouts and common construction procedures like erection of beams. The design engineer does not specify exactly how to build using these materials and techniques. The engineer reviews the materials and procedures during the submittal review process.

These approaches to project specifications may be sufficient in the future after more widespread use of these products becomes commonplace. In the meantime, design engineers should give guidance as to the proper procedures for the installation of grouted couplers. The following pages include specification language text that can be used as the basis of specification for grouted couplers.

Example 14.1.1. Sample Specification Language for Installation of Grouted Splice Couplers.

Basis: Text is based on a recent Massachusetts DOT project construction special provision.

Connection Procedure Using Grouted Splice Couplers:
Use personnel who are familiar with installation and grouting of splice couplers and who have completed at least two successful projects in the last two years. Training of new personnel within three months of installation by a manufacturer's technical representative is an acceptable substitution for this experience. Remote training via internet is acceptable in lieu of on-site training.

Remove and clean all debris from the connections prior to application of non-shrink grout. Keep bonding surfaces free from laitance, dirt, dust, paint, grease oil, or any contaminants other than water.

Saturate Surface DRY (SSD) all connection surfaces prior to connecting the elements.

Use heaters in freezing temperatures to maintain a minimum temperature of 50 degrees F. Monitor the temperature of the covered sleeves until the temporary bracing is removed.

Follow the recommendations of the manufacturer for the installation and grouting of the couplers. The installation shall generally be as follows:

- Determine the thickness of shims to provide the specified elevation within tolerance. It is recommended that the projecting reinforcing bars from the adjacent element be cast longer than required and cut to length in the field after the top of shims have been set. Follow the manufacturer's recommendations for the projection length of the bars measured from the top of the shims to the top of the bars.

- Mix the non-shrink grout according to the supplier's recommendations, including preparation and application.

- Place non-shrink grout on the interface between the two elements being joined prior to setting the element. Crown the thickness of the grout toward the center of the connection so that the grout can be displaced outward as the element is lowered onto the connection. Take precautions to prevent the non-shrink grout from entering the coupler above (e.g. grout dams or seals).

- Set the element in place. Engage all couplers in the connection. Allow the non-shrink grout to seep out of the connection.

- Trowel off excess non-shrink grout to form a neat connection once the element is set, plumbed, and aligned. Pack grout into any voids around the connection perimeter.

- Flush out the coupler with clean potable water.

- Mix the special coupler grout according to the manufacturer’s recommendations for methods and proportions of mix and water.

- Make four sets of three 2-inch-cubed specimens for testing. Cure the specimens according to AASHTO T 106. Test one set of cubes for compressive strength at a minimum of 24 hours (or to determine when to release bracing) and 28 days. Store extra sets for longer term testing, if necessary.

- Pump the coupler grout into the coupler that is cast into the element. Start from the lower port. Pump until the grout is flowing freely from the upper port.

- Cap the upper port first and then remove the nozzle to cap the lower port. Proceed to the next coupler in a defined sequence.

- Cure the connection according to the non-shrink grout manufacturer’s recommendations.

Until the use of grouted couplers becomes more commonplace, agencies may choose to include provisions for a test mockup of a grouted coupler connection. The purpose of this mockup is to provide a hands-on training exercise on a connection that is similar to the actual bridge detail. Construction management personnel have noted that the grouting of the couplers is a relatively simple process, but there is a benefit to practicing on one connection before the actual construction. The connection does not necessarily need to be as large as the actual construction; however, it is recommended that the connection contain multiple couplers and have a similar grout detail. The project specifications should include either a description of the test or a detail. It is not necessary to include the detail in the project plans, since it is not part of the completed construction.

14.1.2. Post-Tensioning Systems

The FHWA has previously published a manual entitled "Post-Tensioning Tendon Installation and Grouting Manual" [80]. This manual contains significant information on post-tensioning systems.

The manual covers the following topics related to construction of post-tensioning systems:

- Post-tensioning system materials and components.
- Post-tensioning duct and tendon installation.
- Grouting of post-tensioning tendons.
- Personnel qualifications.
- Examples of post-tensioning tendon applications.
- Corrosion protection of post-tensioning tendons.

The document provides significant guidance on construction methods and testing during construction. The following is a list of recommended provisions for contract specifications.

- Field Mockup Testing.
- Pumpability and Fluidity (flow cone).
- Simulated Field High Temperature Fluidity Test (optional).
- Wick Induced Bleed Test.
- Wet Density Test.
- Schupack Pressure Bleed Test.
- Production Bleed Test—prior to injection.
- Normal, Non-Thixotropic, Grout Testing—prior to injection at inlet.
- Thixotropic Grout Testing—prior to injection at inlet.
- Normal, Non-Thixotropic, Grout Testing—discharge at final outlet.
- Thixotropic Grout Testing—discharge at final outlet.
- Testing Prior to Injection of Grout.
- Pumping.
- Limiting Grout Injection Pressures.
- Grout Flow Rate.

Other reference materials are available. Contract specifications can reference the following published standard specifications:

- "Guide Specification for Grouted Post-Tensioning" [82].
- "Specification for Grouting of Post-Tensioned Structures" [81].

Requirements for certification of post-tensioning workers can be included in construction specification. The American Segmental Bridge Institute (ASBI) has a certification program for grouting of post-tensioning systems. The ASBI website states that:

"The purpose of the training is to provide supervisors and inspectors of grouting operations with the training necessary to understand and successfully implement grouting specifications for post-tensioned structures."

Agencies may consider specifying that supervisors of construction crews and agency construction inspectors be certified under this program. Prior to this, the agency should coordinate with local contractor organizations to verify that they have the personnel to meet this specification. It should be noted that there is a significant effort to complete this training and the training is not held on a regular basis. Early coordination will also give contractors time to obtain the certification prior to advertising of projects. Planning a program of multiple projects that include post-tensioning will also help to offset the cost and effort required to obtain this certification.

14.2. ASSEMBLY PLANS

Section 11.2 of this manual contains information on the need for an assembly plan for prefabricated bridges. The intent of assembly plans is to demonstrate a method of construction that accounts for the site constraints, the equipment required, and the materials used. The assembly plan is similar to an erection plan, except that the strength gain of the connecting materials needs to be considered.

Typical materials specifications call for final strength of materials. For example, concrete is normally specified to achieve a 28-day compressive strength. On some ABC projects, 28 days represents the entire construction cycle; therefore, other options need to be considered. The reality of most connections is that the design strength is only required for the completed structure after it has been opened to traffic. During construction, it is possible to continue with the construction process with interim strength of materials. Construction specifications should allow for continuation of construction with partial interim strength materials. The contractor is still responsible for delivering materials that will achieve the full design strength; however, the contractor should be allowed to assemble the bridge with interim strengths. One of the key aspects of an assembly plan is to have the contractor's engineer determine the required interim strengths for each step of the assembly.

The following pages include specification language that can be used as the basis of specification for construction assembly plans.

Example 14.2. Sample Specification Language for Construction Assembly Plans.

Basis: Text is based on a recent Massachusetts DOT project construction special provision.

Assembly Plan:
The Assembly Plan is a document prepared and submitted by the contractor prior to the start of work that details the means to which the contractor will construct the bridge. The Assembly shall be prepared by and stamped by a registered P.E., with working knowledge of the contractor’s equipment, approved shop drawings, and selected materials to build the project.

The Assembly Plan submittal requires the contractor to detail the sequence of construction in accordance with the project schedule. This document will be treated as a Construction Procedure and will be reviewed by both the designer and the district construction office. The approval of this document will serve as a guideline to allow relaxation of the certain provisions of the standard specifications (for example, interim concrete strengths and curing procedures).

The following list details the minimum criteria that should be included in the Assembly Plan:

- A detailed schedule showing the sequence of operations that the contractor will follow. The schedule shall include a timeline for installation of all major elements of the bridge, accounting for the installation of temporary works and cure times of closure pour concrete and other selected materials such as grouts.

- Calculations that support the schedule outlined above should be included, verifying that the selected materials have adequate interim strength to proceed from one step to another. Final material strengths are not normally required until the bridge is opened to vehicular traffic. The minimum factor of safety of two (2) will be required for the interim strength of grouts and closure pour concrete before construction is allowed to proceed to subsequent steps. The factor of safety is applied to the service loads that are supported by the elements and materials during various stages of construction. For example, if the contractor calculates that the grout between the precast pier cap and pier wall requires a compressive strength of 100 psi in order to support the dead load of the PBU in the next step. A cylinder break of 200 psi will be required prior to allowing the pier cap to be loaded with the PBU. The required strength of materials for subsequent construction stages shall also be calculated and the material strength verified.

- The contractor is responsible for determining the center of gravity for all elements. Special care shall be used for unusual elements that are not symmetric. These elements may require special lifting hardware to allow for installation in a plumb or flat position.

- Include a work area plan, depicting items such as temporary earth support, utilities within the immediate vicinity of the work, drainage structures, etc. The contractor is required to coordinate the various subcontractors that will need to occupy the same area and ensure that there are no conflicts. For example, if the contractor is having different subcontractors prepare and submit plans for temporary earth support and demolition, and the earth support is required to be installed prior to the demolition, it is the contractor's responsibility to ensure that the Assembly Plan submission allows both operations to be performed without field modification.

- Include details of all equipment that will be employed for the construction of the bridge.

- Include details of all equipment to be used to lift elements, including cranes, excavators, lifting slings, sling hooks, and jacks. Include crane locations, operation radii, and lifting calculations. It is anticipated that the contractor will use the fabricator's lifting inserts, but this needs to be coordinated prior to approval of the precast shop drawings. Follow Chapter 8 of the PCI Design Handbook (seventh edition) for handling and erection bracing requirements.

- Include methods of providing temporary support of the elements. Include methods of adjusting and securing the element after placement.

- Include procedures for controlling tolerance limits—both horizontal and vertical.

- Include methods for curing grout and closure pour concrete.

- Include proposed methods for installing non-shrink grout and the sequence and equipment for the grouting operation.

- Include methods for placement of controlled density fill under precast approach slabs. Add additional grout ports in the footings to facilitate the bedding process if required.

- Submit full size 24x36-inch sheets depicting the assembly procedures for the precast elements.

- Include methods of forming closure pours including the use of backer rods. Do not assume that the backer rods will restrain the pressure from the grout in vertical grout connections. Provide additional forming to retain the backer rod.

14.3. INSTALLATION OF SYSTEMS

The FHWA definition of prefabricated systems is as follows:

Prefabricated Systems are a category of PBES that consists of an entire superstructure, an entire superstructure and substructure, or a total bridge that is procured in a modular manner such that traffic operations can be allowed to resume after placement. Prefabricated systems are rolled, launched, slid, lifted, or otherwise transported into place, having the deck and preferably the parapets in place such that no separate construction phase is required after placement. Due to the manner in which they are installed, prefabricated systems often require innovations in planning, engineering design, high-performance materials, and "Structural Placement Methods."

Early work with the installation of bridge systems focused on the use of SPMTs. In 2007, the FHWA published a manual entitled "Manual on Use of Self-Propelled Modular Transporters to Move Bridges" [2]. This manual contains significant information on the use of SPMTs to install bridge systems. Since 2007, the use of SPMTs has become more widespread, with Utah DOTUDOT leading the way. To date, Utah DOT has installed more than 20 bridge systems using SPMTs.

Installation of bridge systems using lateral slide methods has been used for decades. The recent emphasis on reduction in construction impacts has increased the frequency of bridge lateral slides. Utah DOT has moved more than 10 bridges with lateral slides.

The FHWA EDC initiative is used to progress various technologies in the transportation industry. The second round of EDC initiative includes the use of lateral sliding techniques to install bridge systems. This does not mean that SPMT bridge moves are not appropriate. Lateral sliding/moving requires that the bridge be built adjacent and generally parallel to the existing bridge. If the bridge is over a busy highway, the impacts of constructing the new bridge over traffic may be undesirable. In this case, construction of the system off site and installation with SPMTs might be a better solution. The FHWA manual entitled "Accelerated Bridge Construction—Experience in Design, Fabrication and Erection of Prefabricated Bridge Elements and Systems" [4] includes a chapter in selecting the appropriate methods of ABC and prefabrication for specific sites.

Utah DOT has recently revised their specifications for bridge system installations. The previous specification covered SPMT installations. The new specification covers all types of bridge movement methods, including lateral sliding.

The key features of this specification include:

- Requirements for submission of detailed working drawings.
- Requirements for construction of the bridge system off-line.
- Procedures for corrections and repairs to the bridge.
- Requirements for preparation of the structure for transport.

- Requirements for lifting, transport, and placement of the superstructure including provisions for control of deflection and twisting.
- Requirements for monitoring during the move.
- Tolerance requirements.

The specification is available on the Utah DOT website at: http://www.udot.utah.gov. The specification is entitled “03355S Move Prefabricated Bridge (Superstructure).”

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APPENDIX A—ACCELERATED BRIDGE CONSTRUCTION DEFINITIONS

ACCELERATED BRIDGE CONSTRUCTION (ABC)

ABC is bridge construction that uses innovative planning, design, materials, and construction methods in a safe and cost-effective manner to reduce the on-site construction time that occurs when building new bridges or replacing and rehabilitating existing bridges.

ABC improves:

- Site constructability.
- Total project delivery time.
- Material quality and product durability.
- Work zone safety for the traveling public and contractor personnel.

ABC reduces:

- Traffic impacts.
- On-site construction time.
- Weather-related time delays.

ABC can minimize:

- Environmental impacts.
- Impacts to existing roadway alignment.
- Utility relocations and right-of-way take.

A common reason to use ABC is to reduce traffic impacts because the safety of the traveling public and the flow of the transportation network are directly impacted by on-site construction-related activities. However, other common and equally viable reasons to use ABC deal with site constructability issues. Often, long detours, costly use of a temporary structures, remote site locations, and limited construction periods present opportunities where the use of ABC methods can provide more practical and economical solutions to those offered if conventional construction methods were used.

CONVENTIONAL BRIDGE CONSTRUCTION

Conventional bridge construction is bridge construction that does not significantly reduce the on-site construction time that is needed to build, replace, or rehabilitate a single bridge or group of bridge projects. Conventional construction methods involve on-site activities that are time-consuming and weather dependent.

An example of conventional construction includes on-site installation of substructure and superstructure forms, followed by reinforcing steel placement, concrete placement, and concrete curing, all typically occurring in a sequential manner.

One of the reasons to minimize on-site construction activity is because the long-term presence of contractor-related equipment, labor, and staging areas can present driver distractions and traffic disruptions that reduce the safety and mobility efficiencies of the transportation network.

Time Metrics for ABC

To gauge the effectiveness of ABC, two time metrics are used:

On-site Construction Time

This is defined as the period of time from when a contractor alters the project site location until all construction-related activity is removed. This includes, but is not limited to, the removal of MOT items, construction materials, equipment, and personnel.

Mobility Impact Time

This is defined as any period of time the traffic flow of the transportation network is reduced due to on-site construction activities.

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

<u>*Note*</u>*: "Total project" time is the period of time from when project planning begins until the time that all bridge work is completed. Total project time adds a planning time component to the on-site construction time period. It is not a focused metric because planning time is needed regardless of whether a project is planned using ABC or conventional construction methods. Owners recognize that the use of ABC may require varying degrees of planning effort and resource allocations, but choose the ABC approach due to the site constraints, the many benefits of ABC, or a combination of the two.*

PREFABRICATED BRIDGE ELEMENTS AND SYSTEMS (PBES)

Use of PBES is one strategy that can meet the objectives of ABC. PBES are structural components of a bridge that are built off-site or near the site of a bridge, and include features that reduce the on-site construction time and mobility impact time that occur from conventional construction methods. PBES includes innovations in design and high-performance materials and can be combined with the use of "fast-track contracting" methods. Because PBES are built off the critical path and under controlled environmental conditions, improvements in safety, quality, and long-term durability can be better achieved.

Regardless of the reason(s) to choose PBES, on-site construction time and mobility impact time are typically reduced in some manner relative to conventional construction methods.

Elements

Prefabricated elements are a category of PBES which comprise a single structural component of a bridge. Under the context of ABC, prefabricated elements reduce or eliminate the on-site construction time that is needed to build a similar structural component using conventional construction methods. An element is typically built in a prefabricated and repeatable manner to offset costs. Because the elements are built under controlled environmental conditions, the influence of weather-related impacts can be eliminated and improvements in product quality and long-term durability can be better achieved.

Deck Elements

Prefabricated deck elements eliminate activities that are associated with conventional deck construction, which typically includes on-site installation of deck forms, overhang bracket and formwork installation, reinforcing steel placement, paving equipment setup, concrete placement, and concrete curing, all typically occurring in a sequential manner.

Examples of deck elements include:

- Partial depth precast deck panels.
- Full depth precast deck panels with and without longitudinal post-tensioning.
- Lightweight precast deck panels.
- FRP deck panels.
- Steel grid (open or filled with concrete).
- Orthotropic deck.
- Other prefabricated deck panels made with different materials or processes.

Beam Elements

Prefabricated beam elements are composed of two types: “deck” beam elements and “full width” beam elements.

Deck beam elements eliminate conventional on-site deck forming activities as noted above. To reduce on-site deck forming operations, deck beam elements are typically placed in an abutting manner.

Examples of deck beam elements include:

- Adjacent deck bulb tee beams.
- Adjacent double tee beams.

- Adjacent inverted tee beams.
- Adjacent box beams.
- Modular beams with decks.
- Other prefabricated adjacent beam elements.

<u>Note</u>: *Although not preferred under the context of ABC, a separate construction phase (performed in an accelerated manner) may be required to finish the deck. A deck connection closure pour, overlay, or milling operation using innovative materials can be used to expedite the completion of the deck. In some situations, the placement of overlays can be accomplished during off-peak hours after the bridge is opened to traffic.*

Full-width beam elements eliminate conventional on-site beam placement activities. They are typically rolled, slid, or lifted into place to allow deck placement operations to begin immediately after placement. Given their size and weight, the entire deck is not included.

Examples of Full-Width Beam Elements include:

- Truss span without deck
- Arch span without deck
- Other prefabricated full-width beam element without deck

Pier Elements

Prefabricated pier elements eliminate activities that are associated with conventional pier construction, which typically includes on-site form installation, reinforcing steel placement, concrete placement, and concrete curing, all typically occurring in a sequential manner.

Examples of Pier Elements include:

- Prefabricated caps for caisson or pile foundations
- Precast spread footings
- Prefabricated columns
- Prefabricated column caps
- Prefabricated combined caps and columns
- Other prefabricated pier elements

Abutment and Wall Elements

Prefabricated abutment and wall elements eliminate activities that are associated with conventional abutment and wall construction, which typically includes form installation, reinforcing steel placement, concrete placement, and concrete curing, all occurring in a sequential manner.

Prefabricated abutment and wall elements may be built in a phased manner using conventional construction methods, but under or near an existing bridge without disrupting traffic.

Examples of abutment and wall elements include:

- Prefabricated caps for caisson or pile foundations.
- Precast footings, wing walls, or backwalls.
- Sheet piling (steel or precast concrete).
- Prefabricated full height wall panels used in front, behind, or around foundation elements.
- CIP concrete abutments and walls used with or without precast elements if built in a manner that is accelerated, or has no impact to mobility.
- MSE, modular block, or proprietary walls.
- GRS abutment.
- Other prefabricated abutment or wall elements.

Miscellaneous Elements

Prefabricated miscellaneous elements either eliminate various activities that are associated with conventional bridge construction or compliment the use of PBES.

Examples of miscellaneous elements include:

- Precast approach slabs.
- Prefabricated parapets.
- Deck closure joints.
- Overlays.
    - Includes overlays that can be placed in an accelerated manner that complements or enhances the durability and rideability of the prefabricated element other prefabricated miscellaneous elements.

<u>Note</u>: *Any cast-in-place concrete or overlay placement operation should be performed in a manner that reduces the impacts to mobility. This may require work that is performed under "Fast Track Contracting" methods with incentive/disincentive clauses, nighttime or off-peak hour timeframes, or work done entirely off line. Innovative materials may be needed to expedite placement times such as the use of rapid-set/early-strength-gain materials or ultra-high-performance concrete (UHPC) in closure pours.*

Systems

Prefabricated systems are a category of PBES that consists of an entire superstructure, an entire superstructure and substructure, or a total bridge that is procured in a modular manner such that traffic operations can be allowed to resume after placement. Prefabricated systems are rolled, launched, slid, lifted, or otherwise transported into place, having the deck and preferably the parapets in place such that no separate construction phase is required after placement. Due to the manner in which they are installed, prefabricated systems often require innovations in planning, engineering design, high-performance materials, and “Structural Placement Methods.”

Benefits of using prefabricated systems include:

- Minimal utility relocation and right-of-way take (if any at all).
- Minimal or no traffic detouring over an extended period of time.
- Preservation of existing roadway alignment.
- No use of temporary alignments.
- No temporary bridge structures.
- Minimal or no traffic phasing or staging.

Superstructure Systems

Superstructure systems include both the deck and primary supporting members integrated in a modular manner such that mobility disruptions occur only as a result of the system being placed. These systems can be rolled, launched, slid, lifted, or transported in place, onto existing or new substructures (abutments and/or piers) that have been built in a manner that does not impact mobility.

Examples of superstructure systems include:

- Full width beam span with deck.
- Through-girder span with deck.
- Truss span with deck.
- Arch span with deck.
- Other prefabricated superstructure systems.

Superstructure/Substructure Systems

Prefabricated superstructure/substructure systems include either the interior piers or the abutments, which are integrated in a modular manner with the superstructure as described above. Superstructure/substructure systems can be slid, lifted, or transported into place onto new or existing substructures that have been built in a manner that does not impact mobility.

Examples of superstructure/substructure systems include:

- Rigid frames with decks and parapets.
- Other prefabricated superstructure/substructure systems.

Total Bridge Systems

Total bridge systems include the entire superstructure and substructures (both abutments and piers) that are integral with the superstructure that are built off-line and installed in a manner to allow traffic operations to resume after placement. This excludes projects that are built off-line and, once complete, traffic “shifted” to the new alignment. Total bridge systems typically require innovations in designs, high-performance materials, and “structural placement methods” with or without the use of “fast track contracting” methods.

Examples of total bridge systems include:

- Total bridges of any kind, rolled/launched/slid/lifted into place.
- Rigid frames with decks, parapets, and integrated substructures.
- Other prefabricated total bridge systems.

The following pages contain examples of the most common prefabricated bridge elements and systems:

Table A1. Deck Element Examples.

Deck Elements	Examples
Partial-depth precast deck panels	

Deck Elements	Examples
Full-depth precast deck panels with and without longitudinal post-tensioning Lightweight precast deck panels	 

Deck Elements	Examples
FRP deck panels	
Steel grid (open or filled with concrete)	

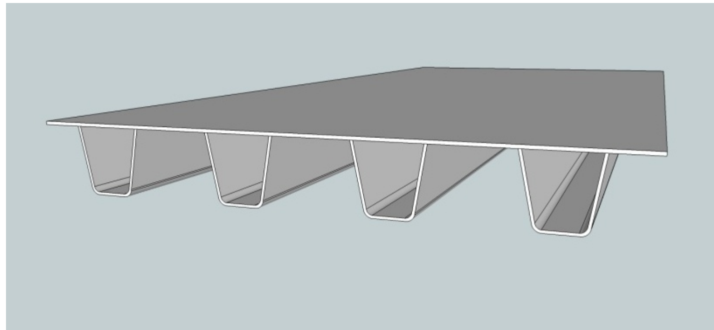
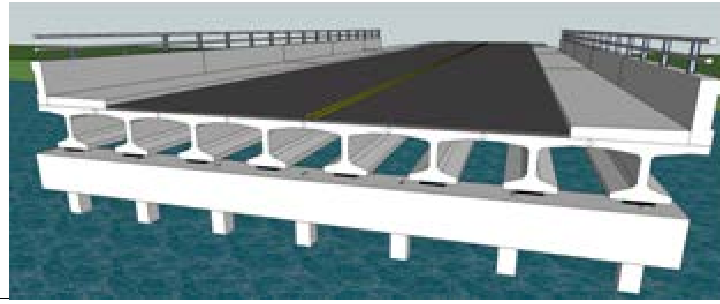
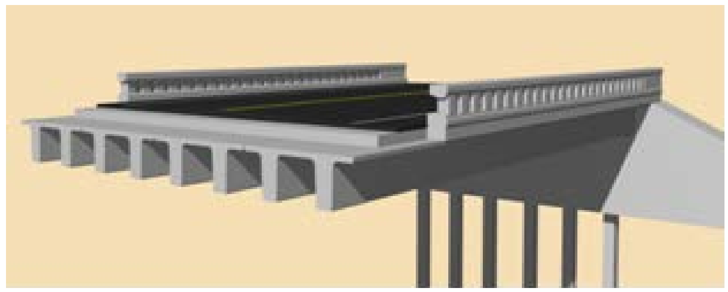
Deck Elements	Examples
Orthotropic deck	

Table A2. Deck Beam Element Examples.

Deck Beam Elements	Examples
Adjacent deck bulb tee beams	
Adjacent double tee beams	
Adjacent inverted tee beams	

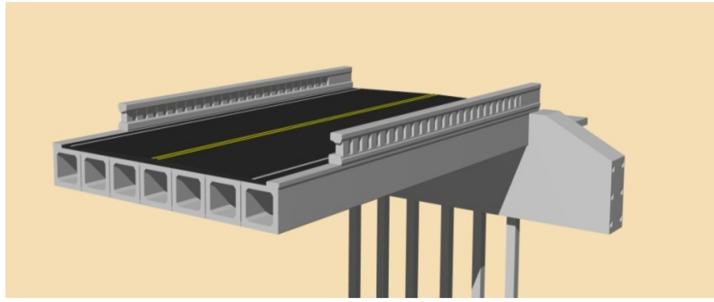
Deck Beam Elements	Examples
Adjacent box beams	
Modular beams with decks	

Table A3. Full Width Beam Element Examples.

Full Width Beam Elements	Examples
Truss span without deck	
Arch span without deck	

Table A4. Pier Element Examples.

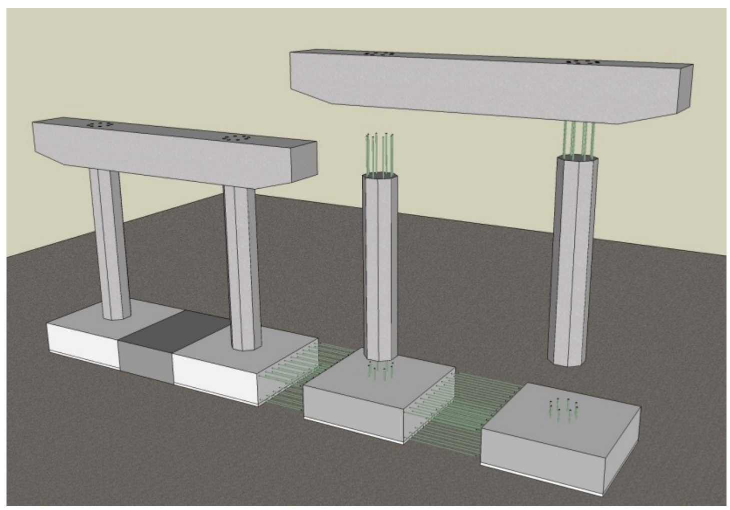
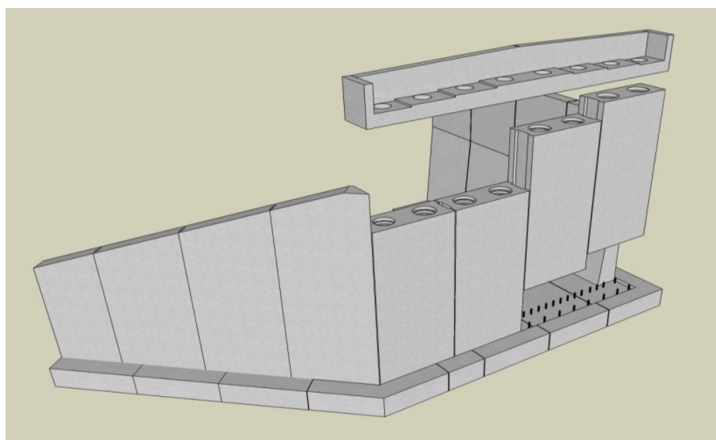
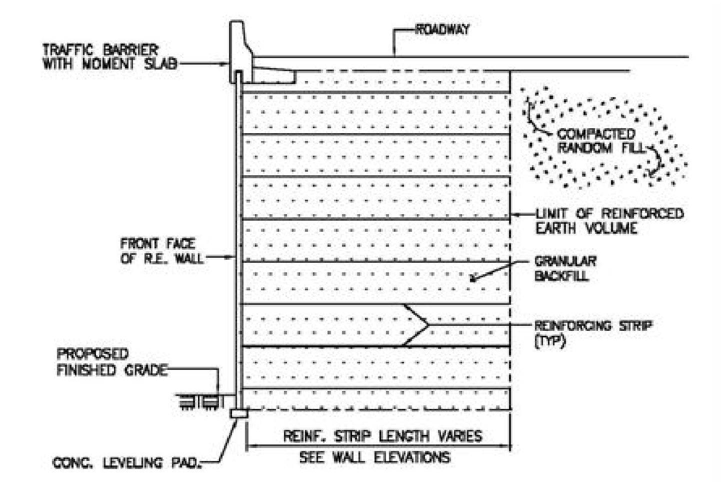
Pier Elements	Examples
Prefabricated caps for caisson or pile foundations	
Precast spread footings Prefabricated columns Prefabricated column caps	

Table A5. Abutment and Wall Element Examples.

Abutment and Wall Elements	**Examples**
Prefabricated caps for caisson or pile foundations	
Precast footings, wing walls, or backwalls	
Sheet piling (steel or precast concrete)	

Abutment and Wall Elements	**Examples**
Prefabricated full height wall panels used in front, behind, or around foundation elements	
MSE, modular block, or proprietary walls	

Abutment and Wall Elements	**Examples**
Geosynthetic Reinforced Soil (GRS) abutment	

Table A6. Prefabricated System Examples.

System	Examples
Superstructure Systems Full-width beam span with deck	 
Total Bridge System Total bridges of any kind rolled/launched/slid/lifted into place	

APPENDIX B—GLOSSARY OF TERMS

The following terms may be used in this document. The description of each term is written in the context of this document.

Term	Description
accelerated bridge construction	Construction methods that result in an overall decrease in construction time when compared to the historic construction methods used to build bridges.
additives	Substances (typically chemical) that are added to a grout mixture to counteract the natural tendency of grouts to shrink.
air release grouts	A type of grout that does not rely on a chemical reaction to achieve expansion. The additive reacts with water to release air and cause expansion of the grout.
anchor rods	Steel rods that are used to transfer loads from the superstructure to the substructure. Often referred to as "anchor bolts", anchor rods differ in that they do not have a hexagonal head. Anchor rods are normally specified according to ASTM F1554.
approach slabs	Structural slabs that span between the bridge abutments and the approach fill. They are used to span across the potential settlement of the approach roadway fills directly behind the abutments.
backwall	A structural wall element that retains the backfill soils directly behind the beam ends on a bridge abutment.
barrier	A structural wall element that is used to contain aberrant vehicles. They can be used on the bridge (parapet), or on the approach roadway.
batching	The process of combining and mixing the materials to form concrete.
bearing	A structural element that connects the bridge superstructure to the substructure, while allowing for movements such as thermal expansion and contraction.
benefit-cost analysis	The comparison of benefits over time and of costs over time for proposed projects. BCA is a tool used to aid in public investment decision making by measuring the

Term	Description
	efficiency of spending from the viewpoint of net benefit to society.
bleed water (grout)	Water that seeps out of the surface of a grout due to expansion of a grout in a confined or semi-confined area.
blockouts	Voids that are cast in prefabricated concrete elements that are used in connecting the elements in the field.
bottom up estimating	A process of construction estimating that breaks the individual tasks into discrete segments where the cost for time, equipment and materials can be determined for each segment. The total of all segments are then combined for the total construction estimate.
breastwall	A wall that is typically non-structural that covers the beam ends at the corners of the bridge abutments. Sometimes referred to as "cheekwalls" by some states.
bridge deck	A structural slab that spans between support elements (typically beams and girders) on a bridge. Bridge decks can be made of many materials, including reinforced concrete, steel, timber, fiber reinforced polymers, etc.
cable restrainers	Structural elements that are used to restrain a bridge superstructure from excessive lateral movement during seismic events. The goal being to prevent the superstructure from falling off the substructure, which is a very common form of failure during seismic events.
camber	A geometric adjustment of a bridge beam that is designed to compensate for the vertical deflection of the beam due to dead loads. Camber is typically built into steel beams during fabrication. Camber is an inherent side effect of prestressed girder construction.
cantilever wall	A cantilever wall consists of a concrete stem and a concrete base slab, both of which are relatively thin and fully reinforced to resist the moments and shears to which they are subjected.
carbon fiber	A materials that is used in fiber reinforced polymer elements (FRP) to provide the structure performance. These fibers are oriented parallel to the direction of stress.

Term	Description
cast-in-place concrete	Concrete that is cast on site (as opposed to cast in a fabrication plant).
cheekwall	A wall that is typically non-structural that covers the beam ends at the corners of the bridge abutments. Sometimes referred to as “breastwalls” by some states.
cofferdam	An enclosure used to retain water and support excavation in order to create a dry work environment. Typically used for bridge substructure construction in rivers and along river banks.
composite beam action	The process of connecting the bridge deck to the beams or girders to form a combined structural element.
composites	The combining of multiple structural materials to form a structural element.
compressive strength	The value of uniaxial compressive stress reached when a material fails.
concrete	A construction material that consists of cement (commonly portland cement), coarse aggregates (such as gravel limestone or granite), fine aggregates (such as sand), and water. Often other materials are added to improve the structural properties such as chemical admixtures and other cementitious materials (such as fly ash and slag cement).
confinement steel	Reinforcing steel used to contain the concrete core of a column when subjected to plastic deformations brought on by seismic loading.
consistency	The state of a mixture of materials where the formulation is of uniform quality.
constructability	The extent to which a design of a structure provides for ease of construction yet meets the overall strength and quality requirements.
construction joints	Joints in structures that are used to facilitate the construction of a portion of the structure. Construction joints typically have reinforcing steel passing from one side of the joint to the other providing continuity of the joined elements.

Term	**Description**
construction stages	A process of building a bridge in segments in order to maintain traffic during construction.
continuity connection	A connection used to connect two longitudinal bridge element (beams) to form a continuous bridge system. Typically these connections are only designed to resist live load.
continuous spans	A structural system where the beams span across a support without a joint.
contraction joints	Joints in structures that are used to allow the concrete elements to shrink without causing excessive cracking. Contraction joints do not have reinforcing steel passing from one side of the joint to the other.
controlled density fill	See “flowable fill.”
conventional bridge construction	Conventional bridge construction is bridge construction that does not significantly reduce the onsite construction time that is needed to build, replace, or rehabilitate a single, or group of bridge projects. Conventional construction methods involve onsite activities that may be time consuming and weather dependent.
cover concrete	The specified minimum distance between the surface of the reinforcing bars, strands, posttensioning ducts, anchorages, or other embedded items, and the surface of the concrete.
critical path	The portion of the sequence of construction activities which represents the longest overall duration. This in turn determines the shortest time possible to complete a project.
cross frame	A transverse structural element connecting adjacent longitudinal flexural element used to transfer and distribute vertical and lateral loads and to provide stability during construction. Sometimes synonymous with the term “diaphragm.”
crown	The apex of the roadway cross slope.
curing compounds	Chemical compounds that are used to prevent the rapid evaporation of water from concrete during curing.

Term	Description
curb	A structural element that is constructed at the edge of the bridge deck that is used to contain rain water runoff. Curbs are often combined with structural railings to retain vehicles.
debonding	The process of disconnecting prestressing strand from the surrounding concrete in a prestressed concrete element. This is done to control stresses in prestressed elements (typically at the ends of the element).
deck	The structural portion of a bridge that is directly beneath the wheels of passing vehicles.
dewatering	The process of removing water from an excavation that is below the water table or surface of adjacent water.
diaphragm	A transverse structural element connecting adjacent longitudinal flexural element used to transfer and distribute vertical and lateral loads and to provide stability during construction. Sometimes synonymous with the term “cross frame.”
differential camber	A variation on the camber of two adjacent beams. See “camber.”
dimensional growth	The phenomenon that results in the change in overall structure width or length when multiple elements are butted together. This is brought on by a buildup of element side variations or tolerances that are a result of the fabrication process.
distribution direction	A direction that is normally parallel to the supporting members and is perpendicular to the direction of beam action in reinforced concrete slabs that are designed for one-way slab action.
drilled shafts	A deep foundation unit, wholly or partly embedded in the ground, constructed by placing fresh concrete in a drilled hole with or without steel reinforcement. Drilled shafts derive their capacity from the surrounding soil and/or from the soil or rock strata below its tip. Drilled shafts are also commonly referred to as caissons, drilled caissons, bored piles, or drilled piers.
dry pack grout	A form of grout that has very stiff consistency that is

Term	Description
	placed by packing the material into voids by hand and hand tools.
effective prestress	The stress or force remaining in the prestressing steel after all losses have occurred.
elastomeric bearing pads	A type of structural bearing that is comprised of virgin neoprene or natural rubber. Sometimes combined with internal steel plates, fiberglass sheets, or cotton duck sheets.
emulation design	A design method where a prefabricated connection is designed and detailed to act as (or emulate) a conventional concrete construction joint.
epoxy adhesive anchoring systems	A method of embedding reinforcing rods into hardened concrete to form a structural connection. The process involves a drilled hole and a chemical adhesive. Note: Epoxy adhesive anchoring systems should not be used in sustained tension applications.
epoxy grouts	Grout materials with chemical adhesives used in place of cementitious materials.
erection engineer	An engineer that is hired by the contractor to perform structural analysis calculations for temporary works, lifting calculations and calculations for prefabricated elements.
ettringite expansive grout	Ettringite is crystal that forms as a result of the byproduct of reactive chemicals that can be inter-ground into the cement in expansive grouts to produce non-shrink grout.
exodermic bridge deck	A bridge deck system that is composed of a steel grid deck combined with a top layer of concrete to form a composite system. This system differs from filled grid decks in that the concrete is placed above the top of the grid to maximize the composite action between the steel and the concrete.
expansion joints	Joints in structures that are used to allow the concrete elements to expand and contract with temperature variation without causing excessive cracking. Expansion joints are similar to contraction joints except they are normally wider and often include a compressible material to allow for thermal expansion. They also do not have reinforcing steel passing from one side of the joint to the other.

Term	**Description**
fiber reinforced polymers (FRP)	A structural matrix of materials used to produce a structural element. FRP is commonly made reinforcing fibers that are combined with polyester, epoxy or nylon, which bind and protect the fibers from damage, and transfers the stresses between fibers. FRPs are typically organized in a laminate structure, such that each lamina (or flat layer) contains an arrangement of unidirectional fibers or woven fiber fabrics embedded within a thin layer of light polymer matrix material. The fibers, typically composed of carbon or glass, provide the strength and stiffness.
filled steel grids	A bridge deck system that is composed of a steel grid deck combined that is either fully or partially filled with concrete.
flowable fill	A material used to rapidly fill a void in embankment backfills or under structures without compaction. It normally has high flow characteristics. It is commonly made up of sand, water and a minor amount of cement. It is also referred to as “controlled density fill.”
flying wingwalls	Walls used to retain embankment soils at the corners of abutments that are cantilevered from the end or rear of the abutment as opposed to being supported on a footing.
foam block fill	A material made with expanded polystyrene (EPS) used to rapidly fill embankments where low unit weight materials are desired. This is often used over highly compressible soils such as clays. This material is also referred to as geofoam.
full-depth precast concrete deck slabs	A bridge deck system that is composed of reinforced concrete elements that when placed, make up the full structural deck system.
gantry crane	A crane type that is characterized by two or more legs supporting an overhead beam with a traveling trolley hoist.
gas generating grout	A type of non-shrink grout that expands due to the production of gas during the curing process. The gas is generated by adding reactive materials to the mix (often aluminum) to produce the gas.
general zone	Region adjacent to a post-tensioned anchorage within which the prestressing force spreads out to an essentially

Term	Description
	linear stress distribution over the cross-section of the component
geosynthetic reinforced soil integrated bridge system	Geosynthetic Reinforced Soil (GRS) technology consists of closely spaced layers of geosynthetic reinforcement and compacted granular fill material. GRS–IBS includes a reinforced soil foundation, a GRS abutment, and a GRS integrated approach. When integrated with a bridge superstructure, the system blends the embankment with the superstructure to act as a single unit with respect to settlement.
girder-floorbeam bridges	A bridge framing system that is composed of main girders that run parallel to the centerline of the roadway combined with transverse floorbeams that support the deck. Often the system includes stringer beams that run between floorbeams (parallel to the roadway).
glue laminated wood	A structural framing material that consists of multiple layers of dimensional lumber glued together to form a large timber element.
gravity wall	A gravity wall depends entirely on the weight of the stone or concrete masonry and of any soil resting on the masonry for its stability. Only a nominal amount of steel is placed near the exposed faces to prevent surface cracking due to temperature changes.
greenfield	A construction area where a bridge or highway is being built on land that previously did not support a roadway or bridge.
grout	A material (often cementitious or epoxy) that is used to fill voids between elements.
grouted reinforcing splice couplers	A proprietary product used to join precast concrete elements by connecting reinforcing steel bars at the ends of the elements. They consist of a steel casting sleeve that is filled with grout. The reinforcing bars are inserted into the ends of the casting and developed by the interaction of the grout with the sleeve.
haunch	The material between the top of a beam element and the bottom of the bridge deck that gaps the space between the two elements (also referred to as the “web gap” in some

Term	Description
	states).
heavy lift engineer	An engineer that is hired by the contractor to perform structural analysis calculations for temporary works, lifting calculations and calculations for large scale bridge system installation.
high early strength concrete	A concrete mixture that gains strength rapidly in order to accelerate construction.
integral abutment	A bridge abutment type that is made integral with the bridge superstructure through a combined shear and moment connection. They are often constructed with a single row of piles that allow for thermal movement and girder rotation. Soil forces behind the abutments are resisted through the strut action of the superstructure.
integral abutment connection	The connection between the superstructure and the integral abutment substructure that can resist both shear and moment.
integral pier connection	The connection between the superstructure and the pier substructure elements that can resist both shear and moment.
intelligent compaction	Intelligent compaction is a process that uses the compaction equipment to measure and record the quality of compaction during the compaction process. The equipment has the ability to vary the force used to compact the soil in real time to increase compaction where needed. Global positioning systems can be integrated into the equipment to create a map that shows the quality of compaction across the entire surface of each lift.
keeper assemblies	Devices that are placed on top of substructures to prevent lateral movement of the bridge superstructure. They are often used to resist lateral seismic forces. They can be constructed with structural steel or reinforced concrete.
leveling bolts	Bolt assemblies embedded in various prefabricated elements that are used to make grade adjustments in the field during construction.
life-cycle cost analysis	A process for evaluating the total economic worth of a usable project segment by analyzing initial costs and

Term	Description
	discounted future costs, such as maintenance, user costs, reconstruction, rehabilitation, restoring, and resurfacing costs, over the life of the project segment.
local zone	The volume of concrete that surrounds and is immediately ahead of the anchorage device and that is subjected to high compressive stresses.
match casting	A process of joining two precast concrete elements with high precision. This is done by casting one element against the adjoining element in the fabrication yard, separating them, and then re-joining them in the field. The field connection is normally made with thin epoxy adhesives combined with post-tensioning.
mechanical splices	Devices used to connect reinforcing through mechanical means. Examples of these systems include grouted sleeves, wedge assemblies, and threaded bar ends.
mechanically stabilized earth (MSE) retaining walls	A soil-retaining system, employing either strip or grid-type, metallic, or polymeric tensile reinforcements in the soil mass, and a facing element that is either vertical or nearly vertical. In this system, the soil mass is engaged by the strips to become a gravity type retaining wall.
mild reinforcement	Steel bars or grids within concrete elements that are used to resist tension stresses. Mild reinforcement normally consists of deformed steel bars or welded wire fabric.
modular block retaining walls	A soil-retaining system employing interlocking soil-filled timber, reinforced concrete, or steel modules or bins to resist earth pressures by acting as gravity retaining walls.
modular deck beam element	A structural bridge deck element that combines structural steel elements with an integral deck (typically composite concrete).
near site fabrication	A process of constructing prefabricated elements near the bridge construction site in order to minimize problems with shipping of large elements.
network arch bridge	A type of tied arch that includes suspender cables that are run diagonally forming a crisscrossing pattern.
non-shrink cementitious	A structural grout used for filling voids between elements

Term	Description
grout	that is formulated with cement, fine aggregates and admixtures. The admixtures are used to provide expansive properties of the material during curing. This expansion counteracts the natural tendency of cement grouts to shrink during curing.
one-way slab	A reinforced concrete slab system that primarily spans between two parallel support members. In this system, the majority of the reinforcing runs perpendicular to the support members.
open grid decks	A bridge deck system that is composed of an open steel grid spanning between supporting members.
orthotropic bridge deck	A steel bridge deck system comprised of a top deck plate supported by open or closed ribs that are welded to the top plate.
parapet	A structural element that is constructed at the edge of bridge deck that is used to contain aberrant vehicles.
partial-depth precast concrete deck panels	A bridge deck system that consists of relatively thin precast concrete panels that span between supporting members that are made composite with a thin layer of site- cast reinforced concrete. The precast panel makes up the bottom portion of the structural slab. The site cast concrete makes up the remainder of the structural slab.
pier box	A prefabricated system that includes a precast concrete box that is placed over driven piles or drilled shafts. The box becomes the form to contain site cast reinforced concrete. Often pier boxes are used in water applications to form a cofferdam for the footing concrete.
pier cap	A structural beam spanning between pier columns.
pier column	The vertical structural element in a bridge pier.
pile bent pier	A bridge pier without a footing that is comprised of driven piles or drilled shafts supporting a pier cap.
pile cap footing	A footing that is supported by driven piles or drilled shafts.
plastic hinge	A method of dissipating lateral seismic forces by allowing

Term	Description
	portions of reinforced concrete pier columns to bend beyond the yield point. Stability of the structure is maintained by providing adequate confinement reinforcement.
post-tensioning ducts	A form device used to provide a path for post-tensioning tendons or bars in hardened concrete.
post-tensioning (PT)	A method of prestressing in which the strands or bars are tensioned after the concrete has reached a specified strength.
precast concrete	Concrete elements that are cast in a location other than their final position on the bridge.
prefabrication	The process of building bridge elements prior to on-site construction in order to accelerate the construction of the bridge.
prefabricated bridge elements	Portions of a bridge structure that are constructed away from the final bridge site.
prefabricated bridge systems	Portions of a bridge structure that are made up of several elements that are combined to form a larger portion of the bridge such as the superstructure, substructure or the entire bridge.
prestressed concrete	Concrete elements in which force is introduced into the element during fabrication to produce internal stresses that are normally opposite of the anticipated stresses in the completed structure. Prestressing can be accomplished with pretensioning or post-tensioning.
pretensioning	A method of prestressing in which strands are tensioned before the concrete is placed, and released after the concrete has hardened to a specified strength.
quality assurance and quality control (QA/QC)	The process of inspection and control during fabrication to ensure that the specified quality is achieved.
reflective cracking	A crack that can form in site cast concrete that is placed over a connection between two elements below the pour.
reinforced closure pours	A method of connecting two prefabricated elements by casting a segment of reinforced concrete between two

Term	Description
	elements. The connection is often made using lap splices or mechanical reinforcing connectors.
reinforced concrete	Concrete elements with reinforcing steel cast into the concrete to form a structural element. The steel is normally used to resist tension stresses in the element.
reinforcing steel	Steel placed in concrete elements (either be mild reinforcement or prestressing steel).
return on investment	Measurement of the efficiency of spending from the viewpoint of net benefit to society. ROI analysis is essentially identical to benefit/cost analysis, incorporating benefit concepts that do not directly result in a revenue stream.
right of way	The land used for the route of a railroad or public road.
road user costs	Costs that incurred by users of a highway network when they are delayed due to construction activities.
saturated surface dry (SSD) condition	A condition that is normally specified for concrete surfaces that are to be grouted. Saturated Surface Dry describes the condition of the concrete surface in which the pores are filled with water; however, no excess water is on the surface. This condition minimizes the absorption of water from the grout into the surrounding concrete.
segregation	A condition where the distribution of course or fine aggregates in the concrete or grout mix become non-uniform.
self-propelled modular transporter (SPMT)	A high capacity transport trailer that can lift and move prefabricated elements with a high degree of precision and maneuverability.
shear key	A shaped connection between two prefabricated elements that can resist shear through the geometric configuration of the connection.
shear studs	Headed steel rods that are welded to elements to provide composite action between two bridge elements. Typically used between beams and the deck slab.
sheeting	A structural system used to retain earth and water and

Term	Description
	allow for excavation during the construction of a bridge substructure.
shims pack	Flat plates placed between two prefabricated elements used to provide a specified separation. Shims are also used to make vertical grade adjustments. Shims are typically made of steel or polymer sheets.
shrinkage (grout)	A property of cementitious concretes and grouts that occurs during curing where the material reduces in size.
skew angle	In most state agencies, this is defined as the angle measured between the centerline of the bridge elements (abutments, piers, joints, etc.) and a line perpendicular to the roadway alignment (i.e. a bridge with zero skew is square bridge). This definition is used in this manual. Several states define the skew angle as the complimentary angle (i.e. a bridge with 90 degree skew is square).
spandrel wall	A wall that is constructed on the sides of earth filled arch structures that are used to retain the fill soils.
spiral reinforcement	Transverse reinforcement used in reinforced concrete columns to resist shear. Spirals are also used for confinement of the concrete core as a plastic hinge forms.
steel stay-in-place forms	Corrugated steel sheeting that is used to support the wet concrete in a bridge deck during construction, and left in place in the permanent structure.
strength direction	A direction that is normally perpendicular to the supporting members and is parallel to the direction of beam action in reinforced concrete slabs that are designed for one-way slab action.
stress laminated timber deck bridges	A timber bridge deck that is comprised of multiple layers of dimension lumber placed on edge and connected with transverse prestressing. Shear transfer between the laminations is accomplished through friction.
stringers	There are two common uses for this term. • Longitudinal steel beams on short span multi-beam bridges. • Secondary framing members on floor beam type bridges that span from floor beam to floor beam.

Term	**Description**
stub abutments	A short cantilever type abutment that is constructed near the top of the approach embankment.
substructure	The portion of the bridge that is below the beam and/or deck elements. It typically includes piers, abutments, and walls.
superstructure	The portion of the bridge that is above substructure. It typically includes bearings, beams, girders, trusses, and the bridge deck.
surface preparation (grout)	The process of preparing a concrete surface for grouting by cleaning or intentionally roughening the surface. This is done to improve the adhesion of the grout to the concrete. It typically includes sand blasting, water blasting, or hand tool cleaning.
sweep	The lateral curvature of a prefabricated element caused by fabrication form irregularities and/or internal stresses.
test pours and test mock-ups	A method of quality control whereas a contractor will build a model of a portion of the bridge structure that includes a void that requires grout placement. These are used to demonstrate proper grout placement in complex voids.
tied arch	An arch structure where the thrust forces at the supports are resisted by a continuous bottom chord that runs from end to end.
timber deck panels	Prefabricated timber panels that are made with glue laminated lumber.
tolerance	Specified allowable dimensional variations in prefabricated elements. The variations are a result of irregularities in formwork and minor deviations in measurements during fabrication.
transverse ties	Reinforcement used in reinforced concrete columns to resist shear. Ties, if properly detailed, can also used for confinement of the concrete core as a plastic hinge forms.
tremie concrete pour	Concrete that is placed underwater and within a cofferdam to resist the vertical pore pressure of the water below a footing during construction.

Term	Description
ultra high performance concrete	Specially formulated concrete that can attain very high strength through the use of specialized mix ingredients including high strength steel fibers. The fibers increase the compressive strength as well as the tensile strength.
variable web gap	See “Haunch.”
water content	The specified amount of water in a concrete or grout mix.
wearing surface	The top portion of the bridge deck that is directly below the vehicle tires. Often, wearing surfaces are designed to be sacrificial and replaceable.
wet curing	Curing is the process of retaining sufficient moisture (water) in freshly placed grout/concrete to complete the hydration reaction which occurs when water is introduced to Portland cement. Wet curing leaves the freshly placed grout/concrete in an environment of 100 percent humidity
working time	The amount of time that a concrete or grout mix remains in a liquid or plastic state so it can be placed and consolidated.
yield strength	The stress at which an elastic material begins to deform in a plastic manner. Prior to yield, the material will deform elastically and will return to its original shape when the applied stress is removed. If loaded beyond yield and then unloaded, the material will not return to its original shape.