SHRP 2 Renewal Project R19A

Design Guide for Bridges for Service Life

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PREFACE

This document is the first version (version 1.0) of the Design Guide for Bridges for Service Life- hereafter, referred to as Guide. Guide is the main product of the Second Strategic Highway Research Program (SHRP2) project, R19A and entitled "Bridges for Service Life beyond 100 Years: Innovative Systems, Subsystems and Components".

The design for service life is gaining more importance, as limited resources demand enhancing service life of existing and new bridges. The cost of addressing service life issues at the design stage is significantly lower than taking maintenance and preservation actions while the bridge is in service. In general, design for service life, is approached by utilizing individual strategies, each capable of enhancing service life of particular bridge element, that historically has resulted to limiting the Bridge's service life. However, design for service life must be approached in a systematic manner using a frame work that is general and applicable for all bridges, while having specifics that are different from one bridge to another. The need for having these differences is reflection of the fact that design for service life is a context sensitive problem- Namely, needing to consider local experiences, practice and owner preferences.

The main objective of the Guide is to provide information and define procedures to systematically design for service life and durability for both new and existing bridges. The objective of the Guide is to equip the user with knowledge that is needed to develop specific optimal solution for a bridge under consideration in a systematic manner using a framework that is universal with specifics being different. This objective is achieved through providing 11 chapters, each devoted to certain part of bridge or aspects of service life design process. It is important to read and comprehend Chapter 1 of the Guide before proceeding with using other chapters of the Guide. The general frame work for design for service life is described in Chapter 1, followed by addressing specifics related to each step of the frame work by topics covered in various chapters.

Guide includes new concepts and approaches that are improvement to current practice and capable of enhancing the service life of bridges. The new concepts and approaches, which are not yet used in practice, are placed in the appendices of various chapters, signifying the need for using these provisions with exercising good engineering judgment.

The Guide is a flexible and living document that can be updated as new information becomes available. Additionally, several new chapters could also be added in future, making the Guide more comprehensive and applicable to wider ranges of bridges. The first version of the Guide (2012) also provides a platform for developing customized manuals by each State DOT or for developing a customized and systematic approach for design for service life for unique bridges.

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CHAPTER 1

DESIGN FOR SERVICE LIFE: GENERAL FRAMEWORK

The design for service life is gaining more importance as limited resources demand enhancing the service life of existing and new bridges. As part of the research project entitled *Bridges for Service Life Beyond 100 Years: Innovative Systems, Subsystems and Components* and supported by *SHRP 2 Project R19A*, a systematic and general approach to design for service life has been developed. The major product of this project is this document, referred to as *Design Guide for Bridges for Service Life*, hereafter referred to as the *Guide*. This chapter provides the general framework used in the *Guide*, primarily for bridges with spans of less than about 300 ft. However, the framework is general and can be adapted and customized for major and complex bridges such as those with much longer spans. It can also be adapted to suit bridges located in any region within the United States, recognizing however, that while the framework remains the same for all bridges, the resulting details for service life could be significantly different.

1.1 BACKGROUND

Providing safety for the public by having adequate strength for constructed facilities has been the cornerstone of the framework used by engineers for bridge design. This design for strength approach has not been restricted to bridges—it has also been the framework one could find in various building codes. In the case of buildings, however, most structural elements are protected from environmental-type loads and as a result the strength framework has served this sector of the industry very well. In the case of bridges or pavements, which are constructed facilities exposed to environmental loads, the story is different.

Significant changes to our contemporary bridge design specifications have also been mainly related to strength issues. The transitions from Allowable Stress Design (ASD) to Load Factor Design (LFD), and more recently to the Load and Resistance Factor Design (LRFD), reflect this line of thinking. It is also important to note that in the early 1970s, bridge engineers developed criteria for steel bridge details to protect against fatigue and fracture failure. These were indeed service life design provisions.

The strength framework did not prevent visionary engineers such as John Roebling to think in terms of service life. A review of bridges that have lasted more than 100 years provides valuable lessons. These bridges are not so much innovative in system or material, but have proven to be:

- Maintainable and well maintained over their 100-year lives due to extreme importance or high capital replacement cost,
- Adaptable to changes in functional use as well as service limit state demands and/or,
- Originally overdesigned.

Examples of bridges with long service lives are New York City's oldest East River bridges, the Brooklyn Bridge (the longest bridge in the world when opened to traffic in 1883) and the Williamsburg Bridge (the longest bridge in the world when opened in 1903), and St. Louis's Eads Bridge (the first steel bridge opened in 1874).

The Brooklyn Bridge has been well maintained and rehabilitated in a timely manner throughout its lifetime. Initial coatings to protect the bridge's steel from corrosion did not provide a 100-year life, but cleaning and repainting the bridge did. The metal deck of the Brooklyn Bridge has not survived its 100 plus year service life, but replacement of the replaceable metal decking has. Figure 1.1 shows the Brooklyn Bridge circa 1890.

Figure 1.1. The Brooklyn Bridge from the South Street Seaport, circa 1890.

The Williamsburg Bridge was not as well maintained as evidenced by its emergency closing in 1988. In April of that year, after a thorough inspection revealed corrosion of the cables, beams and steel supports, the Williamsburg Bridge was closed to all vehicular and train traffic for nearly two months. After engineers performed emergency construction on the bridge and reopened it to traffic, a panel of design experts convened to determine if the Williamsburg Bridge should be replaced, or if it should be rehabilitated. In November 1988, after evaluating several

alternatives, the New York City Department of Transportation (DOT) determined that the Williamsburg Bridge should be repaired while kept open to traffic. This option was deemed to have the least detrimental impact on motorists and nearby communities. In 1991, the New York City DOT began a major rehabilitation of the Williamsburg Bridge. The program was designed to undo the effects of age, weather, increased traffic volumes, and deferred maintenance. Figure 1.2 shows the Williamsburg Bridge circa 1904.

Figure 1.2. The Williamsburg Bridge, circa 1904.

The decision to rehabilitate the Williamsburg Bridge instead of undertaking a costly in-place replacement in downtown Manhattan was made possible by the original conservative design of the bridge cables. The need to rehabilitate the cable was necessitated by a poor corrosion-protection choice. Leffert L. Buck, the designer of the Williamsburg Bridge, chose linseed oil. The 1988 inspection of the Williamsburg Bridge cables revealed significant corrosion, proving the choice of linseed oil to be a relatively poor one. For the Brooklyn Bridge, John Augustus Roebling chose a coating of graphite to protect the individual wires of the bridge cable from corrosion, a choice that provided over 100 years of corrosion protection. Fortunately, the cable design for the Williamsburg Bridge utilized a factor of safety of resistance divided by load of about 5. After the significant loss of section due to corrosion was observed in 1988, the factor of safety was deemed adequate and a cable rehabilitation program to arrest the corrosion was initiated instead of a cable replacement. Thus, original overdesign allowed the bridge and its cables to continue in service.

The Eads Bridge, completed in 1874 and named for its designer and builder, James Buchanan Eads, has proven long-lived by being well maintained and readily adaptable. Figure 1.3 shows the Eads Bridge circa 1983. The scale of the bridge was unprecedented: the more than 500-ft span of the center arch exceeded by some 200 ft any arch built

previously. The arch ribs were made of steel, its first extensive use in a bridge. An additional innovation was the cantilever erection of the arches without falsework, the first example of this type of construction for a major bridge.

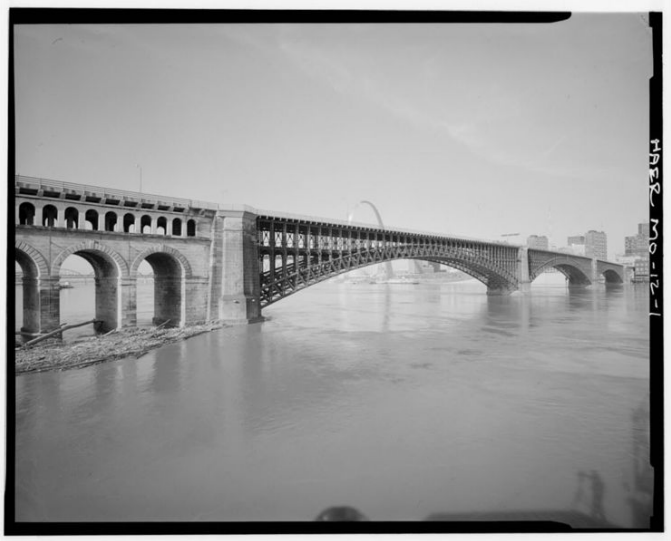


Figure 1.3. The Eads Bridge looking toward St. Louis and the Gateway Arch, circa 1983.

An interesting feature of the history of the Eads Bridge has been its adaptability to varying use. (It should be noted that the Brooklyn and Williamsburg Bridges have also seen varied use.) The bridge was originally a railway bridge carrying pedestrians on an upper deck with two rail lines below. The Eads Bridge eventually carried vehicular and rail traffic and the last train crossed the bridge in 1974. By the early 1990s, traffic on the bridge had dwindled to about 4,000 cars a day and in 1991, the Eads Bridge was closed. For a while, it was unused altogether, but in 1993, new uses were found. MetroLink, the region's new light rail system, began to use the lower deck, which originally served passenger and freight train traffic and in 2003, the upper deck reopened to buses and automobiles. Today, a new lane for pedestrians and bicyclists on the south side of the bridge provides a great place to look at the river and the skyline of the city.

The examples of these three 100-plus yr old bridges illustrates that for bridges to serve a long life, they must be:

- Resistant to environmental and man-made hazards,
- Maintainable (and subsequently maintained) or relatively maintenance-free, and
- Adaptable to changes in traveled-way cross section and usage.

Traditional approaches for enhancing the service life of bridges used in various codes and specifications such as AASHTO specifications, Eurocodes, or British Standards, are mainly in an indirect form, specifying the use of certain details or properties such as cover thickness, maximum crack width, concrete compressive strength, etc.

Recognizing the importance of design for service life has motivated different agencies to undertake new initiatives for developing more formal design approaches for service life, similar to those used for design for strength. However, to date the majority of these efforts have concentrated on addressing concrete durability and service life, and significant advances have been achieved in this field. Designing bridges for service life, however, is more than just addressing service life and durability of concrete.

Furthermore, the design for service life for bridges needs to be approached in a systematic, all-inclusive manner rather than as a series of isolated tasks, each addressing service life of a particular portion of a bridge independently. The interaction between strategies for enhancing the service life of different bridge elements, components, and subsystems must be given critical consideration. In addition, a maintenance program, retrofit or replacement options, and management plan should all be part of this systematic service life design approach. In summary, at the design stage the design for service life should be approached as a comprehensive plan capable of providing the owner with a complete picture of what will be necessary for the bridge to achieve its specified service life.

The most notable efforts to develop a scientific approach for service life and durability of concrete elements (covering buildings, bridges, and tunnels) were a series of studies carried out between 1996 and 1999 in Europe for the fédération international du béton (The International Federation for Structural Concrete). One of the products of these efforts was the publication of ***fib*** *Bulletin 34*, *Model Code for Service Life Design*, (TTD 2006). *Bulletin 34*, however, only concentrated on addressing concrete service life and durability. Further, caution must be exercised when applying the recommendations of this publication to concrete placed in a horizontal configuration, such as a bridge decks. While *Bulletin 34* has many useful recommendations for designing concrete elements for service life and durability, the application of these recommendations to bridge components such as bridge decks remains a point of debate (in particular the use of various solutions to Fick's second law to predict the rate of chloride ingress through deck concrete). The use of recommendations made in *Bulletin 34* is believed to be most applicable for concrete in vertical configuration and under compression, such as in substructure columns or sides of concrete box girders. This

same debate can also be extended to the use of some of the available commercial and noncommercial programs that use the fundamental concepts stated in *Bulletin 34*.

Efforts to address service life of bridges are not limited to Europe. A significant number of research studies have been and are presently (2012) being carried out to develop solutions for various service life issues related to different bridge types.

One of the missing elements for designing bridges for service life is the framework that would approach the problem in a systematic manner and provide a complete solution in a format that could ensure long lasting bridges. Individual solutions to issues that historically have reduced service life, maintenance plans, retrofit or replacement plans, bridge management, and life cycle cost analysis are all just components of this systematic framework and not the framework itself. The steps within this framework should start at the design stage and should provide the owner with complete information for ensuring the serviceability of the bridge for a specified target service life. It is important for the plan to be transparent and identify the challenges for the period of specified service life, at the design stage, so that the owner will encounter no surprises.

1.2 OBJECTIVES OF THE *GUIDE*

The main objective of the *Guide* is to provide information about, and define procedures for systematically designing for service life and durability for both new and existing bridges. The cost of addressing service life issues at the design stage is significantly lower than taking maintenance and preservation actions while the bridge is in service.

1.3 BRIDGE SERVICE LIFE RELATED TERMINOLOGY AND RELATIONSHIPS

The following sections provide service life related terminology and relationships used in the *Guide*.

1.3.1 Service Life and Design Life

Service Life. The time duration during which the bridge element, component, subsystem, or system provides the desired level of performance or functionality, with any required level of repair and/or maintenance.

Target Design Service Life. The time duration during which the bridge element, component, subsystem, and system is expected to provide the desired function with a specified level of maintenance established at the design or retrofit stage.

Design Life. The period of time on which the statistical derivation of transient loads is based: 75 years for the current version of *AASHTO LRFD Bridge Design Specifications* (2012), hereafter referred to as *LRFD Specifications*.

1.3.2 Bridge Element, Component, Subsystem, and System

The term bridge subsystem is introduced by the *Guide*. The terms bridge element, component, and system are the same as that defined by FHWA National Bridge Inventory.

Bridge Element. Individual bridge members such as a girder, floor beam, stringer, cap, bearing, expansion joint, railing, etc. Combined, these elements form subsystems and components, which then constitute a bridge system.

Bridge Component. A combination of bridge elements forming one of the three major portions of a bridge that makes up the entire structure. The three major components of a bridge system are substructure, superstructure, and deck.

Bridge Subsystem. A combination of two or more bridge elements acting together to serve a common structural purpose. Examples include composite girder, which could consist of girder, reinforcement, and concrete.

Bridge System. The three major components of the bridge—deck, substructure and superstructure—combined to form a complete bridge.

1.3.3 Service and Design Life: Basic Relationships

Several basic relationships exist between service lives of bridge components, elements, subsystems and systems, and bridge design life. Following are descriptions of these relationships.

- Predicting service life of bridge systems is accomplished by predicting service life of its elements, components, or subsystems.
- The design life of a bridge system is a target life in years, set at the initial design stage and specified by the bridge owner.
- The service life of a given bridge element, component, subsystem, or system could be more than the target design service life of the bridge system.

- The end of service life for a bridge element, component, or subsystem does not necessarily signify the end of bridge system service life as long as the bridge element, component, or subsystem could be replaced or resume its function with retrofit.

- A given bridge element, component, or subsystem could be replaced or retrofitted, allowing the bridge as a system to continue providing the desired function.

- The service life of a bridge element, component, or subsystem ends when it is no longer economical or feasible to repair or retrofit it, and replacement is the only remaining option.

- The service life of a bridge system ends when it is not possible to replace or retrofit one or more of its components, elements, or subsystems economically or because of other considerations.

- The service life of a bridge system is governed by the service life of its critical elements, components, and subsystems. The critical bridge elements, components, or subsystems are defined as those needed for the bridge as a system to provide its intended function.

In general, the service life, t_s , of the bridge elements, components, and subsystems should be equal to or greater than the design life, t_D of the bridge system defined by Equation 1.1.

$$(t_s)_{C,\,E,\,SS} \geq (t_D)_{BS} \qquad \textbf{EQ 1.1}$$

Where:

$(t_s)_{C,\,E,\,SS}$ = service life of bridge component (c), bridge element (E) or bridge subsystem (SS)
$(t_D)_{BS}$ = design life of bridge system (BS)

The service life of the bridge system is less than or equal to the service life of its governing elements, components or subsystems, as described by Equation 1.2.

$$(t_s)_{BS} \leq [(t_S)_{C,\,E,\,BS}]_{critical} \qquad \textbf{EQ 1.2}$$

The service life of the bridge system must exceed or be equal to the target design life of the bridge system as described by Equation 1.3.

$$(t_s)_{BS} \geq (t_D)_{BS} \qquad \textbf{EQ 1.3}$$

1.4 *GUIDE* APPROACH TO DESIGN FOR SERVICE LIFE

The *Guide* approach to design for service life is to provide a body of knowledge relating to bridge durability under different exposure conditions and constraints, and to establish an array of options capable of enhancing service life. A solution for a particular service life issue is highly dependent on many factors that vary from location to location and state to state. A solution also depends on local practices and preferences. Consequently, use of the *Guide* is not intended to dictate a unique solution for any specific service life problem or identify the "best and only" solution. Rather it equips the reader with a body of knowledge for developing specific solutions best suited to stated conditions and constraints.

In applying the *Guide* framework to a particular bridge, including long-span bridges, an array of solutions can be identified for enhancing the service life of a the bridge element, component, or subsystem, and an optimum solution can be identified through life cycle cost analysis. The solutions can be based on data collected by local DOTs or agencies responsible for maintaining the bridge and, in order to be complete, the life cycle cost analysis should include maintenance, retrofit, replacement, and user costs. It is important that the list of assumptions and feasible solutions considered for a particular bridge element, component, and subsystem be communicated and shared with the owner, especially with respect to the life cycle cost analysis, so that the entire process is fully transparent.

The *Guide* recognizes that not all bridges can or need to have 100 years of service life. Therefore maintenance, rehabilitation, and replacement are part of the service life design process. The *Guide* provides the general framework to achieve this objective in a systematic manner that considers the entire bridge system and all project demands.

There are different ways of achieving enhanced service life of existing and new bridges. Two examples include using improved, more durable materials and systems during original construction that will require minimal maintenance, and improving techniques and optimizing the timing of interventions such as preventive maintenance actions. Interventions can be planned and carried out based on the assessment of individual bridge conditions and needs, or based on a program of preventive maintenance actions planned for similar elements on a group of bridges. A simple example of a preventive, planned maintenance program might include the following activities:

- Washing deicing salts off bridge decks in the spring,
- Cleaning debris from bridge deck expansion joints,

- Cleaning debris from bearings and truss joints,
- Cleaning drainage outlets,
- Spot painting steel structures,
- Sealing decks or superstructures in marine environments, and
- Sealing substructures on overpasses where deicing salts are used on the roadways below.

By acknowledging that service life can be extended by either using more durable, deterioration-resistant materials or by planned intervention, a cost comparison can be made to determine the most cost effective approach for various environmental exposure levels and various levels of available maintenance and preservation actions.

The following sections provide an overview of the general approach used in the *Guide*. The discussion is customized for new bridges; however, use of the *Guide* approach for existing bridges can be accomplished by eliminating some of the steps that are used for new bridges. Because the discussions are general and use very simple examples to demonstrate the point of discussion many intermediate steps are eliminated for the sake of clarity. More detailed procedures and examples are provided in subsequent chapters of the *Guide*.

Three related flowcharts, as shown in Figures 1.4, 1.5 and 1.6, are used to demonstrate the general approach used in the *Guide*. Blocks within each flowchart are numbered. Following each flowchart, a brief discussion explains the intent of each block within that flowchart.

It should be noted that customization of the framework introduced in the *Guide* for a particular bridge could be achieved by developing similar flowcharts, making each step of the process transparent to the owner. For major and complex bridges, various elements of the flowcharts need to reflect specific project requirements.

The sections that accompany Figures 1.4, 1.5 and 1.6 describe elements within the flowcharts. Each block within the flowcharts is numbered and described under the corresponding Step number. For example, Step 1 corresponds to block number 1 in Figure 1.4.

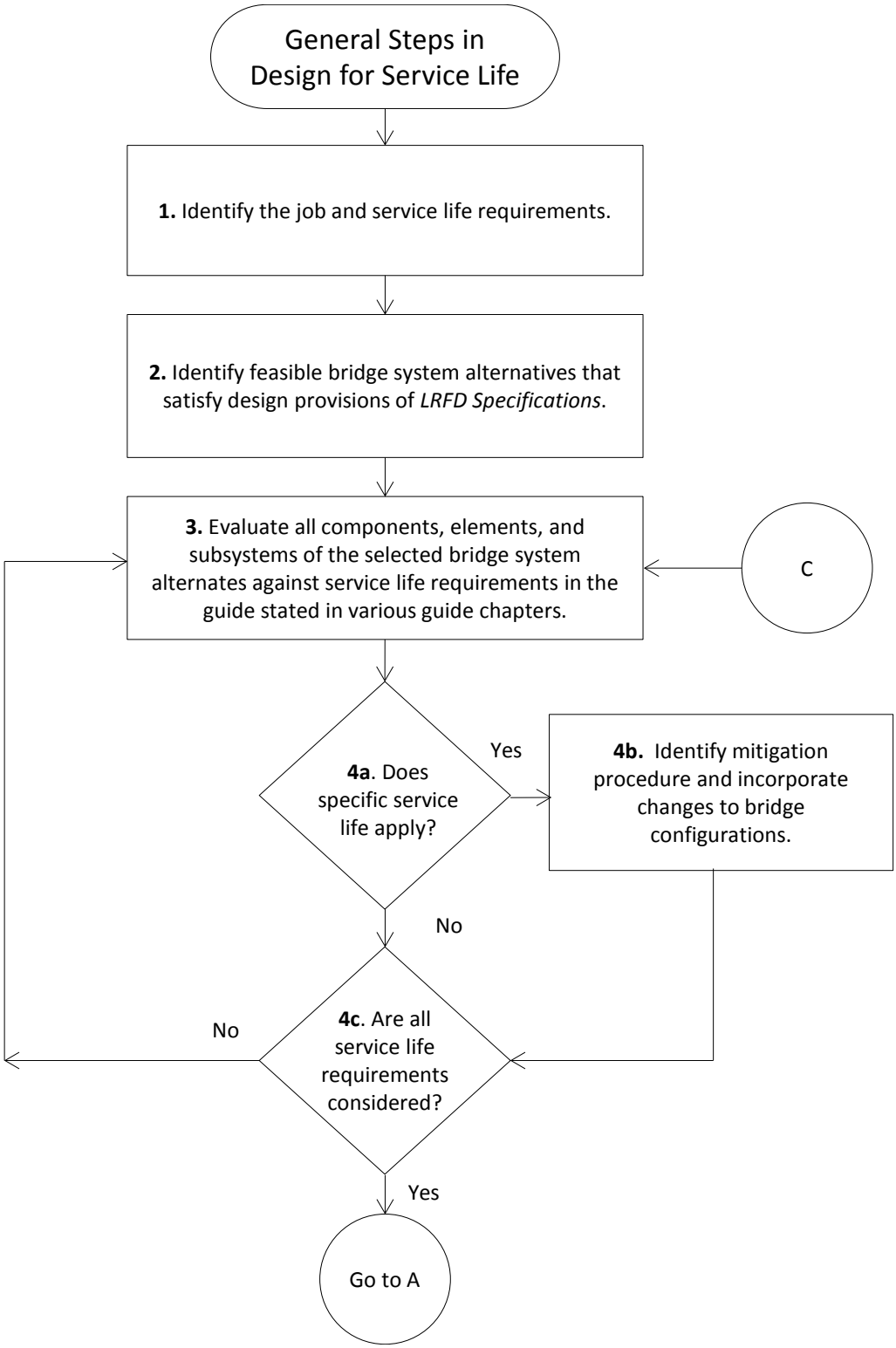


Figure 1.4. General flowchart demonstrating the *Guide's* approach for service life design.

Step 1. The design for service life starts by first considering all project demands set by the owner, including the service life requirements, as stated in Figure 1.4. Chapter 2 provides examples of local operational and site requirements, as well as service life considerations, needing attention.

Step 2. Develop all feasible and preliminary bridge alternatives that satisfy project demands. For example, one might want to consider steel, concrete, and segmental bridge alternates for a particular bridge. The development of the potential bridge systems is carried out in a conventional manner, meeting all the provisions of the *LRFD Specifications*. It is good practice to consider potential service life problems, even at this stage of the design process. It is also feasible to use bridge technologies which do not have a specific design guideline within the *LRFD Specifications*. In such cases, the best available design approach could be used, subject to owner approval.

Steps 3 and 4. The next step in the process consists of evaluating each bridge system alternate one at a time and considering service life issues related to each element, component, and subsystem of that bridge system. For each bridge element, component, and subsystem, the *Guide* provides a framework for incorporating the changes and modifications needed to meet service life requirements.

For example, assume that one of the bridge systems to be considered for a particular project is a steel bridge alternate. The designer will first develop the preliminary bridge configurations using the conventional approaches that meet all *LRFD Specifications*. Then, using procedures depicted in blocks 4a, 4b, 4c, each element, component, or subsystem of the steel alternate will be checked against the service life requirements using the fault tree approach described later in the *Guide*. These evaluation requirements may lead to changes in the details of the element, component, or subsystem under consideration. For example, the preliminary deck configuration may indicate that use of 8-in. thick concrete is sufficient from a strength standpoint. Going through the fault tree corresponding to bridge deck and described in Chapter 4 of the *Guide*, the designer may change the deck thickness to 9 in. to address potential overloads, or may specify sealing the bottom of the deck to protect it from salt spray, if the bridge is located along the coastline. It should be noted that for major and complex bridges, most of these fault trees must be customized to meet specific needs and preferred practices. Examples of fault trees and how they work are provided in later sections of this chapter.

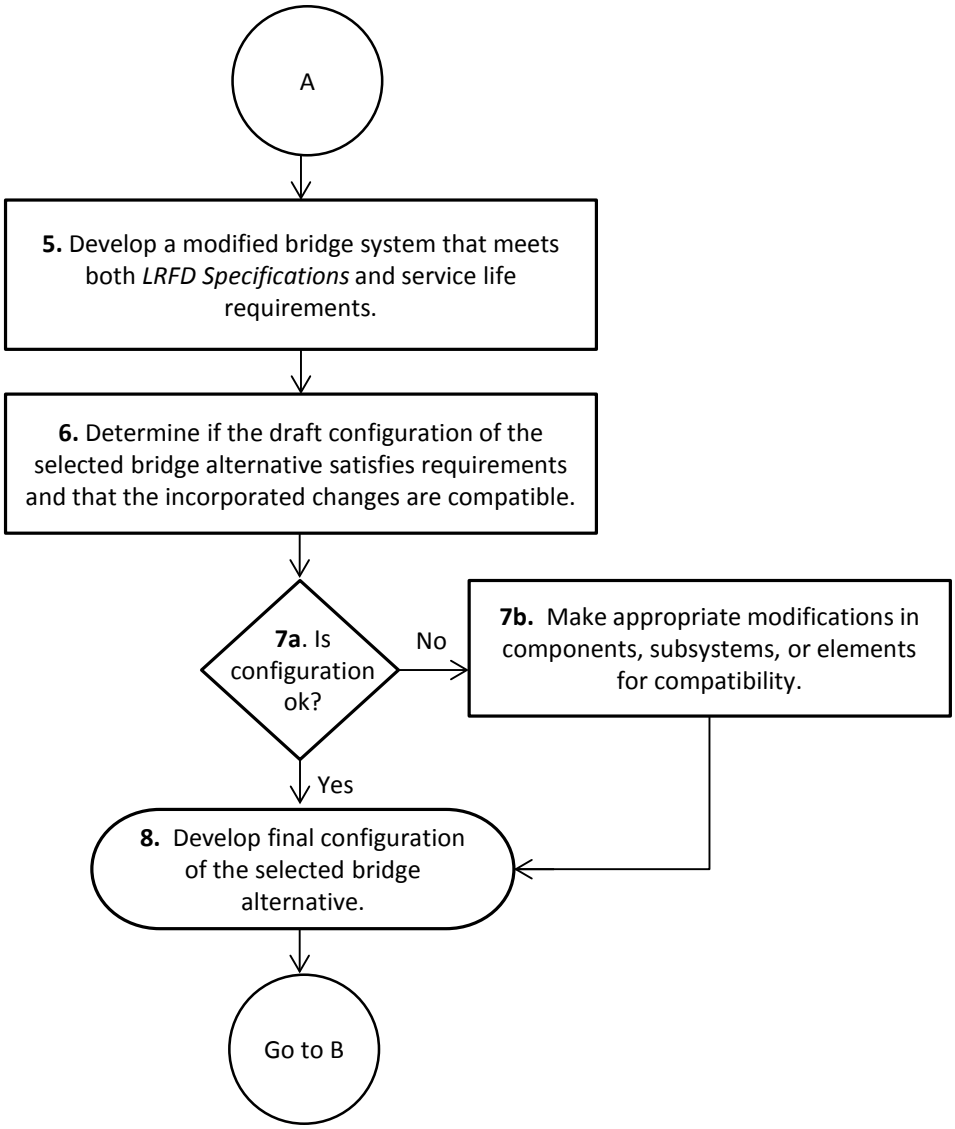


Figure 1.5. General flowchart demonstrating the *Guide's* approach for service life design.

Steps 5 through 8. At the end of Step 4 and after going through appropriate fault trees for various bridge elements, components, and subsystems, the designer will have developed a bridge system that meets both strength and service life requirements, as illustrated by Step 5 in Figure 1.5. To some extent, changes to configurations of various bridge elements, components, and subsystems are carried out separately. Therefore there is a need to make sure that these changes are compatible and not contradictory or overly conservative. Steps 6 and 7 in Figure 1.5 depict this process. For example, in the steel bridge example discussed previously, service life requirements may dictate the use of a jointless, integral abutment system and require metalizing the end of the girder. The designer may then want to consider not metalizing the end of the girder, since leaking joints would be eliminated. Finally,

for the selected bridge system alternate under consideration a final configuration is developed, Step 8, that meets both strength and service life requirements.

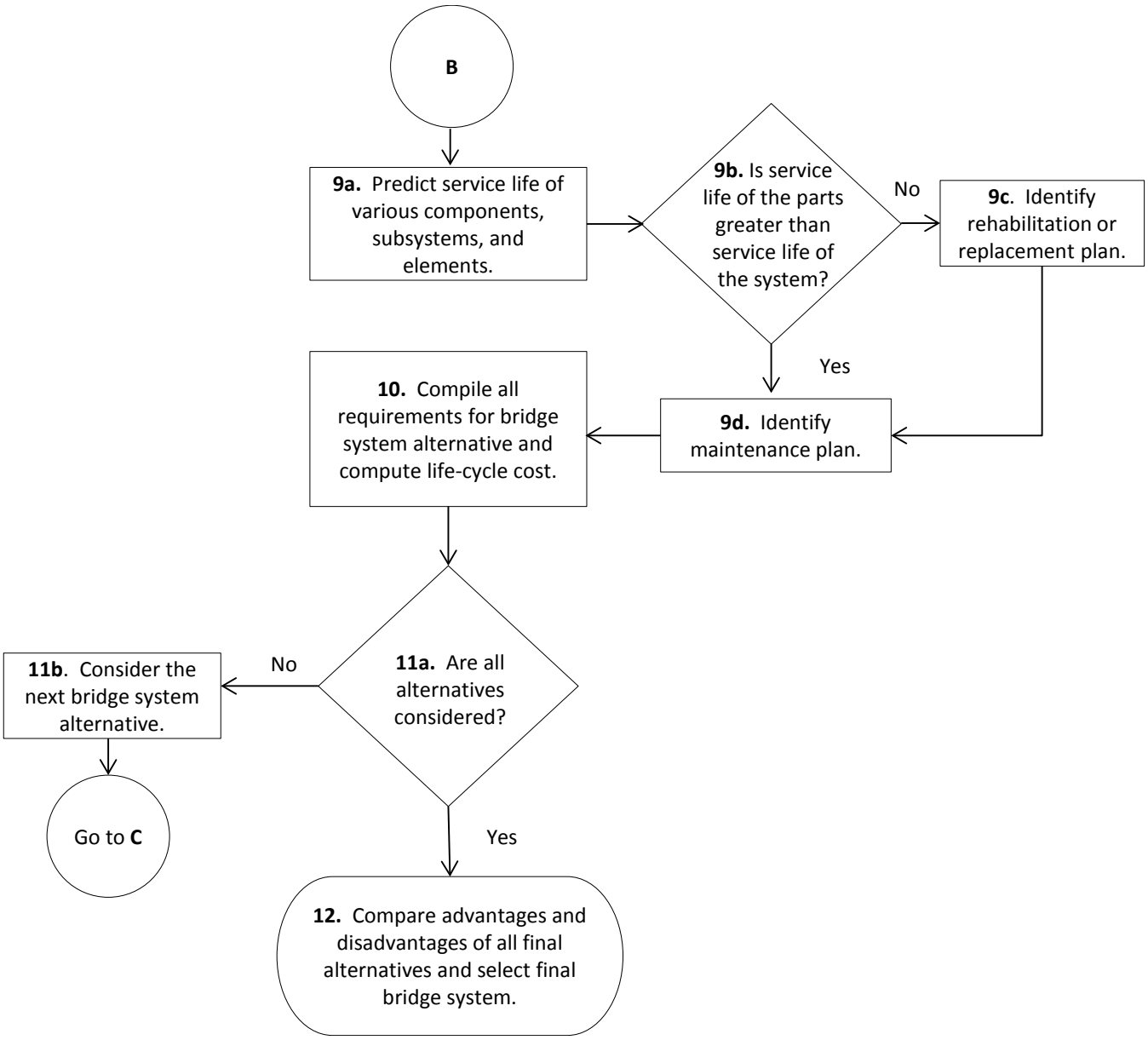


Figure 1.6. General flowchart demonstrating the *Guide's* approach for service life design.

Steps 9 through 12. The next step in the process is to evaluate the service life of the various bridge elements, components, and subsystems of the bridge alternate under consideration and compare it to the owner-specified target service design life of the bridge system. For example, the owner may require that the bridge provide 100 years of service life, whereas the life of a particular bridge element, such as the sliding surface for a bearing, may be limited to 20 years. There would therefore be a need to think ahead and accommodate replacement of the sliding surfaces. Regardless, there will be a need for a

systematic maintenance plan that could require the designer to identify "hot areas" requiring more detailed inspection and maintenance. Blocks 9a through 9d depict the development of a maintenance plan and/or rehabilitation and replacement plan for the bridge system alternate under consideration. The result of this process, as illustrated in Step 10, is a bridge system alternate that meets both strength and service life requirements with an associated maintenance and/or rehabilitation or replacement plan for the bridge. The next step, as illustrated in Step 10, is to carry out life cycle cost analysis considering the final configuration of the select bridge alternate and maintenance plan. The same steps are repeated for all bridge alternate systems as shown by Step 11. After comparing all alternates, the designer can then make a recommendation as to which alternate should be used, allowing the owner to make the final selection.

As described, the selection of the final bridge system within the framework promoted in the *Guide* is mainly based on service life requirements. Some of the details included in the steps presented, such as fault tree analysis, will be described in later sections of this chapter.

A summary of steps for design for service life is provided in Section 1.10 of this chapter.

1.5 ORGANIZATION OF THE *GUIDE*

Included in the *Guide* are 11 chapters, each devoted to particular bridge elements, components, subsystems, or systems. The following is a brief description of information included in each chapter.

Chapter 1. Design for Service Life: General Concepts. This chapter provides an overview of the approach used in the *Guide* for design for service life and describes terminology used throughout the *Guide* and various relationships that exist between service life of bridge element, component, subsystem, and system and bridge design life as used in AASHTO Specifications. The chapter provides an introduction to the different philosophies used to predict service life.

Chapter 2. Bridge System Selection. This chapter provides a description of various bridge systems and factors that affect their service life. Included is a description of a general strategy and rational procedure for selecting the optimum bridge system, subsystems, components, and elements, considering specific project limitations and requirements, such as climate, traffic, usage, and importance.

Chapter 3. Materials. This chapter provides general properties and durability characteristics of the two most commonly used materials in bridge systems, steel and concrete. For each material, a general description of variables affecting the service life is provided, followed by strategies used to mitigate them. Chapter 3 forms the basis for materials used in bridge subsystems and elements specifically addressed in other chapters of the *Guide*.

Chapter 4. Bridge Deck. This chapter provides descriptions of various bridge deck types and essential information related to their service life, such as modes of deterioration and strategies to mitigate them. The chapter concentrates on cast-in-place and precast concrete bridge decks.

Chapter 5. Corrosion Protection of Reinforced Concrete. This chapter looks at basic mechanisms causing corrosion of reinforcement embedded in concrete and provides strategies for preventing corrosion of reinforcement in concrete bridges.

Chapter 6. Corrosion Protection of Steel Bridges. This chapter provides descriptions of various coating systems using paint, galvanizing and metalizing, and descriptions of corrosion resistant steel along with factors affecting service life. Various options for preventing corrosion of steel bridges and general approaches that could lead to bridge coatings with enhanced service life are presented.

Chapter 7. Fatigue and Fracture. This chapter provides the basics of fatigue and fracture and factors that cause fatigue and fracture in steel bridges. Various available options for repairing observed cracking in steel bridges are also presented.

Chapter 8. Jointless Bridges. This chapter provides descriptions, advantages, and disadvantages of various jointless bridge systems, and provides complete steps for design of jointless integral abutment bridges. This chapter provides design procedures to extend the application of jointless integral bridges to curved girder bridges. Also introduced are new details and integral abutment systems, where expansion joints are completely eliminated, even at the end of approach slabs.

Chapter 9. Bridge Expansion Devices. The *Guide* encourages eliminating the use of expansion joints, however, expansion joints may be needed when the total bridge length exceeds practical limits of jointless bridges. This chapter describes various expansion joints used in practice, observed modes of failure for each, and potential strategies to mitigate them.

Chapter 10. Bridge Bearings. This chapter provides descriptions of various bearing types and lists the factors that affect service life of the various bearings with strategies to mitigate them. New high performing sliding surfaces capable of providing long service life are introduced, as well as deterioration models for sliding surfaces. The *Guide* emphasizes use of elastomeric bearing pads.

Chapter 11. Life Cycle Cost Analysis. This chapter provides essential information for incorporating life cycle cost analysis (LCCA) in bridge system, subsystem, component, and element selection. It concentrates on general features and elements of incorporating LCCA in the design process, emphasizing consideration of project costs throughout its service life.

1.6 CATEGORIES OF INFORMATION PROVIDED IN *GUIDE* CHAPTERS

Typically, each chapter consists of five major categories of information as described in the following. Closer examination of the type of data included in each chapter could also assist in developing customized information for addressing design for service life for major and complex bridges.

Description of Bridge Elements, Components, or Systems

These sections of each chapter provide brief descriptions of, and essential information related to, both commonly used and more recently developed types of bridge components, elements, subsystems, and systems.

Factors that Affect Service Life

The factors affecting service life are identified using a fault tree approach, which provides a systematic method of identifying factors in various categories and successive subcategories. Most chapters have fault trees applicable for the types of elements, components, or subsystems covered within that chapter. In the case of major and complex bridges, designers should develop customized fault trees, reflecting the specifics associated with location and traffic conditions.

Following is brief description of what the fault tree is, how it is constructed, and how it works.

The fault tree is used to systematically identify the factors that can affect service life of a particular bridge element, component, or subsystem. A customized fault tree can be developed using data and experiences collected from and available from local agencies.

The fault tree starts with the identification of major factors that can reduce service life of a particular bridge element, component, or subsystem. Each major factor can then be broken down into more detailed subcomponents, each capable of reducing the service life. The fault tree continues branching until each branch ends with factors at the lowest or base levels of influence. The factors with subcomponents are placed inside rectangles, and the identified lowest or base factors are placed inside circles. Figure 1.7, for example, shows a portion of the fault tree used in Chapter 4 for bridge a deck.

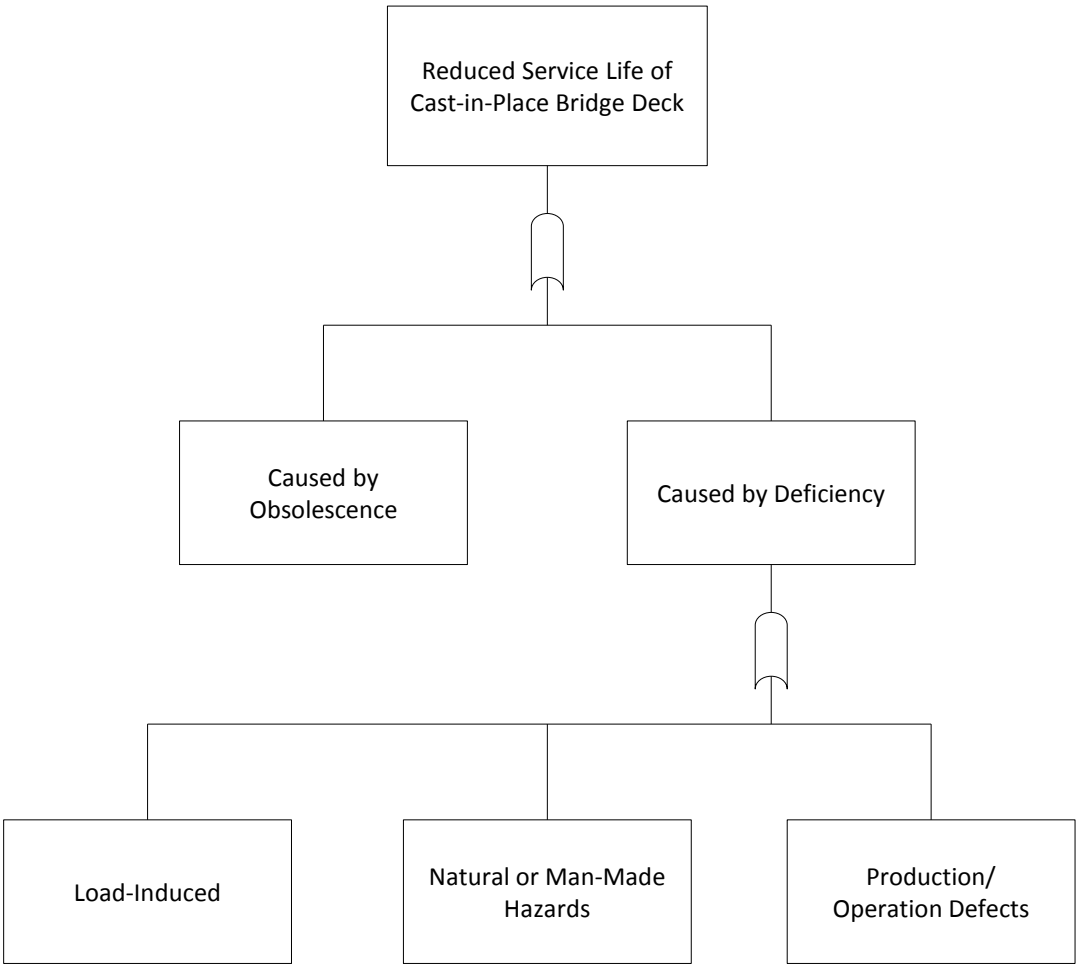


Figure 1.7. Starting point for fault tree for a bridge deck.

In Figure 1.7, either of two main factors is shown to be capable of contributing to reduced service life of a bridge deck: obsolescence or deficiency. The elliptical symbol just above these two factors is referred to as an "or gate," which signifies that either one of the factors below it could result in reduced service life. The fault tree shown in Figure 1.7 continues to list the major categories of factors that could result in reduced service life of bridge deck, those related to induced loads, natural or man-made hazards, and production/operation defects.

Figure 1.8, shows the continuation of the fault tree for breakdown of factors related to load-induced factors for bridge deck.

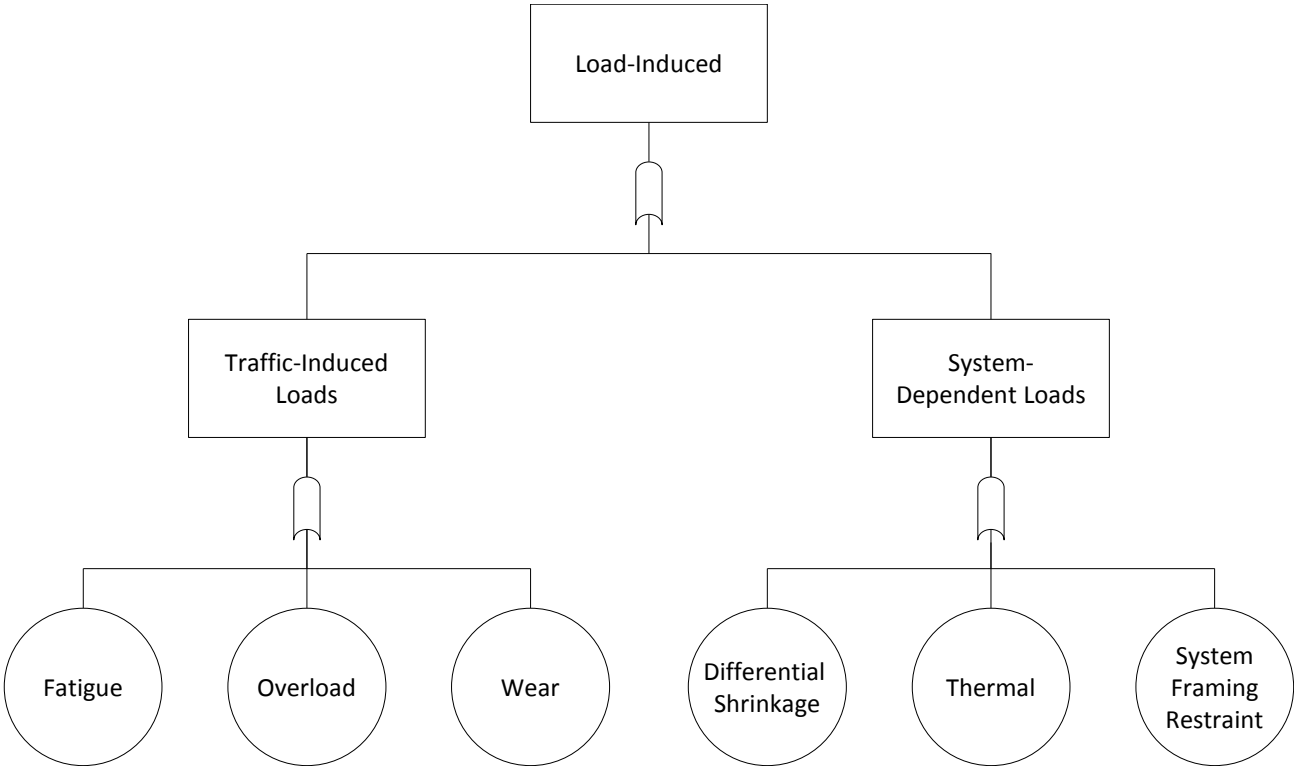


Figure 1.8. Continuation of the fault tree for a bridge deck.

In Figure 1.8, the factors related to load-induced are subcategorized into traffic-induced loads or loads induced by system-dependent loads factors, such as restraints provided by shear studs, etc. As shown in Figure 1.8, two factors are further broken down into subcomponents, each capable of reducing the service life of bridge deck. The factors inside the circles are the basic factors without any further subcomponent. They represent the end of that branch of the fault tree and require the development of individual strategies to mitigate them. This aspect of process is described later.

In the *Guide*, each element of the fault tree is described immediately after introducing each branch of the fault tree. It is advisable to do the same when developing customized fault trees for major and complex bridges. Documentation of factors affecting the service life of bridge elements, components, or subsystems in the form of a fault tree, should be part of the overall plan for design for service life and provided to the owner for future reference.

Mitigation Strategies

Where possible, each chapter provides provable solutions for major factors affecting the service life of a particular bridge element, component, subsystem, or system. Some chapters also include technology tables that summarize major characteristics associated with each solution and provide the potential solutions to factors affecting service life in a form that is easier to comprehend. For example, Figure 1.9 shows the part of the technology tables summarizing solutions for enhancing service life of bridge decks and related to traffic-induced loads as shown in Figure 1.8. In Figure 1.8, as part of the fault tree for bridge deck, traffic-induced loads is identified as one factor capable of reducing the service. In Figure 1.8, below traffic-induced loads, three basic factors capable of reducing the service life of bridge deck are identified: Fatigue, Overload and Wear and Abrasion. Each basic factor needs to be mitigated using a select strategy, and in almost all cases, there is more than one strategy to mitigate these basic factors. It is good practice to collect these strategies in table form and select the optimal strategy, considering its interaction with other parts of bridge. The technology tables provided in various chapters of the *Guide* summarize strategies that can be used to mitigate various basic factors capable of reducing service life. For major and complex bridges the list of strategies could be different and based on local preferences and experiences. Most agencies have access to field data collected over the years, which could be used to construct customized strategy tables for the purpose of mitigating basic factors capable of reducing service life of bridge elements, components and subsystems.

Service Life Issue		Mitigating Strategy	Advantage	Disadvantage
Traffic-Induced Loads	Fatigue	Design per *LRFD Specifications*	Minimizes the possibility of reinforcement failure	May increase the area of steel
	Overload	Increase deck thickness	Minimizes cracking	Adds weight to bridge structure, increases cost
	Wear and Abrasion	Implement concrete mix design strategies	See Chapter 3 - Materials	See Chapter 3 - Materials
		Implement membranes and overlays	Protects surface from direct contact with tires	Requires periodic rehabilitation every 10 to 20 years

Figure 1.9. Technology table for mitigating factors affecting service life of bridge deck and related to traffic-induced loads. (Table 4.3)

The sample table in Figure 1.9 lists the advantages and disadvantages for each possible solution capable of mitigating the adverse service life consequences of traffic-induced loads. In some cases, more information than just advantages and disadvantages are provided, such as qualitative assessment of maintenance cost. For major and complex bridges additional considerations may be included in technology tables.

Optimum Selection Strategies

Overall strategies are provided for achieving enhanced service life. The overall strategy approach provided depends on the particular bridge component, element, subsystem, or system. Figure 1.10 shows an example of the overall strategy for selecting a bearing that meets both strength and service life requirements, taken from Chapter 10 of the *Guide*.

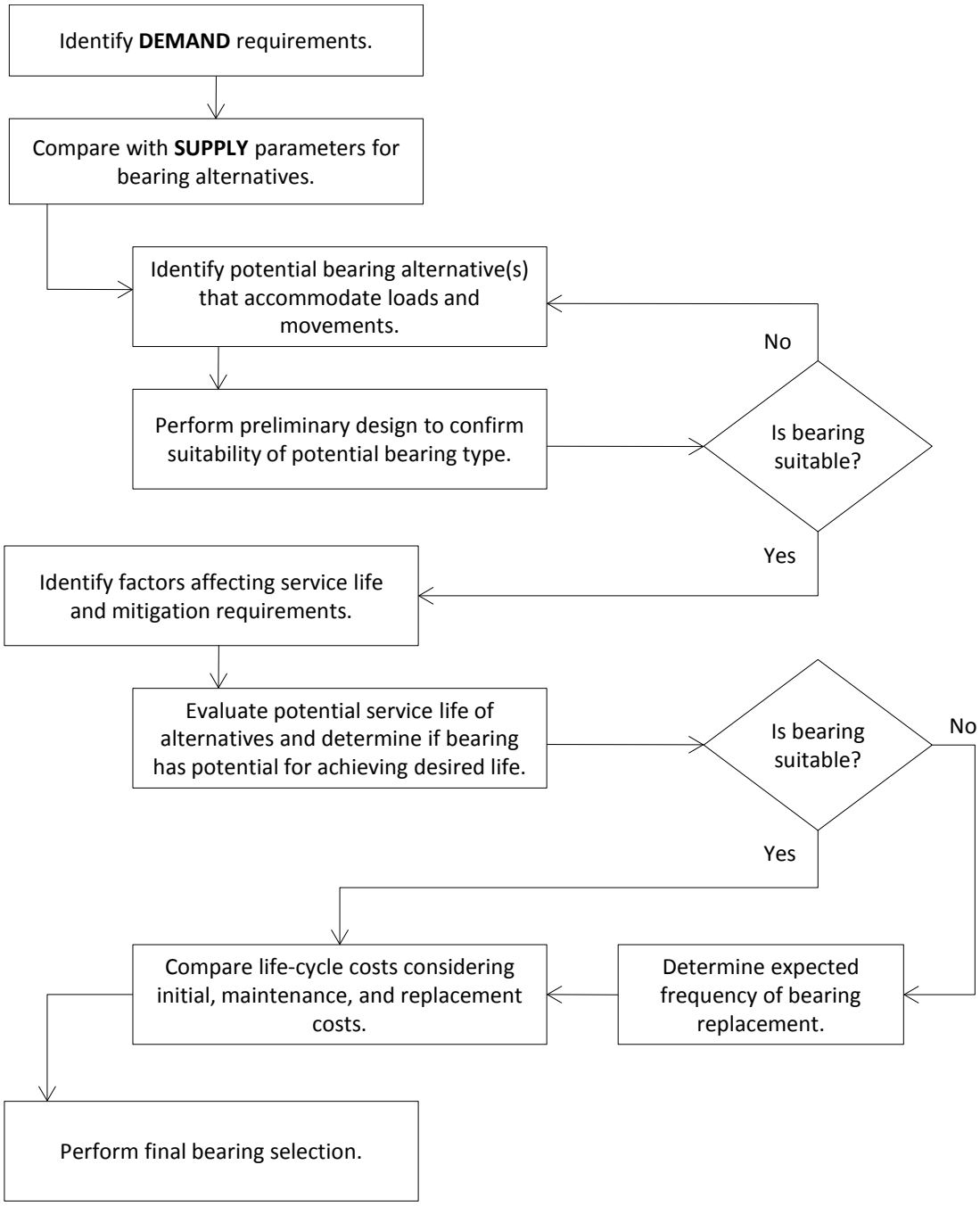


Figure 1.10. Overall strategy for bearing design considering service life.

It should be noted that the chapter devoted to bearings, Chapter 10, identifies factors affecting service life of bearings and provides potential solutions for each. This information, combined with steps outlined in the flowchart, can be used as a rational approach for selecting an appropriate bearing that meets project requirements with emphasis on service life.

Examples and Tools

Most chapters include examples demonstrating the application of strategies in that chapter.

1.7 QUANTIFYING SERVICE LIFE OF BRIDGE ELEMENT, COMPONENT, SUBSYSTEM, AND SYSTEM

One of the important steps in developing a systematic, comprehensive service life design plan for bridges is the capability to predict the expected service life of various bridge elements, components, and subsystems, which in turn will dictate the service life of the bridge system. This is Step 9a shown in Figure 1.6. The service life prediction capability is important for developing maintenance, retrofit, and replacement plans, which are an integral part of service life design process. The objective of this section is to provide an overview of the methodology used in the *Guide* for predicting the service life.

Bridge elements, components, subsystems, and systems are subject to the effects of traffic and the environment. These external sources of deterioration act through various mechanisms to cause actual deterioration of bridge elements and eventually failure. The mechanisms of deterioration are the physical laws that govern such deterioration. Deterioration rates can be described using mathematical expressions or empirical/semiempirical models, which are developed using data collected by field monitoring of bridges, laboratory generated data, expert opinions, or combination of available data. Service life is also affected by risk to damage either from traffic or extreme environmental occurrences. The acceptability of this damage is evaluated based on risk. Service life can be extended by minimizing risk or designing for appropriate levels of extreme occurrences.

Enhanced service life for bridge elements, components, subsystems, and systems can be achieved through:

- Use of durable materials,
- Use of either passive or active protection systems,
- Optimum selection of details,
- Optimum maintenance and repair,
- Reduced service level,
- Increased factor of safety or reduction in stress levels, and
- Isolation from risk damage.

To estimate the service life of bridge elements, components, or subsystems quantitatively, the following information is needed:

- Source of deterioration,
- Deterioration mechanism,
- Deterioration models, and
- Failure modes.

The following sections provide information on each of these items.

Source of Deterioration

Traffic related or environmental effects form the basic external cause of deterioration. For example, deicing compounds, an external source of deterioration, can result in corrosion of reinforcement in bridge elements.

Deterioration Mechanism

Deterioration is governed by a process called the deterioration mechanism. For example, sliding surfaces in bearings experience deterioration through horizontal movement and friction between sliding materials, created by truck passages or temperature fluctuations. The horizontal movement and friction in this instance is the deterioration mechanism. In the case of concrete elements, ingress of chloride through concrete causes initiation of corrosion in unprotected steel reinforcement. In this instance, the ingress of chloride is the deterioration mechanism.

Deterioration Models

Deterioration models are used to describe the rate of deterioration. They describe the relationship between the condition of the bridge (or its element) and its time of use, and show how the bridge deteriorates over time. It assumes that no replacements or major repairs are made, but it usually implies that scheduled maintenance actions are performed as planned. The basic model applies either to a bridge system as a whole, or to any of its subsystems, components or elements.

An example of a deterioration curve is presented in Figure 1.11. If the bridge is placed in service at period T_0, its condition gradually declines, and the deterioration curve represents its condition over time. Initially the condition is

good, but after a period of wear and aging, it eventually (at time T_f) reaches an unacceptably low condition C_f. The time period between T_0 and T_f is called Service Life (SL) of the bridge.

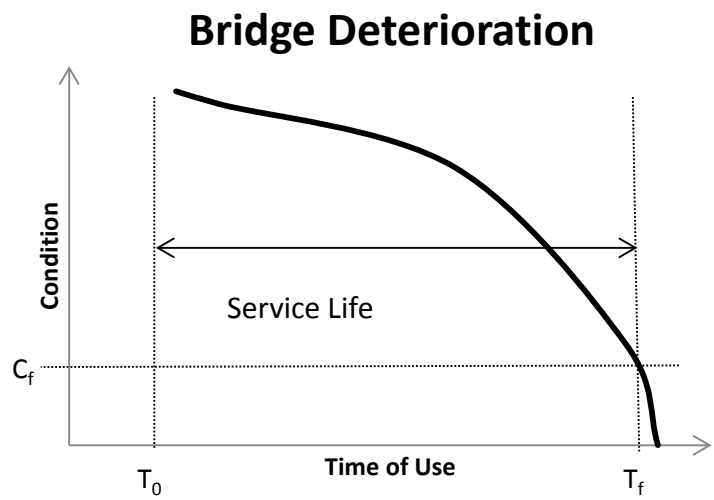


Figure 1.11. An example of bridge deterioration curve.

In practice, the development of realistic behavioral deterioration models is a data-intensive process complicated by lack of knowledge of the underlying physical and chemical processes fostering deterioration, as well as by the data availability. At the present time the available deterioration models, which are based on long-term data collection, are very limited. Further, as time passes, the quality of the bridge design and construction improves. As a result, application of data collected from existing bridges to predict performance of future bridges should be practiced with caution.

Deterioration models capable of quantitatively predicting the service life of bridge elements, components, subsystems, or systems are very limited or nonexistent. The most acceptable deterioration model is in the form of the solution to Fick's second law, used to predict the rate of chloride ingress through concrete cover. This model, including its limitations, is described in Chapter 5 of the *Guide*. It is expected that with time, more deterioration models will become available and will greatly enhance quantification of the service life of bridge elements, components, subsystems, or systems.

As shown on Figure 1.12, if left alone a bridge will deteriorate over the period of its service life. However, in most cases a bridge is not left to follow the basic deterioration path and reach an unacceptable condition without interruption. The agency responsible for the bridge will from time to time undertake repairs, rehabilitations, and

renewals that return conditions to higher levels and extend its service life. During these interventions, the condition rate of the bridge condition increases, as depicted in Figure 1.12.

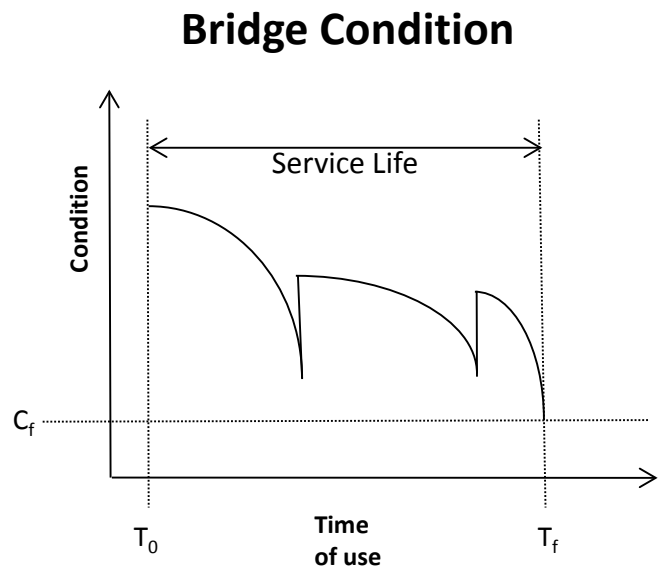


Figure 1.12. Bridge condition life cycle.

Deterioration models can also be based on some level of understanding of the mechanism governing the deterioration and the capability to express the process using a mathematical expression. An example is deterioration of concrete elements because of chloride induced corrosion of reinforcement. The assumption is that ingress of chloride through the concrete element is governed by Fick's second law, which assumes a homogeneous material.

In the case of chloride and carbonation induced corrosion, there is some level of agreement within the scientific community as to the existence of deterioration models. However, for other deterioration modes such as sulfate attack, alkali-silica reactivity (ASR), and freeze/thaw or wear and abrasion, there is a lack of adequate models. Further, as described earlier, the use of deterioration models to predict the time to initiate corrosion of reinforcement embedded within certain distances of the concrete surface because of chloride ingress, should be approached with caution. Following is brief description of a deterioration model for chloride induced corrosion.

There are different approaches to solving Fick's second law. A finite difference approach, or the use of error functions, is reported in published literatures. Equation 1.4 is an error function solution of Fick's second law, capable of predicting the chloride concentration level at various depths within the concrete element.

$$C_{crit.} = C(x = a, t) = C_0 + (C_{S,\Delta x} - C_0).\left[1 - erf\,\frac{a - \Delta x}{2.\sqrt{D_{app,C}.t}}\right] \qquad \textbf{EQ 1.4}$$

Where:

C_{crit}	=	critical chloride content [wt.-%/c]
C(x,t)	=	content of chlorides in the concrete at a depth x (structure surface: x=0 m) and at time t [wt.-%/c]
C_0	=	initial chloride content of the concrete [wt.-%/c]
$C_{S,\Delta x}$:	=	chloride content at a depth Δx and a certain point of time t [wt.
x	=	depth with a corresponding content of chlorides C(x,t) [mm]
a	=	concrete cover [mm]
Δx	=	depth of the convection zone (concrete layer, up to which the process of chloride penetration differs from Fick's 2nd law of diffusion) [mm]
$D_{app,C}$	=	apparent coefficient of chloride diffusion through concrete [mm2/years]
t	=	time [years]
erf	=	error function

Equation 1.4 should be used in conjunction with probabilistic approaches to account for variability of several parameters, such as apparent coefficient of diffusion, chloride concentration, and critical chloride level to start corrosion. Furthermore, the diffusivity of concrete through different layers of concrete element is not uniform. Equation 1.4 predicts the chloride content in the structure at a given depth (x) and time (t). This number is given by the left-hand side of the equation, C(x,t).

The C(x,t) obtained from Equation 1.4 is then compared to the critical chloride content, C_{crit} , which is the value determined to be the point at which corrosion starts. When the chloride level at a given depth, x, of the structure is reached, the critical value, the corrosion is assumed to initiate. The service life of concrete element can then be assumed to consist of the time period to initiate corrosion plus the time period for propagation of the corrosion to the point that will limit the functionality of concrete element. This process is depicted in Figure 1.13.

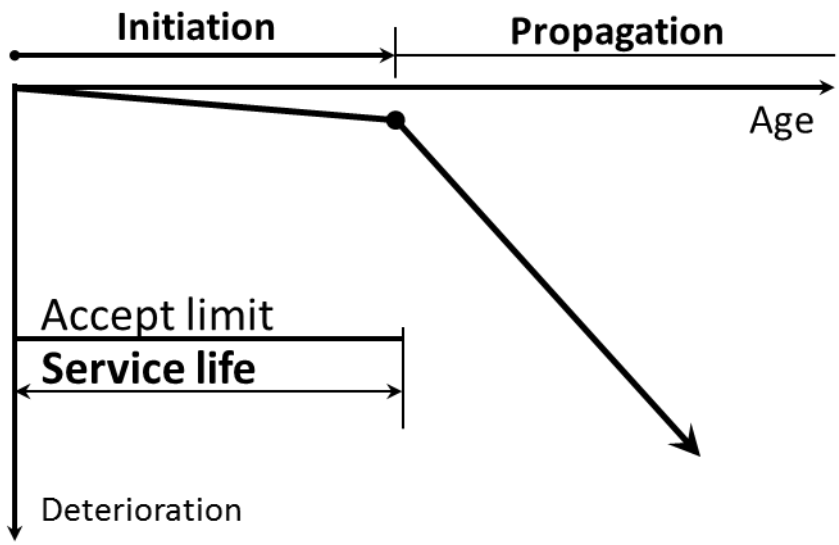


Figure 1.13. Relationship between damage and service life. (Source COWI, Denmark)

Failure Modes

Sources of deterioration (such as deicing compounds), acting through deterioration mechanisms (such as ingress of chloride through concrete cover), and described by deterioration models (such as solution to Fick's second law) result in failure modes (corrosion of reinforcement, causing corrosion induced cracking and loss of strength). The final failure could consist of several stages, such as start and propagation phases.

Service Life Estimation

In the *Guide*, two general philosophies are presented to estimate the service life of bridge elements, components, and subsystems. The end result of quantifying the service life of bridge elements, components, or subsystems is to establish the t_s, the service life of bridge elements, components, or subsystems, and compare it to specified service life of the bridge system to determine the need for retrofit or replacement strategies if needed.

Two general design approaches for service life are:

- Finite Service Life Approach, and
- Target Service Life Approach.

When the design service life, t_s, of the bridge element, component, or subsystem, established through one of the two design approaches for service life philosophies is less than the specified design life of the bridge system, t_D, the

bridge element, component, or subsystem under consideration could be replaced to achieve the specified design life of the bridge system.

The major difference between the two approaches for service life design is the existence of well-accepted deterioration models, which is needed before using the finite service life approach.

Finite Service Life Approach

Bridge elements, components, and subsystems designed using the finite service life approach, should have service life that is greater than or equal to specified bridge system service life. Otherwise, the bridge element, component, or subsystem under consideration must be retrofitted or replaced to allow the bridge to continue providing its intended function until reaching the specified bridge system service life. In the finite service life approach, the service life of the bridge components, elements, or subsystems is estimated using well-accepted deterioration models. The existence of deterioration models is therefore essential for use of the finite service life approach.

The deterioration models are generally developed using one of the following approaches:

- Mathematical models which describe the deterioration rate. These models could be approximate or based on laws of physics.
- Empirical or semiempirical models developed using data collected from laboratory or field performance of bridges. Fatigue models used in the *LRFD Specifications* is an example of empirical deterioration model.
- Empirical models based on expert opinions or experiences. Examples include various models used in Pontis™.

Where deterioration models exist, the service life design could be in the form of:

Full probabilistic approach. This approach requires having probability distribution functions for all variables used in the deterioration model.

Semiprobabilistic or partial load factor approach. This approach is developed using full probabilistic approach. It is equivalent to using load and resistance factors in the *LRFD Specifications* versus using full probabilistic approach (such as using Mont Carlo simulation) to design or rate bridges.

Target Service Life Approach

In many instances the deterioration models are not available or their applicability is questionable. In these situations, available alternatives are 1) use of high performing material that does not deteriorate, such as use of stainless steel, an approach is generally referred to as the avoidance of deterioration method within European practice, or 2) use material that based on experience, or based on expert opinion, could provide a specified or a target service life. If the estimated service life of the bridge element, component, or subsystem is less than the specified service life of the bridge, retrofit or replacement strategies must be specified, allowing the bridge system to continue providing its intended function.

The major difference between finite service life and target service life design approaches is that in finite service life design approach, the condition of the bridge element, component, and subsystem can be traced over time using deterioration models, whereas in target service life design approach, only the total expected service life is estimated. The specified target service life of bridge element, component, or subsystem is mainly established based on experience or expert opinion, and could vary significantly from assumed values. Nevertheless, specifying a target service life for a given bridge element, component, or subsystem allows the bridge owner to plan and anticipate necessary maintenance actions and places demands on the designer to incorporate necessary design features where needed. For example, the service life of PTFE sliding surfaces in bearing devices could be assumed to be about ten years (target service life of 10 years). The designer must then incorporate necessary mechanisms to lift the bridge and replace the sliding surfaces, preferably while maintaining traffic. On the other hand, the bridge owner must plan and anticipate the replacement of sliding surfaces every 10 years.

1.8 OWNER'S MANUAL

In instances where specified by the owner and for major and complex bridges, the final step in the design for service life process is the development of a bridge Owner's Manual, which summarizes the processes used for design for service life and provides complete descriptions of outcomes and recommendations. The intention is to equip the owner with the necessary knowledge to keep the bridge operational for the specified service life period. The bridge Owner's Manual should be provided to the owner at the time of opening the bridge to traffic, following an independent review process described in the next section.

The entire process used for design for service life should be well documented and include assumptions, limitations, and any other information of which the owner should be aware, including complete information with respect to "hot spots" within various bridge elements, components, or subsystems that will require closer inspection, maintenance, retrofit, or replacement. The Owner's Manual should include a complete management plan with respect to service life, including information on timely maintenance actions, and identify replacement items and methodologies for replacement with information on the required level of traffic interruption, if any. In the case of major and complex bridges it is suggested that a bridge instrumentation and monitoring plan be developed and be tied to the bridge service life management plan. Additional information to be included in the Owner's Manual after construction should include the actual material properties of critical bridge elements versus the assumed values used in the design process. Such information is important for future bridge rating.

With respect to major and complex bridges, the designer should use sound engineering judgment for determining the level and extent of information to be included in the Owner's Manual. The bridge Owner's Manual is analogous to the design calculations that are customarily provided to the bridge owner, except that the Owner's Manual contains much more detailed information.

1.9 INDEPENDENT REVIEW OF DESIGN FOR SERVICE LIFE PROCESS

The design for service life processes, results, and recommendations as summarized in the bridge Owner's Manual, should be checked by an independent and knowledgeable third party. This independent check is analogous to an independent design check typically conducted for bridge design.

1.10 SUMMARY OF STEPS FOR DESIGN FOR SERVICE LIFE FOR SPECIFIC BRIDGE ELEMENT, COMPONENT, AND SUBSYSTEM

This section provides a summary of the steps in design for service life. Detailed description of individual steps is provided in Section 1.4.

Bridge elements, components, and subsystems can deteriorate at different rates and have different service lives. This governs the service life of a bridge system, which is reached when the service life of critical bridge elements, components, or subsystems is exhausted beyond being repaired or replaced economically or because of other considerations.

The general steps in design for service life for particular a bridge element, component or subsystem can be summarized are as follows:

Step 1. Identify the project requirements, particularly those that will influence the service life.

Step 2. Identify feasible bridge systems capable of meeting the project demand.

Step 3. Select each feasible bridge system one at a time and complete Steps 4 through 10.

Step 4. Identify the factors that influence the service life of bridge elements, components, and subsystems, such as traffic and environmental factors.

Step 5. Identify modes of failures and consequences. For instance, the corrosion of reinforcement causing corrosion induced cracking and loss of strength.

Step 6. Identify suitable approaches for mitigating the failure modes or assessing risk of damage, through life cycle cost analysis. For example, use of better performing materials for sliding surfaces in bearings or use of material prone to deterioration at lower initial cost.

Step 7. Modify the bridge element, component, or subsystem under consideration, using the selected strategy and ensure compatibility of different strategies used for various bridge elements, components, or subsystems. This step may involve the need to develop several alternatives.

Step 8. For each modified alternative, estimate the service life of the bridge element, component or subsystem using Finite or Target Service Life Design approaches.

Step 9. For each modified alternative, compare the service life of the bridge element, component, or subsystem to the service life of the bridge system and develop appropriate maintenance, retrofit, and/or replacement plan.

Step 10. For each modified alternative, develop design, fabrication, construction, operation, maintenance, replacement, and management plans for achieving the specified design life for the bridge system.

Step 11. For each modified alternative, conduct life cycle cost analysis for each feasible bridge system meeting strength and service life requirements, and select the optimum bridge system

Step 12. When specified by the owner or in cases of major and complex bridges, document the entire design for service life processes in a document called the Owner's Manual. Conduct an independent review of the document and provide it to bridge owner at the time of opening the bridge to traffic.

1.11 APPROACHES TO USING THE *GUIDE*

This section provides a limited example demonstrating the use of the *Guide* and how to implement systematic approaches for design for service life. The example is not inclusive and considers an isolated component of the bridge without considering the remaining bridge elements, components, or subsystems. Further, the example, for the sake of demonstration, uses Life-365, which has limitations when applied to horizontal surfaces, such as bridge decks. Life-365 uses the solution to Fick's second law to predict deterioration of concrete elements subjected to chloride ingress. While this approach has merits for vertical surfaces, such as columns under compression, its applicability to horizontal surfaces, such as bridge deck, is not warranted. In the case of bridge decks, the existence of cracks violates the assumption of a homogeneous material in Fick's second law. The use of Life-365 for the bridge deck example here is for demonstration purposes, as it includes life cycle cost analysis in addition to predicting time to initiate corrosion and propagation.

The following sections provide an overall description of the bridge used for the example, and illustrate the steps taken in design for service life for one component of the bridge in isolation.

1.11.1 Example Bridge Description

The example bridge is a 1400-ft long structure carrying four lanes of high volume traffic with pedestrian sidewalks and bicycle lanes. The bridge crosses over low-volume urban local roads, a railroad, and a navigable waterway. Refer to Figures 1.14 and 1.15 for a rendering of the project concept. Figure 1.16 shows the bridge deck system that will be used for this example.

Figure 1.14. Concept aerial of bridge project.

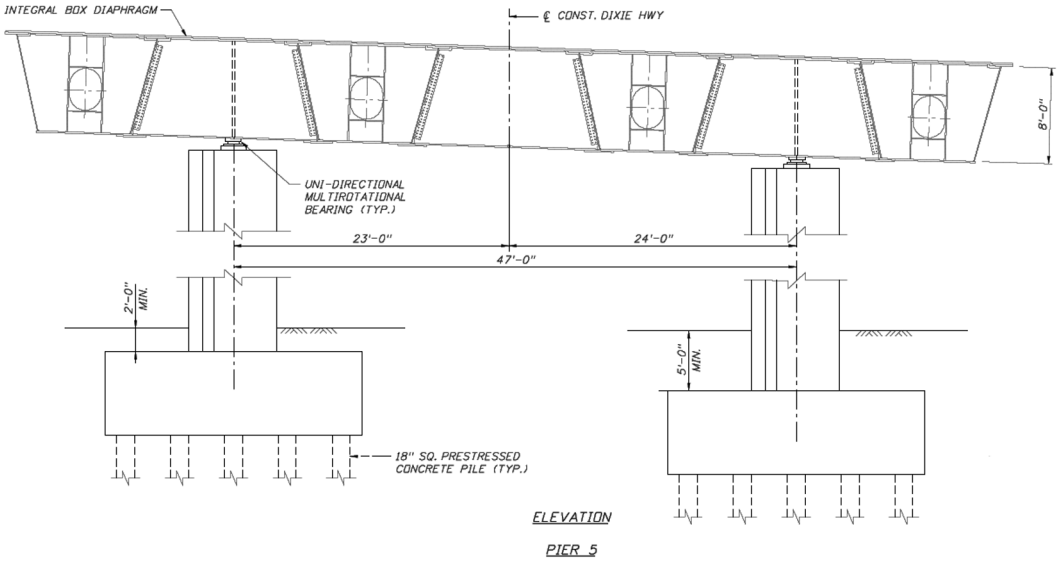


Figure 1.15. Typical superstructure/substructure configuration.

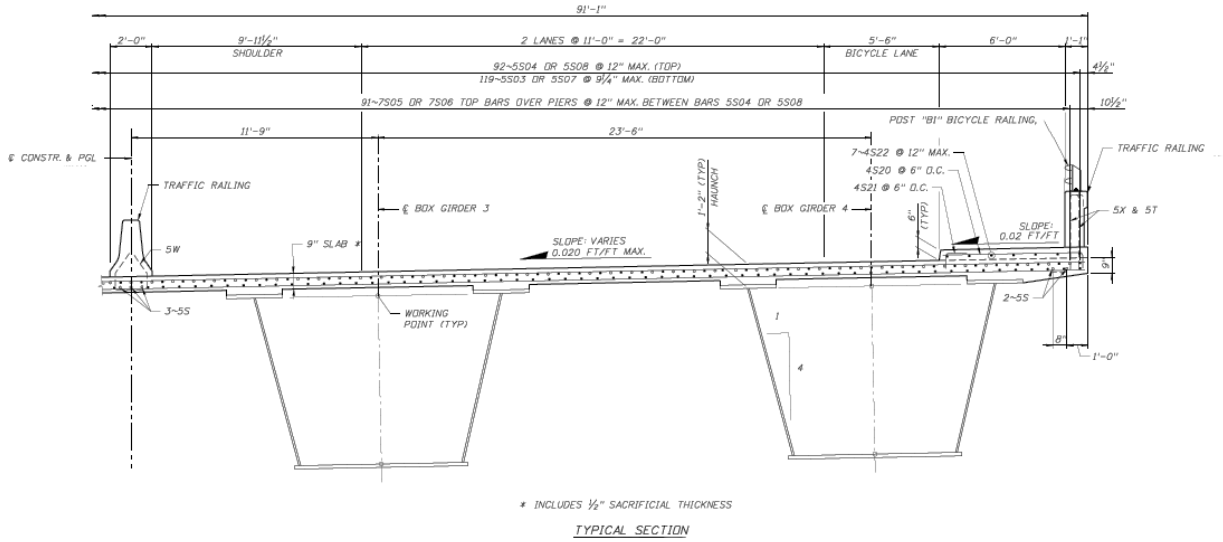


Figure 1.16. Superstructure section—bridge deck—cast-in-place option and designed based on the *LRFD Specifications*.

The bridge setting includes the following characteristics, which have an influence on the service life:

- Located in a cold environment where deicing salts are utilized, and multiple cycles of freeze/thaw are anticipated;
- Located in an area where studded tires are used in the winter;
- Subjected to potential overloads with 20-kip tire loads in an HL93 truck configuration;
- Spans over a navigable waterway with primarily brackish conditions, and located adjacent to a park with water access for jet skis;
- Located near the coastline with possible salt water storm surge with potential hurricane force winds up to 150 mph wind gusts; and
- Located in an area where local aggregates are subject to ASR.

1.11.2 Steps in Addressing Service Life Design

The first step in the process for addressing the service life design issue for bridge deck is to follow the flowchart shown in Section 4.5 of the *Guide* and repeated in Figure 1.17.

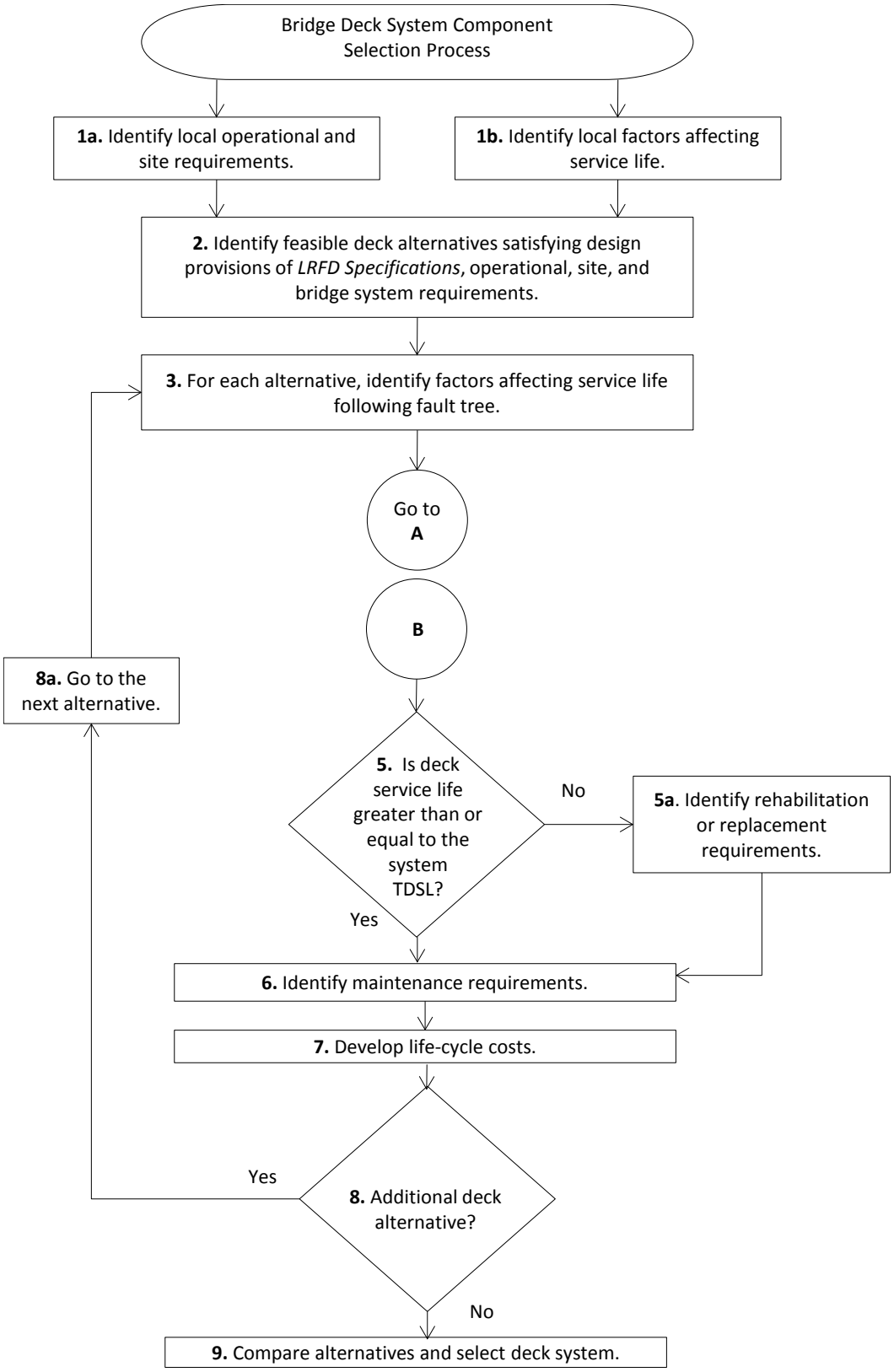


Figure 1.17. Flowchart given in Chapter 4 of the *Guide*. (Figure 4.18)

Table 1.1 is used to extract the information needed to address the requirements of Steps 1a and 1b shown in Figure 1.17.

Table 1.1. Operational and Local Factors to be Considered—Steps 1a and 1b in Figure 1.17.

Operational Category	**Operational Criteria to Be Specified**
Traffic capacity requirements	Urban arterial, 4 lanes, 40 mph
Traffic volumes and required capacity	24000 ADT NB and SB ⇦*
Truck volumes	10% ⇦
Special vehicle uses	Overload possible ⇦
The local environment or man-made hazard category	Maintain 2 existing lanes ⇦
Mixed use requirements	Traffic, pedestrians, bicycle lane
Vehicle loads and special vehicle load requirements	HL 93 with typical legal and permit loads No special construction loads Overload with 20 kip tire loads (HL93 truck configuration) Studded tires used in winter ⇦
Service Life Category	**Service Life Criteria To Be Specified**
Identify bridge importance	Critical
Identify target design service life	100 years ⇦
Potential for future bridge widening	N/A
Potential for future widening of crossed roadways	N/A
Vertical clearance requirements related to future bridge widening or widening of crossed roadways	N/A
Local Site Category	**Local Site Criteria To Be Specified**
Geometry	See plan. Overall bridge length: 1400' Curvature: curve 1 R=1150', curve 2 R=1300', reversing Cross-slope: superelevation transition from +2% to -2% Skew: 35º
Features crossed	Road A: 2 lane urban, 25 mph Road B: 1 lane urban, 25 mph (2 lanes temporary) Railroad: 1 existing track, 4 additional future tracks, potential commuter rail corridor with mixed freight usage Navigable waterway: USCG jurisdiction, small vessels and jet skies
Horizontal clearances	Standard clear zones with no barrier protection No railroad crash walls allowed No piers in waterway
Vertical clearances	Road A: 16.5' plus 2' for raising of tracks (for RR bridge replacement) Road B: 16.5' Railroad: 23.5' plus 2' for raising of tracks (for RR Bridge replacement) Navigable waterway: 15' minimum (bank to bank)
Hydraulic or waterway requirements	Greatest flood (500-year) EL. 7.50 (storm surge) Natural bend in channel Historic erosion of south bank
Navigation requirements	No piers in waterway (bank to bank)
Utility issues, carried or crossed	Bridge lighting Relocate underground fiber optic lines
Other physical boundary conditions	Maintain access to adjacent park
Geotechnical considerations	All foundation types acceptable
Environmental considerations	Water in waterway tests with low chlorides, but adjacent mangroves indicate that water may be brackish at times Subject to salt water intrusion from storm surge
Access for construction	Limited to existing right-of-way and railroad agreement
Aesthetics and sustainability	Closed box system required by city (I-girders unacceptable)

Table 1.1 Continued

Local Environmental or Man-Made Hazard Category	**Local Environmental or Man-Made Hazard Criteria to Be Specified**
Thermal climate	Cold climate, solar radiation, zone 3 Deicing salts used, multiple cycles of freeze/thaw, ice flow
Coastal climate	Brackish conditions
Chemical climate	ASR susceptible
Hydraulic action hazard—flood or scour	500-year water velocity/discharge: 4.8 fps / 11,200 cfs
Wind action hazard	Hurricane zone (150 mph), exposure C
Drainage requirements	50-year storm
Vehicle/vessel collision susceptibility	Vehicle collision to be addressed Vessel collision potential negligible (mostly pleasure craft)
Fire or blast susceptibility	Minimal combustible materials on route
Seismic susceptibility	Seismic zone 1
Construction Constraints Category	**Construction Constraints Criteria to Be Specified**
Constructability requirements	Phasing not required
Construction schedule requirements	Accommodate short schedule demands
Special local construction preferences	Do not use cast-in-place concrete boxes

*Note: Rows identified by this arrow ⇦ point out items used in developing the design example listed in this section.

The information shown in Table 1.1 is developed from project requirements, and exemplifies the type of information necessary for layout and service life evaluation of the entire bridge system. It is well beyond that needed for simply evaluating a bridge deck, but is provided here for completeness. The pertinent issues in Table 1.1, with respect to the bridge deck, are indicated by an arrow at the right side of the table.

The next step is to identify the possible bridge deck alternatives, as indicated by Step 2 in Figure 1.17. Information provided in Section 2.3.2 of Chapter 2 and Section 4.2.1 of Chapter 4 of the *Guide* can be used to obtain advantages and disadvantages of various deck systems. In the case of major and complex bridges the designer may consider feasible bridge deck systems based on local preferences and experiences. In the current example, it is assumed that only the cast-in-place option is selected, as indicated in the typical girder cross section shown in Figure 1.16.

Following are the steps shown in the flowchart given in Chapter 4, Section 4.4 of the *Guide* (Figure 4.19) and repeated in Figure 1.18 is the next step.

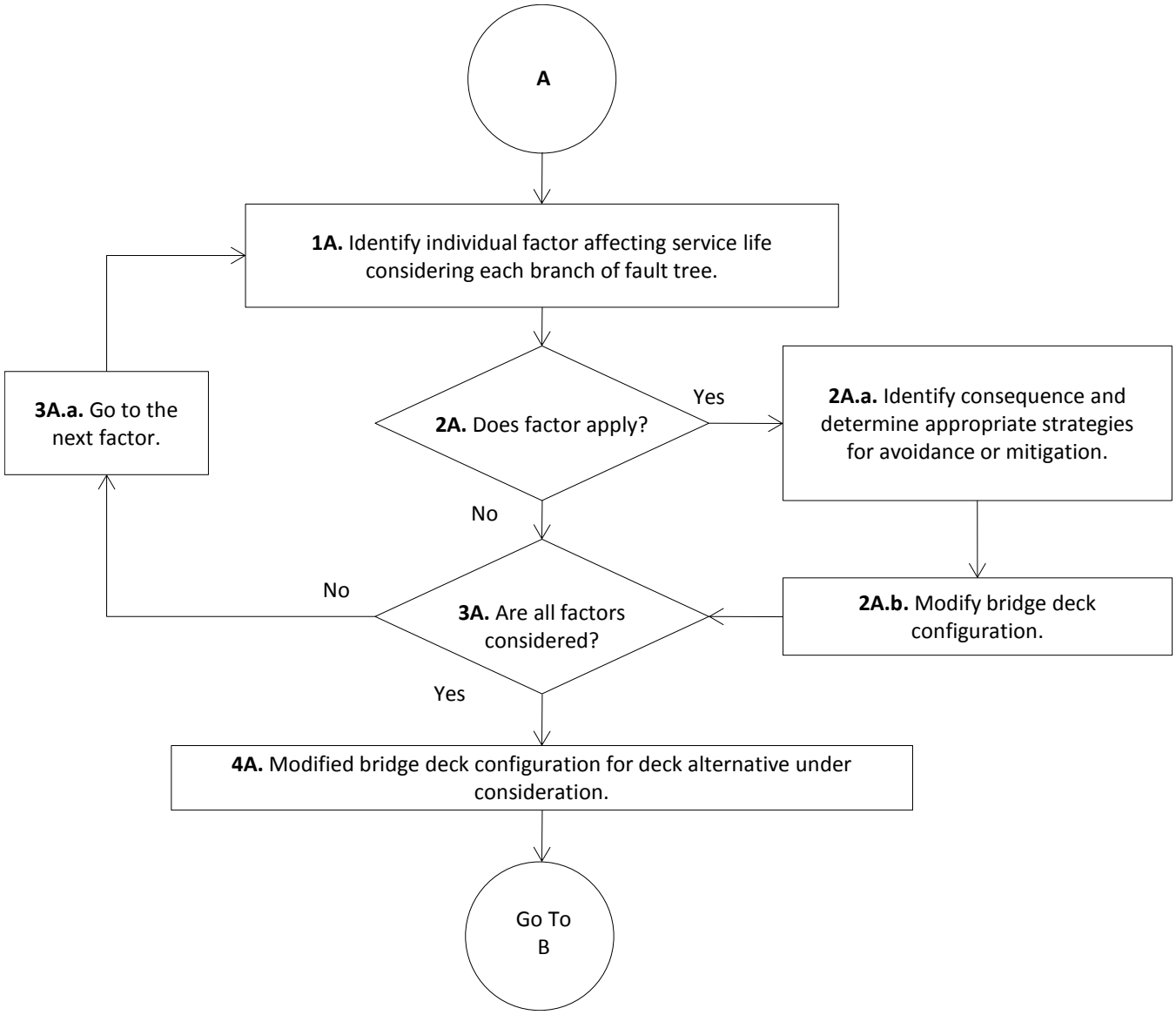


Figure 1.18. Flowchart to identify factors affecting service life. (Figure 4.19 of *Guide*)

Figure 1.18 aids in identifying factors that affect service life of bridge deck and in selecting possible strategies capable of mitigating the adverse effects of these factors. Identifying the factors that affect service life of bridge deck can be accomplished using the fault trees provided in Chapter 4, Section 4.2 of the *Guide*. Navigating through the fault tree can be simplified through the use of software. Figure 1.19 shows an example of what an Excel-based solution could look like. Using the software shown in Figure 1.19, the user selects applicable factors from the first fault tree layer then continues through successive layers until reaching the last levels, which are depicted as circles. The items listed in each circle are the factors that will have to be addressed in the design for service life process. Each factor has the capability of reducing the service life of bridge deck. Chapter 4 identifies one or more strategies capable of mitigating the effect of each particular factor listed in the circles.

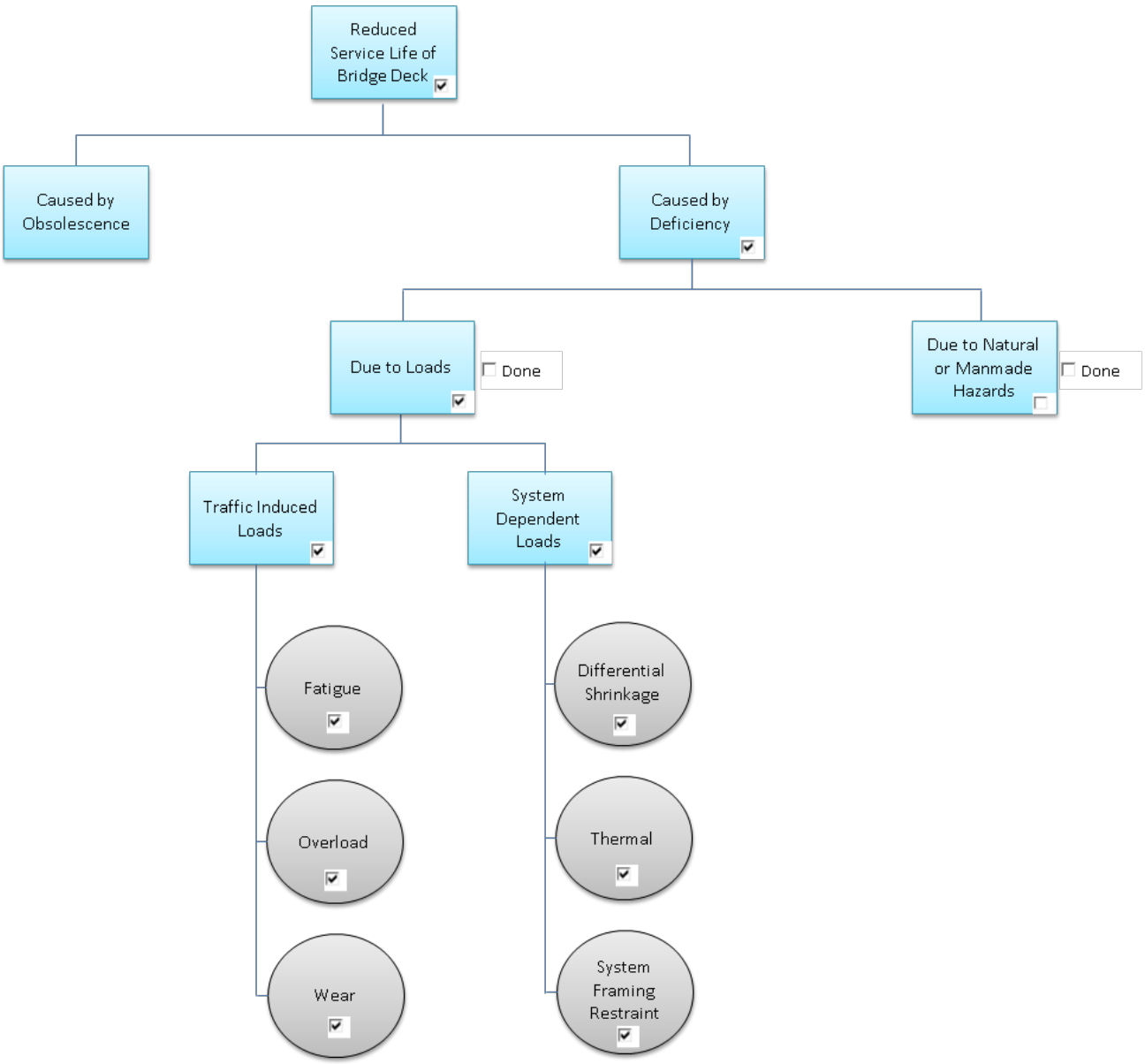


Figure 1.19. Screen shot of navigation through fault tree, using an Excel worksheet.

Figure 1.19 shows branches of the fault tree that are applicable to the example under consideration. In the first layer, based on the project requirements, a decision is made that the service life of bridge deck can only be reduced due to deficiencies. The second layer states that either loads or natural or man-made hazards can cause bridge deck deficiencies. Both are judged to be applicable and are therefore selected. Figure 1.19 then shows the progression through the fault tree on the branch related to the factor, due to loads. The fourth layer in Figure 1.19 states that either traffic-induced loads or system-dependent loads can reduce service life of bridge deck. Again, both factors are judged to be applicable and the boxes are checked. Further, the reduction in service life of bridge deck as a result of traffic-

induced loads can be caused by fatigue, overload, or wear, as depicted by the circles. All circles are applicable to the example. Figure 1.19 also identifies each factor that is capable of reducing the service life of bridge deck as a result of system-dependent loads, as shown in circles and identified as differential shrinkage, thermal, and system framing restraint. Based on project requirements, these factors are also judged applicable for consideration when addressing service life design.

The circles in the fault tree signify:

- Issues capable of reducing service life, and
- Issues for which the designer needs to develop strategy(ies) to mitigate the problem.

The strategies to address the individual items listed in each circle are provided in the *Guide*, in this case in Chapter 4 on bridge decks. It should also be noted that for most factors listed in the circles, the *Guide* identifies more than just one possible strategy.

To complete the process of navigating through the fault tree, all branches applicable to the problem need to be considered and applicable circles checked. Figure 1.20 shows the remainder of the fault tree with the "Due to Loads" branch collapsed for clarity. The decision for selecting the applicable circles is based on specific project conditions and requirements.

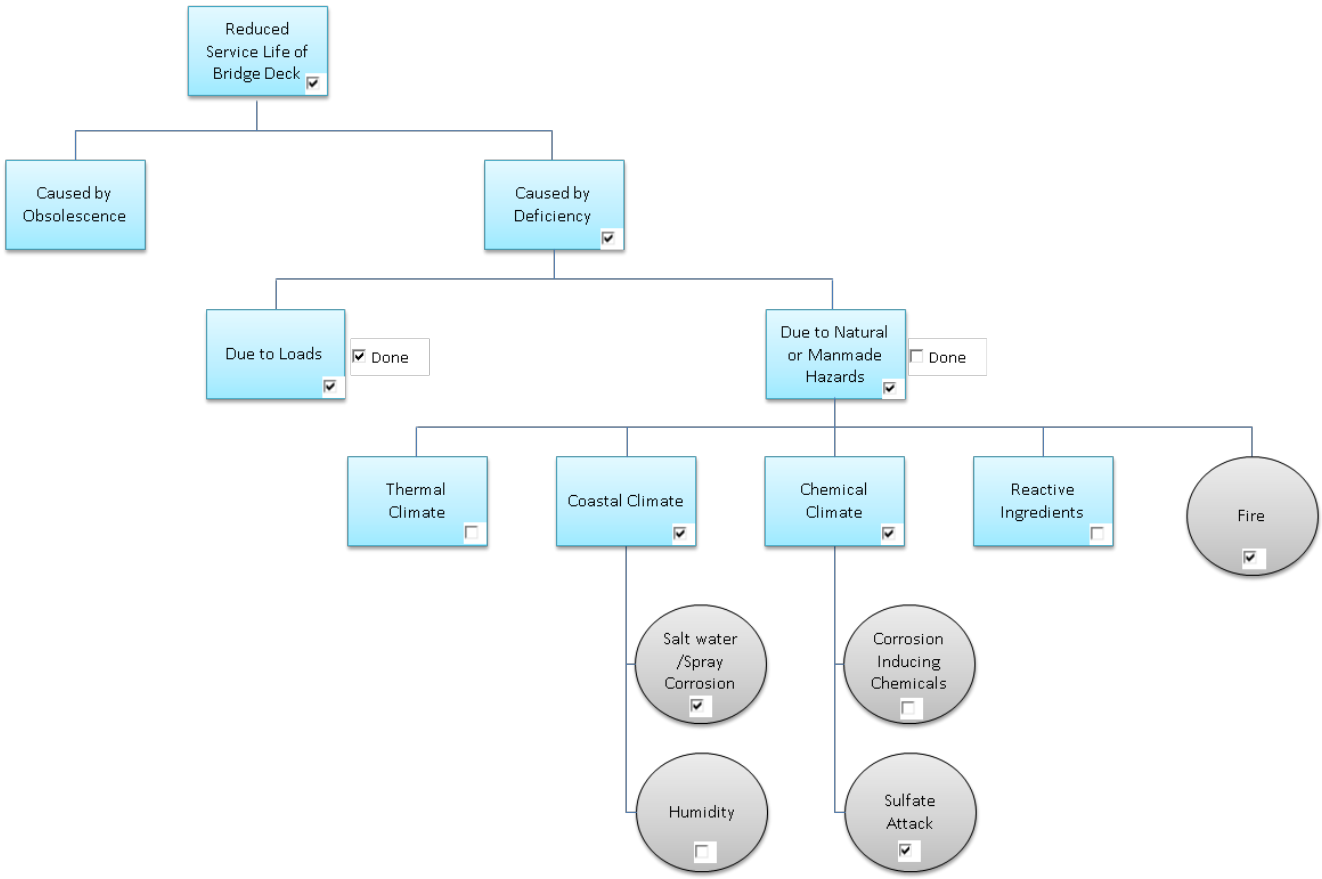


Figure 1.20. Segment of the fault tree with applicable circles checked for the example.

1.11.3 Developing Strategies and Alternative Solutions

Once the fault tree is completed and all applicable factors are identified, the individual strategies capable of mitigating the factors can be collected. If software is utilized to work through the fault tree then this step can be automated based on the selections made. Table 1.2 summarizes the list of individual strategies capable of mitigating the factors developed for the example problem as identified as checked circles in Figures 1.19 and 1.20.

Table 1.2. List of Individual Strategies Capable of Mitigating Factors Affecting Service Life for Example Problem.

Service Life Issue	Corresponding Project Requirements	Section	Mitigating Strategy	Advantage	Disadvantage
Overload	HL93 with 20 kip wheel load, applied once a month	4.3.1.1	Increase deck thickness	Minimizes cracking	Adds weight to bridge structure, increases cost
			Minimize bar spacing for given amount of steel	Improves crack control	More labor to install and higher cost
Fatigue	24000 ADT NB and SB and 10% truck volume	4.3.1.1	Design per *LRFD Specifications*	Minimizes possibility of reinforcement failure	May increase area of steel
Wear and Abrasion	Studded tires on high level of service bridge	4.3.1.1	Implement concrete mix design strategies	Identified in Chapter 3	Identified in Chapter 3
			Implement membranes and overlays	Protects surface from direct contact with tires	Requires periodic rehabilitation every 10 to 20 years
Differential Shrinkage	No Requirements for Example Problem	4.3.2.1	Use low modulus concrete mix design for composite decks	Allows additional strain to be accommodated up to cracking stress	Typically lower in strength and may be subject to wear and abrasion
			Use high creep concrete mix designed for composite decks	Reduces locked-in stresses	Uncommon mix design. Difficult to assess stress relief
			Develop composite action after concrete has hardened	Allows slippage between deck and supporting members, minimizing locked-in stresses	Little experience with experimental systems. Friction reduction difficult to assess. Introduces numerous construction joints. Grout integrity issues in closed void systems.
			Use precast deck panels	Allows slippage between deck and supporting members, minimizing locked-in stresses	Introduces numerous construction joints
System Framing Restraint	Deck shrinkage restraint from shear studs	4.3.2.2	Develop accurate system model	Identifies design criteria for establishing stresses	Restraining force may cause cracking in deck. Refer to Chapter 8.
Reactive Ingredients—ASR/ACR	Local aggregates are reactive	4.3.3	Use materials and mix designs that are not sensitive to aggregate	Refer to Chapter 3	Refer to Chapter 3
Coastal Climate—Humidity	RH average 70%	4.3.3	Use materials that are not sensitive to moisture content	Refer to Chapter 3	Refer to Chapter 3
Thermal Climate—Freeze/Thaw	Multiple cycles of freeze/thaw expected	4.3.3	Refer to Chapter 3 for strategies relating to freeze/thaw	Refer to Chapter 3 for strategies relating to freeze/thaw	Refer to Chapter 3 for strategies relating to freeze/thaw

Table 1.2 Continued

Service Life Issue	Corresponding Project Requirements	Section	Mitigating Strategy	Advantage	Disadvantage
Thermal Climate—Deicing Salts	Potential for high chloride concentrations	4.3.3	Use impermeable concrete	Increases passivity around reinforcement. Refer to Chapter 5.	High initial shrinkage, which can result in cracking
			Use corrosion resistant reinforcement	Eliminates deck spalls, delaminations, and cracking from reinforcement	High cost. Limited availability. Some performance issues as noted in Chapter 3.
			Use waterproof membranes/overlays	Minimizes intrusion of dissolved chlorides into deck. Easily rehabilitated	Requires periodic rehabilitation every 10 to 20 years
			Use external protection methods, such as cathodic protection	Reduces corrosion. Refer to Chapter 5.	High cost. Requires extensive maintenance and anode/battery
			Use effective drainage to keep surface dry, minimize ponding	Minimizes intrusion of dissolved chlorides into deck	Requires maintenance of drainage
			Use periodic pressure washing to remove contaminants	Minimizes intrusion of dissolved chlorides into deck. Low cost.	Requires dedicated maintenance staff and appropriate budget
			Use non-chloride based de-icing solution	Eliminates corrosion from chlorides	High cos.
Coastal Climate—Salt Spray	Splash potential from jet skis (roostertails)	4.3.3	Use impermeable concrete	Increases passivity around reinforcement. Refer to Chapter 5.	Typically lower in strength and may be subject to wear and abrasion
			Use corrosion resistant reinforcement	Eliminates deck spalls, delaminations, and cracking from reinforcement corrosion. Refer to Chapter 3.	High cost. Limited availability. Some performance issues as noted in Chapter 3
			Use waterproof membranes/overlays on travel services of bridge deck	Minimizes intrusion of dissolved chlorides into deck	Requires periodic rehabilitation every 10 to 20 years
			Use external protection methods, such as cathodic protection	Reduces corrosion. Refer to Chapter 5.	High cost. Requires extensive maintenance and anode/battery
			Use sealers on non-travel surfaces of bridge deck	Minimizes intrusion of dissolved chlorides into deck	Requires periodic rehabilitation every 5 to 10 years
			Use corrosion resistant stay in place forms on bottom of bridge deck	Minimizes intrusion of dissolved chlorides into deck	Difficult to inspect
			Use effective drainage to keep surface dry	Minimizes intrusion of dissolved chlorides into deck	Requires maintenance of drainage and periodic cleaning
			Use periodic pressure washing to remove contaminants	Minimizes intrusion of dissolved chlorides into deck	Requires dedicated maintenance staff and appropriate budget

At this point, the designer has developed a complete list of strategies that can be used for mitigating various factors capable of affecting the service life of bridge deck, based on project requirements. Table 1.2 also provides sections of the *Guide* that the user can refer to for more information, and lists advantages and disadvantages of the defined strategies. Some of the strategies may contradict each other while others may result in similar results. Intentionally, the *Guide* does not provide a single strategy or attempt to identify the best strategy. In many cases strategies to mitigate the individual factors capable of reducing service life are context sensitive, meaning that the best strategy is very much dependent on such factors as local practice, environment, or owner preferences.

As mentioned, some of the strategies may contradict each other and some may be more preferable based on local practices or owner preferences. Consequently, the next step for the designer is to select strategies that are desirable for each factor affecting the service life. Table 1.3 shows a narrower list of strategies extracted from the complete list given in Table 1.2. The first row in Table 1.3 shows the applicable factors that can reduce the service life of bridge deck under consideration. These factors were obtained by navigating through branches of the fault tree.

Table 1.3. List of Strategies Specific for Developing Deck Alternatives.

Overload	Fatigue	Wear	System Restraint	Differential Shrinkage	Deicing	Freeze/ Thaw	Salt spray	Humidity	ASR/ACR
Increase Deck Thinness	Design per AASHTO	Concrete mix	Accurate modeling during structural analysis of the bridge system	Concrete mix—use mix with low modulus	Impermeable Concrete	Concrete mix—air content	Stainless steel	Use aggregate that are not sensitive to humidity	Concrete mix non-reactive aggregate
		Membrane and overlay			Stainless Steel		Stay in place metal deck to protect bottom		
		Increase thickness			Specify non-chloride based deicing		Deck bottom sealer and top membrane		
					Membrane and Overlay				

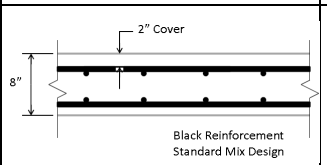
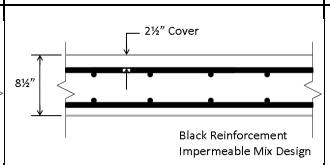
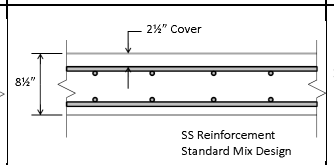
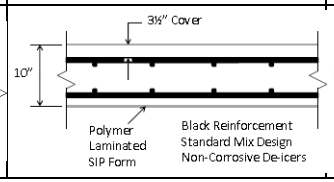
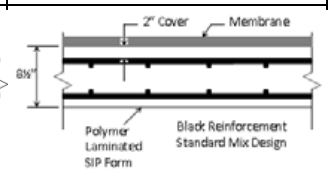
In developing the information shown in Table 1.3, the designer may consider many factors and ensure that there are no contradicting strategies. For instance, appropriate concrete mix is specified as one strategy to address wear, differential shrinkage, freeze/thaw, humidity, and ASR/ACR. The designer must ensure that a concrete mix capable of

addressing all of these issues can be developed. Otherwise, for a particular factor the designer may be forced to use another strategy. As an example for differential shrinkage, a low modulus of elasticity concrete is needed, whereas for wear, a high modulus of elasticity is needed. Consequently, to address wear and differential shrinkage, the same concrete mix cannot be used to mitigate both factors, and within a given deck alternative, one of these factors should be mitigated using a different strategy.

The next step in the process is to develop possible deck alternatives that meet both *LRFD Specifications* requirements and *Guide* requirements.

Using the information provided in Table 1.3 and ensuring there is no contradiction among strategies to mitigate various factors, Table 1.4 shows four possible deck alternatives capable of mitigating all factors affecting the service life of the bridge for the example under consideration. The four alternatives shown in Table 1.4 are project specific solutions. It is also possible to automate this step by first identifying all possible deck alternatives based on all possible combinations of strategies listed in Table 1.3 and then eliminating those judged not feasible, because of contradiction among strategies.

Table 1.4. Developing Deck Alternatives Meeting Both AASHTO and *Guide* Requirements.

	Configuration per AASHTO Design Requirements	Alternative 1	Alternative 2	Alternative 3	Alternative 4
Overload	N/A	Increase thickness by 0.5 in.	Increase thickness by 0.5 in.	Increase thickness by 0.5 in.	Increase thickness by 0.5 in.
Fatigue	N/A	Design per AASHTO	Design per AASHTO	Design per AASHTO	Design per AASHTO
Wear	N/A	Increase thickness by 0.5 in.	Increase thickness by 0.5 in.	Increase thickness by 0.5 in.	Membrane and overlay
System Restraint	N/A	Accurate modeling during analysis of the system	Accurate modeling during analysis of the system	Accurate modeling during analysis of the system	Accurate modeling during analysis of the system
Differential Shrinkage	N/A	Concrete mix—use mix with low modulus	Concrete mix—use mix with low modulus	Concrete mix—use mix with low modulus	Concrete mix—use mix with low modulus
Deicing	N/A	Impermeable concrete—5% silica fume, 10% fly ash	Stainless steel reinforcement	Increase cover by 1 in.	Membrane and overlay
Freeze/Thaw	N/A	Concrete mix—air content	Concrete mix—air content	Concrete mix—air content	Concrete mix—air content and membrane and overlay
Salt Spray	N/A	Impermeable concrete—5% silica fume, 10% fly ash	Stainless steel reinforcement	Seal the bottom using stay in place metal deck—top is protected by increasing cover	Seal the bottom using stay in place metal deck—top is protected by membrane and overlay
Coastal—Humidity	N/A	Use aggregates that are not sensitive to humidity	Use aggregate that are not sensitive to humidity	Use aggregate that are not sensitive to humidity	Use aggregate that are not sensitive to humidity
ASR/ACR	N/A	Concrete mix—nonreactive aggregate	Concrete mix—nonreactive aggregate	Concrete mix—nonreactive aggregate	Concrete mix—nonreactive aggregate
Strategy	As designed (LRFD strength)	As designed with thickened deck and impermeable concrete	As designed with thickened deck and stainless steel reinforcement	As designed with thickened deck, black reinforcement, deck bottom sealed with stay in place form	As designed with black reinforcement, deck bottom sealed by metal deck and top of deck membrane
Configuration					

For each alternative shown in Table 1.4, rows two through 11 show the service life design factors identified in Table 1.3 and corresponding strategies selected for each alternative. Incorporating all of the select strategies listed in rows 2 through 11 for each of the four alternatives results in modified deck configurations shown in row 13 of Table 1.4. Development of the four deck alternatives signifies the completion of step 4a shown in Figure 1.18. It should be mentioned that strategies listed in Table 1.3 could lead to the development of additional deck alternatives. However, for the sake of simplicity, only four alternatives are shown in Table 1.4.

The first option shown in Table 1.4, column 2, represents a design that meets the strength requirements stated in *LRFD Specifications*. The total deck thickness is 8 in., with no considerations for any of the factors capable of reducing the service life. The main feature of the first alternative, Alt. 1 in Table 1.4, is having impermeable concrete, with 5% silica fume and 10% fly ash. The addition of fly ash is assumed to impact the rate of reduction in the diffusivity of concrete, a parameter used in estimating the time to initiate corrosion.

The second alternative, Alt. 2 in Table 1.4, relies mainly on the use of stainless steel reinforcement, in this case Grade 316 stainless steel, to prevent corrosion. Alternative 3, Alt. 3 in Table 1.4, uses regular concrete with increased cover to delay the time to initiate the corrosion. Finally, the fourth alternative, Alt. 4 in Table 1.4, uses a membrane and overlay to address corrosion.

All alternatives use increased thickness to address overload, increasing the deck thickness by 0.5 in. Table 1.4 also addresses additional strategies capable of reducing service life of the various alternatives.

1.11.4 Evaluating Alternatives

As shown in Figure 1.17, the next step is predicting the service life of each alternative, Step 5 in Figure 1.17, and comparing it to the design service life of the bridge system, as specified by the owner and project requirements. Based on the outcome, the development of rehabilitation or replacement requirements, Step 5a in Figure 1.17, and a maintenance plan, Step 6 in Figure 1.17, may be necessary. The last step for the bridge deck alternative under consideration, as shown in Figure 1.17, Step 7, is to perform life cycle cost analysis for comparison.

As described in Chapter 2 on system selection, the designer must also consider the interaction that might exist between various parts of the bridge. This step is not covered for this example.

The potential service life of each deck alternate can be calculated based on the assumption that the main mode of deterioration is ingress of chloride into concrete, which can result in corrosion of reinforcement. One approach is to use the solution to Fick's second law as shown below.

In a one-dimensional case, Fick's Law can be expressed and illustrated as follows:

$$C_{(x,t)} = C_o\left(1 - erf\,\frac{x}{2\sqrt{D_c t}}\right) \qquad \textbf{EQ 1.5}$$

Where:

$C_{(x,t)}$ = chloride concentration at depth x and time t
C_0 = surface chloride concentration (kg/m^3 or lb/yd^3)
D_c = chloride diffusion constant (cm^2/yr or in^2/yr)
erf = error function (from standard mathematical tables)

The use of Fick's Law to determine the time of corrosion initiation is described in Chapter 5, Section 2, of the *Guide*. Equation 1.5 can be used to assess ingress of chloride through the concrete cover. As an example, Figure 1.21 indicates the type of information that can be developed, which shows chloride concentration through the deck thickness for three time periods, after a deck is cast.

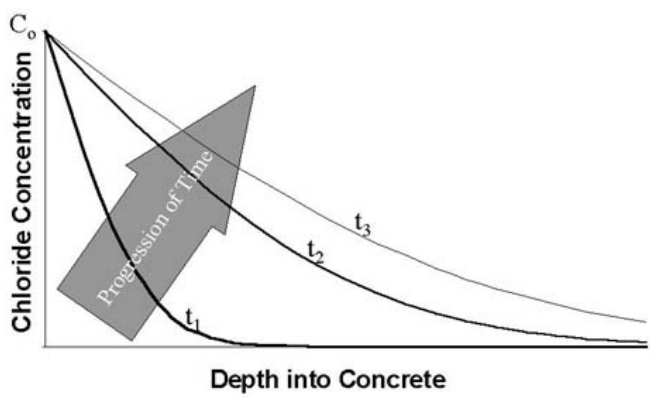


Figure 1.21. Chloride concentration within concrete over time.

The information shown in Figure 1.21 can be used to predict the time when corrosion will be initiated, which in turn can be used to estimate the service life of the bridge deck, if corrosion of reinforcement is the main mode of deterioration. The Fick's law in this case is the deterioration model.

To complete the example, Life-365, a free program developed by the concrete industry, is used to conduct the life cycle cost analysis and assist in selecting an optimum solution. Life-365 uses a finite-difference approach to solve

Fick's second law and to estimate the time to initiation of corrosion. Other approaches such as error function solution to Fick's second law, Equation 1.5, could also be used.

The solution to Fick's second law estimates the time to initiation of corrosion, t_i. For the example, it is assumed (assumption within Life-365) that once the corrosion is initiated, the time to propagate the corrosion to the point at which repair is needed, t_r , is a constant six years, regardless of concrete mix used. Following the time period, $t_i + t_r$, Life-365 assumes repair action at set time intervals, say every 10 or 20 years, and set cost per unit area in square feet. Further, within each repair cycle, it is assumed that only a portion of the deck area will be repaired. For instance, within each repair cycle only 10% or 20% of the deck will need repair.

The time to initiate corrosion depends on concrete mix and preventive measures, such as use of stainless steel, concrete cover, or membranes. Life-365 follows the guidance and terminology in *ASTM E-917 Standard Practice for Estimating the Life Cycle Cost of Building System.* The final number that can be used to select the optimal deck alternative can be the life cycle cost, which is the initial cost plus the present value of all future rehabilitation costs over the desired service life, in this case 100 years.

Table 1.5 shows the input parameters used within Life-365 to conduct a life cycle cost analysis for each of the four alternatives shown in Table 1.4.

Table 1.5. Input Parameters in Life-365 for All Four Alternatives.

Name	Value
Base Units	US units
Concentration Units	(% weight concentration)
Type of Structure	Slabs and walls (1-D)
Third Dimension (ft^2)	10,000
Base Year	2011
Study Period (years)	100
Inflation Rate (%)	1.6
Discount Rate (%)	2
Location	Massachusetts
Sublocation	Boston
Exposure Type	Urban highway bridges
Max Surface Concentration (% weight of concrete.)	0.68
Time to buildup (years)	7.1
Temperature Profile	
January (ºF)	-1.9
February (ºF)	-0.9
March (ºF)	3.7
April (ºF)	8.9
May (ºF)	14.6
June (ºF)	19.8
July (ºF)	23.1
August (ºF)	22.2
September (ºF)	18.2
October (ºF)	12.7
November (ºF)	7.4
December (ºF)	0.9

It is assumed that the bridge is located in Boston, Massachusetts; has a required service life of the deck of 100 years; and, for the sake of comparison, a total surface area of the deck of 10,000 square ft. Table 1.5 also shows the yearly temperature profile used. The diffusion coefficient and ingress of chloride are influenced by temperature fluctuation. The input parameters shown in Table 1.5 are applicable to all four alternatives shown in Table 1.4. The specific input and end results for each alternative are shown in Table 1.6.

Table 1.6. Parameters Specific to Each Alternative.

Analysis Parameters	AASHTO Design	Alternative 1	Alternative 2	Alternative 3	Alternative 4
Concrete Mix Type	Regular	Silica Fume	Regular	Regular	Regular
Water Cement Ratio (w/cm)	0.42	0.35	0.42	0.42	0.42
Slag (%)	0	0	0	0	0
Fly Ash (%)	0	10	0	0	0
Silica Fume (%)	0	5	0	0	0
Steel type	Black Steel	Black Steel	316 Stainless	Black Steel	Black Steel
Steel (%)	1.2	1.2	1.2	1.2	1.2
Propagation Period (years)	6	6	6	6	6
Inhibitor	<none>	<none>	<none>	<none>	<none>
Barrier	<none>	<none>	<none>	<none>	Membrane
D28 (in.×in./s)	1.38E-08	4.09E-09	1.38E-08	1.38E-08	1.38E-08
m	0.2	0.28	0.2	0.2	0.2
Initiation (years)	6.6	34.8	100	15.2	22.4
Propagation (years)	6	6	6	6	6
Service Life (years)	12.6	40.8	106	21.2	28.4
Use user mix cost? (true/false)	FALSE	FALSE	FALSE	FALSE	FALSE
User Mix Cost ($/yd^3)	0	0	0	0	
Depth (in.)	8	9	9	10	8.5
Depth to Reinforcement. (in.)	2	2.5	2.5	3.5	2
Unit Costs					
Area to repair (%)	20	10	10	20	5
Repair cost ($/ft^2)	50	50	50	50	20
Repair interval (years)	10	10	10	10	10
Base Mix Cost ($/yd^3)	80	90	80	80	80
Black Steel Cost ($/lb)	0.45	0.45	0.45	0.45	0.45
Epoxy Steel Cost ($/lb)	0.6	0.6	0.6	0.6	0.6
Stainless Steel Cost ($/lb)	2.99	2.99	2.99	2.99	2.99
Inhibitor Cost ($/lb)	5.68	5.68	5.68	5.68	5.68
Membrane Cost ($/ft^2)	7	7	7	7	7
Sealant Cost ($/ft^2)	0.65	0.65	0.65	0.65	0.65
Results					
Repair interval	10	10	-	10	10
Base Cost ($)	37,215	44,645	152,753	46,519	39,541
Barrier Cost ($)	0	0	0	0	70,000
Repair Cost ($)	737,461	232,905	0	644,595	62,711
Life-Cycle Cost ($)	774,676	277,550	152,753	691,114	172,252

As indicated in Table 1.6, alternatives 1, 2, 3, and 4 use the same concrete mix, hereafter referred to as regular mix, which was used for the base option designed in accordance with *LRFD Specifications* (option designated as AASHTO Design in Table 1.4). Alternative 1 uses 5% silica fume to make the concrete impermeable. Alternative 2

uses stainless steel. Alternative 3 uses increased concrete cover to delay the adverse effects of corrosion. Alternative 4 uses a membrane and overlay.

Following is a brief description of the results for each alternative.

Deck Design Based on *LRFD Specifications*. As shown in Table 1.6, in the case of the AASHTO base design, corrosion starts after 6.6 years (shown as Initiation). Thereafter, the propagation of corrosion to the point of needed repair is 6 years. At the end of 12.6 years, the repair and maintenance actions are assumed to start and continue every 10 years, during which 20% of the surface area is repaired. These assumptions are used for the sake of demonstration and will vary based on various DOT preferences and practices. Figure 1.22 shows the total life cycle cost based on present value.

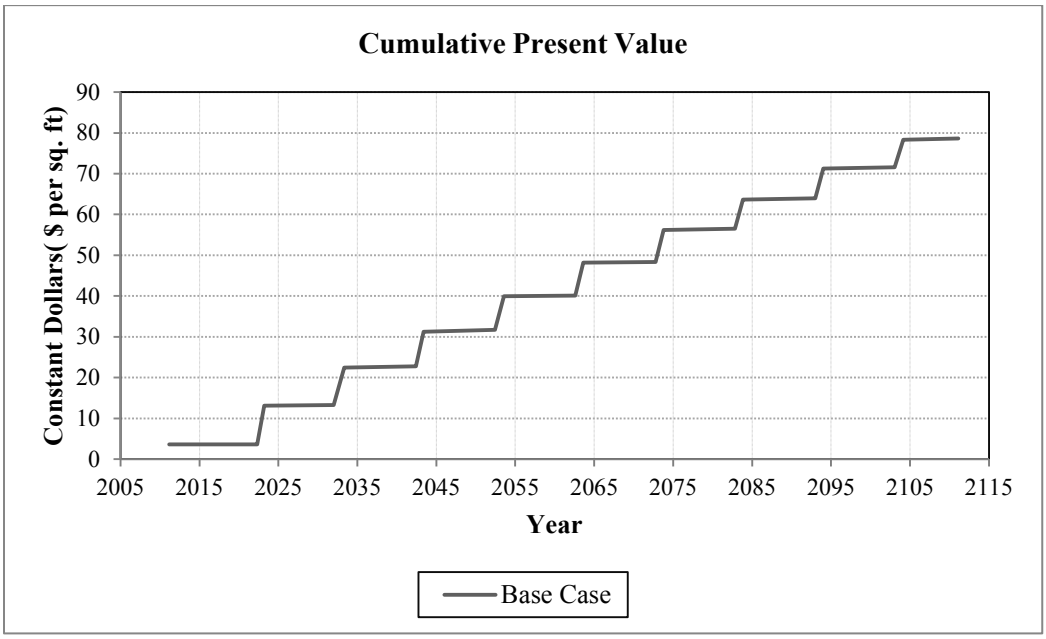


Figure 1.22. Total life cycle cost for *LRFD Specifications* based design.

The total life cycle cost for the AASHTO based design is $774,676, which is shown in the last row of Table 1.6. An inflation rate of 1.6% and a discount rate of 2% were used in developing the total life cycle cost. As indicated in Table 1.6, the initial cost of AASHTO base design is the lowest of the alternatives ($37, 215). However, the total life cycle cost is the highest ($774,676). It should also be mentioned that these life cycle cost analyses ignore the user costs, or any other cost to society during repair and closure to traffic, and these costs can be significant.

Alternative 1. This alternative uses impermeable concrete by incorporating 5% silica Fume in the mix. As a result, the initial unit cost of concrete is assumed to increase by ten dollars per cubic yard. The initiation of corrosion

starts at 34.8 years after casting and the propagation phase lasts six years. Therefore, after 40.8 years the repair procedure is assumed to stat. It is assumed that the repair procedure is to be conducted every 10 years and during each repair cycle, 10% of the surface area is assumed to need repair. Other repair alternatives can consist of complete replacement of deck every 40 years. The total life cycle cost of Alternative 1 for the assumptions made is $277, 550, as shown in Table 1.6. The initial cost of Alternative 1, $44,645, is slightly higher than initial cost for AASHTO base design ($37,215).

Alternative 2. This alternative uses Grade 316 stainless steel to address the corrosion of reinforcement issue. The time to initiation of corrosion for this alternative is more than 100 years and consequently no repair action is needed. The initial cost of using stainless steel is the highest among all alternatives ($152,753). However, the total life cycle cost associated with Alternative 2, $152,753 as shown in Table 1.6, is the lowest among all alternatives.

Alternative 3. This alternative uses increased cover to delay the initiation of corrosion. Using increased cover delays the corrosion initiation from for AASHTO base design from 6.6 years to 15.2 years, as indicated in Table 1.6. The total cost of Alternative 3 is relatively high, $691,114. There does not seem to be much benefit in using this alternative, especially considering that increasing the concrete cover will subject the substructure and foundations to higher dead loads.

Alternative 4. Alternative 4 uses a membrane and overlay to prevent corrosion of reinforcement. The total deck thickness is only 8.5 in. compared to 9 in. and 10 in. used for Alternatives 1, 2 and 3. The repair cost per square foot for this alternative is assumed to be lower at $\$20/ft^2$ as compared to $\$50/ft^2$ for others when a membrane is used. It is assumed that a high quality membrane at $\$7/ft^2$ is used and that it will last 75 years. Consequently, the repair will involve replacing damaged overlay areas, which are assumed to be 5% of the total surface area during each repair cycle—every 10 years after 28.4 years of first installation (see Table 1.6). At $172,252, the total life cycle cost of this alternative, using membrane, is very low, while the initial cost ($39,541 + $70,000) is more than twice the AASHTO base design of $37,215. It should also be \mentioned that the use of a membrane could be much more economical than that indicated by this example. For instance, the calculation leads to the conclusion that corrosion will start after 28.4 years, which is not realistic. The concrete deck below the membrane could last a long time without any need for repair, and any needed repair action would only be for replacing the thin overlay, which could be achieved quickly

with minimal interruption to traffic. These factors are not considered in conducting the life cycle cost analysis for this alternative.

1.11.5 Summary and Conclusion

The table below provides a summary of results for all alternatives. Using this information it is feasible to conclude that use of stainless steel or membrane plus overlay can provide the best economy.

Table 1.7. Alternative Summary.

Alternative	Main Feature to address corrosion	Initial cost	Life cycle cost
AASHTO Base Design	N/A	$37,215	$774,676
1	Impermeable concrete using silica fume	$44,645	$277,550
2	Use of 316-stainless steel	$152,753	$152,753
3	Increasing concrete cover	$46,519	$691,114
4	Using membrane and overlay	$109,541	$172,252

1.12 FUTURE DEVELOPMENT OF THE *GUIDE*

The *Guide* provides a general, comprehensive framework for designing new bridges and rehabilitating existing bridges for service life. The approach presented by the *Guide* is flexible and can be adapted as new information becomes available. The *Guide* also provides a platform for developing customized manuals by individual State DOTs or for developing a customized and systematic approach for service life design of major and complex bridges.

Design for service life is a context sensitive problem and local agency practices and preferences are important. Customizing the *Guide* can be achieved by using the general framework outlined in these pages and incorporating strategies and solutions preferred by each DOT for factors affecting service life of their bridges.

One of the challenges in developing true service life design is the lack of reliable, available deterioration models that are based on either field data or laws of physics governing the deterioration. Several studies are underway to develop deterioration models for various bridge elements, components, and subsystems. Such information can be incorporated into the *Guide* as they become available. These models are needed to further develop reliable life cycle cost analysis.

There is a need to develop specific life cycle cost analysis tools dedicated to bridges, with the ability to incorporate user costs where applicable. These tools must be flexible enough to allow incorporating new information and deterioration models as they become available.

There is a further need to develop more comprehensive examples, which take into account the interaction between solutions that may seem appropriate for an individual bridge element, component, or subsystem when viewed in isolation, and yet are less than optimum when considering service life solutions for the combined bridge system.

Finally, the significant amount of information provided in the *Guide* is time consuming to comprehend in its entirety. There is a need to automate the use of the *Guide* by developing tools that would facilitate navigating through all of the included information.

CHAPTER 2

BRIDGE SYSTEM SELECTION

2.1 INTRODUCTION

Selecting the proper bridge system and incorporating service life design principles into the planning and design process are critical steps in achieving long-term bridge service life. As it is more cost effective to address service life at the design stage, the design for service life must be approached in a systematic, coherent manner.

This chapter provides essential information, steps, and guidelines for selecting and designing optimum bridge systems for both existing and new bridges. More specific details for certain bridge elements, components, subsystems, and materials are provided in other chapters and are referenced herein.

Commonly-used bridge systems are examined along with their associated challenges and solutions, and the focus is on durability and service life. The discussion covers conventional bridge systems and newer, innovative systems involving accelerated and modular construction. Steel and concrete bridge superstructure types are discussed, but when possible they are not directly compared. Instead, the discussion addresses various service life issues within both steel and concrete superstructures.

Section 2.2 provides general information, advantages, and disadvantages of various bridge elements, components, subsystems, and systems currently in use.

Section 2.3 provides a summary of factors affecting service life of bridge elements, components, subsystems, and systems using a fault tree analysis approach. (A detailed description of the fault tree is provided in Chapter 1.) Section 2.4 then provides strategies that can be used to avoid or mitigate most of the factors affecting service life along with options for enhancing service life of those factors.

Section 2.5 describes a systematic approach for selecting the most appropriate bridge systems that will accommodate operational requirements and site conditions while also achieving the desired target design service life. In addition to primary system selection factors relating to function and initial cost, the necessity to consider service life factors including importance, potential for obsolescence, element and material durability, element maintenance and possible replacement, and life-cycle cost are also stressed.

Foremost, the strategies emphasize that durability and service life of all bridge elements, components, subsystems, and systems must be addressed during the system selection process for new bridges as part of a comprehensive approach to service life design. A similar approach should also be implemented for existing bridges as part of a comprehensive plan for extending service life.

2.2 BRIDGE SYSTEM DESCRIPTION

2.2.1 Bridge System Terminology

Definitions for the various terms that are used herein to describe a bridge—bridge element, component, subsystem, and system—are presented in Chapter 1. The bridge system is referred to as the total of all elements, components, and subsystems that make up an entire bridge.

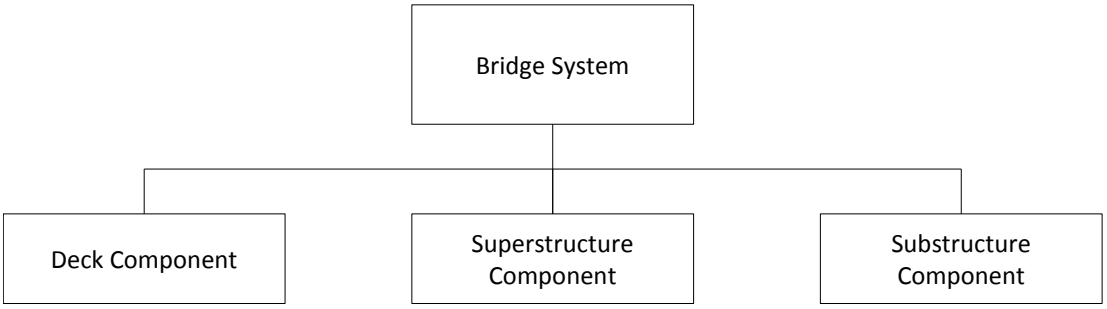


Figure 2.1. Bridge system composition.

As shown in Figure 2.1, a bridge system is initially broken down into three main components: deck, superstructure and substructure. These are the primary categories or groupings of subsystems and elements within a bridge that define specific purpose and function.

The deck component supports and receives live load and must provide a safe, smooth riding surface for traffic. It transfers live load and deck dead load to other components, which in most cases is to the superstructure. The superstructure component supports the deck and transmits loads across the span(s) to the bridge supports.

The substructure component includes all elements that support the superstructure. It transfers vertical and horizontal loads from the superstructure to the foundation material, such as soil or rock. At abutments, additional vertical and horizontal loads applied from the roadway embankment are also resisted.

Often bridge systems are categorized or named by the superstructure type and material. This is discussed further in Section 2.2.3.

2.2.2 Deck Component

2.2.2.1 Deck Elements

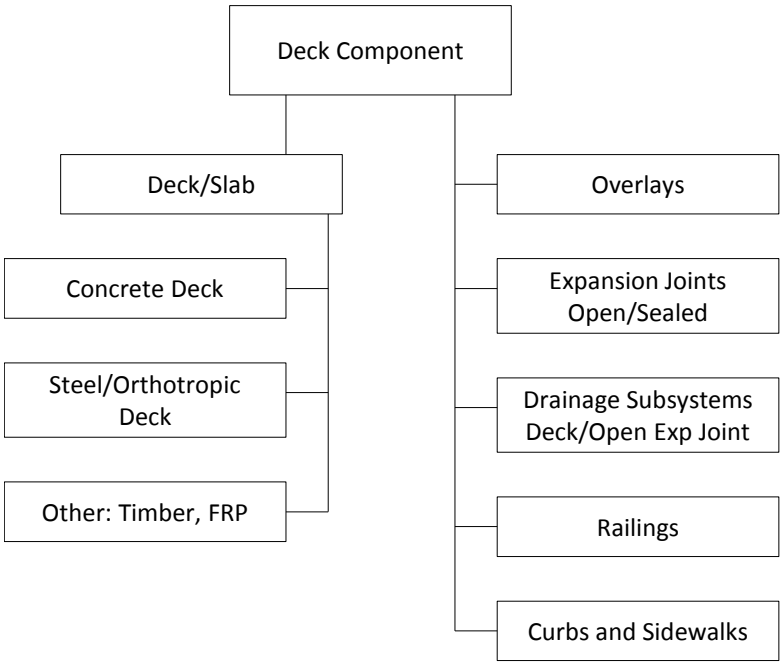


Figure 2.2. Deck component.

Figure 2.2 shows the various elements that make up the deck component, which includes the deck/slab element itself along with other related elements including overlays/wearing surfaces, expansion joints, drainage elements, railings, and curbs and sidewalks. There are various types of deck/slab elements, including concrete decks (either cast-in-place (CIP) or precast), steel/orthotropic decks (including open or concrete-filled steel grids), and other types including timber and FRP.

A detailed discussion of bridge decks and related service life issues is included in Chapter 4. Most decks are composite cast-in-place concrete types but other types composed of precast concrete panels (both partial depth and full depth) and posttensioning have been used, particularly with accelerated construction techniques. Steel deck types, including steel orthotropic decks, are also discussed in Chapter 4. A thorough look at materials used in bridge decks is provided in Chapter 3, and Chapter 9 examines deck expansion devices and joints.

2.2.2.2 Bridge Deck Drainage

The deck drainage subsystem includes inlets or scuppers, pipes and downspouts, and outlets. The main requirement of this subsystem is to remove rainfall-generated runoff from the bridge deck before it collects and spreads excessively in the gutter to encroach onto the traveled roadway. The deck drainage subsystem must be

designed to deter flow and accompanying corrosive deicing chemicals from contacting vulnerable structural members. Proper maintenance of deck drainage elements is essential to avoid clogging and malfunction and such maintenance requirements must be considered in the design.

Open expansion joint drainage includes collection troughs, pipes, and attachments below open expansion joints such as tooth or sliding plate dams to collect drainage, debris, and deicing chemicals that flow through the openings and protect adjacent structural elements. Again, proper maintenance is essential and must be factored into the design.

2.2.2.3 Bridge Railings

Materials used in bridge railing designs include metal, reinforced concrete, and various combinations. Crash testing requirements have been established by FHWA and AASHTO specifications to provide adequate strength depending on vehicle size and speed. Three general categories of bridge railings are typically considered: traffic railings, pedestrian or bicycle railings, and combination railings.

- Traffic railings are designed to contain and safely redirect vehicles.
- Pedestrian or bicycle railings are generally located on the outside edge of a bridge sidewalk, and are designed to safely contain pedestrians or bicyclists. AASHTO specifications require certain heights and limit the opening sizes between members.
- Combination railings are dual purpose railings designed to contain both vehicles and pedestrians or bicyclists, and are generally located at the outside edge of a bridge sidewalk. With this type of railing, there is usually no other barrier between the roadway and sidewalk.

Bridge railings are often located in high splash zones and are often subject to harsh environments that effect steel element corrosion, concrete deterioration and reinforcing bar corrosion. Special protection is necessary for long-term service life of these elements.

Bridge rails are usually cast, following the deck casting. In these instances, special attention should be paid to the cold joint that will be created between deck and cast-in-place rail as it provides a natural path for ingress of moisture and causes corrosion of reinforcement.

2.2.2.4 Curbs and Sidewalks

Curbs and sidewalks are affected similarly to deck slabs. Information relative to these elements is provided in Chapter 4 on bridge decks and Chapter 3 on materials.

2.2.3 Superstructure Component

The superstructure component includes the structural subsystem and bearings. A detailed discussion of bearing elements is given in Chapter 10.

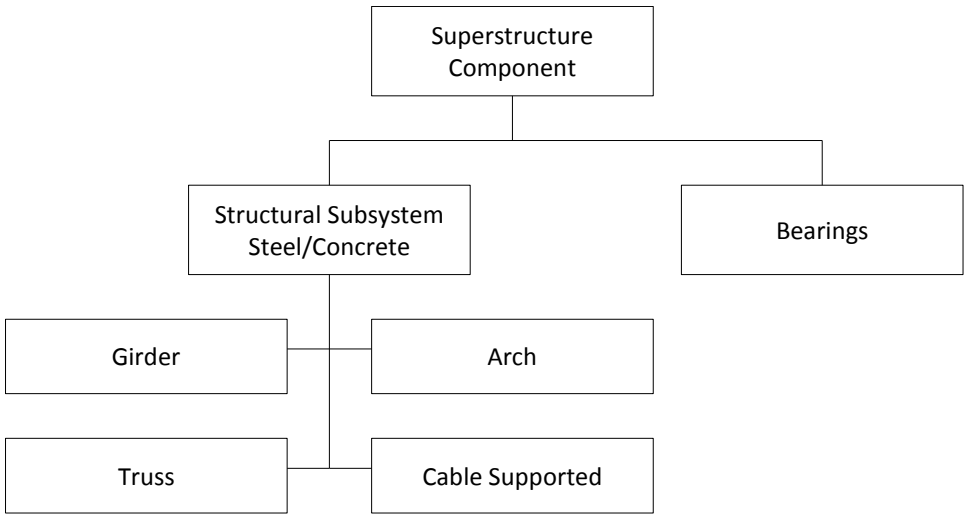


Figure 2.3. Superstructure component.

Figure 2.3 shows the various subsystems and elements that make up the superstructure component.

Superstructures are often categorized by:

- Material type. Steel or concrete are most commonly used.
- Structure subsystem type. Girder subsystems are most often used for common-span lengths within the 300 feet limit. Longer spans typically use girders, trusses, arches or cable-supported types, depending on span length.
- Superstructure continuity. Many older bridges were simple spans, while bridges that are more modern are fully continuous or continuous for live load. The continuous spans provide structural continuity that helps distribute traffic loads in case of excessive deterioration of some of the bridge elements. Structural continuity is especially important in instances of bridges with fracture critical elements.

- Jointless Systems. Integral abutment construction is gaining popularity among many states. Integral pier construction is used only occasionally. Additional information on integral construction is provided in Chapter 8 on jointless bridges.

- Modular Construction. Modular systems using prefabricated superstructure elements such as "topped girders" or preconstructed spans are becoming more popular in instances requiring accelerated construction. Durability of connection details is a concern for long-term service life of these systems. These types of connections are addressed in Chapter 8on jointless bridges.

These categories are often combined in an overall classification of the superstructure, which is frequently used to define the entire bridge system.

Following is a brief discussion of the most common steel and concrete superstructure types.

2.2.3.1 Steel Superstructures

2.2.3.1.1 Steel Girder Superstructures

The most common steel bridge superstructures today are composite multi-girder subsystems that use either rolled beams, plate girders or tub girders. These systems can be single or multi span, and can be either straight or curved. Either of these can also be skewed. Rolled beam superstructures using W-shapes are used in shorter spans up to about 100 ft for simple spans and up to about 120 ft for continuous spans. Recently, deeper rolled shapes (44 in.) for bridge applications have become available. When combined with the simple for dead and continuous for live load concept, these W-shapes can be used for longer spans. Welded plate girders are usually used for spans over 120 ft (NSBA 2008). Figure 2.4 shows typical steel I-girder and tub girder systems. Recently, folded plate beam sections have been developed for short span applications. See Section 2.2.3.1.4 for steel modular systems.

Up until the 1970s, many bridges were designed with systems using two main deck girders, combined with transverse floor beams and longitudinal stringers. The perceived notion that two girder systems are not redundant led to a significant decrease in their use within the United States. However, two girder systems are very popular in Europe. Multi-girder bridges with inherent redundancy are currently preferred by many bridge owners (NSBA 2008). Use of high performance steels with greater fracture toughness, however, has led to re-evaluation of two-girder

systems. Further, a memo dated June 20, 2012 by FHWA has paved the way to more use of two girder systems (FHWA 2012).

Deck I-girder system. (Courtesy HDR)

Tub girder system. (Courtesy Palmer Engineering)

Figure 2.4. Typical steel girder superstructures.

A variation to the typical multi-girder system is the girder-substringer system, which has been used as an economical concept for longer spans beyond approximately 275 ft. This system uses several heavy girders with wide girder spacing and rolled-beam stringers supported midway between the main girders by truss K-type cross frames.

2.2.3.1.2 Continuity in Steel Systems

For many years, bridges were designed as a series of simple spans with expansion joints at each pier because they were easy to design and construct. Leaking joints, however, became a leading cause of structural deterioration and the desire to eliminate joints became prevalent. Multi-span steel girder systems were also shown to be much more efficient when designed as continuous systems, making continuous design more commonplace.

Multi-span systems have typically been fully continuous for both dead load and live load, but new systems, typically with spans up to about 150 ft, have been introduced with a Simple for Dead Load and Continuous for Live Load (SDCL) concept. These systems combine the advantage of simple-span construction with the efficiency of live load continuity and the durability of not having joints that can ultimately leak.

Recently, extensive research studies have been carried out to develop practical details for SDCL steel bridge systems (Azizinamini et al. 2003, Azizinamini et al. 2005a, Azizinamini et al. 2005b; Azizinamini 2013; Lampe et al. 2013; Farimani et al. 2013; Yakel and Azizinamini 2013; Javidi et al. 2013). These studies demonstrate that for the

SDCL steel bridge system, continuity for live load can be provided using steel reinforcement placed over the pier, before casting deck; however, in order to provide continuity, various girder connection details have been used in practice. Figure 2.5 shows two different details in use. Splice plates are sometimes used for top flange connections, which are in tension. The research studies previously referenced, however, do not recommend use of such detail. Bottom flanges in compression are typically butted with plates and wedges.

The disadvantage of the continuity detail with the top flange splice is that the bolts for connecting the top plate have to be tightened after casting the deck. This creates additional construction sequencing with a separate closure pour over the pier.

Continuity with top flange splice. (Courtesy HDR)

Continuity without top flange splice. (Courtesy UNL)

Figure 2.5. Steel bridge system using Simple for Dead Load and Continuous for Live Load (SDCL) concept.

2.2.3.1.3 Long-Span Superstructures

Figure 2.6 shows examples of long-span girder, truss, arch, and cable-stayed bridge systems. Steel plate girder systems have been used for spans up to approximately 500 ft. Spans up to 400 ft have been designed economically with parallel flanges. Variable-depth haunched girders have been used in the 350-ft to 500-ft range. Use of high-performance steel (HPS 70W) has shown economy for plate girder and tub girder systems in most span ranges over 150 ft, particularly in hybrid combinations. Studies have shown that hybrid configurations using conventional grade 50W steel in webs and HPS 70W steel in top and bottom flanges in negative moment regions and bottom flanges in positive moment regions are typically the most economical (Horton et al. 2002). Top flanges in positive moment regions are affected by composite action with the deck and cannot realize enough benefit from the use of higher-strength steel to be economical, except for longer spans. Use of HPS 70W steel in long-span negative moment

ranges can also permit economical parallel flange design without expensive haunches. Trusses, arches, cable-stayed, and suspension systems have also been used for longer-span applications, typically over 500 feet. For spans up to 300 feet, deck girder systems are the most applicable.

Long-span plate girder.

Continuous truss.

Tied arch.

Cable-supported. (Photo by Vince Streano)

Figure 2.6. Long-span steel bridge superstructures. (All Figures Courtesy HDR)

Long-span structures can have special needs for addressing long-term service life relating to unique details, inspection, and maintenance. Access for inspection and maintenance can require elaborate systems of inspection walkways and access ladders, particularly for access to fracture-critical members. Older trusses typically require heavy maintenance because of large surface area to weight ratio, and riveted, built-up members with lacing bars subject to pack-out and other surface corrosion. Truss joint details typically have moisture and debris traps that initiate corrosion, while newer trusses have cleaner surface details that are more easily painted and maintained.

Through structures are subject to splash zone wetting environments for all structural elements near roadway edges. This needs to be considered in a corrosion protection and maintenance plan. Long-span bridges have large

thermal movement requirements that result in large expansion joints. This requires even additional attention to joint maintenance to prevent deck drainage from spilling through. Heavy loads and large thermal movements also require special bearing designs. Navigation channel crossings are subject to vessel collision and need to be protected.

2.2.3.1.4 Steel Modular Systems

New steel systems that provide for accelerated construction include modular construction with pre-topped deck. Modular orthotropic deck systems are also a consideration. These modular systems require special attention to both transverse and longitudinal connection details for achieving long-term durability.

Pre-top modular bridge systems are best suited for accelerated bridge construction applications. In these systems, several units consisting of pre-topped steel or concrete girders are placed side by side and joined together using longitudinal closure pours. The service life of these pre-topped modular systems is significantly influenced by service life of longitudinal closure pour. Pre-topped steel modular systems present two major advantages:

1. The use of steel girders significantly reduces the creep and shrinkage deflections.
2. Pre-topped steel modular units weigh less.

The Folded Plate Bridge System is a new modular system that offers an economical solution for many short-span bridge applications. The system consists of a series of standard shapes that are built by bending flat plates into inverted tub sections using a press break, as shown in Figure 2.7.

Figure 2.7. Making of folded plate girder using break press. (Courtesy UNL)

The maximum span length for this system is currently limited to about 60 feet, reflecting the longest press breaks available in the industry.

The process of bending the girder can take less than an hour. Geometrical variations are obtained simply by changing the bend locations. Varying the depth of the web and the width of the top and bottom flanges allow the folded plate girders to accommodate different span length requirements.

One of the advantages of the Folded Plate Bridge System is its promise for rapid delivery. All folded plate girders are fabricated using either 0.375- or 0.5-in. thick plate. The ability to stock standard plate sizes means the girders can be produced and delivered quickly.

The Folded Plate Bridge System can be constructed using conventional construction techniques as well as using principles of Accelerated Bridge Construction. In the case of conventional construction procedures, readily available construction equipment can be used to build the formwork for casting the concrete deck, as shown in Figure 2.8.

Figure 2.8. Deck forming using conventional approach. (Courtesy UNL)

Recently, the trend within the bridge construction industry has been toward reducing construction activities on the bridge site and eliminating the interruption to traffic. Therefore, an alternate and perhaps better approach when using the Folded Plate Girder System to construct short-span bridges is to use prefabricated elements.

In one scenario, the tributary width of concrete deck for each folded plate girder is cast on the girder prior to shipping to the site. In this case, each prefabricated girder unit is a folded plate girder with a precast deck as shown in Figure 2.9. The steel girder can be supported at the ends, or continuously supported along the length during casting, in which case all dead loads are carried by the composite section, thereby reducing deflections.

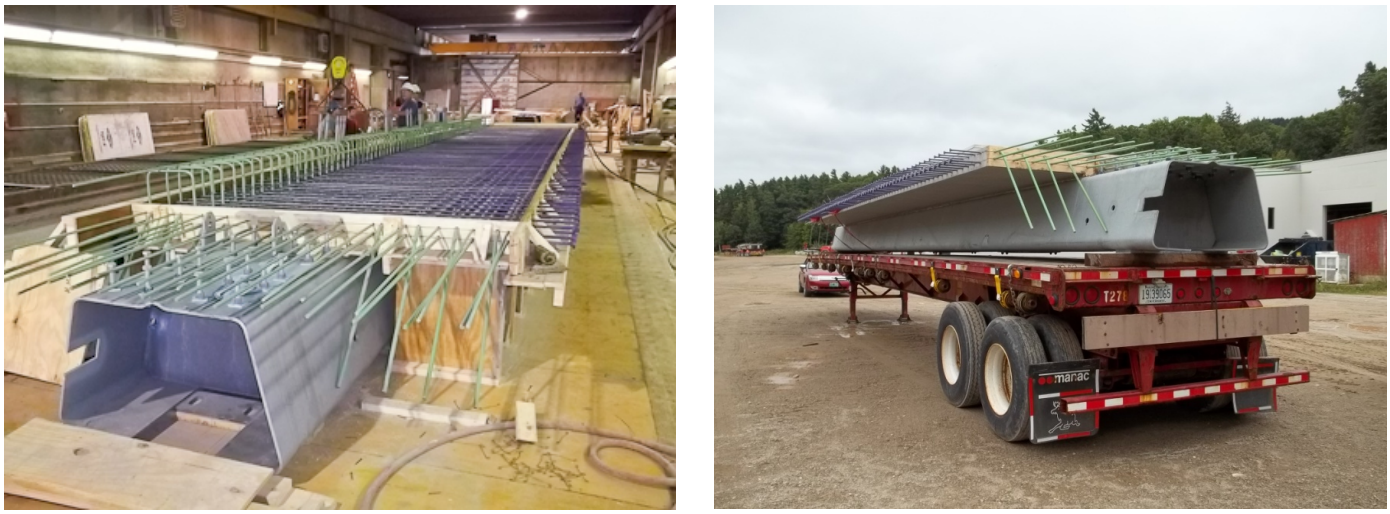


Figure 2.9. Precast folded plate girder unit. (Courtesy Massachusetts DOT)

A typical two-lane county-type bridge will require three or four prefabricated folded plate girder units placed side by side and connected longitudinally at the deck, as shown in Figure 2.10. The connection between the units can be accomplished using a number of methods. A 40-ft-long folded plate girder with precast deck weighs about 24,000 pounds, allowing use of a relatively lightweight crane on the construction site.

Figure 2.10. Folded plate girder system. (Courtesy Massachusetts DOT)

2.2.3.2 Concrete Bridge Superstructures

There are several commonly used reinforced concrete bridge systems in the United States. The type of system implemented at a particular site is generally dictated by economics and the system's ability to accommodate the required span, or geometric requirements such as curvature.

The most commonly used concrete bridge superstructures are:

- Cast-in-place concrete slabs;
- Precast concrete box beams including both spread and adjacent box beams;
- Precast concrete I-girders including standard I-girders, bulb-tee girders, and U-beams;
- Precast concrete spliced girders, including spliced I-girders, U-beams, and box girders;
- Cast-in-place posttensioned box girders;
- Segmental posttensioned concrete box girders, including both precast or cast-in-place;
- Concrete arches; and
- Modular pre-topped concrete girder units, which are typically used for accelerated bridge construction.

2.2.3.2.1 Cast-in-Place (CIP) Concrete Slabs

Full-depth, cast-in-place concrete slab superstructures consist of a concrete slab spanning between substructure units without the aid of supporting stringers, as shown in Figure 2.11. Concrete slab bridges commonly span less than 50 feet and are typically used over minor water crossings. This bridge system was traditionally constructed as a series of simple spans, but in recent years, the use of continuous spans has gained favor, eliminating the joints over the substructure units. This system is commonly reinforced conventionally, but can also be posttensioned to increase the span length range. The haunched posttensioned concrete slab system used in Kansas can span up to about 100 feet. Many states, especially in the Midwest, own many older concrete slab bridges, mainly constructed in 1930s, that have shown a very good performance history. When rated, these older concrete slab bridges usually demand posting. However, research results indicate that older concrete slab bridges possess reserve capacity significantly more than that indicated by routine rating calculations (Azizinamini et al. 1994a, 1994b). The main reason for such high capacity of older concrete slab bridges is the higher yield strength of reinforcement used versus the assumed value in rating calculations. This higher capacity of existing older concrete slab bridges coupled with their good performance record can be advantageous when developing maintenance plans.

Cast-in-place concrete flat slab bridge.

Transversely posttensioned prestressed slabs.

Figure 2.11. Short-span concrete bridge applications. (Courtesy Atkins North America, Inc.)

2.2.3.2.2 Precast Concrete Box Beams

This type of superstructure consists of adjacent precast concrete box beams with non-composite deck, adjacent box beams with composite cast-in-place (CIP) concrete deck, and spread box beams with composite CIP concrete deck. Shallower precast solid and voided rectangular slabs also fall into this category. Precast concrete box beams are typically plant manufactured standard AASHTO-PCI sections that range in depth from 27 in. to 42 in. and are available in 36-in. or 48-in. widths. These precast girders are plant-produced, which generally results in high quality products.

Precast adjacent box beam bridges are the most prevalent box girder system for short and medium-span bridges, typically 20 to 127 feet, especially on secondary roadways. These bridges consist of multiple precast concrete box beams that are butted against each other to form the bridge deck and superstructure. Their advantage is that they eliminate the need for forming when using a composite CIP deck, or can be used directly with a bituminous overlay in the non-composite state. Adjacent box beams are generally connected using partial or full-depth grouted shear keys along the sides of each box. Transverse ties are usually used in addition to the grouted shear keys and may vary from a limited number of threaded rods to several posttensioned tendons. In some cases, no topping is applied to the structure while in other cases a non-composite topping or a composite structural slab is added. Problems have been encountered with adjacent non-composite box beam superstructures and are further discussed in Section 2.3.3.2.2.

2.2.3.2.3 Precast Concrete Girders

Precast concrete I-girders with composite cast-in-place deck are commonly used concrete bridge superstructures in the 50- to 150-ft-plus span range. These girders are made of high-performance, plant-produced materials and are generally very durable and result in high quality products. In a bridge system consisting of I-girders with composite CIP slabs, commonly referred to as beam-slab bridges (see Figure 2.12), the longitudinal stringers are often prestressed concrete I-girders using one of six standard AASHTO-PCI sections, Types I through VI, which vary in depth from 28 to 54 in. In addition, newer standard AASHTO-PCI bulb-tee (BT) shapes are used in 54-, 63-, and 72-in. depths. These standard I and BT shapes accommodate various span requirements up to about 170 ft.

Bulb-tee shapes were developed to provide increased efficiency over original I shapes. They have wide top flanges similar to Type V and VI girders that increase stability for handling and shipping and reduce deck forming. However, bulb-tees also have thinner top flanges, webs and bottom flanges that reduce weight, and have other flange geometric and proportioning modifications that optimize the sections. A number of states including Washington, Colorado, Florida and Nebraska have developed special bulb-tee shapes, modifying the standard AASHTO-PCI BT shapes, to accommodate local needs and practice.

Figure 2.12. Prestressed concrete I-girder bridge with composite concrete deck. (Courtesy Atkins North America, Inc.)

A noteworthy I-girder advancement is the NU I-girder, which was developed by the University of Nebraska (UNL) in cooperation with the Nebraska Department of Roads, and has a series of eight standard shapes with depths

ranging from 29.5 to 94.5 in. (Geren and Tadros 1994; Hanna et al. 2010). The NU girders have several section efficiency enhancements such as wide and thick bottom flanges that enable increased strand capacity for simple spans and provide increased negative moment capacity for continuous spans. The wide bottom flanges also provide increased stability in shipping and handling. Curved fillets in top and bottom flanges reduce stress concentration and aid the flow of concrete during fabrication. With the increased section efficiencies, these girders have been used for spans greater than 200 feet (see Figure 2.13).

Figure 2.13. A 9-ft x 3-in. deep, 213-ft-long, 130-ton NU I-girder being shipped. (Courtesy of Con-Force Structures, a division of Armtec Limited Partnership, Calgary, Alberta, Canada)

In many instances, precast concrete I-girders are erected as simple spans and then connected over the piers to form continuous for live load systems that eliminate deck joints.

A newer alternative to concrete I-girders is the U-beam, or concrete tub girder, concept, first developed in Texas and now used in other states including Florida and Washington State, that provides economic and aesthetic spread beam systems. The Texas U54 beam is 54-in. deep, similar to an AASHTO Type IV beam, and can span up to about 140 feet (Ralls et al. 1993). The Florida U-beams have four depths ranging from 48 in. to 72 in., and can be used in spans ranging from about 100 ft to 160 ft (FDOT 2012). Washington State (WSDOT) U-beams are similar and have four depths varying from 54 in. to 72 in., and bottom flanges that are either 4-ft or 5-ft wide. Similar to Florida, these

beams can accommodate span lengths up to 160 ft. With a composite concrete deck, U- beams form a trapezoidal box shape, similar to steel tub girders. These beams are typically designed to act as simple spans under both dead load and live load, even when the deck is placed continuous across intermediate supports. As with I-girders, these beams are plant produced and result in high quality products.

2.2.3.2.4 Precast Concrete Spliced Girders

Spliced girders are precast concrete girders fabricated in several segments that are then assembled longitudinally, typically using posttensioning, into a single simple-span or continuous girder for the final bridge structure. They have been used to extend the span lengths of regular short- to medium-span precast concrete girders and are designed to utilize the economy and high quality of plant-produced precast girders for longer span applications. The length and weight of typical precast girders prevents them from being effectively used on spans greater than about 150-feet due to the limitations of transportation equipment and available cranes. However, with spliced girders, precast girder segments with manageable weights and lengths are transported to the construction site and then joined together. This can either be done by splicing girder segments on the ground and erected them into their final position, or by placing girder segments on temporary supports and then splicing them in their final position. Spliced girders have been used for simple spans up to about 220 ft, and for continuous spans up to about 320 ft, and have been found to provide an economical concrete superstructure type in span ranges between that of conventional precast girders and segmental box girders. Figure 2.14 shows a typical spliced girder span under construction using temporary supports.

Figure 2.14. Spliced I-girder construction. (Courtesy HDR)

Spliced girders are typically used on relatively straight alignments; however, in recent years they have also been used for curved alignments in Nebraska and Colorado. Figure 2.15 shows a spliced-box girder bridge recently built in Denver, Colorado.

Precast spliced girders have some similarity with segmental box girders in that both structure types consist of smaller girder segments that are assembled and connected by posttensioning to form a final, longer girder, and both types are erected by staged construction. However, spliced girders and segmental box girders are quite different in the length of segments, type of splices, types of sections, tendon locations, and construction methods. Also, a composite concrete deck is typically cast on spliced girders, while the deck slab is typically cast as an integral part of a segmental box girder. Spliced girders use bulb-tee, I-beam, U-beam, or box shapes, while segmental box girders are typically box shapes.

Figure 2.15. Spliced concrete curved boxes. (Courtesy Summit Engineering Group)

2.2.3.2.5 Cast-in-Place (CIP) Posttensioned Concrete Box Girders (on Falsework)

Posttensioned concrete box girders cast continuously on falsework, have become very popular in several states, particularly California, Arizona, and Nevada, and have been used in spans up to about 350 feet. This type of construction lends itself to local construction industry practices in which contractors can economically provide the required falsework. Similar to segmental construction, CIP on falsework offers the advantage of longer spans than conventional girders, and can easily accommodate curved alignments. CIP construction also allows clean lines and architectural finishes that improve the aesthetics. The use of posttensioning further enhances concrete durability by providing a superstructure that will remain essentially crack-free under service loads. Designing the structures as a frame and utilizing monolithic connections between the superstructure and piers also eliminates bearings, which further eliminates associated future maintenance. A potential disadvantage of this type of construction is difficulty in replacing the deck or widening the bridge. Figure 2.16 shows a CIP box girder bridge under construction.

Figure 2.16. Cast-in-place box girder bridge on falsework. (Courtesy Atkins North America, Inc.)

2.2.3.2.6 Segmental posttensioned concrete box girders (CIP and Precast)

Segmental concrete box girder systems have been used when span requirements are greater than what can be achieved with conventional stringer-type girders or spliced girders, in instances of sharp horizontal curvature, or when special aesthetics are required. They have been economical in span ranges from about 250 feet to 500 feet. This system is further divided into cantilever construction and span-by-span construction, and can be either precast or cast-in-place. They can be cast to match the shape of any alignment making them particularly suited to curvature. Figure 2.17 shows a cast-in-place segmental bridge recently built in Florida using balanced cantilever construction.

Segmental box girder bridges have been observed to improve deck performance due to the pre-compression of the deck. Refer to Chapter 3 on materials for additional information on these bridge deck systems and durability issues relating to details currently in use.

Figure 2.17. Cast-in -place segmental concrete box system. (Courtesy HDR, right photo by John Rupe)

2.2.3.2.7 Concrete Arches

Concrete arches have been used for bridges with short spans of about 100 ft to long spans of over 1,000-ft span, but are typically considered today only in certain long span applications because of the relative economy of I-girder and segmental box girders in shorter spans, or when special aesthetics are required. True arches are efficient structural systems because vertical dead load produces axial member compressive forces that are resisted by a thrust at the arch abutments. Concrete has been useful for arches because of its inherent efficiency in compression.

Concrete arches have typically been used in deck-type systems where the arch ribs are below the deck, but they have also been used in some through-type applications where the arch ribs extend above the deck. Deck arch systems are either closed spandrel types or open spandrel types. Closed spandrel types typically use barrel arches with longitudinal walls along the outside edges of the arches that are either filled or unfilled. Open spandrel types have a series of spandrel columns that transmit deck loads to the arches.

Concrete arches in the U.S. have typically been constructed using either cast-in-place on falsework methods or cable-stayed segmental methods. Figure 2.18 shows the cable-stayed, cast-in-place segmental construction method used for the Hoover Dam Bypass concrete arch bridge.

Figure 2.18. Hoover Dam Bypass concrete arch bridge constructed using cable-stayed segmental methods.

(Courtesy HDR, photo by Keith Philpott)

2.2.3.2.8 Modular Pre-topped Concrete Girders

These types of systems utilize precast beam elements that are fabricated with a portion of the deck in place as a unit and are erected side by side and connected with a CIP closure joint, posttensioning, composite concrete topping, or a combination of these methodologies. The precast elements commonly consist of conventionally-reinforced or

prestressed sections that include T beams, double Ts, and deck bulb-tees. This system is expected to gain popularity with accelerated bridge construction as pressure mounts to expedite construction and to minimize field forming and placing of concrete. Refer to Chapter 4, Bridge Deck, for information concerning CIP closure connections.

A recent concept is the NEXT Beam (Northeast EXtreme Tee), which was developed by the Precast/Prestressed Concrete Institute North East (PCINE) along with the Departments of Transportation for New York, Connecticut, Massachusetts, Vermont, Maine, New Hampshire, and Rhode Island (Culmo and Seraderian 2010). It is a precast, prestressed double-tee section with 8-ft or 12-ft deck widths, and beam depths from 24 in. to 36 in. that is applicable for approximately 40-ft to 90-ft spans. It is available with a thick top flange that comprises the deck, or with a top flange that creates a form for a composite CIP deck. The NEXT beam is considered as an alternative to traditional adjacent concrete box beams, providing improved durability, lower cost, easier inspection and accelerated bridge construction.

2.2.4 Substructure Component

The substructure component includes all structural elements required to support the superstructure and is typically defined from the underside of bridge bearings down through the foundation. The function of these elements is to transfer all vertical loads from the superstructure to the foundation supporting strata and to resist horizontal forces acting on the bridge. The transfer of load to the supporting ground can be either through spread foundations, piles, or drilled shafts, depending on the strength and stability of subsurface geotechnical conditions. This component typically includes pier and abutment subsystems, each including several elements, as shown in Figure 2.19.

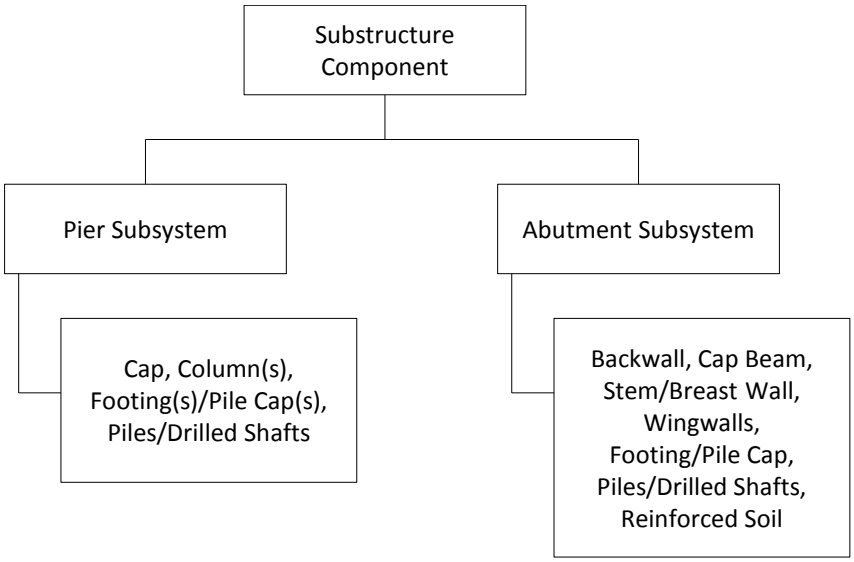


Figure 2.19. Substructure component.

2.2.4.1 Piers and Bents

Piers are intermediate supports for multi-span bridges. They can have multiple configurations, but typically fall into two major categories, piers and bents, as illustrated in Figures 2.20 and 2.21. A pier subsystem consists of several elements, including a cap beam supporting the main load-carrying elements of the superstructure, which in turn is supported on one or more columns. The columns are supported by foundations that are typically located at or below the finish grade of the adjacent ground. The foundation can be a footing bearing directly on rock or soil, or a deep foundation using piles or drilled shafts.

T-piers are examples of single column piers with a cap element. Solid- or wall-type piers are also single column piers, but are wide enough to support the superstructure without having a separate cap element.

A bent consists of a cap beam supporting the main load-carrying elements of the superstructure, which in turn is supported directly on deep foundation elements such as piles or drilled shafts that extend up from finish grade. In some cases, the term “bent” is also used to describe a multi-column pier.

Common practice is to construct piers with reinforced concrete, although some steel piers with pier caps have been used. When deep foundations are required to support the bent caps, they normally consist of timber, prestressed concrete square, solid round or hollow cylinder piles, CIP concrete drilled shafts, or steel HP or steel pipe sections.

Modular, precast concrete pier elements have also been used for accelerated construction.

Multi-column pier.

Pile bent.

Figure 2.20. Pier types: multi-column supported pier and pile bent. (Courtesy Atkins North America, Inc.)

Integral pier cap construction was also developed as a way to avoid sharp skews or associated longer spans in interchange ramp bridges, and to lower overpass profiles. Integral pier caps also have the advantage of eliminating bearings, which can minimize future maintenance requirements. Figure 2.21 shows a ramp bridge with conventional stacked T-pier construction and a similar ramp bridge with integral pier construction. Integral pier system is advantageous in seismic areas by integrating the super structure and substructure and creating frame action.

Ramp bridge with conventional T-piers.

Ramp bridge with integral piers.

Figure 2.21. Conventional and integral pier types. (Courtesy HDR)

2.2.4.2 Abutments

Abutments are provided in multiple configurations, but can be defined in two major categories as illustrated in Figure 2.22:

- Stub or spill-through abutments, and

- Full abutments.

Stub abutments are characterized by sloped embankments under the end span of the bridge and provide support to the superstructure through a shallow bent cap resting on a pile foundation.

Traditionally, full abutments are characterized by a vertical wall that retains the embankment fill and also transfers the bridge load to the supporting foundation at the base of the wall. Full abutments can also be in the form of a mechanically-stabilized earth (MSE) system, which employs a fascia wall connected to a system of reinforcing elements in multiple layers that work with the backfill material to create a composite soil mass. This composite soil mass can then support vertical load and/or act as an earth retention system. There are two types of MSE abutments: true or mixed (Anderson and Brabant 2010). In a true MSE abutment, the bridge superstructure is supported on spread footings bearing directly on the top of the reinforced soil mass. In a mixed MSE abutment, a shallow bent cap with a row of piles is used to support the superstructure behind the MSE fascia wall, and the reinforced soil mass is used to retain the fill behind and adjacent to the end of the bridge. A MSE full abutment is pictured in Figure 2.22.

Another recent form of abutment system is the Geosynthetic Reinforced Soil Integrated Bridge System (GRS-IBS), which is described in FHWA publication *FHWA-HRT-11-027* (Adams et al. 2011). This is a relatively new abutment system that has been used for accelerated bridge construction, and typically for short spans up to about 140 feet. The abutment uses alternating thin layers of compacted fill and geosynthetic reinforcement sheets that combine to form a reinforced soil mass foundation that directly supports the bridge superstructure without the need for piles. The geosynthetic reinforcement is connected into layers of precast facing blocks that are placed with the reinforcement and soil backfill. Once completed, the reinforced soil mass is ready to support the bridge.

Traditional abutments are typically concrete construction. When deep foundations are required to support the bent caps, they normally consist of timber, prestressed concrete square, solid round or hollow cylinder piles, CIP concrete drilled shafts, or steel HP or pipe pile sections.

Stub or spill-through abutment.

MSE full abutment.

Figure 2.22. Abutment types. (Courtesy Atkins North America, Inc.)

Types of abutments used also characterize the way the entire bridge system responds to thermally-induced longitudinal movements. There are three distinct abutment types:

- Integral abutment,
- Semi-integral abutment, and
- Abutment using expansion devices.

In integral abutment systems, are attached directly to abutment, and thermally-induced longitudinal movements are accommodated by flexibility of the piles. The piles are subject to both axial and flexural moments. In semi-integral abutment systems, girders and piles are not directly connected and the bearings used to support girders and piles are mainly subject to axial loads. Integral and semi-integral abutment systems are part of different continuous bridge systems. Chapter 8 provides a more in-depth discussion, as well as detailed design provisions for integral and semi-integral abutment systems.

2.3 FACTORS AFFECTING SERVICE LIFE

All of the elements, components, and subsystems that make up the overall bridge system are adversely affected in various degrees by both external and internal factors that contribute to reduced service life. External factors typically refer to loads or hazards, which can be both natural and man-made. Internal factors can pertain to such items as structure type (e.g. fracture critical), materials, and design/details.

Following is a discussion of critical factors that affect bridge service life using a fault tree analysis approach. Section 2.3.1 discusses factors that affect the overall bridge system. The following sections, 2.3.2 through 2.3.4, address specific factors affecting deck, superstructure, and substructure components. Section 2.4 addresses options to avoid or mitigate these factors.

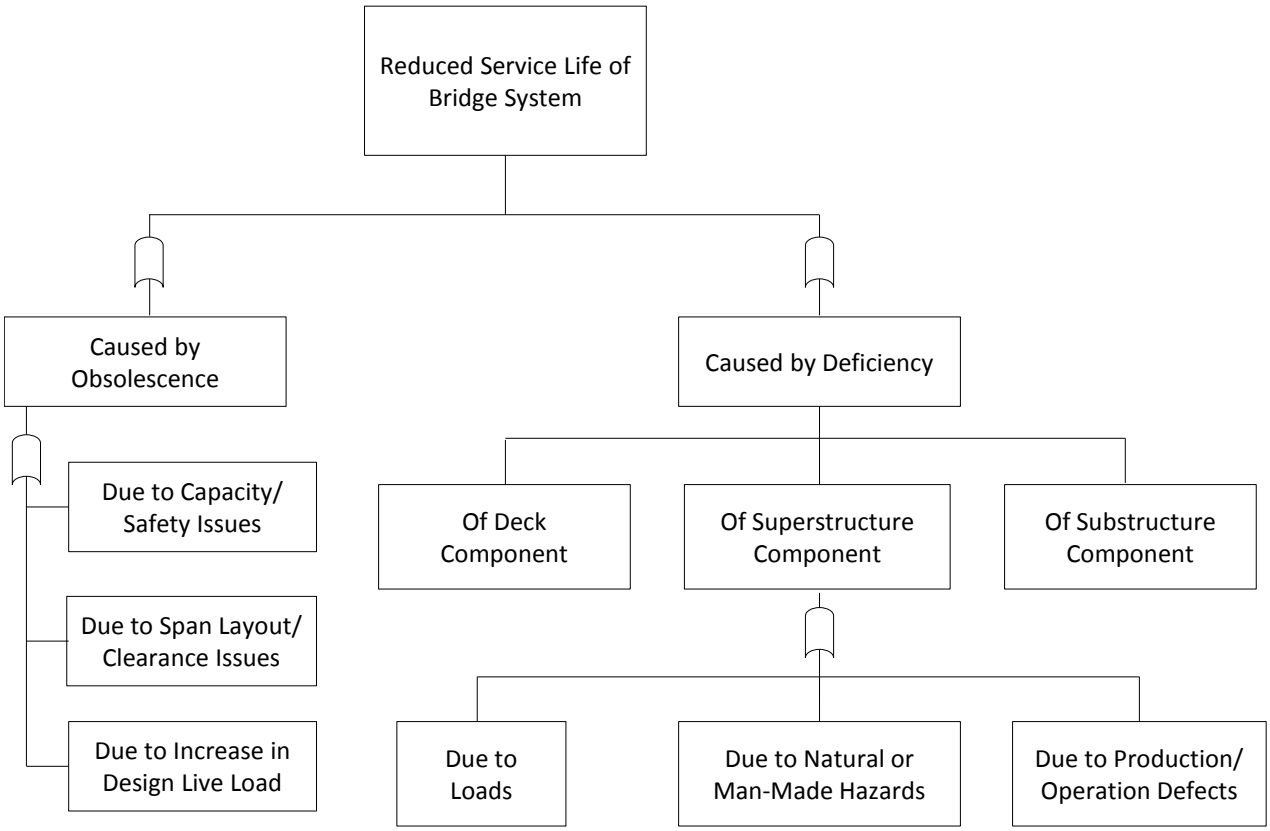


Figure 2.23. Factors affecting service life.

2.3.1 Bridge System Fault Tree Analysis

Figure 2.23 shows the initial fault tree diagram that identifies factors affecting service life for a complete bridge system. (A detailed discussion of the fault tree process is included in Chapter 1.) The diagram identifies causes of, or factors affecting service life and categorizes them for consideration. In following sections, these categories are successively sub-categorized at descending levels to identify multiple contributing factors. The fault tree analysis is then continued until the basic events or lowest levels of resolution are reached and discussed. The lowest level or basic events require strategies for mitigations.

2.3.1.1 Obsolescence or Deficiency

At the highest fault level, reduced service life of a bridge system can be attributed to either obsolescence or deficiency.

Obsolescence refers to reduced service life of a bridge due to issues related to how it functions, which can be further categorized as:

- Operational issues related to reduced traffic capacity and safety,
- Physical issues related to span layout and clearances, or
- Loading issues related to increases in design live load.

Many bridges are replaced because of functional issues well before their full-potential service life is achieved. Significant increases in corridor traffic demand, caused by such factors as urban planning, land use, and development, can ultimately result in the functional inability of a bridge to provide required level of service, necessitating bridge widening or replacement. Vertical clearance limitations sometimes prevent existing bridges from being widened. Increased corridor traffic can also require replacement of overpass bridges to accommodate widened roadways and increased span requirements below. Major interchanges are sometimes reconstructed because of the need for increased traffic capacity.

Often, safety issues relating to inadequate lane and shoulder widths, sharp curves, and inadequate sight distances have a significant effect on service life. Changes in design live load over the life of a bridge can affect service life as it relates to the structure's ability to safely accommodate increased load.

Service life design considerations should evaluate the potential of future operational needs, and how those needs might impact the service life of the planned facility. Risks of future obsolescence should be considered and appropriate choices should be made concerning mitigation or acceptance. Those choices should be incorporated into the design as appropriate considering life-cycle cost analysis.

Deficiency refers to reduced service life of a bridge due to damage or deterioration that can be caused by a number of primary factors and sub-factors, each of which can lead to reduced service life if un-mitigated. Deficiency

can occur in all three bridge components: deck, superstructure, and substructure. In Figure 2.23, the fault tree continues below the superstructure component, but it applies equally to all three components.

Within a bridge system, the interaction between components, deficiencies, or failures within a particular component can have a significant effect on other components. A primary example is deterioration of superstructure and substructure below leaking joints in the deck component (see Section 2.3.1.3.1). Another example is damage to substructure and other superstructure elements caused by failed bearings in the superstructure component (see Section 0)

Deficiency can be further attributed to any of three major causes:

- Load-induced,
- Natural or man-made hazards, or
- Defects in production/operation.

2.3.1.2 Reduced Service Life due to Loads

Load-induced deficiencies can be further categorized as those caused by traffic-induced loads or system-dependent loads (see Figure 2.24). The fault tree is continued for each of these load types to identify the basic or lowest levels causing damage or deterioration.

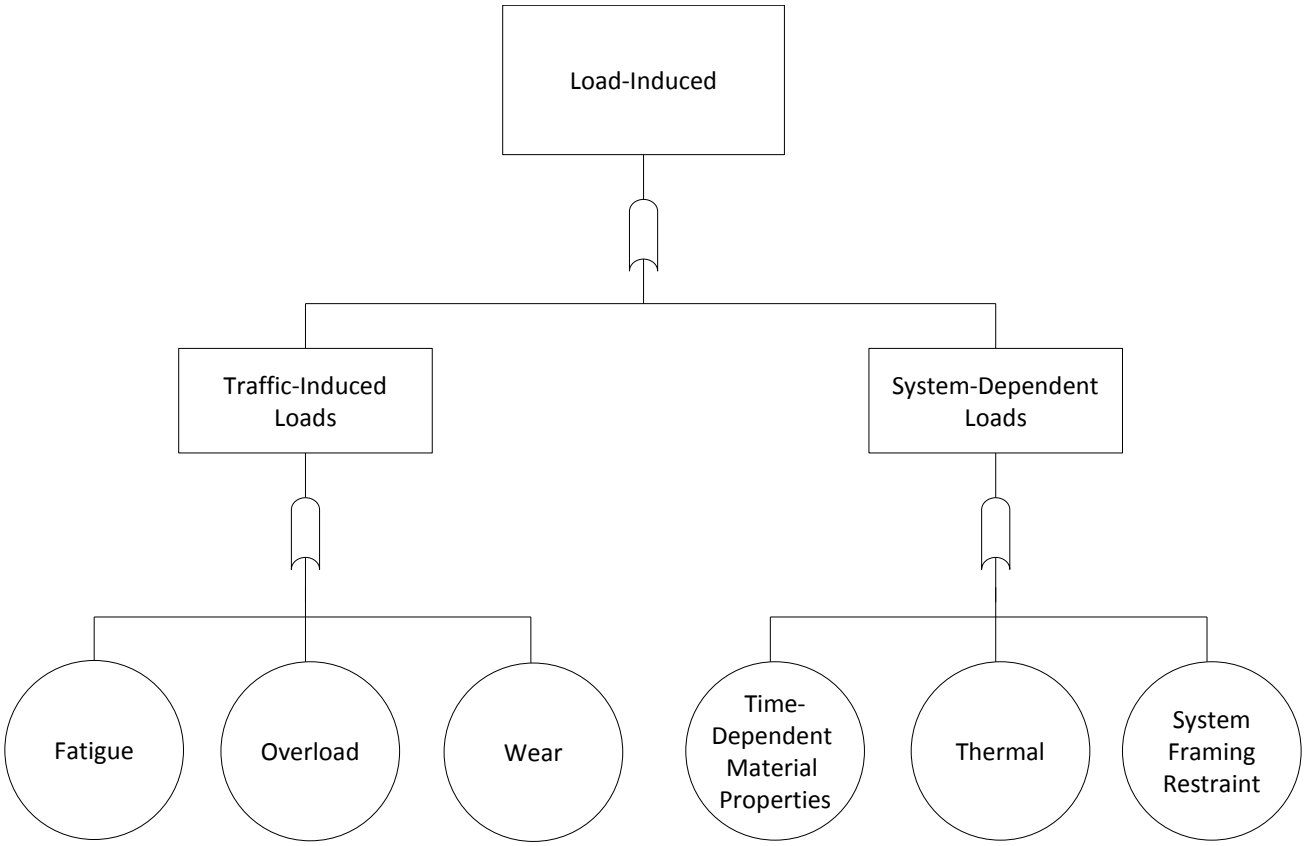


Figure 2.24. Load-induced deficiencies

2.3.1.2.1 Traffic-Induced Loads

Traffic-induced loads include the effects of truck and other vehicle traffic that are applied to the bridge deck and transmitted throughout the bridge system. Traffic load can ultimately cause damage to bridge elements through fatigue, overload, or wear.

Fatigue is structural damage to an element resulting from cyclic loading that results in the initiation and propagation of cracks, and can occur at stress levels considerably below the yield stress. Although fatigue can occur in reinforced concrete and structural steel elements, it is more predominant in steel elements. Chapter 3 on materials discusses fatigue deterioration in reinforced concrete.

Early-welded steel structures have a history of cracking at certain types of weld details due to load-induced and distortion-induced fatigue. Newer design provisions and recommended details have been developed that provide solutions for both load-induced and distortion-induced fatigue that will achieve desired service life. Section 2.3.3.1.1 provides additional information of fatigue in steel structures, and Chapter 7 provides a comprehensive discussion on fatigue and fracture in steel bridges.

Overload refers to element overstress or damage resulting from over-weight vehicles that exceed maximum gross vehicle weight restrictions or individual axle or tire restrictions. Overload often results from illegal, non-permitted vehicles and is the third leading cause of bridge failure in the U.S. behind hydraulic and impact causes (Wardhana and Hadipriono, 2003). Overload produces higher stress in members than what was considered in design, and can significantly reduce safety factors against failure, and can cause cracking in concrete elements. Multiple applications can also affect fatigue behavior and also result in excessive deflection that can affect certain elements, particularly in cases of differential deflection.

Because overload occurs on many bridges, the risk of overload should be considered on certain vehicular routes when planning new bridges. It may also be necessary to consider special owner-specified loads to avoid or mitigate this risk.

Wear refers to element damage and gradual loss of material caused by friction or rubbing. Decks are susceptible to wear from vehicle tires, especially with the use of studs or chains. Deck wear and abrasion is further discussed in Chapter 4. Wear has been a factor in steel structures, particularly on pins and pin plates in connections, and in bearings, with surface wear in sliding bearings, brass sealing ring wear in pot bearings, and pin wear in steel bearings.

2.3.1.2.2 System-Induced Loads

System-induced loads include the effects of the bridge system configuration on the behavior of the structure. These effects are accentuated by restraints provided through bridge boundary conditions and can result in significant locked-in stresses. The system-induced loads can be the result of movements due to time-dependent material properties, thermal movements, or system framing restraint.

Time-dependent material properties refers mainly to shrinkage and creep-related deformations in restrained concrete elements and can result in concrete cracking. This phenomenon is discussed further in Chapter 4 for bridge decks and in Chapter 3 for concrete materials in general.

Thermal Conditions refers to effects caused by temperature change, which can result in significant stresses in restrained structural members, and can be as large as live load stresses in some cases. The effects can be the result of uniform stress across a bridge member, or the result of a temperature gradient throughout the depth of a member.

System-framing restraint refers to effects caused by boundary condition restraints that prevent normal or intended structural behavior. Improper function or seizing of bearings can result in unintended movement restraint, which can further cause pier cracking and distress. Another example occurs at ends of skewed integral abutments, where lateral movement resulting from the skew can cause cracking and distress in corner details if adequate clearance is not provided to allow for the movement.

2.3.1.3 Reduced Service Life due to Natural or Man-Made Hazards

Environmental hazards from both natural and man-made sources can have a significant influence on bridge service life. These also include effects from areas with adverse thermal climate, coastal climates, and chemical climates, as well as from chemical properties of the materials themselves. Other hazards such as hydraulic action, collisions, fire/blast, or seismic events can also have considerable effect. These natural and man-made hazards are introduced in the fault tree shown in Figure 2.25.

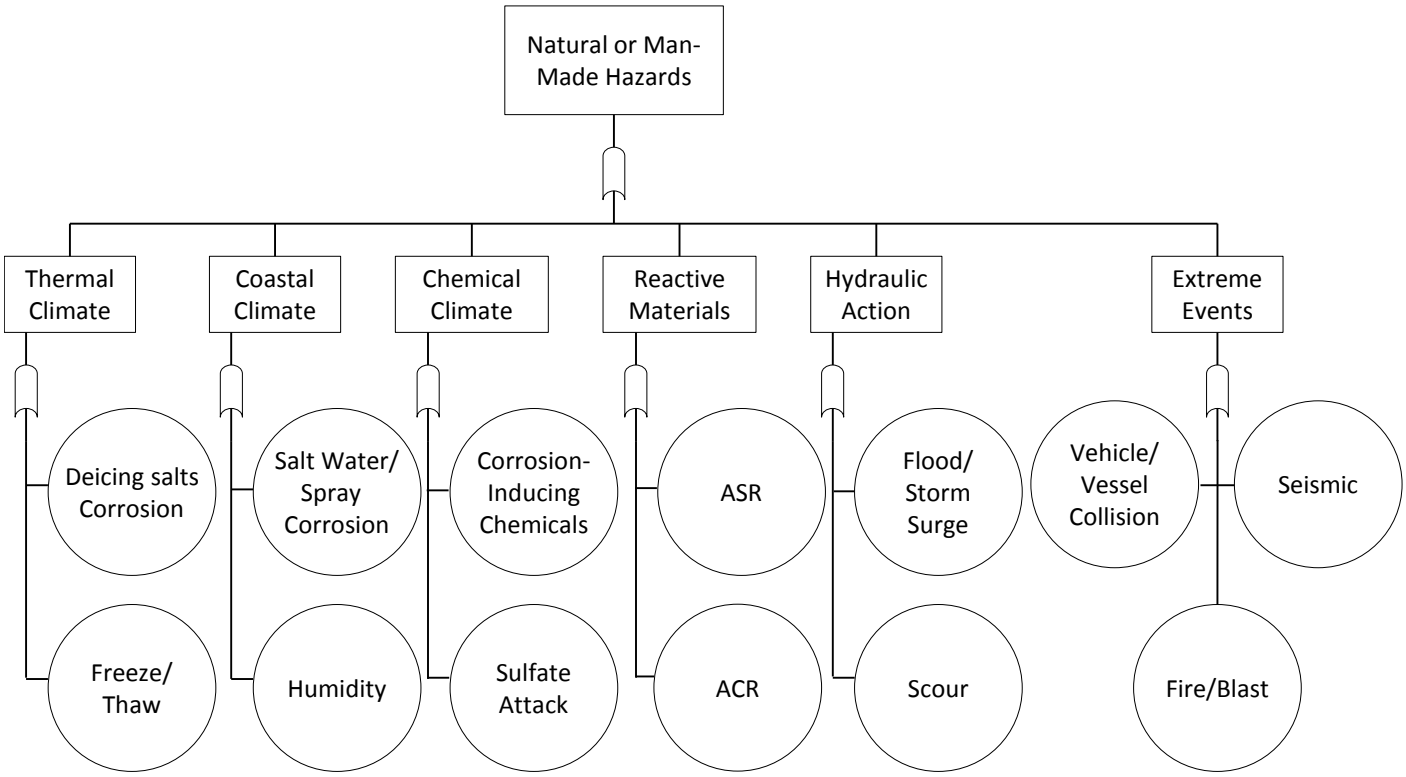


Figure 2.25. Natural or man-made hazards.

2.3.1.3.1 Thermal Climate

Deicing salts corrosion. Bridge service life is typically severely affected in cold, wet climates due to heavy use of roadway deicing salt. Salt-contaminated moisture penetration directly affects bridge deck service life by initiating

and propagating corrosion in unprotected reinforcing steel, and by accelerating concrete deterioration due to cracking and freeze/thaw damage. All unprotected bridge elements below open or leaking deck joints are subject to salt-contaminated roadway drainage, which causes unprotected structural steel corrosion, concrete reinforcing bar corrosion, and associated concrete cracking and spalling.

On overpass bridges, salt spray rising from roadways below affects superstructures and undersides of decks, causing concrete penetration and corrosion of unprotected reinforcing, and corrosion of unprotected structural steel. The salt spray from vehicles passing underneath the bridge can also affect the service life of weathering steel by keeping the steel continuously wet.

Decks, barriers and deck joints are also susceptible to damage from snow plows used to clear roadways for traffic.

Freeze/Thaw. Water absorbed into concrete surfaces and contained in cracks can freeze in cold weather conditions. The frozen water tends to expand causing stresses within the concrete. Cyclic freezing and thawing of the water absorbed in the deck surface fatigues the concrete, resulting in cracking, scaling, and spalling. Refer to Chapter 3 on materials for additional information on freeze/thaw in concrete.

2.3.1.3.2 Coastal Climate

Salt water/spray corrosion. Coastal saltwater environments also have severe effects on bridge service life due to wetting and chloride penetration causing corrosion of unprotected reinforcing, and corrosion of unprotected structural steel.

Structures in these areas are subjected to a chloride-laden saltwater environment and a combination of wind and wave action that causes these chlorides to become airborne as salt spray. The susceptibility of various bridge components to these environmental influences depends on their degree of direct contact or their height above the water elevation. Pier columns with direct contact in areas of continual wetting are most susceptible to damage.

Wave action hitting substructure units and seawalls or abutments under the bridge also tend to cause the salt spray to explode upward, wetting the bottoms of lower level superstructures and decks. The salt spray can also deposit itself on the bridge deck surface, particularly on windy days. The salt spray wets the surfaces leaving a chloride residual that can absorb into the concrete, resulting in reinforcement corrosion that in turn causes cracking,

spalling, and/or delamination. The wet salt spray can also be deposited on the sides of structural steel members affecting service life of coatings, and directly causing and accelerating steel corrosion.

Humidity. High humidity in coastal regions also results in cyclic wetting and drying of bridge surfaces. Concrete materials sensitive to repeated wetting, such as those in which reactive aggregates are utilized, can have an adverse effect on concrete elements. Continuous wetting and drying also affects coatings on structural steel members and causes steel corrosion.

2.3.1.3.3 Chemical Climate

Corrosion-inducing chemicals. Chemical climate influences on bridge service life performance can be attributed primarily to airborne corrosion-inducing chemicals as a result of nearby industrial facilities such as chemical plants or oil and coal-burning facilities. Chapter 3 provides further discussion on chemical influences on concrete, and Chapter 6 discusses the influence of corrosion-inducing chemicals with respect to steel coatings and steel corrosion.

Sulfate attack. Exposure to sulfates can cause expansion of concrete material that can cause spalling and cracking, and the loss of bond strength between the cement paste and aggregate. Refer to Chapter 3 for additional information on sulfate attack in concrete.

2.3.1.3.4 Reactive Materials

Reactive ingredients with the concrete mix can affect concrete service life performance by altering the volumetric stability of the concrete mix design. These influences primarily occur naturally.

Alkali-silica reactivity (ASR) results in swelling of aggregate particles within concrete that can lead to spalling, cracking and general concrete deterioration. Chapter 3 provides additional information on ASR in concrete.

Alkali-carbonate reactivity (ACR) results in aggregate expansion within concrete that can lead to spalling, cracking and general concrete deterioration. Refer to Chapter 3 for additional information on ACR in concrete.

2.3.1.3.5 Hydraulic Action

Hydraulic action is the leading cause of bridge failure in the United States (Wardhana and Hadipriono 2003). The two principal base elements are flood/storm surge and scour.

Flood/storm surge. Floods and storm surges can significantly affect bridge service life, and can dislodge spans from their bearings and wash them away. Storm surges during major hurricanes are most often the cause of bridge damage, as occurred in 2005 during Hurricane Katrina, which devastated the Gulf coastline from Louisiana to the Florida Panhandle, and damaged nearly 45 bridges (Padgett et al. 2008). Most of the damaged bridges were adjacent to water and damage resulted from storm surge-induced loading. Much of the damage was to superstructures, where typical damage included unseating or shifting of decks and failure of bridge parapets. Several bridges suffered damage due to impact from loose barges and debris. The most common severe failure was unseating, which often occurred in low elevation spans. The deck displacements were attributed primarily to a combination of buoyant forces and pounding waves. Superstructure damage largely depended on the connection type between the decks and bents and the bearings often provided no apparent positive connection between the superstructure and substructure.

Figure 2.26 shows the I-10 bridges across Escambia Bay in Florida that were dislodged during a storm in 2006. Damage due to superstructure unseating was similar to that experienced in the Katrina bridge damages.

Bridges with low vertical clearance over a waterway can also be vulnerable to damage resulting from debris flow in a flood.

Figure 2.26. Florida I-10 Escambia Bay bridges washed out during storm. (Courtesy Florida DOT)

Scour. Scour is defined as the erosion or removal of streambed or bank material from bridge foundations due to flowing water. Although scour can occur at any time, bridge scour most often results during floods where swiftly flowing water has more energy than calm water to lift and carry sediment down river. A hole is created adjacent to the pier or abutment when material is washed away from the river bottom exposing or undermining footings, which

can compromise the integrity of the structure and lead to failure. Figure 2.27 shows an example of abutment scour. See Section 2.3.4 for factors affecting service life of bridge substructures.

Figure 2.27. Abutment scour. (Courtesy U.S. Geological Survey, photo by Bill Colson)

2.3.1.3.6 Extreme Events

2.3.1.3.6a Vehicle/Vessel Collision

Vehicle/vessel collision is second to hydraulic effects as the leading cause of bridge failures (Wardhana and Hadipriono 2003). Bridges crossing other roadways with minimum or low clearance are subject to various types of vehicle collision, particularly involving over-height vehicles. Figure 2.28 shows the effects of a collision in which a truck transporting a hydraulic crane with the boom inadvertently raised struck a concrete bridge and cut halfway through the entire width of the superstructure. Piers with minimum offset from edge of roadway or shoulder are also subject to vehicle collision if not adequately protected by barriers.

The risk of vehicle impact should be considered in the design of new bridges, particularly bridges crossing heavy truck routes where a greater probability exists for over-height vehicles, and where there has been a history of impacts from over-height vehicles. Possible mitigation strategies include:

- Using higher clearances,
- Using sacrificial beams to protect load carrying members, and
- Using laser detection systems that set off warning signals if an over-height vehicle is detected.

Figure 2.28. Bridge impacted by truck transporting hydraulic crane. (Courtesy Kansas DOT)

Bridges crossing water bodies or waterways are subject to ships colliding with either piers or superstructure. They are rarely occurring extreme events, but have potentially high consequences. Figure 2.29 shows the aftermath of a ship collision with the original Sunshine Skyway Bridge in Florida. The ship collided with one of the end piers in the main channel 3-span unit, took out the pier and subsequently the superstructure unit.

Considerations for new bridges should evaluate span openings required for safe navigation, including horizontal and vertical clearances, and consider appropriate mitigation measures to reduce the risk of collision. Adequate fender systems or other pier protection devices also need to be considered where there is risk of ship collision.

The current *AASHTO LRFD Bridge Design Specifications* (*LRFD Specifications*) provide requirements for new bridge design for both vehicle and ship impact (AASHTO 2012).

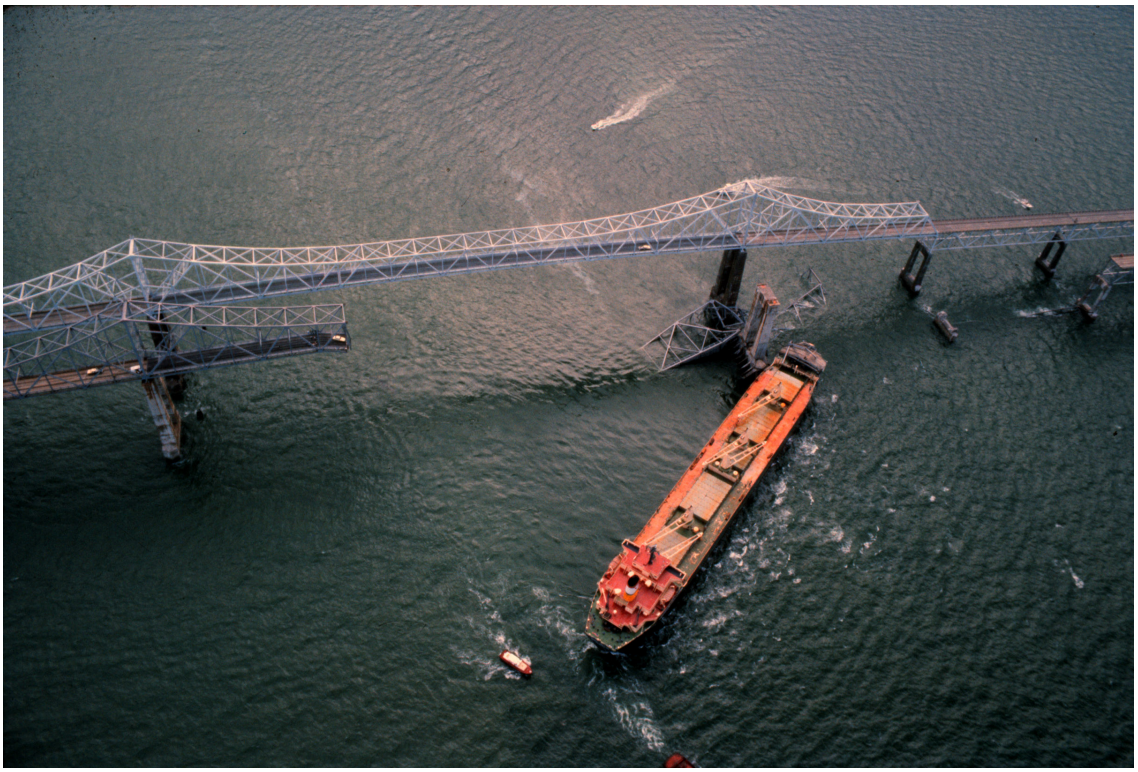


Figure 2.29. Sunshine Skyway Bridge collapse from ship collision. (Courtesy Tampa Bay Times)

2.3.1.3.6b Fire

Fires, as extreme-event hazards for bridges, have a low probability of occurrence, but can cause significant damage to affected bridge components including the deck, superstructure, and substructure, and can cause collapse of entire spans. Although considered a low risk hazard, a recent study by the New York Department of Transportation (DOT) in 2008 showed that nearly three times more bridges have collapsed because of fire than earthquakes (Kodur et al. 2010).

Fires affecting bridges most typically occur due to vehicle accidents either on a bridge or on a roadway or railway crossing below a bridge, but can also occur due to fires in adjacent buildings or facilities. Fires can vary in intensity, the most intense due to accidents with tanker trucks or railroad tanker cars carrying large quantities of highly flammable fuels or chemicals. The temperature of a recent fire below a bridge that was caused by a railroad tanker car collision loaded with 30,000 gallons of Methyl Alcohol was estimated to be approximately 3000°F (Stoddard 2002). Recent bridge fires involving tanker trucks carrying diesel fuel and gasoline were reported to have reached temperatures over 2,000°F (Kodur et al. 2010).

The extreme high temperatures generated in these types of bridge fires for prolonged periods of time can significantly affect both steel and concrete structures. Figure 2.30 shows examples of dramatic bridge fires caused by gasoline tanker truck accidents.

Steel bridge elements are especially vulnerable to high temperatures because of steel's high thermal conductivity in which the temperature of unprotected steelwork will vary little from that of the fire. These cases can result in loss of strength, significant sagging, and possible collapse. Steel starts to loose strength at about 600° F, and its strength is reduced to about half its yield strength at about 1,100°F (Brandt et al. 2011). At about 1,700° F, the yield strength is only about 10% or less. When fires at steel bridge elements reach these extreme temperatures, significant deformation and sagging usually occurs (if not total collapse) and the affected bridge elements will typically have to be replaced. Figure 2.31 shows extreme sagging in a steel bridge span and heavy concrete pier deterioration after a gasoline tanker truck fire.

Figure 2.30. Intense bridge fire due to tanker truck accident. (Courtesy U.S. Fish and Wildlife Service)

Figure 2.31. Steel bridge heavily damaged by fire after gasoline tanker truck collision. (Courtesy Alabama DOT)

In cases where damage to steel bridges sustained during a fire is not obvious (i.e., no clear signs of distress such as sagging or buckling) the question is often raised as to whether permanent material property effects in heat affected areas has occurred. It has been reported that steel will begin to encounter phase changes at temperatures greater than 1,300°F, whereby undesirable material properties such as reduced ductility and toughness can result during uncontrolled cooling. The Pennsylvania DOT sponsored a study to examine the effects of fire damage on the structural properties of steel bridge elements (Brandt et al. 2011). The study performed fire tests on painted steel plate specimens at various temperatures up to 1,200°F and exposure times to evaluate changes in surface conditions and discoloration and to then test for changes in material properties after cooling. The results showed that up to steel surface temperatures of 1,200°F, the fire-exposed material after cooling still satisfied AASHTO material specifications. They further concluded that if excessive distortions or deformations occurred, the steel would likely have been subjected to steel temperatures well in excess of 1,200°F, and the corresponding sections would require replacement.

Concrete bridge elements are typically able to withstand high temperatures with less distress than unprotected steel elements. Concrete has inherent fire resistant properties due to its relatively low thermal conductivity, which insulates interior portions of the member including reinforcement and/or prestressing steel from high surface temperatures. Concrete does experience a reduction in strength and modulus of elasticity with high temperature. Strength reduction is largely a function of type and size of aggregate. Concrete with siliceous aggregate (materials consisting of silica and including granite and sandstone) begins to lose strength at about 800°F, and is reduced to

about 55% at 1200°F. Concrete containing light-weight aggregates (manufactured by heating shale, slate, or clay) and carbonate aggregates (include limestone and dolomite) retain most of their compressive strength up to about 1200°F (Bilow and Kamara 2008). The following compares concrete temperature with typically encountered signs of distress and concrete color (Shutt 2006).

- Up to 200°F—little or no concrete damage;
- 500°F—surface crazing, localized cracks, iron bearing aggregates begin to acquire pink/red color;
- 700°F—cracks appear around aggregate, numerous micro cracks present in cement paste;
- 900°F—purple/gray color appears if enough iron and lime are present;
- 1,000°F—serious cracking of paste and aggregates occurs due to expansion. Purple/gray color may become more pronounced;
- 1,500°F—cement paste is completely dehydrated with severe shrinkage cracking and honeycombing. Concrete may begin to become friable and porous;
- 2,200°F—some components of concrete begin to fail; and
- 2,500°F—Concrete is completely failed.

In all cases, fire damaged structures should be evaluated as quickly as possible once the fire is extinguished to determine the extent and severity of damage. Limits of concrete damage can often be tested with an impact-rebound hammer. Concrete core samples can be taken for petrographic examination, which will determine the extent of damage within the overall concrete matrix. Steel coupons can be taken to evaluate changes in material properties.

2.3.1.3.6c Blast

The possibility of terrorism against our nation's bridges is an ever-increasing threat in today's society. The risk of blast attack is typically considered very low for most bridges, but major bridges or bridges along major corridors that have high economic and/or socio-political impact can have greater risk. By their nature, bridges are very accessible to vehicles carrying explosive devices traveling either on the bridge or below on crossed roadways. They

are also susceptible to ships or boats carrying explosive devices below. Because bridges also vary in type and size, the assessment of blast vulnerability can be very complicated. Until recently, there has been little work done or information available concerning the effects of blasts on highway bridges or methods of analysis.

Extensive research is being undertaken by the FHWA (Duwadi and Munley 2011) to further understand the behavior and effects of blast loadings on bridge elements. Part of these studies is also to develop methods for evaluating risk and risk-mitigation strategies. The most significant research in the area of blast-resistant design guidelines for highway bridges is being conducted under the NCHRP, project number *NCHRP 12-72*, which has been recently documented in *NCHRP Report 645* (Williamson et al. 2010). The response evaluation of reinforced concrete bridge columns was a key part of this investigation. Other recent research reported by Agrawal and Yi, 2008, dealing with blast load effects on highway bridges developed computer models and showed through simulation analysis that seismic capacities and blast-load effects are strongly correlated. Kiger et al. 2010, reported on bridge vulnerability assessment and mitigation against explosions, and focused on response of posttensioned box girder bridges under blast loads.

Blast loads are considered one of the extreme hazards affecting bridges, and even a small amount of explosive can produce severe localized damage to a bridge element. In some cases, this localized damage can potentially progress to global collapse of the structure (Kiger et al. 2010). There are a number of factors that affect the potential damage to a bridge due to blast, including:

- Size and type of explosive charge. Small explosive devices can have varied effects depending on placement and size of bridge element, but large truck bombs can be disastrous. The Oklahoma City Federal Building bombing in 1995 is an example of the devastating effect a large truck bomb.

- Proximity to blast—standoff distance. The distance from the blast to a bridge element is a critical parameter in determining the blast effect. For a given size blast, the effect will reduce significantly with relatively small increases in distance from the blast.

  Depending on the size and standoff distance, three blast categories exist: contact, close in, and plane wave (Williamson et al. 2010). Contact blasts are very close and create high intensity, non-uniform loads where breaching, or complete loss of material at a section in a bridge element, can occur. In this case, there can be

extensive local destruction. A close in blast is farther away but still results in a localized spherical shock wave striking the structure to produce a non-uniform load and impulse-dominated response. A plane wave blast is far enough away to produce essentially planar shock waves and a uniform load on the structure. In this case, the structure will be loaded in a manner that leads to global deformation, and will be resisted by the entire structure or a number of combined elements.

- Location of blast. Blasts can occur above or below deck. Above deck blasts can affect the deck itself, and any structural elements above deck such as in a through truss, arch, or cable supported bridge. Blasts below deck would typically have more effect on pier columns, but can affect superstructure if sufficiently large. Below deck blasts can also have greater intensity because of the enclosed effect created by the overhead structure, whereas above deck blasts have more freedom to dissipate without shock wave reflection.

- Type and size of bridge element. Members with greater mass, hardness, and flexibility have greater blast resistance.

- Structural redundancy. Having multiple load paths is a key factor in resisting overall structural collapse with any type of individual member failure. Multi-column piers or multi-girder superstructures are typically able to re-distribute internal forces and provide greater resistance to overall structure collapse.

A risk management approach can be taken for bridges with greater potential of fire or blast hazard. These bridges can be identified by reviewing major corridors that would experience the greatest economical and socio-political impact if damaged by these extreme event hazards. Potential mitigation including local protective measures, alternative routes, or accelerated reconstruction strategies can be evaluated for these higher-risk bridges.

2.3.1.3.6d Seismic

Earthquakes, including those of moderate intensity, are extreme hazard events that can cause significant damage to bridges, and particularly to existing bridges that were designed under older codes and have not been retrofitted.

The 1971 San Fernando earthquake in California, which resulted in numerous bridge collapses, has often been cited as a watershed event in bridge engineering since it demonstrated the inadequacy of seismic bridge design

practices of the time (Buckle et al. 2006). The FHWA became a major sponsor of bridge seismic research shortly afterward, including research on retrofitting existing bridges.

Other subsequent major California seismic events such as the 1989 Loma Prieta and 1994 Northridge earthquakes, and the 1995 Kobe, Japan earthquake caused significant bridge damage and collapse, which also led to further research and understanding of bridge seismic behavior (Azizinamini and Ghosh 1997).

The observed damage and knowledge gained from these previous events, along with extensive research undertaken since 1971, have led to significantly improved seismic bridge design specifications (*AASHTO Guide Specifications for LRFD Seismic Bridge Design,* 2nd Edition, 2011 and *AASHTO LRFD Bridge Design Specifications*, 6th Edition, 2012), advanced concepts for seismic retrofit (*Seismic Retrofitting Manual for Highway Structures: Part 1—Bridges*, Buckle et al. 2006), and guidance for seismic design of foundations (*LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations*, Kavazanjian et al. 2011).

The current approach adopted in the *LRFD Specifications* (2012) is to design new conventional (or ordinary) bridges for a design earthquake, or level of ground motion, that represents the largest motion that can be reasonably expected during the life of the bridge. It implies that ground motions larger than the design earthquake could occur during the life of the bridge, but their likelihood of happening is small. This likelihood is usually expressed as the probability of exceedance, and it may also be described by a return period in years. The specifications call for a design earthquake that has a 7% probability of exceedance in 75 years (approximately 1000 year return period). Bridges designed and detailed under these provisions may suffer damage but should have low probability of collapse. Key principles used for the development of these specifications are that small to moderate earthquakes should be resisted within the elastic range of the structural components without significant damage, and that large earthquakes should not cause collapse of all or part of the bridge, although they may cause significant damage requiring replacement.

One of the key considerations in seismic design is repairability of the damage to bridges during moderate seismic events. Oftentimes the so-called “minor damage” may require complete replacement of the bridge. In the case of the 1995 Hyogoken-Nanbu earthquake in Kobe, Japan (Bruneau et al. 1996; Chung 1996; Shinozuka et al. 1995; Azizinamini and Ghosh 1997) steel bridges suffered damages to superstructure elements including inadequate cross-frame detailing leading to lateral bending of the girder webs near girder ends. This resulted in major retrofit activities

and the closing of major highways, such as the Hanshine Expressway, which was closed for more than a year. The Kobe experience demonstrated that even "minor damage" to steel bridges during seismic events can result in damage that could be very difficult to repair. Among the lessons learned is that critical elements of the bridge that are difficult to inspect and repair must be protected from any level of damage and remain elastic during the entire seismic excitation.

Service life design philosophy needs to be considered when following seismic design principles, by examining the effects of repair on traffic interruption after small to moderate earthquakes. In particular, the areas with potential to form plastic hinges, as described below, must be detailed so that the repair can proceed with little or no interruption to traffic. The major areas of concern are substructure elements where most plastic hinges are anticipated. The superstructure elements of the bridge are mainly kept elastic during entire seismic event.

Seismic load behavior is largely un-known; therefore, the design philosophy for buildings and bridges is to work on behavior of the structure under known conditions. Specifically, the design objective is to predefine the damage locations and design them accordingly by providing adequate levels of ductility. In the case of bridges, the preferred damage locations are at the ends of pier columns (formation of plastic hinges). In the direction of traffic, it is preferred to put columns in double curvature as shown in Figure 2.32. This allows larger portions of the pier column (two plastic hinges vs. one for single curvature) to participate in energy dissipation.

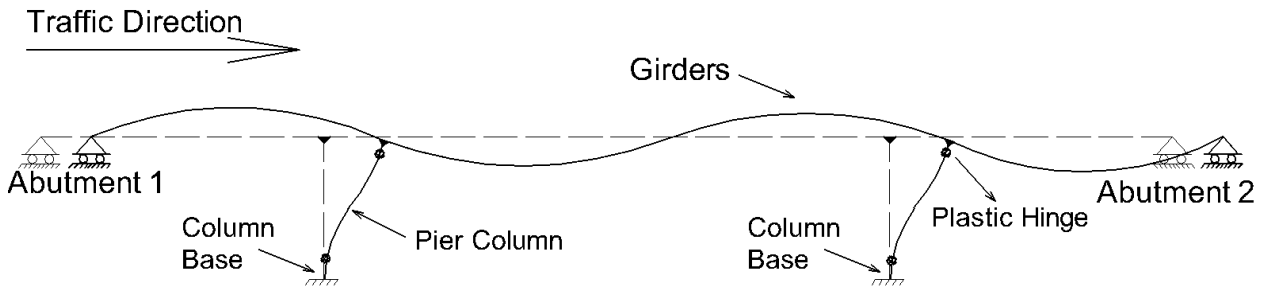


Figure 2.32. Deflected shape of a 3-span bridge under longitudinal (along traffic) direction.

In the transverse direction, pier columns are usually designed to act in single curvature, as shown in Figure 2.33.

(a) Longitudinal Deformation **(b) Transverse Deformation**

Figure 2.33. Deflected shape of pier column in longitudinal and transverse directions.

Under longitudinal excitation, plastic hinges are located near the top and bottom of the columns, while under transverse excitation; the plastic hinge is located near the bottom of the pier column.

The main design feature in seismic design of bridges is to keep the superstructure elements completely elastic during an entire seismic event. Repairing any elements of the superstructure, even "minor" damage, could be very time consuming and result in a major interruption to traffic. The elements that should remain elastic are referred to as protected elements in capacity design approach. The inelasticity is then forced to take place at predefined locations within the substructure. The predefined damage locations are the weak links or fuses that control the level of forces to be transmitted to superstructure elements. This design approach is referred to as the capacity design approach and is routinely used for designing bridges in seismic regions.

In the capacity design approach, protected elements are designed for the largest possible force effects they might experience, considering the over-strength that may exist because of higher actual material strength than that specified in design.

Areas with Seismic Risk

While earthquakes are sometimes considered primarily a California or West Coast problem in the continental United States, data produced by the United States Geological Survey (USGS) National Seismic Hazard Mapping indicates that at least 40% of the United States is subject to damaging, ground shaking levels (Kavazanjian et al. 2011). Since 1996, the USGS has developed and updated maps that have depicted areas in the United States with

various levels of seismic risk. These maps display earthquake ground motions for various risk levels including 2%, 5%, and 10% probability of being exceeded in 50 years. Figure 2.34 shows the USGS seismic hazard map depicting peak ground acceleration (PGA) levels with a 2% probability of being exceeded in 50 years, or a return period of approximately 2500 years. Along with areas along the west coast, these maps further show areas of high seismic risk in the central and eastern United States.

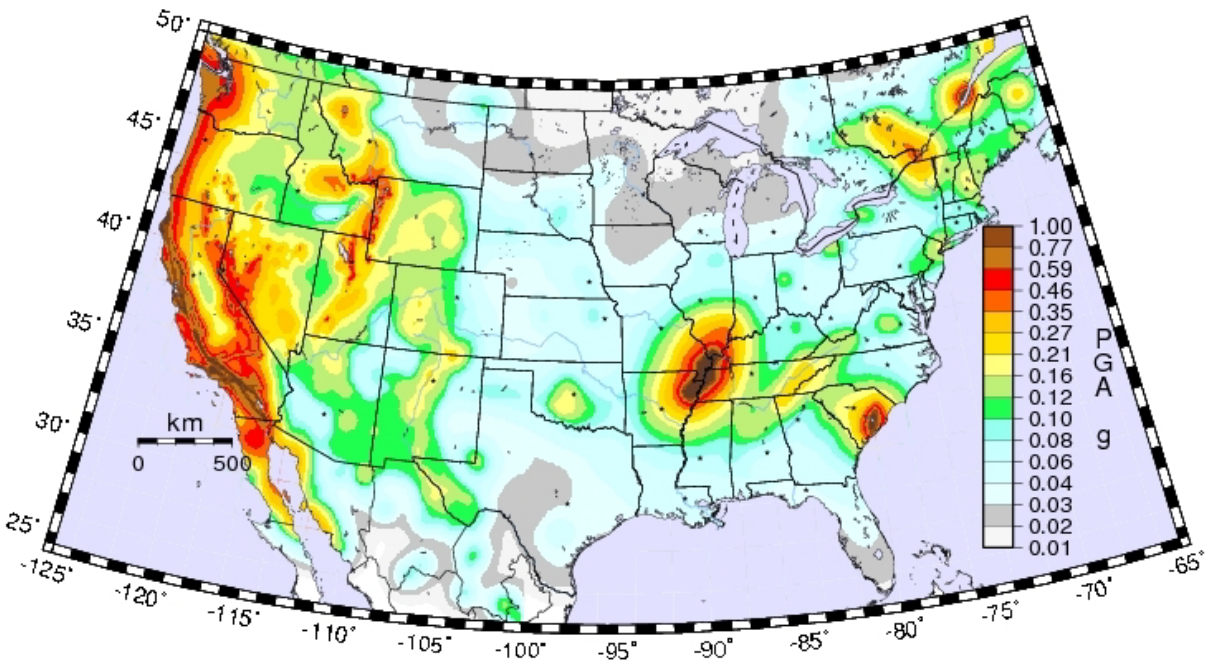


Figure 2.34. Seismic hazard map, PGA, 2% in 50 years. (Courtesy U.S. Geological Survey)

Performance During Earthquakes

Moehle and Eberhard, 2000, discuss various causes and types of damage that bridges experience during earthquakes. Key factors that affect the type and severity of bridge damage include:

- Close proximity to fault rupture. This results in ground motions having high horizontal and vertical ground accelerations as well as large velocity pulses.

- Soil conditions. Soft soil sites can significantly amplify ground motion. Soil liquefaction and lateral spreading results in permanent substructure deformations and loss of superstructure support.

- Structural Configuration. Bridges are most vulnerable that have excessive deformation demands in rigid, nonductile elements; complex or non-uniform structural configuration; curved or skewed configuration; or non redundancy..

Major types of damage include:

- Unseating at joints. Superstructure expansion joints introduce a structural irregularity that can have catastrophic consequences. These joints can occur within a span or at substructure supports. Irregular ground shaking can induce superstructure movements that can cause a span to unseat. Unrestrained superstructures can be toppled from their supporting substructures due to shaking or differential support movement associated with ground motion. Bridges with short seats are especially vulnerable. Use of restrainers has been effective in minimizing this risk. Figure 2.35 shows a span unseating failure on the Oakland Bay Bridge in San Francisco during the Loma Prieta Earthquake in 1989.

- Superstructure damage. Superstructures typically have sufficient strength to remain elastic during earthquakes, and are unlikely to be the primary cause of collapse of a span. However, certain types of superstructure damage have been observed, including bearing damage, pounding of adjacent units at expansion joints, and transverse bracing or diaphragm damage.

- Substructure damage. Substructures typically sustain the most damage, and can be categorized by column failure and abutment damage.

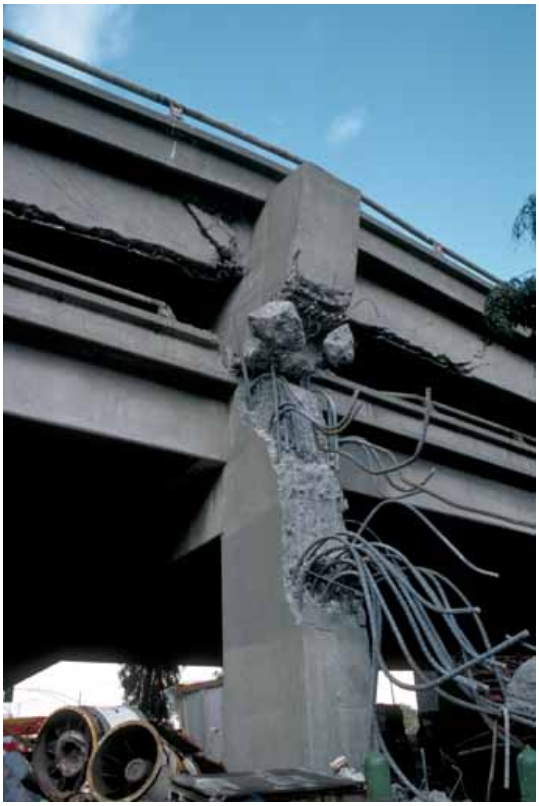
  - Column failure. The lateral load capacity of a pier is limited by the shear or flexural strength of its columns. For non-ductile reinforced concrete columns, shear failure is often the primary mode of failure when subject to large inelastic demands during strong earthquakes. Column failure is often the primary cause of bridge collapse during earthquakes (Moehle and Eberhard 2000).

    Most damage to columns can be attributed to inadequate detailing, which limits the ability of the column to deform inelastically. In concrete columns, detailing inadequacies can produce flexural, shear, splice, or anchorage failures. In steel columns, local buckling has been observed to lead progressively to collapse (Moehle and Eberhard 2000).

Figure 2.35 shows a non-ductile column shear failure on the Cyprus Street Viaduct in San Francisco that occurred during the Loma Prieta Earthquake in 1989.

- Abutment damage. Damage to shear keys and wing walls is often prevalent.

Oakland Bay Bridge upper roadway span unseating and collapse.



Cyprus Street viaduct support column collapse.

Figure 2.35. 1989 Loma Prieta earthquake damage near San Francisco, California. (Courtesy U.S. Geological Survey)

2.3.1.4 Reduced Service Life due to Production/Operation Defects

Decisions made for the production of bridges and activities during its operation can have a significant influence on overall bridge service life. These production and operation influences are introduced in the fault tree provided in Figure 2.36, and include decisions made during the design and detailing of the bridge, quality of fabrication and /or manufacturing, quality of construction, the level of inspection performed during operations, and the level and quality of maintenance. Each of these categories can be further developed to identify the lowest or basic levels causing deficiency, but these lower levels can vary significantly for each bridge system, component, or element type. The discussion here will address general issues that are common to all.

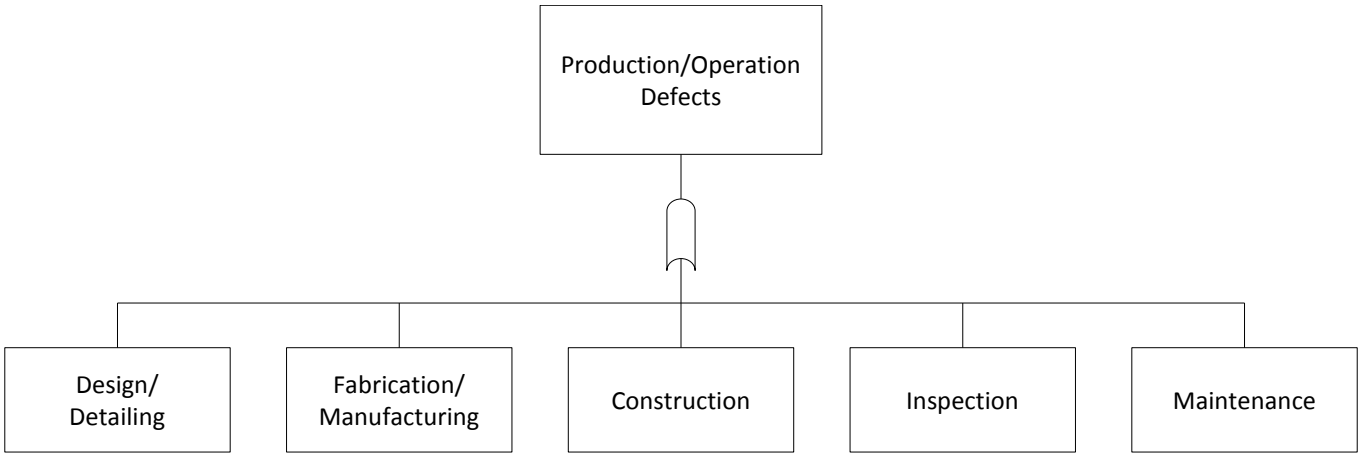


Figure 2.36. Production or operation defects.

2.3.1.4.1 Design/Detailing

Decisions made during the system selection, design, and detailing phase of a bridge project can significantly impact the service life of the bridge. It is incumbent upon designers to understand the implications of these decisions in order to help in making rational choices that will improve service life.

Examples of design/detail issues causing reduced service life include:

- Using bridge systems with deck joints that can ultimately leak and cause service life issues below. See Chapter 8 on jointless bridges.
- Providing inadequate drainage systems that allow moisture to remain on bridge decks, leading to deterioration, or improper layout, capacity, or slopes on drainage elements that ultimately lead to clogging and malfunction.
- Dealing with moisture trap details on steel bridges that hold water and debris resulting in coating damage and steel corrosion. See Chapter 6 on corrosion protection of steel bridges
- Using fatigue-prone details.
- Considering design errors. Effective quality assurance (QA) and quality control (QC) in design process is necessary to avoid errors.

2.3.1.4.2 Fabrication/Manufacturing

Defects in fabrication or manufacturing can lead to reduced service life in steel or concrete bridge elements. Undetected fabrication defects can lead to fatigue damage in steel structures.

2.3.1.4.3 Construction

Defects or damage in construction can reduce service life in steel or concrete bridge elements. Poor concrete placement and curing practices can have significant effects. See Chapter 3 on material and Chapter 4 on decks for further discussion on concrete placement.

Transportation and erection of both steel and concrete girders can become an issue if not handled properly. As high-performance materials are increasingly used for bridge construction (HPS and/or HPC), girders tend to become longer and their webs slimmer. Transportation of these girders can then cause higher stresses or out-of-plane bending, which can result in cracking. Girder stability during erection, particularly in curved steel girders, needs to be carefully addressed.

2.3.1.4.4 Inspection

Proper inspection during bridge operation is essential to identify defects and issues early, before more serious conditions develop.

2.3.1.4.5 Maintenance

Lack of or inadequate maintenance can allow deterioration to initiate throughout the bridge system and develop into serious conditions whereby the only alternative is costly component or element replacement. Applying the appropriate bridge preservation treatments and activities at the appropriate time can extend bridge service life at a lower lifetime cost (FHWA 2011).

FHWA publication *FHWA-HIF-11042* (FHWA 2011) provides general guidance on the importance and benefits of preventive maintenance as part of an overall bridge preservation program. Examples of various cost-effective preventive maintenance activities that can be applied to decks, superstructure, and substructure components are further discussed in Section 2.4.1.3.

2.3.2 Factors Affecting Service Life of Deck Component

The deck component includes several elements as described in Section 2.2.2.

Bridge deck service life and the factors affecting service life are described in detail in Chapter 4. Concrete bridge decks are particularly affected by thermal and coastal environments in which chloride penetration can cause reinforcing steel corrosion leading to concrete cracking and spalling. This cracking can cause the concrete surrounding the steel reinforcement to reach the corrosion threshold limit in environments where the top of the bridge deck is exposed to chlorides, such as deicing salts. Other concrete deck issues include wear and freeze/thaw damage.

Bridge expansion devices, commonly referred to as bridge joints, are discussed in Chapter 9.

Drainage systems are most affected by lack of maintenance that causes clogging and malfunction.

Bridge railings are affected by wet chloride environments that cause corrosion of reinforcing steel and concrete cracking and spalling in concrete railings. This condition is exacerbated at cold joints between the concrete barrier and top of slab, where salt moisture can easily penetrate and cause reinforcing corrosion. The same environments cause corrosion of steel railings.

2.3.3 Factors Affecting Service Life of Superstructure Component

2.3.3.1 Steel Superstructures

The principal causes of steel element deterioration in steel bridge systems are fatigue and fracture, and corrosion. These issues are addressed in additional detail in Chapter 7 on fatigue and fracture, and Chapter 6 on corrosion protection of steel bridges.

2.3.3.1.1 Load-Induced Deficiency—Fatigue and Fracture

Early-welded steel structures have a history of cracking at certain types of weld details due to load-induced and distortion-induced fatigue. Cracking at I-beam cover plate terminations or at other longitudinal weld terminations in tension zones has been particularly evident. Cracking in girder webs due to out-of-plane bending within stiffener web gap regions next to cross frame attachments also became a common problem. Subsequently, extensive research and laboratory testing has provided an understanding of fatigue behavior, and different weld detail types were found to have varying levels of fatigue susceptibility. Newer design provisions and recommended details were developed that provide solutions for both load-induced and distortion-induced fatigue that will achieve desired service life.

Steel bridges can fail by fracture, which is the rapid, unstable propagation of a larger flaw, most likely the result of fatigue. Fatigue crack initiation is independent of steel type and strength, but possible brittle fracture is influenced

by steel toughness among other variables. Early steels were more susceptible to brittle fracture, but in recent years, new high-performance steels—HPS 50W, 70W, and 100W—have been developed with very high toughness characteristics. Although somewhat more costly than conventional-grade steels, high-performance steels are now encouraged where applicable, particularly in non-redundant or fracture-critical applications. Use of high-performance steel allows time for any fatigue cracks that may have developed to be found during regular bridge safety inspections before fracture can occur.

Fatigue should not be an issue in new steel bridges designed in accordance with the latest *LRFD Specifications*. Extensive research has been done in recent years to identify causes and solutions for fatigue- and fracture-related problems. When using proper details and fabrication methods, both load-induced and distortion-induced fatigue problems should not be an issue in achieving desired service life.

2.3.3.1.2 Deficiency Due to Natural or Man-Made Hazards—Corrosion

Corrosion is a fundamental limitation of steel as a construction material, and is the result of exposure to oxygen and moisture. The process is greatly accelerated in the presence of chloride ions from roadway deicing salt or salt spray in a marine environment. Deck drainage with deicing salt leaking through open deck joints is a leading cause of steel element corrosion in bridges.

Corrosion control should be designed into the overall steel bridge system. Use of systems that eliminate or minimize deck joints will have a significant effect in reducing corrosion. Details that serve to protect and keep the steel dry should be included in the design. Among these are bridge system solutions that eliminate deck joints, preventing salt-contaminated drainage from reaching steel elements below.

Salt marine environments or locations subject to deicing salt spray from below also create harsh environmental conditions subject to corrosion; thorough cleaning and zinc-rich primer coating systems can provide long-term protection. However, requirements for related long-term coating maintenance cannot be overlooked, and must also be designed into an overall corrosion protection plan.

To achieve long-term bridge durability, a corrosion resistance plan must be a design requirement for every new or rehabilitated steel structure. This plan should include the use of “best painting practices” and a maintenance plan that addresses painting priorities and timetables.

Best painting practices now include paint systems that contain metallic zinc as the corrosion-resistant pigment. Zinc coatings provide "galvanic protection" to the steel, in which zinc (the more noble metal) will oxidize (corrode) in preference to the steel. To protect the zinc-coating layer from oxidation, additional coating layers are applied over the zinc-rich primer.

There are many studies that demonstrate the value of zinc coatings as a steel protection system. In addition to zinc-rich paint, these zinc coatings also include galvanizing and metalizing. Their use should be considered carefully as part of the best plan for achieving extended service life.

Weathering steel has also found widespread use in steel bridges, and has been used in both unpainted and painted applications. Weathering steel is corrosion resistant in some circumstances, but it is adversely affected by continual drainage and roadway salts, particularly below joints. Typically, special coatings are applied in these locations when using weathering steel.

In addition to weathering steel, a new structural stainless steel for bridges, ASTM A1010, has been developed for use in severe corrosive environments.

2.3.3.2 Concrete superstructures

2.3.3.2.1 General Deficiencies

There are numerous causes of deterioration in concrete superstructure elements. Chapter 3 on materials discusses the various factors influencing durability of concrete. Typically, the deficiencies are due to three main factors that were described in the fault tree analysis in Section 2.3.1. These are:

- Load-Induced:
    - Traffic causing vibration, impact, or wear; and
    - Restrained thermal movement causing internal stress and cracking.
- Natural or Man-Made Hazards:
    - Environmental influences including effects from moisture and freezing and thawing, and reinforcing corrosion due to chloride exposure; and

- Chemical influences including exposure sulfates, carbon dioxide, alkalis, and various acids.

- Defects in production/operation, primarily defective placement, curing and maintenance.

The degree and severity of concrete deterioration depends on the level of load and environmental influences to which the bridge is subjected.

The durability of concrete exposed to these influences is highly dependent on design practice, materials, and their proportioning and workmanship during construction. While all of these influences are important, the principal deterioration of concrete elements is the corrosion of steel reinforcement, which results in severe cracking, spalling, and delamination of the surrounding concrete. Cracking of the concrete often compromises the protection provided by the depassivated zone around the steel reinforcement.

Concrete superstructures in thermal or marine environments are not exposed to the same concentration of chlorides as the top of a bridge deck that may be directly in contact with deicing salts. They are, however, susceptible to the same type of corrosion, only at a slower rate through the same mechanism of failure.

2.3.3.2.2 Adjacent Box Beam Deterioration

Bridges constructed using adjacent box beams have been in service for many years and have generally performed well. However, a recurring problem with non-composite beams is cracking in the grouted joints between adjacent units, resulting in reflective cracks in the wearing surface. The development of these longitudinal cracks jeopardizes the durability and structural behavior of adjacent box beam bridges. In most cases, the cracking leads to leakage, which allows chloride-laden water to penetrate the sides and bottom of the beams and cause corrosion of the non-prestressed and prestressed reinforcement, see Figure 2.37. In addition, the load distribution among the beams is adversely affected, requiring the loaded beams to carry more load than originally intended.

Underside of beam cracking.

Top of roadway reflective cracking.

Figure 2.37. Longitudinal cracking in adjacent box beam bridge. (Courtesy HDR)

Ultimately, these conditions have led to severe beam deterioration and failure causing premature replacement of entire bridges. On December 27, 2005, the east-side fascia beam of the Lakeview Drive Bridge over I-70 in Washington, Pennsylvania failed near mid-span and fell to the highway below, as shown in Figure 2.38. Inspection of the bridge revealed heavy spalling and corrosion of the strands on the bottom flange of the failed non-composite prestressed concrete box beam. Additional corrosion was revealed on other box beams and the bridge was subsequently removed from service.

Figure 2.38. Failure of adjacent box beam bridge in Pennsylvania. (Courtesy Pennsylvania DOT)

2.3.4 Factors Affecting Service Life of Substructure Component

There are numerous causes of substructure deterioration, which can be categorized in three areas:

- Improper detailing and improper consideration of appropriate forces due to applied mean recurrence level event forces, such as scour, vessel collision and earthquake;
- Deterioration caused by corrosion and section loss, primarily from chloride intrusion; and
- Seized bearings and unintended movement restraint.

2.3.4.1 Mean Recurrence Level Event Forces

As bridge service life increases, bridges are subjected to environmental conditions for a longer period of time. Many of these conditions are accounted for in design by the use of traditional recurrence values of extreme environmental conditions such as those for hydraulic stages, wind loads, and seismic events. Design for vessel impact is treated similarly. The increased service life from 50-year (pre-LRFD) to 75-year (LRFD) to 100-year service life increases the statistical probability of exceeding the design recurrence level event.

The importance of considering these events has been exemplified by numerous bridge incidents, including, but not limited to:

- The 1987 collapse of the I-90 Bridge over Schoharie Creek emphasizing the importance of providing adequate hydraulic openings and flow characteristics under bridges. Undermining of embankments, such as shown in Figure 2.39, and undermining of pier foundations due to scour remain primary causes of bridge failures around the nation.

Figure 2.39. Scour undermining of bridge abutment. (Courtesy Atkins North America, Inc.)

- Multiple catastrophic vessel and bridge accidents around the world from the 1960s to the mid-1980s (Knott and Larsen 1990), including the 1964 and 1974 collapses of the Pontchartrain Bridge in Louisiana and the 1980 collapse of the Sunshine Skyway Bridge in Florida

- Impact forces from motor vehicles, similar to vessel collision, and subsequent fires that have resulted in structural damage to substructure units. Susceptibility to damage from these aberrant vehicle impacts underscores the importance of pier protection.

- The 1971 San Fernando Earthquake, and subsequent major earthquakes identified in Section 2.3.1.3.6d, which caused catastrophic damage to numerous bridges and stimulated research relating to bridge performance during seismic events (AASHTO 1983).

2.3.4.2 Substructure Deterioration due to Material Deterioration, Corrosion, and Section Loss

Corrosion deterioration in substructure elements is due to numerous causes, including:

- Chloride intrusion due to leakage of expansion joints and bridge drainage where deicing salts are utilized to remove snow and ice from bridge decks,

- Chloride intrusion due to direct salt splash from traffic travelling on roadways below the bridge where deicing salts are utilized to remove snow and ice from the pavement,

- Chloride intrusion found in marine and brackish water environments affecting exposed elements such as those shown in Figure 2.40, and

- Corrosion due to concrete cracking induced by alkali-silica reaction (ASR) and other concrete quality issues.

Figure 2.40. Marine pile degradation—jacket failure on concrete pile and corroded steel pile. (Courtesy Atkins North America, Inc.)

Many of the issues affecting durability of the substructure are similar to the issues affecting the bridge in general. Leakage of expansion joints and bridge drainage creates a major problem for the superstructure and substructure below the leak. Strategies to address the causes and possible relief of these leaks are addressed in Chapter 9 on joints. Strategies to address concrete quality issues, such as ASR and others, are addressed in Chapter 3 of this *Guide*. This section of the chapter addresses issues related to substructure in a marine environment or in grade separations where deicing salts can be splashed on supporting members.

Degradation of concrete and steel structures in aggressive corrosive environments, such as the splash zone in a marine environment, has historically led to a reduction in service life of numerous structures. The areas particularly susceptible to chloride intrusion are the splash zone, areas of poor concrete consolidation, spalls, and pile splices.

The degradation of piles and other deep foundation elements in marine environments has spawned an enormous concrete and steel protection and repair industry, which has developed numerous products dealing with the preservation of these deteriorating structural elements (Heffron 2007). Many of these products are relatively new, and their long-term effectiveness and expected life are not verifiable with historic data. Some of the more promising techniques include:

- Cathodic protection with embedded sacrificial anodes;
- Pile jacketing, as shown in Figure 2.41;
- Metalized coatings;
- Crystalline admixtures for crack sealing;
- Repassivation through the removal of chloride ions; and
- Various combinations of the these techniques.

Figure 2.41. Pile restoration with pile encapsulation and epoxy grout fill. (Courtesy Atkins North America, Inc.)

2.3.4.3 Potential Effect of Climate Change and Service Life of Bridges in Coastal Areas

Climate-related impacts that require adaptation are already being observed in the United States and its coastal waters (USGCRP 2009) and empirical evidence suggests that many of these and other impacts will grow in severity

in the future (USGCRP 2009). Sea level has increased along most of the U.S. coast over the past 50 years, with some areas along the Atlantic and Gulf coasts experiencing increases of greater than 8 in.

Figure 2.42 shows the projected sea level rise through 2100. The data is based on IPCC temperature projections for three different GHG emissions scenarios (shaded areas, labeled on right). The outer light gray area represents additional uncertainty in the projections due to uncertainty in the fit between temperature rise and sea level rise. All of these projections are considerably larger than the sea level rise estimates for 2100 provided in IPCC AR4 (vertical bars), which did not account for potential changes in ice sheet dynamics and are considered conservative. Also shown are the observations of annual global sea level rise over the past half-century (line), relative to 1990.

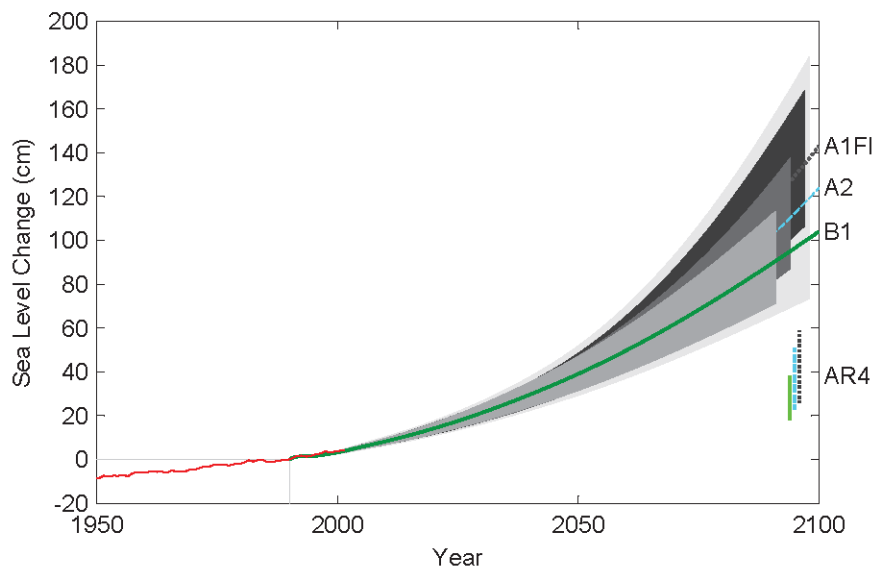


Figure 2.42. Projection of sea level rise from 1990 to 2100. (Vermeer and Rahmstorf 2009)

Although there is still debate on the level of severity that will be experienced over the next decades because of climate changes, it is important to consider this factor when planning for new bridges or retrofitting existing bridges and located in areas that could be effected.

Currently there are no organized, formal plans for considering the possible effects of climate changes on existing and new bridges located in coastal areas. However, the major impact could be substructure and splash zone of the columns located in the water.

2.3.4.4 Seized Bearings and Unintended Movement Restraint

The structural design of substructures is in part based on a distribution of longitudinal and transverse forces associated with the allowable movement of the superstructure. Fixed bearings provide an anchor that is intended to

restrict "walking" of the superstructure resulting from shrinkage and cycling of expansion and contraction. The bearings are usually located longitudinally near the point of zero movement of a supported multi-span superstructure unit. Care must be exercised in locating the so-called "point of zero movement," especially in the case of curved girder bridges. Existence of such a point could be viewed as an assumption, more than reality. Chapter 8 on jointless bridges provides a detailed discussion of the point of "zero movement" for curved girder bridges. Point of "zero movement" is more meaningful in instances of straight bridges with zero skew. Existence of skew or curvature complicates the determination of point of zero movement. These fixed bearings, used in combination with bearings designed to allow the superstructure to either move or slide over the top of the substructure, reduce restraining forces that would otherwise be required to resist the movement. Improper function or seizing of the bearings results in unintended movement restraint that can raise the force resisted by the substructure well above the intended design. This unintended restraint can cause unanticipated cracking with greater potential for corrosion. Proper bearing performance is essential to durability of substructure and is addressed in Chapter 10 of this *Guide*.

2.4 OPTIONS FOR ENHANCING SERVICE LIFE

This section describes options for enhancing service life and addresses factors identified in Section 2.3.

2.4.1 System-Related Options

2.4.1.1 Functional Options

Sometimes bridges are replaced because of functional issues well before their full potential service life is achieved. The following considerations should be incorporated into a bridge system where there is probability that future bridge widening or crossed roadway widening may be necessary:

- Use bridge system types that can be widened, particularly superstructures. Multi-girder superstructures typically lend themselves to this ability. Cast-in-place concrete structures provide additional challenges for widening.

- Consider longer spans when crossing roadways that have the potential for widening.

- Consider additional vertical clearance when setting limits for bridges that have the potential for future widening. Bridge widening along a deck cross slope can infringe on minimum vertical clearances if additional clearance is not provided at the beginning.

2.4.1.2 System Configuration Options

How the bridge system is configured will have significant impact on service life. Leaking deck joints have been identified as one of the leading causes of system deterioration. The following considerations should be incorporated into the system selection to avoid or mitigate this risk (see Chapter 8 on jointless bridges):

2.4.1.2.1 Integral Abutments

Consider integral abutments to eliminate joints at abutments.

- Fully-integral abutments eliminate deck joints and bearings, and

- Semi-integral abutments eliminate deck joints

It should also be noted that in addition to many other advantages with respect to enhanced service life, jointless integral abutment bridges provides lower initial cost. Jointless integral abutment bridges are increasing in popularity and their use is encouraged where appropriate. Chapter 8 on jointless bridges provides step-by-step design provisions for jointless integral abutment bridges. Unlike the current practice there is no need to impose arbitrary limits on maximum length of bridges.

2.4.1.2.2 Maximum Length Limits for Continuity

Using the procedure specified in Chapter 8, establish the maximum lengths for continuity to minimize number of joints in long, multi-span viaducts.

- Superstructure with integral abutments and no joints. Use design provisions stated in Chapter 8 on jointless bridges to establish the maximum length for integral abutments.

- Long continuous superstructure with joints only at abutments. Consider the maximum length for the structure type that can accommodate joints only at abutments without any intermediate joints.

- Multiple continuous units with interior joints (viaduct construction). Consider the maximum length for unit layout between interior joints to minimize number of joints.

- Where deck joints have to be used. Consider joint systems that are more leak resistant. See Chapter 9 on expansion joints.

2.4.1.2.3 Provide Continuity Over Piers

Various methods for providing continuity over piers that eliminate deck joints should be evaluated. These include:

- Fully continuous. These systems are continuous for dead and live load and are suitable for all span lengths, but are typically more economical for longer spans where the benefit of dead load continuity is better realized.

- Simple for dead load, continuous for live load. These systems are becoming very popular in the 150-ft span range because of the ease and speed of construction, but girders must carry all dead load in positive bending.

- Link slab. This is a very economical and popular concept where the deck is made continuous over intermediate supports while the beams remain simple span without any continuity. This concept is further discussed in Chapter 8.

- Integral pier caps. This concept eliminates joints and bearings while lowering the roadway profile, which can add further economical benefit. However, it requires special details for the integral connection, and must consider the system interaction between superstructure and substructure.

2.4.1.2.4 Fixed- and Expansion-Pier Layout

Proper layout of fixed and expansion pier locations can help balance loads to piers while minimizing superstructure thermal movements. Considerations include:

- Traditional layout (single pier fixed, others expansion). Providing a single fixed pier near the center of the bridge focuses longitudinal loads to one location, which is usually acceptable for minimum height bridges and balances thermal movements at adjacent piers and abutments as much as practicable.

- Multiple pier fixity. This has benefit in taller pier situations where longitudinal loads can be distributed to additional piers and tall pier flexibility minimizes temperature loads that develop. The relative stiffness of multiple fixed piers must be considered in distributing longitudinal loads and in determining temperature forces.

- Integral piers. This creates a fixed pier condition and has the benefit of eliminating joints and bearings and lowering roadway profile. Depending on the type of detail, it can also provide longitudinal frame action in resisting longitudinal loads.

- Orientation of expansion bearings on curved and skewed alignments.

2.4.1.3 Maintenance Considerations

FHWA publication *FHWA-HIF-11042*, August, 2011, provides general guidance on the importance and benefits of preventive maintenance as part of an overall bridge preservation program. Examples of various cost-effective preventive maintenance activities that can be applied to decks, superstructure, and substructure components are presented, including:

- Sealing or replacing leaking deck joints before deterioration can begin on elements below;

- General bridge cleaning including decks, joints, drainage systems, bearings, tops of piers and all elements below deck joints;

- Placing deck overlays on aging decks;

- Installing cathodic protection or electromechanical chloride extraction;

- Applying concrete sealants or coatings;

- Spot- and zone-painting steel elements;

- Retrofitting fatigue-prone details;

- Lubricating bearings;

- Jacketing concrete piles in marine environments;
- Installing scour countermeasures; and
- Removing large debris from stream channels.

2.4.1.4 Access Considerations

Proper accessibility to all components and elements below deck for inspection and future maintenance is essential for achieving long-term service life. Accessibility and maintainability considerations must be included as part of the overall bridge system configuration.

2.4.2 Deck Component Options

See Chapter 4 on bridge decks for options related to deck components.

See Chapter 9 on expansion joints for options related to joint elements.

2.4.3 Superstructure Component Options

2.4.3.1 Steel Superstructures

See Chapter 7 on fatigue and fracture for options related to controlling fatigue.

See Chapter 6 on corrosion protection of steel bridges for options related resisting corrosion, including paint systems, galvanizing, metalizing, and use of corrosion-resistant steels.

2.4.3.2 Concrete Superstructures

2.4.3.2.1 General Strategies

Several strategies have been developed to address the durability of concrete systems, subsystems, and components, and are fully described in Chapter 3 on materials. These strategies include:

- Proportioning concrete to provide low permeability and low cracking potential;
- Using non-corrosive materials for reinforcement, such as stainless steel, and protective coatings such as epoxy coating;
- Prestressing/post-tensioning elements to eliminate cracking;

- Applying other solutions, such as cathodic protection and electrochemical chloride extraction; and

- Using various combinations of these strategies.

The use and application of these strategies is highly dependent on the environment to which the concrete systems are exposed. A single strategy that fits all conditions within the United States does not exist. These strategies must be reviewed for applicability by each governing agency.

2.4.3.2.2 Solutions for Adjacent Concrete Box Beams

Adjacent box beam bridges have experienced service life issues in recent years, and have resulted in failures as illustrated in Section 2.3.3.2.2. A survey on the current practices in the design and construction of adjacent box girder bridges in the United States and Canada conducted by the Precast/Prestressed Concrete Institute (PCI) subcommittee found that 29 states and three provinces are currently using adjacent box girder bridges. Most of these transportation agencies have experienced premature reflective cracks in the wearing surface on the bridges built in the late 1980s and early 1990s. These agencies have emphasized the importance of eliminating these cracks that allow the penetration of water and deicing chemicals leading to the corrosion of reinforcing steel in the sides and bottoms of concrete boxes. The following are examples of the preventive actions that the states and provinces have recommended based on lessons learned in the last two decades:

- Use of cast-in-place deck on top of the adjacent boxes to prevent water leakage and to uniformly distribute the loads on adjacent boxes.

- Use of non-shrink grout or appropriate sealant instead of the conventional sand/cement mortar in the shear keys, in addition to blast cleaning of key surfaces prior to grouting. A few states have also recommended the use of full-depth shear keys due to their superior performance over traditional top flange keys.

- Use of transverse posttensioning to improve load distribution and minimize differential deflections among adjacent boxes. Adequate posttensioning force should be applied after grouting the shear keys to minimize the tensile stresses that cause longitudinal cracking at these joints.

- Use of end diaphragms to ensure proper seating of adjacent boxes and intermediate diaphragms to provide the necessary stiffness in the transverse direction.
- Use of wide bearing pads under the middle of the box to eliminate the rocking of the box while grouting the shear keys. Using sloped bearing seats that match the surface cross slope is also recommended.
- Use of adequate concrete cover and corrosion inhibitor admixtures in the concrete mix to resist the chloride-induced corrosion of reinforcing steel.
- Eliminating the use of welded connections between adjacent boxes and avoiding dimensional tolerances that result in inadequate sealing of the shear keys.

Although the use of posttensioned diaphragms to transversally connect adjacent box girders is an effective and practical solution in many cases, it has some disadvantages. Posttensioning of skewed bridges is difficult and may have to be staggered and done in stages. Staged construction leads to a significant increase in construction cost and duration, due to the variation in diaphragm location, the large number of post-tensioning operations, and excessive traffic control required for replacement projects. Moreover, posttensioned diaphragms depend on the shear keys to achieve the desired continuity. Shear keys need to be properly cleaned, sandblasted, sealed, and grouted, which adds complexity to the system and makes it susceptible to cracking and leakage.

A revised approach was developed at the University of Nebraska that eliminates diaphragms and uses top and bottom transverse ties. This work was done as part of *SHRP 2, Project R19A*, and is summarized in the forthcoming final report. Grade 75 threaded rods are used every eight feet to connect each pair of adjacent boxes at the top and bottom flanges. These rods provide continuous connection that transfer shear and moment between adjacent boxes more efficiently than the mid-depth transverse ties at discrete diaphragm locations. A slight modification is made to the standard box cross-section by developing full-length horizontal and full-depth vertical shear keys as shown in Figure 2.43, and is referred to as the narrow joint system. The boxes are fabricated with a plastic duct at the top and bottom flanges to create openings for the threaded rods, as shown in Figure 2.44. The bottom duct is inserted between the two layers of prestressing strands, while the top plastic duct is located 3 in. from the top surface to

provide adequate concrete cover. Vertical vents are provided at one side from each box to allow the air to get out while grouting the ducts.

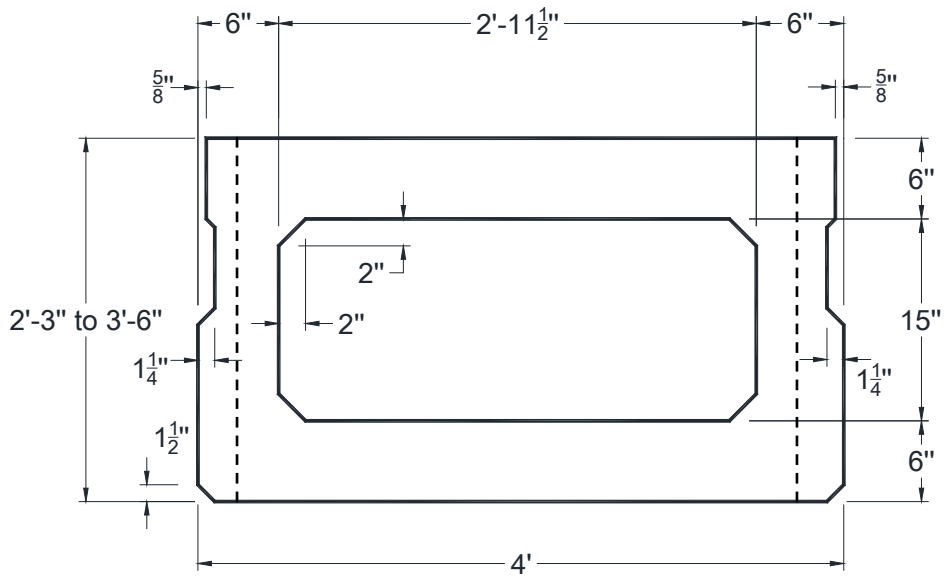


Figure 2.43. Narrow joint system box dimensions.

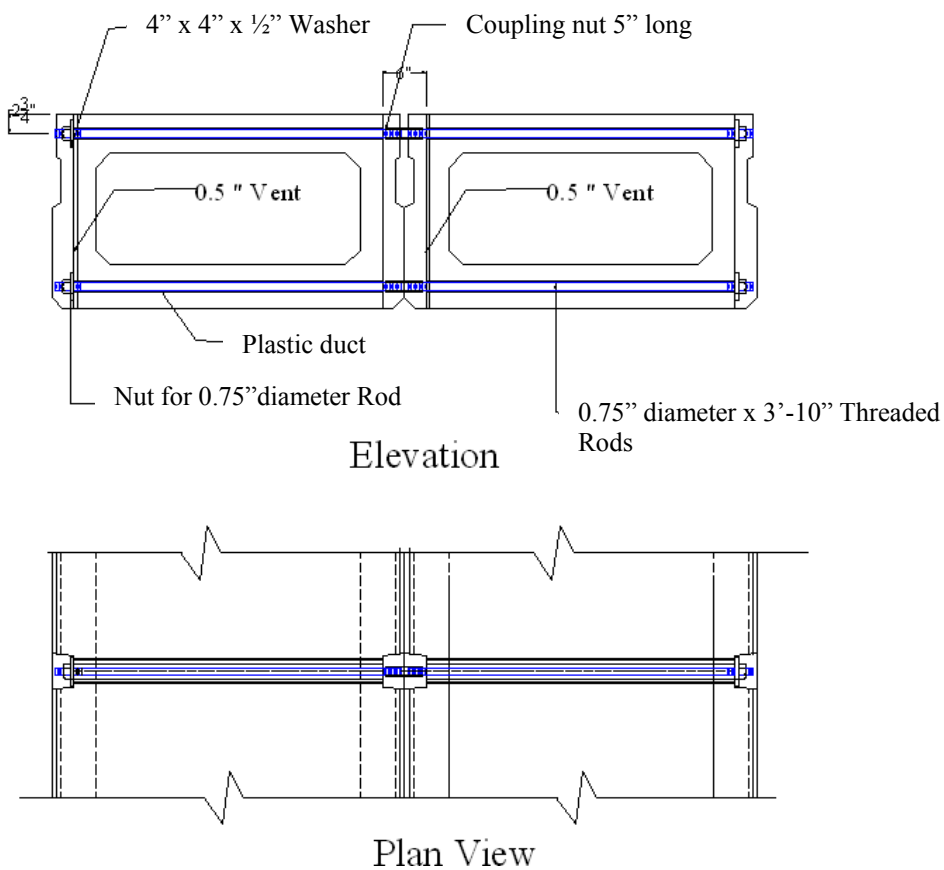


Figure 2.44. Narrow joint system connection details.

2.4.3.3 Bearing Options

See Chapter 10 for a complete discussion of bearing options. Use of steel-reinforced elastomeric bearings is considered the best option for achieving long-term service life.

2.4.4 Substructure Component Options

Substructure deficiencies are primarily due to natural or man-made hazards, and operational issues.

Hazards issues include:

- Element material deterioration due to thermal, coastal, or chemical climate, and reactive materials.

  Strategies for mitigating these effects are discussed in:

  - Chapter 3 on materials,
  - Chapter 5 on corrosion protection of reinforced concrete, and
  - Chapter 6 on corrosion protection of steel bridges.

- Hydraulic action, which includes flood/storm surge and scour; and
- Vessel collision.

Operational issues include frozen or locked expansion bearings and is discussed in Chapter 10 on bridge bearings.

2.4.4.1 Hydraulic Action

2.4.4.1.1 Flood/Storm Surge

Following hurricanes Ivan (2004) and Rita (2005), which damaged numerous bridges along the Gulf coast, the FHWA along with 10 states sponsored a study that culminated in the development of the *AASHTO Guide Specifications for Bridges Vulnerable to Coastal Storms* (AASHTO 2008). This guide specification recommends that where practical, a vertical clearance of at least 1-ft above the 100-year design wave crest elevation, which includes the design storm water elevation, should be provided. The study further recommends additional freeboard due to the large uncertainty in the basic wave and surge data needed to determine the wave crest elevation. If this

vertical clearance is not possible, the bridge should be analyzed and designed to resist storm wave forces, and other wave force mitigation measures should be implemented, such as venting to reduce buoyancy forces.

Florida DOT issued a Temporary Design Bulletin C09-08 (FDOT 2009), which required the implementation of the *AASHTO Guide Specifications for Bridges Vulnerable to Coastal Storms* discussed above.

In the Florida DOT bulletin, the importance or criticality of bridges is a factor in evaluating the risk of damage and potential consequences.

Table 2.1. Bridge Importance Level.

Importance	Level of Design
Extremely Critical	Strength limit state for little or no damage
Critical	Extreme event level state for repairable damage
Non-Critical	Evaluation of wave forces not required

Florida DOT further recommends that for all bridges subject to coastal storms, simple and inexpensive measures that enhance a structure's capacity to resist storm forces should be implemented. For example, placing vents in all diaphragms and venting all bays will reduce the effects of buoyancy forces on a structure. Anchoring the superstructure to the substructure to reduce or prevent damage from storm surges should also be considered.

2.4.4.1.2 Scour

The *LRFD Specifications* require that scour at bridge foundations be designed for 100-year flood event or from an overtopping flood of lesser recurrence interval. Additionally, the bridge foundations are to be checked for stability for the 500-year flood event or from an overtopping flood of lesser recurrence level.

FHWA *Hydraulic Engineering Circular No. 18: Evaluating Scour at Bridges* (*HEC-18*) (Arneson et al. 2012) provides guidelines for designing bridges to resist scour and improving the estimation of scour at bridges.

Riprap remains the counter-measure most commonly used to prevent scour at bridge abutments. A number of physical additions to the abutments of bridges can help prevent scour, such as the installation of gabions and stone pitching upstream from the foundation. The addition of sheet piles or interlocking prefabricated concrete blocks can

also offer protection. These countermeasures do not change the scouring flow and are temporary since the components are known to move or to be washed away in certain flood events.

FHWA recommends design criteria and countermeasures in *HEC-18* and in *Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance* (*HEC-23*) (Lagasse et al. 2009), such as avoiding unfavorable flow patterns, streamlining the abutments, and designing pier foundations resistant to scour without depending upon the use of riprap or other countermeasures, as is available. To reduce the potential for scour, the bottom of spread footings should be placed below the scour design depth, and piles or drilled shafts should be designed by assuming all material above the maximum scour depth is unavailable for load resistance.

Floods also place extreme lateral loads on piers and bents, which should be considered in design for bridges in locations with high risk of flooding. In these cases, the presence of soil and any corresponding load or resistance should only be considered below the minimum scour elevation.

2.5 STRATEGY SELECTION

The intent of this section is to outline an approach for selecting the most appropriate bridge systems to accommodate operational requirements and site conditions, while also achieving the desired target design service life. The process combines the requirements for selecting bridge systems on the basis of operational needs and initial construction cost with requirements for service life and life-cycle cost. The approach presented in this section must be developed in conjunction with strategies presented in subsequent chapters, which address materials, and specific components and elements in additional detail, such as bridge deck, joints, or bearings.

Providing bridge systems with enhanced service life requires a complete understanding of the potential deterioration mechanisms, or factors affecting service life. These mechanisms, described in Section 2.3, are associated with load-induced conditions, local environmental hazards, production-created deficiencies, and lack of effective operational procedures. The avoidance or mitigation of these deterioration mechanisms through the appropriate selection of enhancement techniques is described in Section 2.4. The overall system selection process involves a detailed evaluation of these mechanisms as they would affect each major bridge component, subsystem, and element, and identification of a group of individual strategies that together define an optimum bridge system

configuration. This integrated approach of combining operational and service life requirements will result in the optimum bridge system with the greatest potential for enhanced service life.

2.5.1 Service Life Design Methodology

Chapter 1 provides information concerning design methodologies for service life.

2.5.2 System Selection Process Outline

A process for selecting the optimum bridge system is shown in Figures 2.45a through 2.45c, in which the various steps involved are flow-charted. The outlined process involves four major steps, which are described in the various flow chart blocks:

1. Identifying demand, which includes requirements that the bridge must satisfy—Block 1;
2. Identifying feasible bridge system alternatives that satisfy requirements—Block 2;
3. Evaluating alternatives for service life—Blocks 3 through 12; and
4. Comparing and selecting the optimum alternative—Block 13.

Each of these steps and subsequent flowchart blocks are described in Section 2.5.3 following the flowchart, and further examples corresponding to the blocks are provided in Table 2.2.

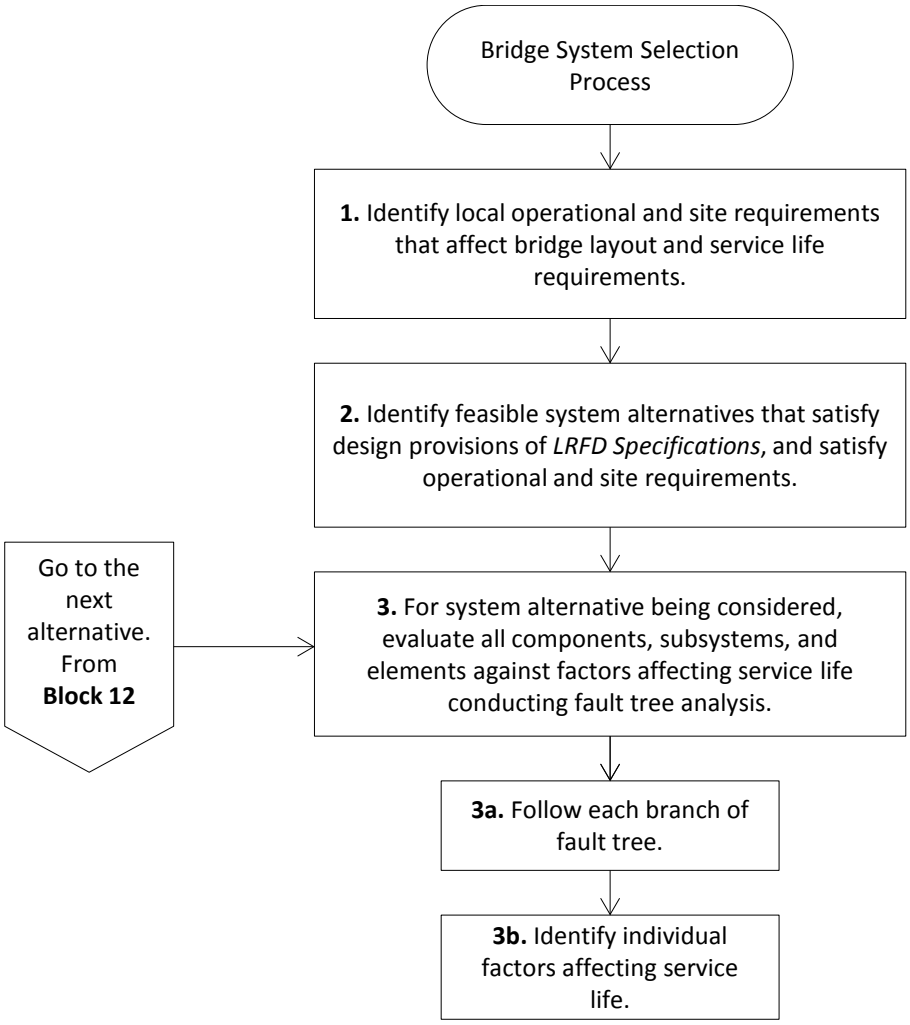


Figure 2.45a. Integrated system selection and design process.

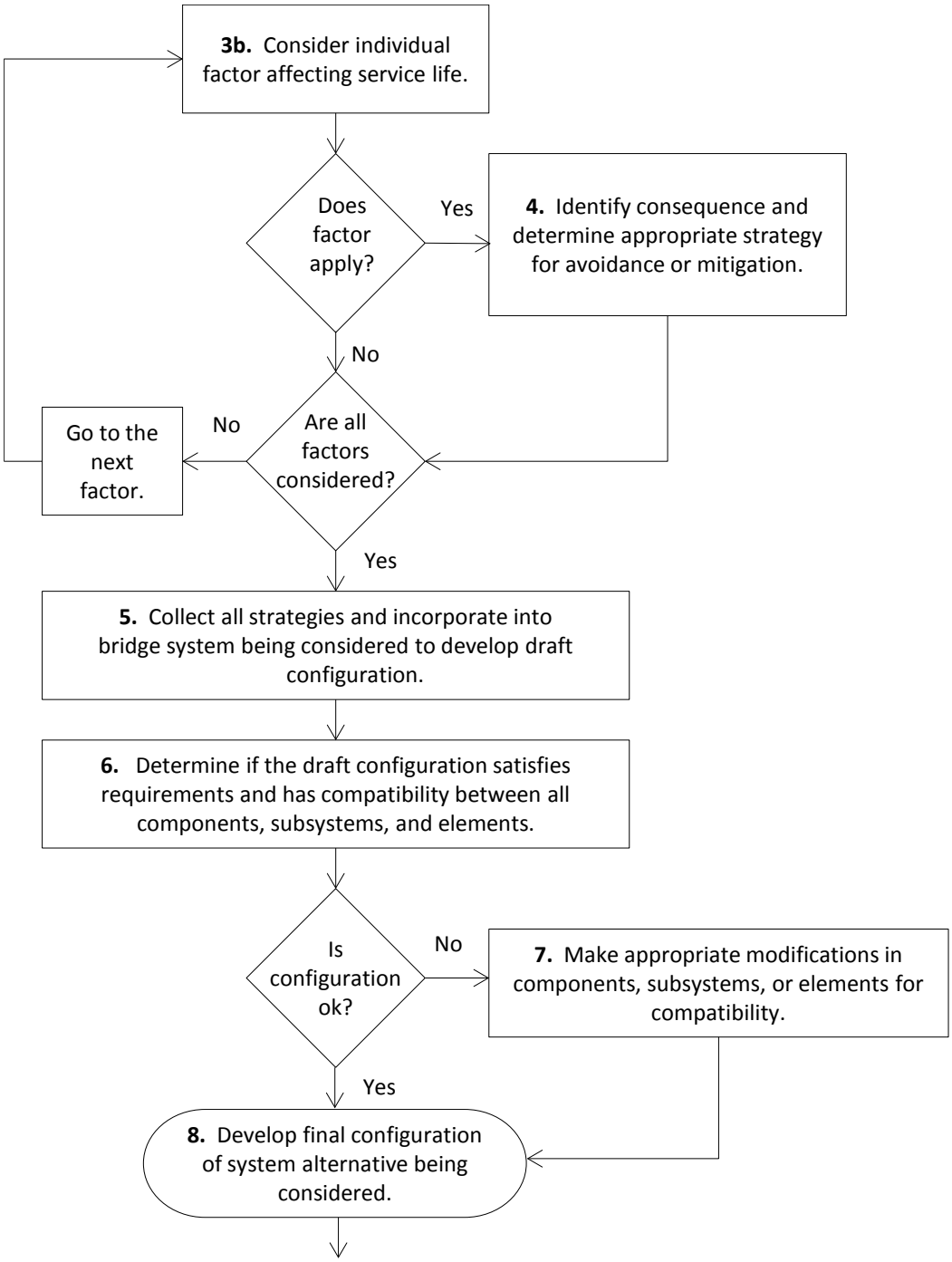


Figure 2.45b: Integrated system selection and design process.

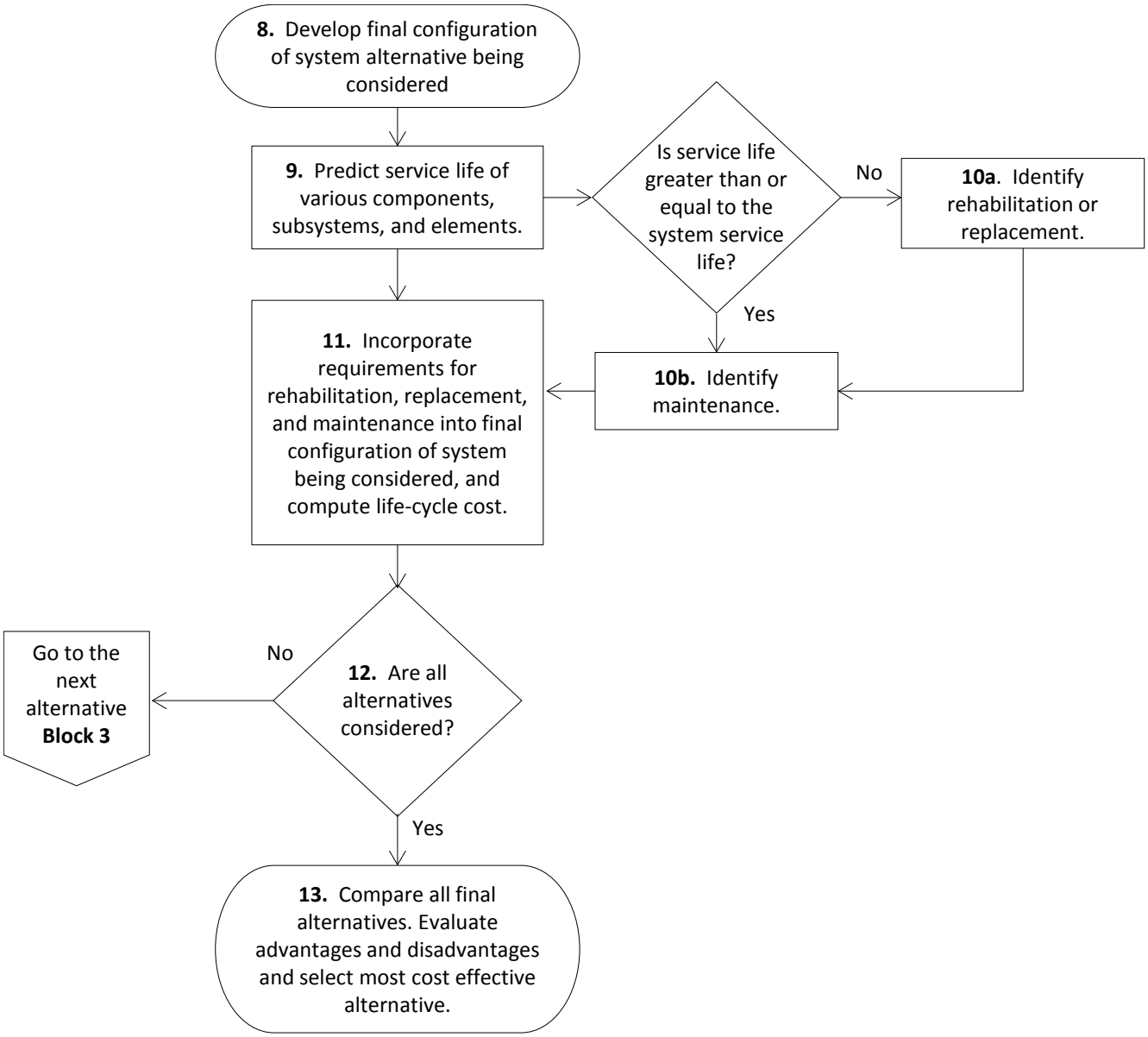


Figure 2.45c: Integrated system selection and design process.

2.5.3 System Selection Process Description

The following is a discussion of the steps in the selection process relating to the flowchart blocks in Figure 2.45.

Block 1. Identify demand parameters that the bridge must satisfy which includes:

a. Operational requirements. These relate to issues as the type of corridor, traffic and truck percentages, vehicle loads, and mixed-use requirements. This information establishes requirements for capacity, number of lanes, and other operational issues.

b. Site requirements. These typically relate to issues that affect bridge layout including, features crossed, geometrics involving curvature or skew, geotechnical data, and other constraints.

c. Service Life requirements. Based on the type of corridor and traffic demand, a judgment is made, usually by bridge owner, as to the importance of the bridge and the target service life the bridge should be designed for.

d. Future considerations. Based on the type of corridor, an evaluation is made regarding the potential for future needs. This might include the probability of future traffic demand that would require bridge widening, or the potential of having to widen any crossed roadways that might affect span layout.

Overall, these requirements relate to how the bridge must function and how long it should last. They may also include limitations on how a bridge might be constructed, how much it should cost, or how it might look in a given setting. Further examples of these items are listed in Table 2.2.

Block 2. The next step is to identify feasible system alternatives that satisfy operational and site requirements, while also satisfying various provisions of *LRFD Specifications*. This step would typically include:

a. Preparing alternative bridge span arrangements that accommodate features crossed, horizontal and vertical clearances, geometric requirements, and other construction constraints. Geotechnical requirements impact foundation costs and where possible, span layouts need to consider the relative costs between superstructure and substructure in achieving optimum span lengths. Optimum span lengths will also vary for different superstructure types, and the interaction between optimal span and superstructure type needs to be considered in setting span layouts.

b. Identifying feasible deck alternatives, such as CIP concrete, precast concrete, or other. This is further discussed in Chapter 4.

c. Identifying feasible superstructure alternatives that accommodate geometric and span-length requirements. Various bridge systems are described in Section 2.2. Feasible superstructure alternatives might include steel or concrete girders, or concrete segmental box girders, among others.

d. Identifying feasible substructure and foundation types that are compatible with superstructure and geotechnical requirements.

e. Verifying preliminary member or element sizes at this stage through preliminary design.

Block 3. After feasible alternative bridge systems are identified, the next step is to evaluate each alternative, against factors that affect service life using the fault tree analysis described in Section 2.3. Within each system, each component, subsystem, and element should be considered. Procedures in other chapters also need to be followed in evaluating certain specific components or elements such as bridge deck, joints, bearings, or in evaluating effects on materials such as ASR or freeze/thaw. A system-level evaluation of these components and elements is necessary first in order to assure compatibility among all components within the system.

Block 3a. In evaluating the various components, subsystems and elements within a system alternative, each branch of the fault tree illustrated in Section 2.3 of Figure 2.23, should be followed. The initial categories to be considered include obsolescence, mostly pertaining to inadequate capacity, or deficiency, pertaining to damage or deterioration. Deficiency is further subdivided into that caused by loads, natural or man-made hazards, or production/operation defects, which are shown on Figures 2.24, 2.25, and 2.36, respectively. The fault tree branches end with basic events or lowest levels of resolution, which are the individual factors to be considered. The individual factors within fault trees for which the specific mitigation strategies need to be developed are placed in circle symbols.

Block 3b. Each factor, is systematically examined and evaluated in regard to its application to the various bridge system components, and a decision is made as to whether that factor applies. For example, in the case of natural or man-made hazards, is the bridge in a thermal climate, which is a cold, wet climate with heavy use of roadway deicing salt? If the bridge is not in this type of climate, this factor would not apply and the evaluation continues to the next factor. If the bridge is in this type of climate, the evaluation proceeds to the next step.

Block 4. If the individual factor applies, the potential consequence of that factor should be evaluated and an appropriate strategy(ies) should be identified to either avoid the factor or mitigate its influence. For example, if the bridge is in a thermal climate as just described, strategies to avoid or mitigate the potential of corrosion due to deicing salts would need to be identified. Special strategies such as stainless steel deck reinforcement or metalizing ends of steel girders under deck expansion joints need to be included. Strategies to mitigate various factors affecting service life are given in Section 2.4. After determining the appropriate mitigation strategy, the evaluation proceeds to consider the next factor.

Block 5. After all factors are considered, the identified strategies are summarized and integrated into the bridge system to develop a draft final configuration.

Block 6. This draft configuration then needs to be checked as to whether all requirements are satisfied and whether all identified strategies to improve service life are consistent with one another and are compatible between various components and elements.

Block 7. If inconsistencies or incompatibilities are found, they need to be resolved by making appropriate modifications to the strategies or to the affected components or elements. For example, in evaluating a steel superstructure in a thermal climate, a strategy might be to metalize girder ends below deck expansion joints at abutments to mitigate the possibility of corrosion. A substructure evaluation however, might identify a strategy to eliminate expansion joints and use integral abutments. These two strategies are inconsistent in that if the integral abutment strategy is implemented, the strategy for metalizing steel girder ends could be unnecessary. A resolution is made as to which strategy to implement.

Block 8. After all inconsistencies are identified and resolved, and the final configuration satisfies all requirements, the final configuration of the system alternative is developed. It is feasible to develop more than one configuration, capable of meeting the service life requirements. See Chapter 1 for an example.

Block 9. The next step in the process is to predict the service life of the various bridge components, subsystems, and elements within the final bridge system alternative under consideration. Deterioration models to quantitatively predict service life are limited or non-existent, so often the prediction is made on the basis of experience or expert opinion. The predicted service life of the various components, subsystems, and elements is then checked to see if they will be equal to or greater than the owner-specified service life of the bridge system.

Block 10a. If they are not equal to or greater than the owner-specified service life of the bridge system, future rehabilitation and/or replacement of these components, subsystems or elements will have to be anticipated at certain intervals during the system service life. For example, if the bridge deck is predicted to have a service life less than the system service life, the rehabilitation plan might include milling and overlaying at an anticipated interval. A replacement plan would include the anticipated extent of replacement and interval.

Block 10b. Future maintenance requirements will have to be identified whether the component, subsystem or element service life meets the system service life or not. In the case of bridge decks, maintenance might include washing the deck surface to remove salt, or cleaning gutters and drains to permit proper drainage.

Block 11. After all requirements for replacement/rehabilitation and maintenance are determined, these requirements are incorporated into the final alternative system configuration, which now includes the system layout with a compatible and consistent set of service life strategies, and with a replacement/rehabilitation and maintenance plan. The corresponding initial construction cost and life-cycle cost for this complete alternative system configuration is then computed.

Block 12. When the evaluation is completed for this alternative, the engineer should return to Block 3 and repeat the evaluation steps for the next identified bridge system alternative. When all identified system alternatives have been evaluated, the designer should proceed to the last step.

Block 13. The last step is to compare the final alternative bridge system configurations and select the optimum alternative. Often this is done in a matrix-type evaluation in which various key performance categories, determined specifically for the bridge, are weighted and evaluated for each alternative. Example performance categories might include service life, traffic impact, construction duration, construction complexity, site suitability, local preference, or aesthetics. In this process, the advantages and disadvantages of the various alternatives are compared. Initial construction costs and life-cycle costs are also compared. With this type of evaluation, the optimum bridge system can be identified as part of a complete cost-benefit selection process.

2.5.4 System Process Tables

This process is further expanded in Table 2.2 , which illustrates the process design phases of the preliminary planning or Type, Size and Location (TS&L) stage, and final design stage for a new bridge. Table 2.2 supplements the information and examples provided in Sections 2.5.2 and 2.5.3.

Table 2.2. System Selection Process for Operational and Service Life Requirements.

Preliminary Planning Stage or TS&L	
1. Identify Demand Requirements	**Operational Demand Requirements (Functionality) Corridor Related Items** Identify corridor and function or traffic related demand requirements. Examples include: • Corridor type such as interstate, urban arterial rural, or other; • Traffic volumes and required capacity; • Truck volumes; • Special vehicle uses such as over-size vehicles, or tanker trucks; • Traffic maintenance requirements; • Mixed use requirements; and • Vehicle loads and special vehicle load requirements.

	Service Life Requirements (Durability and Long-Term Performance) Corridor-Related Items
	Examples include:
	• Identify bridge importance;
	• Identify target design service life;
	• Based on corridor type, evaluate potential future needs by considering:
	  ○ Potential for future bridge widening,
	  ○ Potential for future widening of crossed roadways, and
	  ○ Vertical clearance requirements related to future bridge widening or widening of crossed roadways.

	Local Site-Related Requirements
	
	Identify local site-related issues that the bridge must accommodate. Examples include:
	
	• Geometrics, curvature, and skew;
	• Features crossed;
	• Horizontal and vertical clearances;
	• Hydraulic or waterway requirements;
	• Navigation requirements;
	• Utility issues, either carried or crossed;
	• Other physical boundary conditions;
	• Geotechnical issues;
	• Environmental issues;
	• Drainage requirements and special criteria;
	• Access for construction; and
	• Aesthetics and sustainability.
	
	Local Items
	
	Identify local relevant environmental or man-made hazards that affect service life (follow fault tree). Examples include:
	
	• Climate type;
	  ○ Thermal,
	  ○ Coastal, and
	  ○ Chemical.

	• Potential for hydraulic action hazard—flood or scour;
	• Potential for wind hazard;
	• Potential for other extreme event hazards;
	  ○ Vehicle/vessel collision,
	  ○ Fire or blast, and
	  ○ Seismic event potential.

	Construction Constraints Identify construction requirements that can affect service life, including: • Construction phasing requirements, • Construction schedule requirements (such as accelerated bridge construction), and • Special local construction preferences.
2. Identify Alternative Solutions	Identify feasible alternative bridge systems including span layouts, structure types and materials that accommodate operational and site requirements including: • Accommodate span requirements and constraints. • Accommodate curvature, profile and skew requirements. • Provide optimal span balance. and • Provide optimal superstructure /substructure cost balance. In addition, • Identify superstructure options, • Identify alternative bridge deck options,

	• Identify superstructure-substructure connection options, • Identify expansion joint location and type options • Identify bearing type options, • Identify substructure options, and • Identify foundation options.
3. Evaluate and Compare Alternatives	Identify system service life improvement strategies that avoid future potential obsolescence • Consider bridge types that can be widened, • Consider span lengths that accommodate widening of crossed roadways, and • Consider additional vertical clearance for bridges that may need to be widened. Evaluate factors affecting service life and identify strategies to avoid or mitigate all hazards. For each alternative bridge system, estimate potential service life of components, subsystems, and elements, and compare with target design service life of bridge system. Determine rehabilitation, replacement,, and maintenance requirements over target design service life. Determine estimated life-cycle costs over target design service life.
4. Select Optimal Alternative	Compare operational advantages and disadvantages of bridge system alternatives • Identify local preferences for structure types and construction; • Compare estimated life-cycle costs; and • Select optimal cost-effective system considering operational and service life

	requirements and cost benefit analysis.
Final Design Stage	
1. Design	Design in accordance with strength and serviceability provisions of *LRFD Specifications* Include specific design and details to address service life issues identified in preliminary stage. Examples include: • Design provision and details that allow for potential future deck replacement; • Plan and access for inspection and future maintenance and rehabilitation; • Drainage plan; • Bridge deck protection plan; • Reinforcing bar, or prestressing steel corrosion protection plan for all reinforced or prestressed concrete elements; • Concrete element protection plan; • Fatigue and fracture resistance plan for steel superstructure; • Corrosion protection plan for steel superstructure; • Design provisions and details for potential bearing replacement; and • Mitigation plan for applicable extreme event hazards.
2. Maintenance Issues	Identify recommended future maintenance requirements for achieving design service life related to deck, superstructure, and substructure components. Examples include: • Deck maintenance plan, • Drainage system maintenance plan

	• Steel superstructure coating maintenance plan, • Expansion joint maintenance plan, • Bearing maintenance plan and • Substructure maintenance plan.

2.5.5 Existing Bridges

Many of the service life considerations for new design are also applicable to existing bridges. An inherent difference for existing bridges however, is that the bridge system has already been selected, designed, and constructed, and may have been in service for a considerable time. This means that it will have been subjected to factors affecting service life and may have already experienced some level of deterioration. The level of deterioration will dictate the type of rehabilitation required for restoration. The process for restoring and extending the service life of an existing bridge involves a detailed system evaluation rather than a system selection. The process for existing bridges will follow a similar flow chart to that shown in Section 2.5.2, except that it will start at Block 3 and will entail an evaluation of the existing system components, subsystems, and elements against the various factors in the fault tree branches. Strategies for mitigating the factors are more limited and are more focused on protecting the existing elements and materials. However, they could also consider various forms of retrofit, for example, converting simple spans with deck joints to continuous spans without deck joints. The final evaluation is to determine the optimal protection strategy that will extend the existing bridge service life.

2.6 BRIDGE MANAGEMENT

A major component of the *Guide* and this chapter is address the type of data that should be maintained for a bridge from design through fabrication, construction, and operation. The framework for this documentation is the introduction of the Owner's Manual described in Chapter 1. A bridge Owner's Manual should be provided for unique bridges or when requested by bridge owner. The information to be included in the Owner's Manual is essential for proper future inspection and maintenance of the bridge in order to achieve the bridge's target design service life. The Owner's Manual, should be provided to bridge owner just before opening the bridge to traffic. The bridge Owner's Manual should be treated similar to a design calculation document that is usually provided to the bridge owner and it

is recommended that the Owner's Manual be reviewed by an independent engineer. Chapter 1 provides a more detailed description of bridge Owner's Manual.

Engineering judgment must be exercised in identifying the type of information to be included in the bridge Owner's Manual. Following is partial list of information that could be included in the bridge Owner's Manual.

- Target design service life of the bridge as determined by owner;
- Overall process and philosophies used to address the service life design;
- List of all assumptions, special data, etc. used in service life design process;
- All factors affecting service life that were identified in the initial and final service life design with adequate justification;
- All strategies that were designed into the bridge to avoid or mitigate factors affecting service life;
- Procedure used to estimate the service life of bridge element, component, and subsystem;
- Description of any special steps or requirements that must be followed during construction
- Specific maintenance needs for various bridge element, components, subsystems, and elements in order to achieve their expected service lives;
- All considerations that were incorporated into the overall bridge system design to accommodate rehabilitation or replacement of those items, including expected schedule;
- Identification of all "hot spot" areas of the bridge that would require special inspection or data to be collected during inspection that could be coordinated with FHWA-sponsored Long-Term Bridge Performance Program (LTBPP); and
- Health monitoring of unique bridges to develop a comprehensive bridge management system might be needed and should be described in detail.

The bridge Owner's Manual should also describe how the bridge was designed, constructed, and intended to function from an operational perspective including:

- Design loads, particularly any special vehicle types;
- Expected superstructure deflections;
- Expected movements at expansion joints and bearings;
- Relevant as-built data; and
- Construction methods and procedures.

In summary, the bridge Owner's Manual should provide a clear picture of procedure used to address service life design, and what will be needed to keep the bridge operational for the intended service life.

CHAPTER 3

MATERIALS

3.1 INTRODUCTION

This chapter provides essential information on materials used for constructing durable bridge structures. It focuses on durability and service life issues related mainly to concrete and steel, materials widely used in bridge construction, and provides limited information on other material types used in bridge construction.

Information for enhancing the service life of materials used in bridge systems, subsystems, and components is summarized. Figure 3.1 identifies the materials-enhancement selection process used, and this chapter follows the structure of that process which begins with developing an understanding of the types of materials. These viable materials are further evaluated for the factors that adversely affect their service life. Individual strategies are then developed to mitigate these adverse effects. The overall strategy selection is then developed, blending these individual strategies that are sometimes in conflict with each other. The components of an overall strategy should:

- Identify appropriate design methodologies;
- Select durable material types considering life cycle costs;
- Consider additional protective measures, such as cathodic protection and electrochemical chloride extraction;
- Specify best practices for construction; and
- Develop an effective maintenance plan.

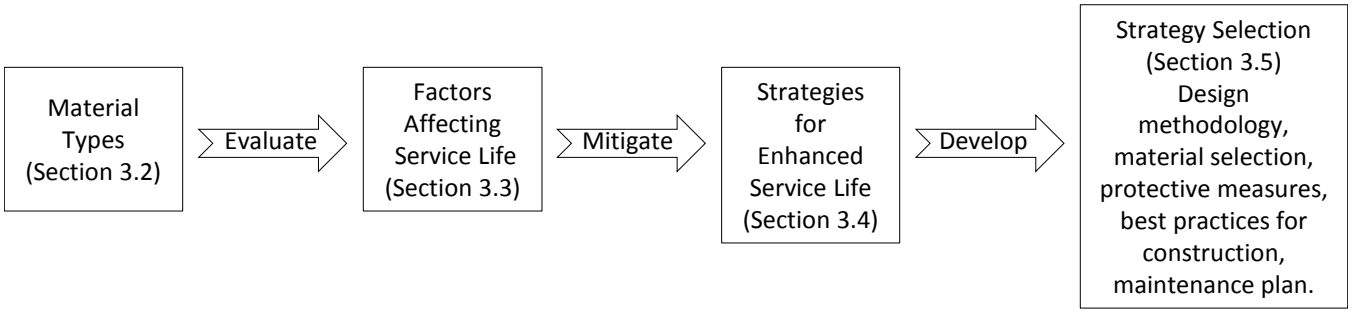


Figure 3.1. Materials enhancement selection process.

Section 3.2 provides a general description of material types used for bridge systems, subsystems, and components, primarily concrete, reinforcement, and structural steel. Section 3.3 addresses factors known to adversely affect service life of these materials and Section 3.4 presents a fault tree approach for considering these factors. Section 3.5 provides available solutions, methods, technologies, and other information helpful in developing individual strategies to mitigate factors adversely affecting service life. Section 3.6 identifies a process to select the overall material selection and protection strategy for providing materials with enhanced service life; however, much of this selection process is highly dependent on the application and is addressed in subsequent chapters.

3.2 DESCRIPTION OF MATERIAL TYPES

This section provides a general description of the materials used for bridge systems, subsystems, and components, primarily concrete, reinforcement and structural steel.

Different concretes for bridge elements require different degrees of durability depending on the exposure environment and the properties desired. There are many examples of longevity of concrete from ancient times, for example the Pantheon in Rome, which was built around 126 A.D. and remains intact. The Confederation Bridge, which joins the eastern Canadian provinces of Prince Edward Island and New Brunswick, was constructed in 1997 and is a good example of a modern concrete bridge designed to resist the harsh marine environment for at least 100 years (Figure 3.2).

Figure 3.2. Confederation Bridge.

Concrete has high compressive, but low tensile strength, which makes it prone to cracking. Early concrete structures were designed to be subjected only to compression, thereby avoiding tensile failures. Arch shapes were used to span distances. In modern structures, reinforcement is common for providing tensile capacity, crack control,

and ductility. The service life of reinforced concrete structures depends on the durability of concrete and the durability of the reinforcement.

The major distress in reinforced concrete is due to the corrosion of the reinforcement. Reinforcement must be protected from aggressive environments either through measures such as the use of the low permeability concrete, adequate cover, and corrosion inhibitors, or the use of noncorrosive reinforcement. Preventive measures could also be employed to prolong the service life of concrete structures, such as incorporating cathodic protection systems.

Structural steel is another common material used for bridge systems, subsystems, and components. It provides high compressive and tensile strengths with considerable ductility, which makes it particularly suitable for long-span bridges. The most common steel bridge systems used today are composite multi-girder deck systems which use either rolled beams, plate girders, or tub girders. These systems can be single- or multi-span, and either straight or curved. Simple-span systems were often used in the past, but most multi-span systems today are continuous. Rolled beam bridges using W-shapes are used in shorter spans up to about 100 ft for simple-spans and up to about 120 ft for continuous spans. Welded deck plate girders are most often used for spans over 120 ft.

3.2.1 Concrete

Concrete consists of cementitious material, aggregate, water, and admixtures. Concrete may also include fibers. The following sections provide general descriptions of each concrete ingredient.

3.2.1.1 Cementitious Material

Cementitious materials include portland cements, blended cements, other hydraulic cements, specialty cements for repairs, and supplementary cementitious materials (SCM). These cementitious materials have different chemical and physical properties that affect the durability of concrete.

3.2.1.1.1 Portland Cement

Portland cement is produced from a combination of calcium, silica, aluminum, and iron (Kosmatka and Wilson 2011). The raw materials reach temperatures of 2600ºF to 3000ºF, forming clinker. Clinker and gypsum are ground to a fine powder such that nearly all of it passes a No. 200 mesh (75micron sieve). Cement has four main compounds: tricalcium silicate ($3CaO.SiO_2=C_3S$), dicalcium silicate ($2CaO.SiO_2=C_2S$), tricalcium aluminate ($3CaO.Al_2O_3=C_3A$), and tetracalcium aluminaferrite ($4CaO.Al_2O_3.\ Fe_2O_3=C_4AF$). These form the different types of

portland cements conforming to the requirements of ASTM C150. Ten types of portland cement are summarized in Table 3.1.

Table 3.1. Cement Types and Uses.

Type of Cement	Use
Type I	General purpose
Type IA	Type I + air entraining
Type II	General use, moderate sulfate resistance
Type IIA	Type II + air entraining
Type II(MH)	General use, moderate heat of hydration, moderate sulfate resistance
Type II(MH)A	Type II(MH) + air entraining
Type III	High early strength
Type IIIA	Type III + air entraining
Type IV	Low heat of hydration
Type V	High sulfate resistance

Some cements are designated with a combined classification, such as Type I/II indicating that cement meets all of the requirements of the specified types.

3.2.1.1.2 Blended Cement

Blended cements are produced by intergrinding or blending two or more types of fine material such as portland cement, ground granulated blast furnace slag, fly ash, silica fume, calcined clay, other pozzolans, hydrated lime, and pre-blended combinations of these materials (Kosmatka and Wilson 2011). ASTM C595 includes three classes of blended cement:

- Type IS (X)—Portland blast furnace slag cement
- Type IP (X)—Portland-pozzolan cement
- Type IT(AX)(BY)—Ternary blended cement

The letters X and Y stand for the percentage of SCM included in the blended cement, and A and B are the Types of SCMs, either S for slag or P for pozzolan. Type IS(X) can include up to 95% slag cement. Type IP(X) can include up to 40% pozzolans. Special properties are designated as follows, and added after the percentages. For example: IP(25)(HS) indicates 25% pozzolans and high sulfate resistance.

- Air entrainment—(A)
- Moderate sulfate resistance—(MS)
- High sulfate resistance—(HS)
- Moderate heat of hydration—(MH)
- Low heat of hydration—(LH)

As an example for ternary cements, Type IT(S25)(P15) contains 25% slag and 15% pozzolans. Type IT can meet MS, HS, and LH options.

3.2.1.1.3 Other Hydraulic Cement

All portland and blended cements are hydraulic cements. ASTM C1157, is a performance specification that includes portland cement, modified portland cement (specialty cement providing characteristics of more than one type of portland cement), and blended cements. ASTM C1157 recognizes six types of hydraulic cements:

- Type GU—general use;
- Type HE—high early strength;
- Type MS—moderate sulfate resistance;
- Type HS—high sulfate resistance;
- Type MH—moderate heat of hydration; and
- Type LH—low heat of hydration.

3.2.1.1.4 Expansive Cements

Expansive cements are modified hydraulic cements that expand slightly during the early hardening period after setting. They are used to compensate for volume decrease due to drying shrinkage, to induce tensile stress in reinforcement, and to stabilize long-term dimensions of post-tensioned concrete. ASTM C845 designates E-1 with

three varieties—K, M, and S—and only K is available in the United States. Type E-1 (K) contains portland cement, anhydrous tetracalcium trialuminosulfate, calcium sulfate, and uncombined calcium oxide (lime).

It has been demonstrated that using lightweight aggregate in concrete with expansive cements is very beneficial for achieving the expansion from expansive cements (Russell 1978). This effect is attributed to the continued presence of internal moisture that has been absorbed by the lightweight aggregate prior to batching. This internal moisture allows a prolonged reaction time for the expansive cement, so it achieves greater expansion. Therefore, it is suggested that lightweight aggregate be used in conjunction with expansive cements.

3.2.1.1.5 Specialty Cements for Repairs

Specialty cements for repairs are needed to achieve high early strengths, and cements other than the portland cements may be used. They may be rapid-setting hydraulic cement, gypsum-based cement, magnesium phosphate cement, or high-alumina cement for use in partial depth concrete repairs (Caltran 2008; ACPA 1998).

Gypsum-based cement mixtures contain calcium sulfates which accelerate strength gain and may be used in temperatures above freezing up to 110°F. They are not recommended for placement in rainy and freezing weather (NRC 1977), and may promote steel corrosion in reinforced pavements (Good-Mojab et al. 1993).

Magnesium phosphate cement mixtures are characterized by high early strength, low permeability, and good bonding to clean dry surfaces. However, this concrete is extremely sensitive to water content and aggregate type, especially limestone. Significant strength reduction can be obtained with very small amounts of excess water (Good-Mojab et al. 1993).

High alumina cement mixtures produce a rapid strength gain concrete with good bonding properties (to dry surfaces) and very low shrinkage. However, significant strength loss is expected due to chemical conversions in the calcium aluminate cement during curing (ACPA 1998).

Other rapid setting materials are also available that may perform adequately (ACPA 1998). However, some rapid-hardening repair materials are affected by high alkaline-bearing materials. These materials may react with certain siliceous aggregates to form alkali-silica reactivity (ASR). Therefore, it is important to make sure that no chemical incompatibilities exist between the patch material and the aggregates used in the mixture.

3.2.1.1.6 Supplementary Cementitious Material (SCM)

Supplementary cementitious material generally consists of byproducts from other processes or natural materials. They exhibit hydraulic or pozzolanic activity and contribute to the properties of concrete. Pozzolanic materials do not possess cementitious properties, but when used with portland cement they form cementitious compounds. SCMs modify the microstructure of concrete and reduce its permeability. They can reduce internal expansion due to chemical reactions. They can also reduce heat of hydration that can cause thermal cracking. Typical examples are natural pozzolans, fly ash, slag cement, and silica fume. They are used individually with portland or blended cements or in different combinations. Fly ash is a finely divided residue that results from the combustion of ground or powdered coal and is transported by flue gases. It is a by-product of power generating stations. Natural pozzolans found in nature may be calcined to induce satisfactory properties. Fly ash and natural pozzolans are covered by ASTM C618. Slag cement is a by-product of iron blast furnaces and consists essentially of silicates and alumina silicates of calcium and other bases. Slag cement conforms to ASTM C989. It is classified by performance in the slag activity test into three grades: Grade 80, Grade 100, and Grade 120. Silica fume is a finely divided residue resulting from the production of elemental silicon or ferro-silicon alloys that is carried from the furnace by the exhaust gases. It conforms to ASTM C1240. ASTM C1697 covers blended supplementary cementitious materials that result from the blending or intergrinding of two or three ASTM-compliant supplementary cementitious materials. SCMs are summarized in Table 3.2. With regard to durability, SCMs reduce permeability and improve chemical resistance; however, the level of durability improvement depends on the type, chemical and physical characteristics, and amount used. For example, concrete with Class F fly ash is expected to have better sulfate resistance then the one with Class C fly ash.

In some areas, a superfine (ultrafine) grade of fly ash is available which exhibits characteristics midway between normal fly ash and silica fume in cost, effectiveness, and desirable dose rate. It does not require large-volume batching facilities as normal fly ash, and is not as difficult a material to handle and disperse effectively as silica fume (Day 2006).

Superfine fly ash is generally processed from a Class F fly ash by passing the parent ash through a classifier in which the coarse and fine particles are separated. The average particle size of raw or the unprocessed fly ash is around 20 to 30 microns and the largest size about 100 microns; however, an ultrafine fly ash has a maximum size

less than 10 microns and an average particle size of 2 to 4 microns. The finer size provides additional reactive surface area contributing to the high early strength and low permeability concrete. Similar early strengths and durability measures as silica fume concrete were observed when a slightly higher dosage of ultrafine Class F fly ash was used with a small reduction (10%) in water content (Obla et al. 2003).

Table 3.2. SCMs, ASTM Standard Specifications, and Use.

SCM	ASTM Std.	Use
Fly ash, Class F	C 618	Low permeability, chemical resistance
Fly ash, Class C	C 618	Low permeability, chemical resistance
Fly ash, Class N	C 618	Low permeability, chemical resistance
Slag cement	C 989	Low permeability, chemical resistance
Silica fume	C 1240	Low permeability, chemical resistance
Blended SCMs	C 1697	Low permeability, chemical resistance

3.2.1.2 Aggregates

Aggregates are granular material, such as sand, gravel, crushed stone, or iron blast furnace slag, used in a cementing medium to form hydraulic cement concrete. Aggregates are divided into two categories: coarse and fine. Coarse aggregates are those predominantly retained on the No. 4 sieve. Fine aggregate are which pass the No. 4 sieve and are predominantly retained on the No. 200 sieve. However, the combination of coarse and fine aggregates is important in concrete. The combined grading indicates the amount of paste needed to affect the water demand and cement content. In comparison to aggregates, paste is more porous, enabling easier transport of water and solutions, which can be detrimental to durability. Normal-density fine and coarse aggregates meet the requirements of ASTM C33. Structural low-density aggregates conform to the requirements of ASTM C330. The aggregate characteristics that affect the properties of concrete are the grading, durability, particle shape and surface texture, abrasion and skid resistance, density, absorption, and surface moisture. Angular, elongated, and rough textures aggregates have a high water demand that can lead to a high water-cementitious materials ratio (w/cm). The absorption and porosity of the aggregate may affect the resistance to freezing and thawing. Aggregates should be free of potentially deleterious materials such as clay lumps, shales, or other friable particles that can affect the water demand and bond. The chemical composition of the aggregates is important due to the possibility of expansive reactions.

3.2.1.2.1 Normal Weight Aggregates (NWA)

Most commonly used normal weight aggregates—sand, gravel, and crushed stone—produce concrete with a density of 140- to 150-lb/ft^3. Aggregates must be clean, hard, strong, durable particles, free of absorbed chemicals, coatings, and other fine material that can adversely affect hydration and bond of the cement paste (Kosmatka and Wilson 2011). Grading, shape, and texture of aggregates affect the water demand. Concretes with angular and poorly graded aggregates are also difficult to pump. Aggregates may also be reactive causing ASR or alkali-carbonate reactivity (ACR). If reactive aggregates are used, pozzolans and lithium-based admixtures can be added to minimize the expansion and provide resistance to ASR. The space in the aggregate also provides a place for the reaction products, if any expansion occurs due to other aggregates in the concrete. In ACR, diluting the aggregates or changing the source, or selective quarrying, could minimize the expansion. In selective quarrying, the layers that contain reactive aggregate are avoided (Ozol 2006). Reactivity is measured using ASTM C1105.

3.2.1.2.2 Lightweight Aggregates (LWA)

In the United States, lightweight aggregate is typically manufactured by expanding shale, clay, or slate by firing the materials at high temperatures in a rotary kiln. At high temperatures, gases are evolved within the pyroplastic mass, forming bubbles that remain after cooling (ACI 213R, 2003). The cellular structure of LWA results in a density that is lower than normal weight aggregate. When used in concrete, LWA reduces the density of concrete.

The porous structure of LWA absorbs more water than the normal weight aggregates. Pores close to the surface are readily permeable, but interior pores are hard to fill. Many of the interior pores are not connected to the surface at all. Water absorbed in the surface pores by pre-wetting prior to batching is released into the paste during hydration of the cement, which provides internal curing (Holm and Ries 2006) which in turn results in improved properties and reduced cracking within the concrete.

Lightweight aggregate used in structural lightweight concrete (LWC) must be capable of producing concrete with a minimum 28-day compressive strength of 2,500 psi with an equilibrium density between 70- and 120-lb/ft^3 (ACI 213R, 2003). The strength of LWA varies with type and source and will affect the strength of lightweight concrete that can be achieved.

The density of LWA varies with particle size, increasing in density for the smaller particles. Due to this density variation, the grading requirements for LWA (ASTM C330) deviate from those of NWA (ASTM C33) by requiring a

larger mass of the lightweight aggregates to pass through the finer sieve sizes. This modification yields the same volumetric distribution of aggregates retained on a series of sieves for both LWA and NWA.

3.2.1.3 Water

ASTM C1602 covers mixing water used in the production of hydraulic cement concrete. Mixing water consists of batch water, ice, water added by truck operator, free moisture on the aggregates, and water introduced in the form of admixtures. Potable and non-potable water is permitted to be used as mixing water in concrete. Non-potable water, including treated wash water and slurry water, is not used in concrete unless it produces 28-day concrete strengths equal to at least 90% of a control mixture using 100% potable water or distilled water, and time of set meets the limits in C1602. Some excessive impurities may cause durability problems; therefore, optional chemical limits for combined mixing water are given for chloride, sulfate, alkalis, and total solids. Small solid particles increase the water demand due to large surface area.

3.2.1.4 Admixtures

Chemical admixtures are the ingredients in concrete other than portland cement, water, and aggregate, that are added to the mix immediately before or during mixing.

Admixtures are primarily used to achieve following objectives (Kosmatka and Wilson 2011):

- To reduce cost of concrete construction;
- To achieve the properties of hardened concrete effectively;
- To maintain the quality of concrete during mixing, transporting, placing, and curing in adverse weather conditions; and
- To overcome certain emergencies during concrete operations.

Chemical admixtures must conform to the requirements of ASTM C494, or C1017 when flowing concrete is applicable. ASTM494 covers the materials and the test methods for use in chemical admixtures to be added to hydraulic-cement concrete mixtures in the field. The admixtures given in ASTM C494 and C1017 are summarized in Table 3.3. Words superplasticizer and high-range water-reducing admixture are used interchangeably. There are other admixtures such as shrinkage-reducing or viscosity-modifying admixtures, which are covered by Type S,

specific performance admixture, in ASTM C494. Corrosion inhibiting admixtures are covered by ASTM C1582 and the cold weather admixtures by ASTM C1622.

Admixtures help in achieving workable, low w/cm, and low permeability concretes. Entrained air voids provide resistance to freezing and thawing, improved workability, and reduced bleeding and segregation. Shrinkage-reducing admixtures reduce shrinkage and cracking potential. Viscosity-modifying admixtures improve stability of the mixture minimizing segregation. Corrosion-inhibiting admixtures increase the resistance to corrosion. They can form a protective layer at the steel surface, as a result of chemical reaction with ferrous ions as with inhibitors containing calcium nitrite; or they can provide a protective layer and reduce chloride ion ingress, as with inhibitors containing amine/ester. ASTM specifications are available to provide guidance in using admixtures in concrete, as summarized in Table 3.3.

Table 3.3. Admixtures, ASTM Standard Specifications and Use.

Type	ASTM	Use
A	C 494	Water-reducing
B	C 494	Retarding
C	C 494	Accelerating
D	C 494	Water-reducing and retarding
E	C 494	Water-reducing and accelerating
F	C 494	High-range water-reducing
G	C 494	High-range water-reducing and retarding
S	C 494	Specific performance admixtures
I	C 1017	Plasticizing
II	C 1017	Plasticizing and retarding

3.2.1.5 Fibers

Fibers are added to concrete to control cracking. Steel fibers conform to ASTM A820, while synthetic fibers meet the requirements of ASTM C1116, 4.1.3, Type III. Fibers, generally synthetic, are added to concrete at low volume dosage, about 0.1%, to reduce plastic-shrinkage cracking. At high volume percentages, up to 2%, they can increase resistance to cracking in hardened concrete and decrease crack width (Kosmatka and Wilson 2011). Plastic shrinkage cracks are addressed in bridge structures by proper curing. Crack control in hardened concrete has been

attempted in decks and overlays (Ozyildirim 2005; Sprinkel and Ozyildirim 2000; Baun 1993). The cost and handling of fibers have limited their use in bridge structures.

One other advantage of synthetic fibers is in the reduction of spalling during fires. This has been a concern, especially in tunnels (Parsons 2006). In a fire, spalling of concrete can occur due to high vapor pressure. Spalling becomes more severe with an increase in strength. Synthetic fibers such as polypropylene fibers reduce the risk of spalling. At high temperatures these fibers melt leaving pores or channels in the concrete for the vapor to escape.

3.2.1.6 Concrete Types

Types of concrete used in bridge structures include normal weight concrete; high performance, concrete including self consolidating concrete; ultra-high performance concrete; fiber-reinforced concrete; and lightweight concrete. In repairs, overlay concretes (latex modified concrete, low-slump, silica fume, polymer concrete, and very early strength concretes), shotcrete, and grouts have been used. The following sections describe different types of concretes and provide information on their characteristics that can be considered in selecting the proper type of concrete for a given application.

3.2.1.6.1 Normal Weight Concrete (NWC)

Normal weight concretes have a wide range of ingredients and performance characteristics including air content, slump, and temperature. The density (unit weight) of NWC is approximately 145 lbs/ft^3. The density of concrete varies, depending on the amount and density of the aggregate, the air, water, and cement contents. Generally, strength is the selected parameter.

Concrete ingredients, proportions, handling, placing, curing practices, and the service environment determine the ultimate durability and life of concrete (Ozyildirim 2007).

The durability of concrete depends largely on its ability to resist the infiltration of water and aggressive solutions. Concretes that are protected from the environment can provide a long service life. For longevity, concretes with low permeability are needed. Concrete may deteriorate when exposed to cycles of freezing and thawing, especially in the presence of deicing chemicals, and become critically saturated. The addition of an air-entraining admixture to critically saturated concrete can improve the resistance to freezing and thawing.

Concrete can resist most natural environments and many chemicals. However, certain chemicals can attack concrete and cause deterioration. For example, sulfate attack, ASR, ACR, acid attack, corrosion, and wear can damage concrete and reduce its service life. However, in such environments proper material selection, proportioning, and construction practices can protect concrete from the unwanted attack and distress. For example, if reactive aggregates are used, pozzolans and lithium based admixtures can be added to minimize the expansion due to ASR.

3.2.1.6.2 High Performance Concrete (HPC)

High performance concretes exhibit high workability, strength, and/or durability. FHWA proposed to define high performance concrete using long-term performance criteria (Goodspeed et al. 1996). The goal was to stimulate the use of higher quality concrete in highway structures. HPC has been used in decks, superstructures, and substructures to extend the service life. The definition of HPC developed for the FHWA (Goodspeed et al. 1996) had four performance parameters related to durability as shown in Table 3.4:

- Resistance to freezing and thawing,
- Resistance to scaling,
- Resistance to abrasion, and
- Resistance to chloride ion penetration.

The four structural design characteristics were compressive strength, modulus of elasticity, shrinkage, and creep. The tensile strength which is related to compressive strength, modulus of elasticity, shrinkage, and creep is an important factor affecting the cracking potential. For each characteristic, standard laboratory tests, specimen preparation procedures, and grades of performance were presented. Later additions to this definition and changes to the grade limits were recommended (Russell and Ozyildirim 2006) and are summarized in Table 3.5. These included ASR- and sulfate-resistance. Workability was also added as a characteristic and would affect durability since concrete should be well consolidated (either through vibration or self consolidation) in order to achieve the desired hardened concrete properties. Another characteristic to be considered is density. Concretes with varying densities varying from light weight to normal weight can be prepared to address the dead load in spanning long distances.

Table 3.4. Grades of Performance Characteristics for High Performance Structural Concrete. (Goodspeed et al. 1996)

Performance Characteristic	Standard Test Method	FHWA HPC Performance Grade			
		1	2	3	4
Freeze/Thaw Durability (x = relative dynamic modulus of elasticity after 300 cycles)	AASHTO T 161 ASTM C 666 Proc. A	$60\% \leq x \leq 80\%$	$80\% \leq x$		
Scaling Resistance (x = visual rating of the surface after 50 cycles)	ASTM C 672	x = 4,5	x = 2,3	x = 0,1	
Abrasion Resistance (x = avg. depth of wear in mm)	ASTM C 944	$2.0 > x \geq 1.0$	$1.0 > x \geq 0.5$	$0.5 > x$	
Chloride Permeability (x = coulombs)	AASHTO T 277 ASTM C 1202	$3000 \geq x > 2000$	$2000 \geq x > 800$	$800 \geq x$	
Strength (x = compressive strength)	AASHTO T 22 ASTM C39	$41 \leq x < 55$ MPa ($6 \leq x < 8$ ksi)	$55 \leq x < 69$ MPa ($8 \leq x < 10$ ksi)	$69 \leq x < 97$ MPa ($10 \leq x < 14$ ksi)	$x \geq 97$ MPa ($x \geq 14$ ksi)
Elasticity (x = modulus of elasticity)	ASTM C 469	$24 \leq x < 40$ GPa ($4 \leq x < 6 \times 10^6$ psi)	$40 \leq x$ 50 GPa ($6 \leq x < 7.5 \times 10^6$ psi)	$x \geq 50$ GPA ($x \geq 7.5 \times 10^6$ psi)	
Shrinkage (x = microstrain)	ASTM C 157	$800 > x \geq 600$	$600 > x \geq 400$	$400 > x$	
Creep (x = microstrain/pressure unit)	ASTM C 512	$0.52 \geq x > 0.41$	$0.41 \geq x > 0.31$	$0.31 \geq x > 0.21$	$0.21 \geq x$

http://www.fhwa.dot.gov/bridge/HPCdef.htm

Table 3.5. Additional Grades of Performance Characteristics. (Russell and Ozyildirim 2006)

Performance Characteristic	Standard Test Method	FHWA HPC Performance Grade		
		1	2	3
Alkali-silica reactivity (ASR = expansion at 56 days, %)	ASTM C 441	$0.20 \geq ASR > 0.15$	$0.15 \geq ASR > 0.10$	$0.10 \geq ASR$
Sulfate resistance (SR = expansion, %)	ASTM C 1012	$SR \leq 0.10$ at 6 months	$SR \leq 0.10$ at 12 months	$SR \leq 0.10$ at 18 months
Workability (SL = slump, SF = slump flow)	AASHTO T 119 ASTM C 143 ASTM C 1611	$SL >$ 7-1/2 in. $SF <$ 20 in.	$20 \leq SF \leq 24$ in.	24 in. $< SF$

3.2.1.6.3 Self-Consolidating Concrete (SCC)

Self-consolidating concrete is a highly-flowable, non-segregating concrete that can spread into place, fill the formwork, and encapsulate the reinforcement without any mechanical consolidation (ACI 237R, 2007). SCC contains a large amount of fine material to obtain a stable mixture; sometimes a viscosity modifying admixture is used to provide the stability instead of, or in combination with, the fine material. The high range water reducing admixtures (HRWRA) used are generally based on polycarboxylate ethers; they provide large water reductions and slow slump loss. Eliminating the consolidation problem would enhance the strength and reduce the permeability of concretes, essential characteristics for longevity. SCC has been used in Japan since the late 1980s (ACI 237R, 2007) and is used widely in the precast industry in North America, while its use in the ready mixed concrete industry has been slower. Some of the benefits of SCC are decreased labor requirements, increased construction speed, improved mechanical properties and durability characteristics, ability to be used in heavily reinforced and congested areas, consolidation without vibration, and a reduced noise level at manufacturing plants and construction sites (Okamura and Ouchi 1999). The flow characteristic of SCC is measured using the slump flow test (ASTM C1611). The stability of the mixture can be qualitatively assessed in accordance with ASTM C1611 using the visual stability index (VSI). A VSI value of zero indicates a highly stable mix with no evidence of segregation or bleeding; 1 is a stable mix with no evidence of segregation but with slight bleeding; VSI 2 and 3 indicate unstable mixtures. To determine the ability of SCC to pass through the reinforcement, a J-Ring test is used (ASTM C1621). ASTM C1610 addresses the determination of static segregation of SCC by measuring the coarse aggregate content in the top and bottom portions of a column. SCC is placed in a column separated into three sections. The coarse aggregate in the top and bottom sections is removed by washing. Another ASTM test method, C1712, covers the rapid assessment of static segregation resistance of normal-weight SCC. This test does not measure static segregation resistance directly, but rather provides an assessment of whether static segregation is likely to occur.

There are some concerns about the use of SCC, among them segregation due to a high flow rate, a poor air void system due to the high fluidity, and high amounts of HRWRA resulting in coarse air bubbles, shrinkage due to smaller maximum size aggregate and lower amount of coarse aggregate, and form pressure and tightness due to high fluidity (ACI 237R, 2007). SCC has been used successfully in beams (Ozyildirim 2008), drilled shafts (Schindler and Brown 2006), substructure repairs, and tunnel sections (ACI 237R, 2007).

3.2.1.6.4 Ultra-High Performance Concrete (UHPC)

Ultra-high performance concrete is a type of concrete that has high strength and high ductility. A particular proprietary UHPC that is commonly available is discussed in this section and is formulated by combining portland cement, silica fume, quartz flour, fine silica sand, high-range water reducer, water, and steel or organic fibers. Its superior durability and negligible permeability is expected to reduce maintenance and extend the service life. Current applications (2012) include beams and connections as in the explanation that follows.

UHPC is expected to achieve compressive strengths greater than 21.7 ksi (150 MPa) and contains fiber reinforcement for improved ductile behavior (AFGC 2002). Small brass-coated steel fibers, with a diameter of 0.007 in (0.185 mm) and a length of 0.55 in (14 mm), are commonly used as fiber reinforcement in UHPC. Synthetic fiber, poly-vinyl alcohol (PVA), has also been used (Parsekian et al. 2008). To achieve very high strengths, exceeding 30 ksi, UHPC is steam-cured. UHPC exhibits strain-hardening that results in numerous tight cracks rather than one large crack prior to failure. It has a very low w/cm (about 0.2) and very dense matrix leading to negligible permeability. The high amount of binder and very low w/cm in UHPC make it very cohesive; however, it flows within the forms without the need for vibration. The high strength and low permeability of UHPC are attributed to a very dense packing of fine material and a low w/cm (Graybeal 2006). UHPC contains no coarse aggregate. Fine sand, typically between 0.006 and 0.024 in (150 and 600 micrometers) is the largest particle size, followed by crushed quartz, cement, and silica fume. The resulting gradation allows for tight packing of these particles. The unit weight of UHPC is approximately 155 lb/ft^3. The coefficient of thermal expansion is about 50% higher than conventional concrete (Graybeal 2006).

Whether steam cured or not, commonly available UHPC exhibits enhanced durability compared to normal weight concrete (Graybeal 2006; Graybeal and Tanesi 2007). UHPC characteristics have been studied under different curing conditions in a study by Graybeal and Tanesi 2007. Four curing regimes were examined in this study: steaming at 60°C (194°F) and 95% relative humidity for 48 h (recommended by the manufacturer), untreated (no steam curing, specimens kept in the standard laboratory environment from demolding until testing), tempered steam treated (temperature in steam chamber limited to 60°C (140°F)), and delayed steam treated (steam treatment initiated after 15 days of casting). Regardless of the curing treatment applied, UHPC exhibited enhanced durability

properties over normal concretes. Thus, field casting and curing can provide the desired properties without the need for steam curing. Steam curing can improve the properties even more, but are relevant to precast operations.

In terms of freeze/thaw resistance, the relative dynamic modulus of UHPC was at least 96% after being subjected to 690 cycles of freezing and thawing (more than two times the normal number of 300 cycles indicated in ASTM C666). The concrete was innocuous to ASTM C1260 ASR deterioration, to ASTM C672 scaling deterioration, and to AASHTO T259 chloride penetration. The ASTM C1202 Coulomb test result was negligible, less than 40 coulombs, if any steam-based curing treatment was applied, and ranged from very low (averaging 360 coulombs) at 28 days, to negligible (averaging 76 coulombs) at 56 days in the absence of any steam curing. The ASTM C1202 test is based on electrical conductance of concrete, and the presence of steel fibers affects the charge passed; however, the very dense matrix of UHPC isolates the steel fibers and provides very high resistance to current flow. Ponding test results (AASHTO T259) have shown that the volume of chlorides that penetrated is extremely low. No scaling was observed when tested in accordance with ASTM C672. The abrasion resistance (ASTM C944) of steam treated UHPC was very low; it abraded 0.1 to 0.3 g. The untreated UHPC lost 10 times more (1 to 3 g) in the abrasion test. Although air-entraining admixture is not added during casting, UHPC exhibits satisfactory resistance to cycles of freezing and thawing.

The first bridge with UHPC beams was constructed in Wapello County, Iowa (Moore and Bierwagen 2006). The three 110-ft beams were modified 45-in. Iowa bulb-tee beams. To save material in the beam section, the web width was reduced by 2 in., the top flange by 1 in., and the bottom flange by 2 in. Virginia Department of Transportation (DOT) used five 45-in.-tall bulb-tee beams with UHPC in the bridge on Route 624 over Cat Point Creek (Ozyildirim 2011). The bridge has ten 81.5-ft spans. One of the spans contained UHPC beams. The steel fibers provided adequate shear resistance, so the UHPC beams did not contain the conventional stirrups normally used as shear reinforcement; however, confinement steel was included at the beam ends (Ozyildirim 2011). Other applications in bridge structures have been accomplished or recommended. Garcia (2007) detailed UHPC flexural behavior and offered a design methodology for two-way ribbed, precast bridge decks. Under the FHWA Highways for LIFE Program, a two-lane bridge on a secondary road in Wapello County, Iowa, was constructed using prestressed concrete girders and 14 UHPC waffle deck bridge panels (Heimann and Schuler 2010). UHPC has very high bond strength. At the Virginia project, the extra UHPC flowing to the top of the steel form had bonded strongly to the form after hardening, making

the removal of the form very difficult (Ozyildirim 2011). The high bond strength, low permeability, and crack resistance make UHPC highly desirable in joints and connections and such applications have been successfully completed (Perry and Royce 2010).

3.2.1.6.5 Fiber Reinforced Concrete (FRC)

Fiber reinforced concrete is expected to improve tensile strength; provide crack control; and increase durability, fatigue life, resistance to impact and abrasion, shrinkage, and fire resistance (ACI 544.1R, 1996). Crack control is critical for longevity in reinforced concrete structures, and one effective solution is to use fiber reinforced concrete. Fiber reinforced concretes containing steel and synthetic fibers are common. Two or more fiber types can also be used to produce hybrid fiber reinforced concrete to obtain improved properties or cost reduction. For example, steel and/or macro-synthetic fibers that enhance toughness and post-crack load-carrying capacity can be combined with micro-fibers that help control plastic shrinkage cracking.

Aspect ratio of fibers, which is the ratio of length to diameter, affects the workability and the hardened concrete properties. Typical aspect ratios range from about 20 to 100 (ACI 544.1R, 1996). Fibers with high aspect ratios can improve the ductile behavior of concrete and also increase strength and stiffness. However, fibers with high ratios tend to interlock to form a mat, or ball, which is very difficult to separate by vibration alone. On the other hand, short fibers with ratios less than 50 are not able to interlock and can easily be dispersed by vibration.

FRC can reduce the amount of cracks in concrete; however, wide cracks (>0.004 in. (0.1 mm)) still exist. Cracks less than 0.004 in. (0.1 mm) wide inhibit the intrusion of corrosive chemicals and are expected to hinder the intrusion of harmful solutions (Wang et al. 1997; Lawler et al. 2002). To obtain tight cracks, high performance fiber reinforced concrete (HPFRC) is needed

High performance fiber reinforced concrete undergoes large deflection and exhibits deflection or strain hardening, causing multiple micro-cracks instead of one large localized crack. As deflection occurs strain hardening is exhibited after the first crack is initiated, an increase in stress occurs with further deformation. One such concrete is SIFCON (Slurry Infiltrated Fiber Concrete) (Naaman and Homrich 1989). It is produced by filling an empty mold with loose steel fibers (about 10% by volume) and filling the voids with high strength cement-based slurry. The resulting composite exhibits high strength and ductility. UHPC with steel or PVA fibers and engineered

cementitious composite (ECC) with PVA fibers developed by Dr. Victor Li also exhibit tight cracks (Li 2002; Li 2003). Both the UHPC and ECC are mortar mixtures without the coarse aggregate. HPFRC with hybrid fibers (steel and PVA) containing #8 coarse aggregate with a nominal maximum size of 3/8 in. was developed and exhibits deflection hardening needed for tight crack formation (Blunt and Ostertag 2009).

FRC can be used in bridge decks, deck repairs, and overlays. For example, Ohio DOT has used steel fibers in bridge deck overlays containing silica fume or dense concrete (Baun 1993). HPFRC can be used in joints, connections, and link slabs.

3.2.1.6.6 Lightweight Concrete (LWC)

Lightweight concrete is typically used to reduce the dead load of a structure in order to improve structural efficiency, thus allowing reduced element sizes, less reinforcement, increased span lengths, fewer piers, or reduced foundation elements (ACI 213R, 2003). As prefabrication becomes more widely used for bridge elements, LWC can save money by reducing handling, shipping, and erection costs. It has also been shown that LWC provides enhanced durability by reducing the permeability and cracking tendency of concrete and has a higher fire resistance than conventional concrete.

LWC consists of lightweight aggregate or a blend of lightweight and normal-density aggregate. Standard procedures and admixtures are used to proportion LWC mixtures. Standard batching and transporting equipment are also used for LWC. LWC can have an equilibrium density between 90 and 125 lb/ft^3, but values from 110 to 120 lb/ft^3 are most common. Equilibrium density is defined in ASTM C567 as the density reached after exposure to relative humidity of 50+5% and a temperature of 73.5+3.5°F for a period of time sufficient to reach constant mass. The fresh density is used for quality control in the field, and should also be used to compute precast element weights for handling and shipping.

Lightweight aggregate is sometimes used in combination with normal weight aggregate to create structural concretes with densities between 120 and 145 lb/ft^3. The properties such as strength and modulus of elasticity would vary and should be addressed by performance requirements.

While design compressive strengths of 3,000 to 5,000 psi are common for LWC, design strengths up to 10,000 psi have been used in bridge beams (Liles 2010). The maximum strength that can be achieved using an LWA source

may be increased by reducing the maximum aggregate size. As is typical with NWC, the use of HRWRA enables w/cm reduction, and with the addition of supplementary cementitious materials SCMs, LWC with high workability, strength, and durability can be achieved.

LWA costs more than normal weight aggregate because of the thermal processing used to manufacture it. The impact of the increased cost of LWA on the difference in cost between LWC and NWC depends on the cost of the LWA and the cost of the NWA that it is replacing. As sources of good NWA dwindle, the use of LWA will become more cost effective since NWAs will also be shipped greater distances. In spite of the increased cost of LWC, the benefits that can be achieved using LWC can make it a cost-effective solution for concrete structures.

LWC can be placed and finished using conventional equipment. It exhibits a lower slump than normal weight concrete with the same workability due to the reduced aggregate density. In high workability mixtures, lower density LWA particles may rise to the surface contrary to NWA where aggregates segregate by settling to the bottom. Therefore, LWC mixtures should be designed to be cohesive and excess vibration should be avoided. Slump, air content, and temperature requirements for LWC are similar to those for NWC. LWA should be pre-wetted prior to use in concrete that will be delivered by pumping. Without adequate moisture, the aggregate may absorb mixing water and cause slump loss during pumping. With proper preparation, LWC has been successfully pumped for long horizontal and vertical distances (Valum and Nilsskog 1999). Lightweight aggregate suppliers can provide additional guidance in preparing for pumping LWC.

Experience shows that lightweight concrete with a proper air void system can be durable and exhibit satisfactory performance expected of normal weight concretes (Holm and Ries 2006). Resistance to freezing and thawing of LWC in the presence of deicing salts is similar to that of NWC (ACI 213R, 2003). Since LWC contains more absorbed water than NWC, LWC should be allowed to dry before it is subjected to freezing and thawing. ASTM C330 requires 14 days of drying for the LWC tested in accordance with ASTM C666.

Because of the porous structure of LWA particles, the resistance to wearing forces may be less than that of a solid particle. However, in many cases, LWC bridge decks have exhibited wearing performance similar to that of NWC (ACI 213R, 2003) because the aggregate is a vitrified material with hardness comparable to quartz. LWA is non-polishing, so it has excellent skid resistance.

The hardened properties of LWC are equal to, or somewhat lower than NWC in many cases. The modulus of elasticity of LWC is reduced from NWC. The splitting tensile strength and the modulus of rupture values are lower for LWC, about 60 to 85% of the NWC (ACI 213R, 2003). Poisson's ratio may be assumed as 0.20, which is similar to NWC. The creep and shrinkage of LWC is similar or a little higher than the NWC (Davis 2008). In general, the creep and shrinkage are higher at low strength LWC (i.e. for bridge deck). However, it has been found that conventional methods can be used to estimate prestress losses for LWC bridge girders (Kahn and Lopez 2005). While values for some properties may be lower for LWC than for NWC, designs can usually be adjusted to account for the differences. In some cases, differences may be offset by the reduced dead load in the structure.

The increased modulus of elasticity of LWC results in a high ultimate strain capacity which is beneficial in reducing the cracking tendency of concrete. The low level of micro-cracking observed in LWC provides high resistance to weathering and corrosion.

Alkali-silica reactivity (ASR) is not expected in concretes with lightweight aggregates, as the surface of the aggregate acts as a source of silica. Silica reacts with the alkalis at an early stage to help counteract any potential long-term disruptive expansion. The space in the aggregate also provides a place for the reaction products if any expansion occurs due to other aggregates in the concrete.

LWC is more fire-resistant than NWC due of lower thermal conductivity, lower coefficient of thermal expansion, and the fire stability of the aggregate that is already exposed to high temperatures (over 2000ºF) during processing (ACI 213R, 2003).

3.2.1.6.7 Overlay Concrete

The use of overlays is described in additional detail in the bridge deck section. The following includes a brief dexription of the types of concretes that have been used in overlays.

Latex-modified concrete (LMC) consists of a conventional concrete supplemented by a polymeric latex emulsion (styrene-butadine latex) (Russell 2004). The water in the emulsion contributes to hydrating the cement. It has low permeability and, consequently, good durability, and also has good bonding characteristics. It outperforms conventional and low-slump dense concrete overlays and can be expected to last up to 25 years. Latex-modified

concrete requires special mobile mixers, and proper curing is needed. It is typically applied in thicknesses of 1.5 in. to 2 in.

Low-slump dense concrete has moderate to high cement content and low w/cm ratio (ACI 546R, 2004; Russell 2004). It has increased resistance to chloride-ion penetration. The main problems are that it is difficult to place, expensive, and prone to surface cracking. It requires special equipment for proper consolidation, proper curing is critical, and the standard has been to apply a 2-in. overlay.

Silica fume concrete is widely used to produce concrete with greater resistance to chloride penetration (ACI 234R, 2006). It can be used effectively in thin overlays, in similar thickness as the LMC, to provide resistance to the penetration of chlorides similar to latex-modified concrete. It is mixed in stationary mixers at the plant or in ready-mixed concrete trucks. Proper curing is necessary.

Polymer concrete is a composite material in which the aggregate is bound together in a dense matrix with a polymer binder (ACI 546R, 2004). It provides low permeability to water and aggressive solutions. Performance is highly dependent on the strength of the bond between the overlay and the concrete underneath, which is also dependent on surface preparation, cleanliness, and field application techniques. Most failures are attributed to workmanship or improper handling of materials. Epoxy-polymers have a minimum thickness of .25-in. and are expected to last 10 to 15 years. Polymer concrete has been used in applications up to 1.5 in. Certain types have an expected life of 20 years, depending on the thickness of treatment.

Very high early strength concrete is achieved using special blended cement with high fineness and high Al_2O_3 and SO_3 (Sprinkel 1998). The LMC-VE (very early) provides a reliable driving surface within a few hours and reduces traffic interruption.

3.2.1.6.8 Shotcrete

Shotcrete is mortar or concrete pneumatically projected at high velocity onto a surface (ACI 506R, 2005). The high velocity of the material striking the surface provides the compactive effort necessary to consolidate the material and develop a bond to the existing surface. It contains coarse and fine aggregates, water, admixtures, and fibers. The use of an air-entraining admixture improves the resistance of shotcrete to freezing and thawing. The use of fibers improves toughness and gives load-bearing capacity after cracking. Fibers also help in minimizing plastic shrinkage

cracking. Fibers used in shotcrete are generally divided into two groups by their diameter (ACI 506.1R, 2008). Fibers with equivalent diameters greater than 0.012 in. (0.3 mm) are known as macrofibers; they are either steel or synthetic fibers. Macro fibers reduce crack propagation, increase flexural toughness, and improve ductility and impact resistance. They can provide resistance to drying shrinkage cracking and control crack widths at dosages as low as 0.25% by volume. Fibers with diameters less than 0.012 in. (0.3 mm) are known as microfibers. Microfibers used in shotcrete are mainly polypropylene or nylon. They can provide resistance to plastic shrinkage cracking due to excessive moisture loss at early ages at volume percentages as low as 0.1%.

Shotcrete is capable of being placed in vertical and overhead applications without the use of forms (ACI 546R, 2004). There are two basic shotcrete processes: wet mix and dry mix. In wet-mix shotcrete, ingredients are mixed and pumped through a hose to a nozzle where air is added to project the material onto the surface. In dry-mix shotcrete, cementitious material and aggregate are premixed and pumped and water is added at the nozzle and projected onto a surface.

Shotcrete is frequently used for repairing deteriorated concrete bridge substructures. It is also used for reinforcing structures by encasing additional reinforcing steel added to beams, placing bonded structural linings on walls, and placing additional concrete cover on existing concrete structures (ACI 546R 2004). The success of shotcrete depends on the material used and the skill of the nozzle operator. In repairs with irregular shapes, shotcrete may be preferred since formwork is not needed. Shotcrete failures occur mainly due to inadequate preparation of the existing surface, poor workmanship, and not accounting for the relatively impermeable nature of shotcrete which may trap moisture and contribute to critical saturation which is harmful during cycles of freezing and thawing. Wet-mix shotcrete is generally used where high production rates are needed. Concrete trucks usually supply concrete for wet-mix shotcreting. In substructure repairs, with small quantities of material, dry-mix is commonly used.

3.2.1.6.9 Grouts

Grout is a mixture of cementitious material and water, with or without aggregate, proportioned to produce a pourable consistency without segregation of the constituents. Grout is a common material used in repairs to fill cracks, honeycombed areas, and interior voids, and as a bonding agent. In new construction, it is used in open joints and to fill tendon ducts (ACI 546R, 2004). It can be a hydraulic-cement grout or other chemical grout such as the polymer-cement slurry, epoxy, urethane, and high molecular-weight methacrylate (HMWM). Grouting can

strengthen a structure, arrest water movement, or both. Grout can be injected into an opening from the surface of a structure or through holes drilled to intersect the opening in the interior. When injected from the surface, short entry holes (ports), a minimum of 1 in. in diameter and a minimum of 2 in. deep, are drilled into the opening. The surface of the opening is sealed between ports and grout injected under pressure. Grouting is usually started with a relatively thin grout, but is thickened when possible. Narrow cracks would be filled by injection under pressure; however, wider cracks can be filled by gravity. Even though grouts are used as a bonding agent, work has shown that concrete bonds well to existing concrete, provided that proper surface preparation has been made.

Selection of type of grout depends on the magnitude of stress at the location, the movement of the crack, the presence of solutions, crack width, required internal grout pressure, setting characteristic, heat liberation (high for epoxy types) cost, compatibility with the existing concrete, penetrability, and bonding in the presence of moisture (ACI 546R, 2004). Chemical grouts have different mechanical properties and are more expensive than cement grout. Also, a high degree of skill is needed for satisfactory use of chemical grouts. Some epoxy systems do not bond in the presence of moisture. Chemical grouts can fill cracks as narrow as 0.02 in. (0.5 mm), whereas for cement grouts the minimum crack width is 3 mm. ASTM C1107 covers three grades of packaged, dry, hydraulic-cement grouts (nonshrinkable) intended for use under applied load, such as to support a structure or a machine, where change in thickness below initial placement thickness is to be minimized. Rigid chemical grouts, such as epoxies, exhibit excellent bond to clean, dry substrates, and some bond to wet concrete. These grouts can restore the full strength of a cracked concrete member. ASTM C881 covers two-component, epoxy-resin bonding systems for application to concrete that are able to cure under humid conditions and bond to damp surfaces.

Grouts in tendon ducts hinder the penetration of chlorides to reach the steel and bond the internal strands to the structure (Corven and Moreton 2004). Complete filling of the duct is essential for proper protection. The primary constituent of grout is ordinary portland cement (Type I or II). Supplementary cementitious materials (SCMs) are added to lower the permeability. Prepackaged materials are preferred since more uniform product can be obtained. Total chlorides in grouts should be less than the specified limit of 0.08% by weight of cementitious material as specified by *AASHTO LRFD Bridge Construction Specifications (Construction Specifications)* (Table 10.9.3-2) (AASHTO 2010). Chlorides are limited to ensure that the protective layer over the strand is not compromised. The bleeding and resulting voids in grouts have been a concern. Usually the voids are interconnected and facilitate the

intrusion of harmful solutions. Similarly, shrinkage should be controlled by using non-shrink grouts, so that cracks that can facilitate the intrusion of chlorides are eliminated or minimized. The grout properties are given in *Construction Specifications* Table 10.9.3-2, along with the maximum total chloride ions.

Grouts can be placed by pumping and vacuum injection. Vacuum injection is generally used after initial grouting and requires that the duct system be sealed. To ensure that the duct is filled completely during construction, thixotropic grouts, which have very low viscosity after agitation making them easy to pump, are used. They stay fluid when mechanically agitated or moving during pumping, but stiffen when at rest.

3.2.2 Reinforcement Material

Carbon steel (black steel) bars are commonly used as reinforcement in concrete. In the high alkaline environment of concrete (pH ≈ 13.0–13.8) a protective oxide layer forms on the steel protecting the reinforcement from corrosion (Hartt et al. 2009). Chlorides penetrating concrete break down this protective layer initiating corrosion. Carbonation can lower the pH increasing the corrosion rate. The iron corrosion products that form on steel have much greater volume than the metal that is consumed in the corrosion reaction. This increase in volume causes tensile stresses in the concrete. When the stresses exceed the strength of concrete cracks, spalls and delaminations occur.

Alternatives to black steel are introduced to minimize the corrosion distress. These corrosion-resistant reinforcements (CRR) are expected to extend the service life of structures. They include epoxy-coated steel, galvanized steel, low carbon chromium steel, stainless steel (solid or clad), nickel-clad reinforcement and copper-clad reinforcement, titanium, and fiber reinforced polymers.

3.2.2.1 Carbon Steel

Reinforcing steel is available in different grades and specifications. They vary in yield strength, ultimate tensile strength, chemical composition, and percent elongation. The grade designation is equal to the minimum yield strength of the bar in ksi (1000 psi). For example, grade 60 rebar has a minimum yield strength of 60 ksi.

Carbon steel are covered by ASTM specifications: ASTM A615/A615M: Deformed and plain carbon-steel bars for concrete reinforcement (covers grades 40 and 60); ASTM A996 (replaces ASTM A616 Rail-Steel (covers grades 50 and 60) and ASTM A617Axle-Steel (covers grades 40 and 60); and ASTM A706 (grade 60) for enhanced

weldability. A706 includes Grade 80; however, its use may be outside of consideration of consensus design codes and specifications as noted in the ASTM specification.

3.2.2.2 Epoxy-Coated Reinforcement (ECR)

Organic coatings were first investigated as a possibility for rebar protection in the 1970s, when the FHWA began a project to find an organic coating that would be best suited for that purpose (Ramniceanu et al. 2008). After looking at 47 different coatings, it was determined that the best candidates were four fusion-bonded epoxy powders (Kepler et al. 2000). The method of coating reinforcing steel with epoxy was adapted from the method used by utility companies for coating pipes in the petroleum industry. The bar is cleaned by blasting with grit to a near-white finish to remove millscale, rust, and contaminates. It is then heated to the temperature required for the application of the epoxy powder, typically 450°F (230°C), and is passed through an electrostatic spray that applies charged, dry epoxy powder to the steel. The epoxy melts, flows, and cures on the bars, which then are quenched, usually with water spray bath (Manning 1995)

3.2.2.3 Galvanized Reinforcement

Evidence began to surface in the early 1960s that hot-dipped zinc-coated steel reinforcement may provide superior performance to that of uncoated steel (Kepler et al. 2000). Zinc-coated, or galvanized bars are produced by a hot-dip process (Xi et al. 2004). Typically, in accordance with ASTM A767, galvanized bars are dipped in a chromate bath after coating to passivate the zinc surface and to prevent it from reacting with the hydroxide in fresh cement paste (Virmani and Clemeña 1998).

3.2.2.4 Low Carbon Chromium Steel

Typical carbon steels form a matrix of chemically-dissimilar materials—carbide at the grain boundaries and ferrite. In a moist environment, a microgalvanic cell forms initiating corrosion. Low carbon chromium steel matrix is almost carbide-free, minimizing the galvanic action. Low carbon chromium steel has low carbon content, less than 0.15%, and contains from 8-10.9% chrome (ASTM A1035). It exhibits strength and toughness (not brittle), and is also significantly more corrosion-resistant than conventional steel. Chloride corrosion threshold for initiation of corrosion was found to be four times that of black bar (Hartt et al. 2009).

3.2.2.5 Stainless Steel

Stainless steels are chromium-containing steel alloys with a minimum chromium content of 10.5% (Markeset et al. 2006). Corrosion resistant stainless steel contains a minimum of 12% chromium (Scully and Hurley 2007). The chromium creates an invisible surface film that helps to make stainless steel corrosion resistant by resisting oxidation. Other metals can be added to increase the corrosion resistance. Stainless steels are divided into four types: ferritic, ferritic-austenitic, austenitic, and martensitic (Kepler et al. 2000). Ferritic steels are low carbon steels with less than 17% chromium. Austenitic steels are low carbon steels with around 18% chromium and 8% nickel. Ferritic-austenitic steels (duplex steels) typically contain 22-28% chromium and 4-8% nickel. Martensitic stainless steels have a chromium content of 12% to 18% and a carbon content as high as 1.2%. Austenitic and ferritic-austenitic steels can be produced as ribbed bars within the normal range of strength and deformability. However, the strength of these bars "as rolled" is not sufficient. Either special heat treatment or cold and warm working, where the temperature of the steel is raised sufficiently so that the steel is easier to form but not so high as to change any of the metallurgical properties, is generally used to increase the strength of the bar (Kepler et al. 2000).

ASTM Specification A955 covers three grades of stainless steel bars with minimum yield strengths of 40, 60, and 75 ksi. Another type of stainless steel reinforcement, stainless steel-clad bars, is available. The cladding is a barrier coating used to prevent corrosive agents from coming in contact with the core of the bar. The improved corrosion resistance of stainless steel is due to a thin chromium oxide film that is formed on the steel surface. Oxygen is required for the film formation. Other typical alloying elements are molybdenum, nickel, and nitrogen. Nickel is mostly alloyed to improve the formability and ductility of stainless steel. Following are the four major types of stainless steel (Markeset et al. 2006):

- Martensitic (not for reinforcement).
- Ferritic. This has properties similar to mild steel but with better corrosion resistance, even though in the lower range of corrosion resistance for reinforcement.
- Austenitic. This is the most widely used type of stainless steel and rated in the higher range of corrosion resistance for reinforcement.

- Austenitic-Ferritic (Duplex). The duplex structure delivers both strength and ductility, and is rated in the very high range of corrosion resistance.

It is apparent that stainless steel has varying properties and corrosion-resisting potential, and studies are continuing to identify them for use as reinforcement (Hartt et al. 2009).

3.2.2.6 Nickel-Clad Reinforcing Bars

Nickel clad bars are produced by applying a heavy layer of nickel to a billet before it is hot rolled. The result is a continuous coating of wrought nickel on the surface of the bar with a diffusion zone of alloyed nickel and iron underneath. The alloyed zone provides additional protection should a break occur in the wrought nickel barrier.

3.2.2.7 Stainless Steel-Clad Reinforcing Bars

Stainless steel-clad bars are an attractive alternative to solid stainless steel from the standpoint of both corrosion mitigation and cost. One would ideally gain the resistance to corrosion of solid stainless steel at a fraction of the cost of solid single-phase stainless steel (Scully and Hurley 2007). Concerns about using stainless steel-clad reinforcement include 1) the difficulty encountered bonding the cladding to the bars, and 2) if areas of carbon steel are exposed because of mechanical damage (e.g., construction site handling or unsealed cut ends) those localized areas will corrode (Darwin et al. 1999; Scully and Hurley 2007). The corrosion resistance of clad bars is dependent on any defects that expose the carbon steel core. The performance is similar to solid stainless steel when intact, and when defective, it is similar to that of carbon steel rebar (Scully and Hurley 2007).

3.2.2.8 Copper-Clad Reinforcement

Copper-clad reinforcing steel bars were first tested in concrete in 1980 in connection with an FHWA study (Virmani et al. 1983). Copper, lead and zinc salts have been known to retard the hydration of cement. Upon examination of cores taken from the slabs, it was found that the copper-clad bars had discolored the surrounding concrete, turning it a gray-green color. Petrographic observation indicated that there was a significantly higher amount of unhydrated cement around the copper-clad bars than elsewhere in the slab. This change extended 0.01 in. to 0.02 in. (0.25 mm to 0.5 mm) from the copper-clad bars into the surrounding concrete. "The paste surrounding the copper-clad rod is still considered to be hard, even though relatively unhydrated." (McDonald et al. 1996). Before copper-clad reinforcement can be put to use in any structures, additional research needs to be performed on the

structural effect of the retardation of cement hydration that is caused by these bars (McDonald et al. 1996; Virmani and Clemeña 1998).

3.2.2.9 Titanium

Titanium is highly resistant to corrosion; however, the high cost can be six times that of stainless steel.

3.2.2.10 Fiber-Reinforced Polymer (FRP)

Fiber-reinforced polymer composite bars and strands are commercially available, have been used in structures, and their use in highway infrastructure is discussed in NCHRP Report 503 (Mertz et al. 2003).

FRP composite materials are light in weight and easy to construct; provide excellent strength-to weight characteristics; and can be fabricated for "made-to-order" strength, stiffness, geometry, and other properties (ACI E2, 2000; Mertz et al. 2003). In addition, FRP is nonmagnetic. FRP composite materials may be the most cost-effective solution for repair, rehabilitation, and construction of portions of the highway infrastructure (Mertz et al. 2003).

FRP is composed of a polymer matrix, either thermoset or thermoplastic, reinforced with fiber or other reinforcing material (ACI 440R, 2007). An anisotropic material, its most favorable properties are in the direction of the fibers. The performance of the FRP depends on the fiber, the matrix, and the interaction of the two. The resin or the polymer holds the fibers in place, the fibers provide the mechanical strength, and the fillers and additives aid in processing and performance. Principal types of fibers used in structural applications are glass, carbon, and aramid.

The FRP bars are generally made of glass fibers embedded in a matrix (thermoset or thermoplastic resins). The properties of the FRP bars, such as high temperature performance, corrosion resistance, dielectric properties, flammability, and thermal conductivity, are mainly derived from the properties of the matrix. Depending on the matrix used, the mechanical properties of FRP bars (e.g. tensile strength, ultimate strain, and Young's modulus of elasticity) might degrade under specific environmental conditions such as alkaline environment and moisture (water and salt added to water) as follows.

- Effects of the alkaline environment. The alkaline water contained in concrete pores may cause degradation, which might cause damage to the polymeric matrix.

- Effects of moisture (water and salt added to water). The main effects of moisture absorption by the matrix may result in strength reduction and stiffness reduction (less pronounced) in the FRP. The absorption of moisture depends on the type of polymeric matrix, the matrix interface, and the quality of the bars.

Glass fibers are available as E-glass (electrical grade), high strength (S-2 glass), improved acid resistance (ECR) and alkali resistance (AR glass) (ACI 440R 2007), and generally range in diameter from 9 to 23 microns. Fibers are drawn in at high speed through small holes in electrically-heated bushings forming the individual filaments. The filaments are coated with a chemical binder or sizing for protection and to enhance the composite properties at the fiber-matrix interface. The filaments are gathered into groups or bundles called strands or tows.

E-glass fibers are the most susceptible to degradation due to alkalinity and moisture and must be protected by the appropriate resin. Carbon fibers are inert to the environment. Different fiber systems and resin systems provide different levels of resistance to environmental conditions such as moisture, alkaline solution, UV radiation, or extreme temperatures. Selection of the proper reinforcement and the proper resin is needed for longevity. The coefficient of thermal expansion of the FRP composites with glass fibers is higher than that of the concrete. The difference should be considered when FRP is in direct contact with concrete. At low temperatures and exposure to cycles of freezing and thawing fibers are not affected, but the resin and the fiber-resin interface are affected. Cycles of freezing and thawing and the presence of road salts may result in microcracks and gradual degradation.

Polymer resins exhibit high creep and relaxation behavior; addition of fibers increases the creep resistance. Thermosetting resins are more resistant to creep than the thermoplastic resins. FRP generally has good fatigue performance since the fibers have minimal defects and the matrix resists the propagation of cracks. Several drawbacks of FRP is that it does not exhibit ductile failure mode as steel, it is the lower elastic modulus, and the strands are difficult to grab, making anchorage details critical.

3.2.3 Structural Steel

This section describes the various steel types, including currently used types and older types found in older bridges, and describes various characteristics that are important for durability and long term service life.

3.2.3.1 Current Steel Grades

Currently, there are seven structural steel grades for bridges covered by ASTM A709 (AASHTO M-270) specifications with four yield strength levels. Table 3.6 identifies these current types. Included are ASTM reference standard grades, along with the steel descriptions and yield and tensile strengths. The letter W is attached to the grade number to designate steel that has weathering capability. The letter S is attached to designate special steel grade for structural shapes.

Table 3.6. Current ASTM A709 Steel Grades.

Grade	Additional Reference Standard	Name/Designation	Yield Strength	Tensile Strength
36	A36	Structural Carbon Steel	36 ksi min	58 to 80 ksi
50	A572	High strength-low allow Columbium-Vanadium steel	50 ksi min	65 ksi min
50S	A992	Structural Steel Shapes	50 – 65 ksi	65 ksi min
50W	A588	High strength-low allow Weathering Steel	50 ksi min	70 ksi min
HPS 50W		High Performance Steel	50 ksi min	70 ksi
HPS 70W		High Performance Steel	70 ksi min	85 to 110 ksi
HPS 100W (to 2.5”thick)		High Performance Steel	100 ksi min	110 to 130 ksi
HPS 100W (over 2.5” to 4”)		High Performance Steel	90 ksi min	100 to 130 ksi

Grades 36, 50, 50W, HPS 50W and HPS 70W are available with properties shown in plate thicknesses up to 4 in. Properties for HPS 100W will vary depending on plate thickness as shown in Table 3.6.

Grade 36 steel has been a highly used grade over the years, and includes basic carbon steel shapes, plates, and bars for use in riveted, bolted, or welded construction.

Grade 50 and 50S steel includes High-strength low-alloy Columbium-Vanadium structural steel for shapes, plates, bars and sheet piling, intended for riveted, bolted, or welded construction.

Grade 50W includes high strength low alloy weathering steel shapes, plates, and bars for welded, riveted, or bolted construction, but is intended primarily for use in welded bridges in which savings in weight and added durability are important.

In addition, new high performance steels have been added with three grades as shown in Table 3.6. These new steels are further described in Section 3.2.3.6.

Table 3.7 identifies older steel grades that are still available, but AASHTO and A709 have replaced them with HPS 70W and HPS 100W grades because of improved properties.

Table 3.7. Older Structural Steel Grades Still Available.

Grade	Additional Reference Standard	Name/Designation	Yield Strength	Tensile Strength
70W	A852	High strength-low allow quenched and tempered steel	70 ksi min	90 ksi min
100/100W	A514	High strength quenched and tempered alloy steel	100 ksi min	110 to 130 ksi

3.2.3.2 Steel Properties

Steel's mechanical properties are those that characterize its elastic and inelastic behavior under stress and strain. Such properties include parameters that are related to the material's strength, ductility, and toughness. Other parameters, such as weldability, machinability, and weathering are also important in regard to the ease in which the material can be fabricated, and achieve long-term durability.

Strength is represented by the material's yield strength and ultimate tensile strength.

Ductility is the ability of a material to undergo large plastic deformations without fracture. It is represented by the amount of elongation that the material can experience after yielding and before reaching its ultimate strength. Ductility is an important material property because it allows redistribution of high local stresses that occur in welded connections and at regions of stress concentration such as holes or geometric changes.

Toughness is the capacity of a material to absorb large amounts of energy prior to fracture. It is related to the area under the stress-strain curve, and is dependent on strength and ductility. The larger the area under the stress-strain curve, the tougher the material.

Weldability is the material's capability to withstand welding without seriously impairing its mechanical properties. Weldability varies considerably for different types of steels and different welding processes.

Machinability is the ease with which a material can be sawed, drilled, or otherwise shaped without seriously impairing its mechanical properties.

Weathering is the material's capability to resist corrosion in a given environment.

The steel's mechanical properties are affected by three factors:

1. Chemical composition,

2. Processes used to transform the base metal into the final plate or structural shape product, and

3. Heat treatment.

The following sections describe these factors.

3.2.3.3 Chemical Composition

The most important single factor in determining the properties of a steel type is its chemical composition. Structural steels are made up of iron and carbon with varying amounts of other elements, primarily manganese, phosphorus, sulfur, and silicon. These and other elements are either unavoidably present or added intentionally in various combinations to achieve specific characteristics and properties.

In carbon steels, the elements carbon and manganese have a controlling influence on strength, ductility, and weldability. Most carbon steels are over 98% iron, with about 0.25% carbon and about 1% manganese by weight at the Grade 36 level. These percentages change dramatically among the various grades. Carbon increases the hardness or tensile strength, but has adverse effects on ductility and weldability. Therefore, small amounts of various alloying elements are sometimes used to increase the ability of a steel to achieve higher strength while maintaining a lower percentage of carbon content. Phosphorus and sulfur typically have a harmful effect, especially on toughness, and the fractional percentages of these elements must therefore be kept low. Small amounts of copper increase corrosion resistance and help develop the "weathering" steel grades; fractional percentages of silicon are used mainly to eliminate unwanted gasses from the molten metal; and Nickel and Vanadium have a generally beneficial effect on steel behavior.

Table 3.8 describes advantages and disadvantages of commonly used alloy elements, and includes approximate chemical composition used in plate steel grades for comparison. It is interesting to note how each element percentage changes in progressing from Grade 36 to 50 to 50W to HPS 50W and finally to HPS 100W. The progression in steel grades increases in strength, weathering characteristics, and ultimately in toughness and weldability, and the various chemical percentages illustrate how these properties are partially achieved. Chemical composition of Grade HPS 70W is same as HPS 50W. Grade 50S is not shown.

Table 3.8. Effects and Composition of Alloying Elements.

Alloy Element	Advantage	Disadvantage	Steel Grade	Typical Composition
Carbon	Principal hardening element in steel. Increases strength and hardness.	Decreases ductility, toughness and weldability.	36 50 50W HPS 50W HPS 100W	.25% .23% .19% .11% .08%
Manganese	Increases hardness and strength. Controls harmful effects of sulfur	High amounts can cause embrittlement and reduce weldability	36 50 50W HPS 50W HPS 100W	.8% to 1.2% 1.35% .8% to 1.25% 1.1% to 1.35% .95% to 1.5%
Phosphorus	Increases strength and hardness. Improves corrosion resistance	Generally considered an impurity. Decreases ductility and toughness.	36 50 50W HPS 50W HPS 100W	.04% .04% .04% .02% .015%
Sulfur	Improves machinability.	Generally undesirable. Decreases ductility, toughness, and weldability. Adversely affects surface quality.	36 50 50W HPS 50W HPS 100W	.05% .05% .05% .006% .006%
Silicon	Used to deoxidize molten steel. Increases strength and hardness	Decreases weldability	36 50 50W HPS 50W HPS 100W	.4% .4% .3% to .65% .3% to .5% .15% to .35%
Aluminum	Used to deoxidize molten steel. Refines grain size, thus increasing strength and toughness.	None	HPS 50W HPS 100W	.01% to .04% .02% to .05%
Vanadium	Used to refine grain size. Small additions increase strength and toughness	High amounts reduce hardness. At high finishing temperature may be detrimental to finishing	50 50W HPS 50W HPS 100W	.01% to .15% .02% to .1% .04% to .08% .04% to .08%
Columbium	Small additions produce finer grain size which increases strength and toughness	None	50 HPS 100W	.005% to .05% .01% to .03%
Nickel	Increases strength and toughness and improves corrosion resistance.	Cost	50W HPS 50W HPS 100W	.4% .25% to .4% .65% to .9%
Chromium	Increases strength and corrosion resistance. Used in “weathering” steel	Cost	50W HPS 50W HPS 100W	.4% to .65% .45% to .7% .4% to .65%
Molybdenum	Increases strength, weldability, toughness, and corrosion resistance	Cost Delays rather than eliminates temper embrittlement	HPS 50W HPS 100W	.02% to .08% .4% to .65%
Copper	Increases corrosion resistance. Used in “weathering” steel	May adversely affect notch toughness	50W HPS 50W	.25% to .4% .25% to .4%

Alloy Element	Advantage	Disadvantage	Steel Grade	Typical Composition
			HPS 100W	.90% to 1.2%
Nitrogen	Increases strength and hardness.	Decreases ductility and toughness.	HPS 50W HPS 100W	.015% .015%

Advantages/disadvantages from AISC Manual of Steel Construction (AISC 1994) and metals handbook of the materials information society (ASM 1998); Percentages from ASTM A709

3.2.3.4 Rolling and Shaping Processes

The process used to transform the base metal into the final plate or structural shape product is another factor affecting steel's mechanical properties. Shaping steel by rolling consists of passing the material between two rolls revolving in opposite directions, with the distance between the rolls less than the thickness of the original entering material. The rolls grip the piece and reduce its cross-sectional area, increasing its length and to a lesser extent, its width. Because of the rolling, the steel fracture toughness is greater in the direction of rolling than in the direction perpendicular to it. Further, a greater reduction of cross section during the normal hot rolling process will produce a greater yield and tensile strength.

Some steels are purposely cold rolled to obtain higher strength levels. The cold working strain hardens the material.

3.2.3.5 Heat Treatment

There are a variety of heat treatments used in the steel making process to develop certain desirable characteristics. These heat treatments can be divided into two groups: slow cooling and rapid cooling. Slow cooling treatments include annealing, normalizing, and stress relieving, which decrease hardness and promote uniformity of microstructure, and improve ductility and fracture toughness. They also improve machinability or facilitate cold forming, and relieve internal stresses. Rapid cooling treatments, such as quenching and tempering, increase strength, hardness, and toughness.

Quenching and tempering is a process consisting of heating steel to a high temperature (about 1650°F) for a time needed to produce a desired microstructure change, then quenching by immersion in water. After quenching, the steel is tempered by re-heating to a specific temperature (usually between 800°F and 1250°F), holding for a specified time, and then cooling under suitable conditions to obtain the desired properties. The rapid cooling by quenching increases strength but reduces ductility. Tempering then restores part of the ductility, but gives up some of the

strength gained by the quenching. This process permits attainment of higher strengths while retaining relatively good ductility.

3.2.3.6 High Performance Steel (HPS)

In 1994, a cooperative research program between the FHWA, the American Iron and Steel Institute (AISI) and the U.S. Navy was launched to develop new high performance steels for bridges. The driving force was the need to develop improved higher strength, improved weldability, and higher toughness steels that would improve the overall quality and ease of fabrication of bridge steels used in the United States. Further, the steel would be weathering grade with the added designation "W." Three grades were developed and are now available for general use: HPS 50W, HPS 70W, and HPS 100W. Grades HPS 70W, and HPS 100W have now replaced the older high strength low allow-quenched and tempered steel, AASHTO M270 Grade 70W, and the high strength quenched and tempered alloy steels, Grade 100 and 100W. It is the intent that the newer HPS steels should be used at these higher strength levels because of their enhanced properties. The older steels are still available, but their use is discouraged.

HPS 70W is the most widely used grade in the HPS group. HPS 70W is produced by quenching and tempering (Q&T) or thermal-mechanical controlled processing (TMCP). Because Q&T processing limits plate lengths to 50 ft in the United States, TMCP practices have been developed to produce HPS 70W up to 2 in. thick and to 125 ft long, depending on the weight. HPS 50W steel has the same chemistry as HPS 70W, and is produced using conventional hot-rolling or controlled rolling up to 4 in. thick in lengths similar to Grade 50W steel.

A major advantage of HPS is its increased fracture toughness, which is much higher than that of conventional bridge steels. This is evident from Figure 3.3, which shows the Charpy V-Notch (CVN) transition curves for HPS 70W and conventional Grade 50W steel. The brittle-ductile transition of HPS occurs at a much lower temperature than conventional Grade 50W steel. This means that HPS 70W remains fully ductile at lower temperatures while conventional Grade 50W steel begins to show brittle behavior.

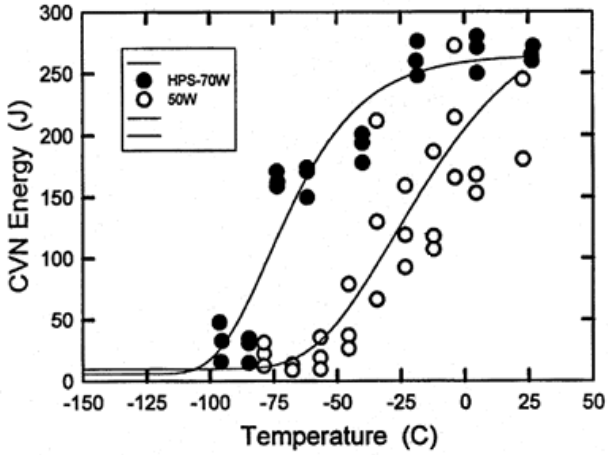


Figure 3.3. CVN transition curves for 50W and HPS 70W steels. (FHWA 2002)

The current AASHTO CVN toughness requirements are specified to avoid brittle failure in steel bridges above the lowest anticipated service temperature. The HPS 70Wsteels tested so far show ductile behavior at the extreme service temperature of -60°F, which corresponds to Zone 3, with minimum service temperature of below -30°F to -60°F.

With higher fracture toughness, high performance steels have much higher crack tolerance than conventional grade steels. Full-scale fatigue and fracture tests of I-girders fabricated of HPS 70W in the laboratory showed that the girders were able to resist the full design overload without fracture even when the crack was large enough to cause a 50% loss in net section of the tension flange. This is a major feature for service life application because large crack tolerance increases the time for detecting and repairing fatigue cracks before the bridge becomes unsafe.

3.2.3.7 Weathering Steels

Uncoated weathering grade steels have been available for bridge application on a large scale since the mid 1960s. These steels have a chemical composition containing small amounts of copper, phosphorus, chromium, nickel, and silicon to attain their weathering properties. Weathering steel forms a tightly adhering patina during its initial exposure to the elements. The patina is essentially an oxide film of corrosion by-products about the same thickness as a heavy coat of paint. FHWA issued *Technical Advisory T-5140.22* (1989), which provides guidelines for proper application of weathering grade steels in highway structures and recommendations for maintenance to ensure continued successful performance.

Weathering grade steels are currently supplied under AASHTO Specification M270, ASTM A709 Grade 50W, and in high performance steel grades HPS 50W, 70W and 100W. When used in the right environment, these steels are both short- and long-term cost-effective as they eliminate the need for shop and field painting, and can provide over 100 years service life with minimal maintenance.

The initial corrosion of weathering steel depends on the presence of moisture and oxygen, but as corrosion continues, a protective barrier layer forms that greatly reduces further access to oxygen, moisture, and contaminants. This stable barrier layer greatly resists further corrosion, reducing it to a low value. Weathering steel bridges initially look orange-brown in color; however, the color will darken as the patina forms. In two to five years, depending on the climate, the steel will attain a dark, rich, purple-brown color that many think is attractive.

Several factors can impact the satisfactory performance of weathering steel. Experience has shown, for example, that weathering steel requires alternating cycles of wet and dry conditions in order to form a properly adhering protective layer. This would generally rule out areas of high rainfall and humidity or persistent fog. Extreme marine conditions, the presence of roadway deicing salts, pollution, surrounding vegetation, and "tunnel-like" conditions can also lead to unsatisfactory performance, as can poor detailing and maintenance.

Bridge engineers should avoid specifying weathering steel that will be exposed to sea water spray, salt fogs, and immediate coastal salt environments because the salt film that is deposited on the metal surface, being hygroscopic, tends to maintain continuously damp conditions, preventing the formation of a proper patina.

Heavy use of deicing salts over and under weathering steel bridges may cause problems. For example, salt-laden runoff that flows through leaking expansion joints and directly over the steel has been identified as a cause of poor weathering steel performance.

Tunnel-like conditions result from a combination of a narrow road with minimum shoulders between vertical retaining walls, or a wide bridge with minimum headroom and full-height abutments. Such situations may be encountered at urban or suburban grade separations. In these cases, the lack of air currents to dispel roadway spray leads to excessive salt deposits on the bridge girders.

In addition, weathering steel bridges should not be located where ambient atmospheres contain high concentrations of pollution and industrial fumes, especially sulfur dioxide, but moderate industrial environments typically speed the weathering process and more quickly achieve the mature dark color.

In all probability, bridge expansion joints will eventually leak, so many states recommend painting the steel beam ends to a length 1.5 times the girder depth. Painting the ends also eliminates staining of the concrete piers below the joints.

3.2.3.8 Structural Stainless Steel

In recent years, a new steel has been developed, ASTM A1010, which is a 12% chromium structural steel with superior corrosion resistance. A1010 is currently widely used in aggressive structural applications such as coal rail cars and coal processing equipment in thicknesses to 0.5 in. Because of A1010's superior corrosion resistance, it is also being considered for bridge applications in corrosive environments. Data on thicknesses to 4 in. are now available. A1010 steel is available in typical plate sizes in 50-ksi and 70-ksi strength levels.

Laboratory corrosion testing to the SAE 52334 standard has shown that A1010 performs in a superior fashion in a wet/dry salt-water environment when compared to weathering or galvanized specimens. This is summarized in Figure 3.4. In addition, long-term exposure to seaside locations has shown that A1010 performs significantly better than a variety of weathering steels. Although this material is slightly more expensive than conventional steels, its ability to perform in highly corrosive environments may give it an advantage when considering life-cycle cost over a long-term service life. Precautions must be taken, however, when using stainless steel to avoid galvanic corrosion when in contact with carbon steel, zinc, or aluminum.

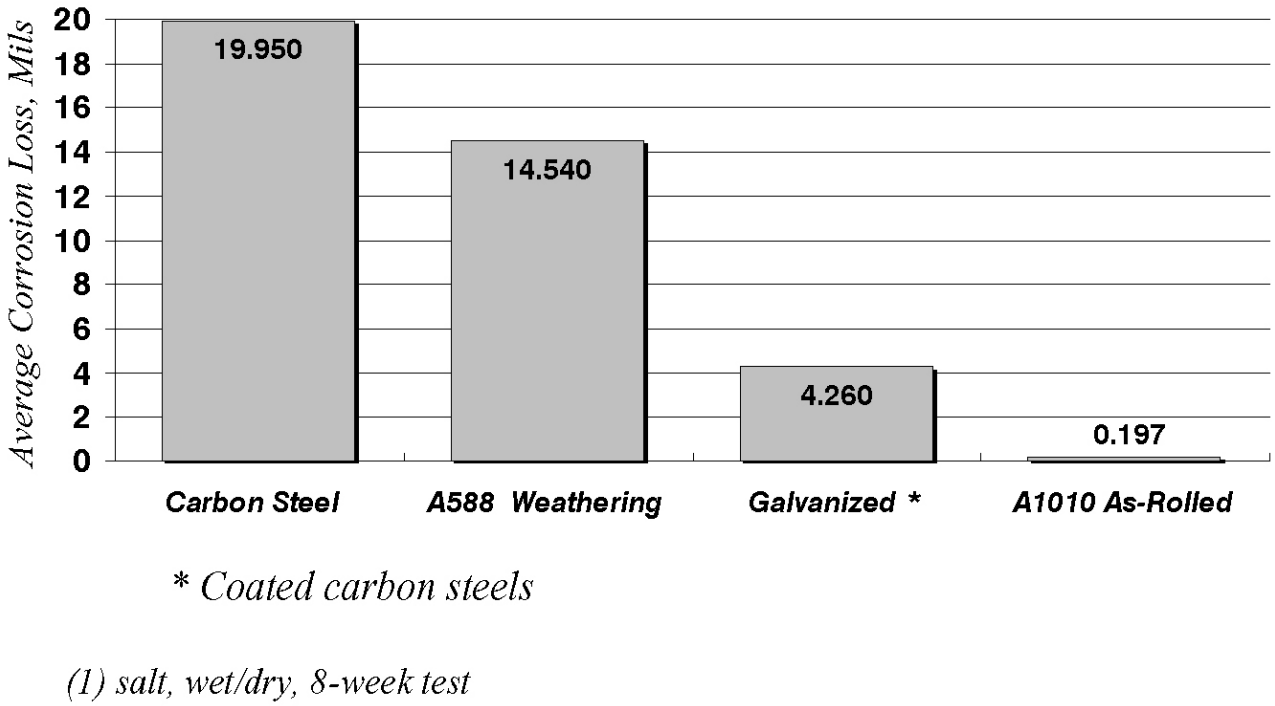


Figure 3.4. Corrosion resistance of A1010 Steel. (Wilson 1999)

3.3 CONCRETE AND STEEL DISTRESSES AND SOLUTIONS

The common distress factors that affect durability and the relevant solutions are given in Section 3.3.1 and Methods for protecting reinforcement against corrosion are given in Section 3.3.2.

3.3.1 Common Concrete Distress Factors and Solutions

Concrete is subject to deterioration due to physical factors (moisture, temperature, freeze and thaw); chemical factors (ASR, alkali-carbonate reaction [ACR], carbonation, chlorides, sulfates, and acids); and functional factors (vibrations, impact, concrete consolidation, concrete curing, and concrete placement). Stresses induced in concrete due to volumetric changes resulting from the chemical, physical, and/or functional factors can exceed the strength of the material leading to cracks, delaminations and spalling. Proper steps should be taken to protect concrete from cracking.

3.3.1.1 Factors

The most common outcome of a distress is the formation of cracks in concrete. Volumetric changes that occur in concrete, the restraint in the bridge system, and the elastic modulus, creep, and tensile strength characteristics lead to cracking (TR Circular E-C107, 2006). The lower the modulus of elasticity, the lower will be the amount of the

induced elastic tensile stress for a given magnitude of volumetric change, thus reducing the possibility of cracking. On the other hand, high elastic modulus is associated with brittle concretes which are prone to cracking and crack propagation. In concretes with low elastic modulus, such as lightweight concrete (LWC), stresses are less for a given strain compared to conventional normal weight concretes with higher elastic modulus. Reduced stresses minimize the occurrence and severity of cracks (number and width). Cracks facilitate the intrusion of aggressive solutions that adversely affect the durability of concrete. In columns and prestressed beams reduced modulus of elasticity would indicate higher deformation and prestress losses. Concrete can be engineered to have tight cracks (Sahmaran and Li 2001). In such concrete cracks formed are numerous and very tight, less than 0.004 in. (0.1 mm), in width. It is difficult for harmful solutions to penetrate such tight cracks. Research has shown that concrete with crack widths less than 0.004 in. (0.1 mm) performs like sound concrete and does not allow water to penetrate the cracks easily (Wang et al. 1997; Lawler et al. 2002). ACI provides guidance on tolerable crack widths for reinforced concrete exposed to different exposure conditions (ACI 224R, 2001). For example, the reasonable crack width for reinforced concrete exposed to seawater and seawater spray, wetting and drying is 0.006 in. (0.15 mm). A note to the table in the ACI document (ACI 224R, 2001) indicates that a portion of the cracks in the structure is expected to exceed these values; with time, a significant portion can exceed these values. The provisions contained in the *AASHTO LRFD Bridge Design Specifications* are based on a maximum crack width of 0.004 in (0.1 mm) for reinforced concrete exposed to a marine environment (AASHTO 2012).

Volumetric changes can occur due to moisture loss as in plastic shrinkage or drying shrinkage, or due to high temperatures and high temperature differentials. As cement and water react, heat is generated and this is known as heat of hydration (Kosmatka and Wilson 2011). The rate of heat generation is greatest at early concrete ages. For thin concrete elements heat generation is generally not a concern since the heat is dissipated quickly. However, in mass concrete (Figure 3.5), heat is not easily dissipated and a significant rise in temperature occurs. As a general rule of thumb, any placement of structural concrete with a minimum dimension equal to or greater than 3 ft should be considered mass concrete (Gajda 2007). Similar considerations should be given to other concrete placements that do not meet this minimum dimension, but contain ASTM C150 Type III or HE cement, accelerating admixtures, or cementitious materials in excess of 600 lb/yd^3 of concrete (Gajda 2007). Mass concrete specifications, generally, limit the temperature difference between the center and surface of the concrete to 35°F. This is a conservative limit

which has been effective (Gajda 2007). Performance based temperature difference limit tailored to the particular job can indicate temperature differences higher than 35°F without adverse effects. Finite element modeling and detailed calculations are often used to develop temperature difference limits.

Figure 3.5. Mass concrete; footings and columns. (Courtesy Virginia DOT)

Also, high temperatures exceeding 160°F during the first few hours following placement can lead to delayed ettringite formation (DEF) at later ages. Ettringite forms when gypsum and other sulfate compounds react with calcium aluminate in cement in the first few hours after mixing with water (Kosmatka and Wilson 2011). However, at high temperatures the normal formation of ettringite during the first few hours is impeded and DEF occurs in hardened concrete, which can crack the concrete. Heat related cracking can also occur in bridge decks that are typically not considered to be mass concrete (Figure 3.6). In this case the bridge beams restrain temperature related movement of the deck. The heat of hydration causes the deck to expand and then contract during cooling and the beams restrain the movement. To prevent the strains and resulting tensile stresses, a maximum temperature differential of 22°F between the beams and the deck is recommended for at least 24 hours following concrete placement (Babaei and Fouladgar 1997). As in the mass concrete, higher temperatures may be justified, but require detailed analysis to justify. To minimize the potential for temperature-related cracking, the amount of portland cement should be minimized, concrete delivery temperature reduced, and pozzolans and slag included in the mixture. With proper temperature management thermal cracks can be minimized and controlled.

Figure 3.6. Thermal cracks in bridge deck. (Courtesy Virginia DOT)

Volumetric changes also occur due to freezing and thawing cycles and chemical (corrosion, ASR, ACR) reactions.

3.3.1.2 Prevention of Distress

3.3.1.2.1 Surface Treatments

Surface treatments, membranes, sealers, and overlays are widely used to reduce solution infiltration. They protect the deck concrete from deterioration induced by freeze/thaw cycles, and protect the reinforcement from corrosion (Kepler et al. 2000). Currently, there are three types of waterproofing membranes used in North America: preformed sheets, liquid membranes, and built-up systems. Preformed systems can be labor intensive to install, and are vulnerable to poor workmanship at critical locations, such as curves, expansion joints, and drains. Defects and blisters have to be repaired by puncturing and patching the membrane. Liquid systems are less vulnerable to poor workmanship, and blisters and pinholes are easy to repair in self-sealing materials, but not in thermosetting materials. Built-up systems are not often used because they are generally labor intensive and expensive (Manning 1995).

To perform well, a membrane must be well bonded and undamaged, characteristics that depend on the quality of workmanship during installation. Waterproof membranes cannot stop corrosion that is already underway in existing decks, but when combined with an asphalt overlay, they can extend the service life of a deck by providing a smooth riding surface, preventing the development of potholes, and possibly slowing the rate of corrosion by limiting additional chloride contamination (Manning 1995).

Sealers can be used to protect all of the exposed concrete surfaces of the structure, including bridge decks, substructure members, and deck undersides (Zemajtis and Weyers 1996). Sealers can either be pore blockers, forming a microscopically thin (up to 2 mm) impermeable layer on the concrete surface, or they can penetrate into the concrete slightly (1.5 to 3 mm) and act as hydrophobic agents (Zemajtis and Weyers 1996). Most pore blockers are not appropriate for use on bridge decks because they do not offer good skid resistance and do not hold up under traffic wear (Sherman et al. 1993).

An important property of a sealer is its vapor transmission characteristics. Moisture within the concrete needs to be able to pass through the sealer and escape to prevent high vapor pressures from building up in the concrete during drying periods which could cause the sealer to blister and peal (Sherman et al. 1993).

3.3.1.2.2 Control of Volumetric Changes due to Moisture and Temperature

To minimize volumetric changes due to shrinkage and temperature, low cementitious material, low water content, and low paste content are desirable. This can be achieved by using well-graded aggregates, reducing mix temperature, and the admixtures.

3.3.1.2.3 Control of Wear and Abrasion

Traffic on bridge decks cause abrasion. Moving water also carries objects that can cause abrasion. Studded tires are very damaging to the surface of the concrete and are not permitted in some states and many other states have seasonal restrictions. Chains also have damaging effect, though, not as much as the studs. Chains are encouraged in winter in many areas and some states require chains for commercial trucks. Abrasion resistance of concrete is a function of the water cementitious material ratio (w/cm) at the surface and the aggregate quantity and quality. The Los Angeles abrasion test indicates the abrasion resistance of aggregates and in general, siliceous aggregates provide satisfactory abrasion resistance. Proper finishing and curing are also factors which influence abrasion resistance.

3.3.1.2.4 Control of Freeze/Thaw Damage

Concrete that can become critically saturated and exposed to cycles of freezing and thawing must be properly air entrained, have sound aggregates and have the maturity to develop a compressive strength of about 4000 psi to avoid cracking and scaling (Mather 1990). Air voids must be small in size, closely spaced and uniformly distributed to ensure adequate resistance to freezing and thawing and satisfactory strength. A spacing factor less than 0.008 in. is

needed for adequate protection during freezing and thawing. However, with the use of high range water reducing admixtures and low permeability concretes, higher values may be acceptable and should be verified by relevant tests such as ASTM C 666 Procedure A.

3.3.1.2.5 Control of Chemical Reactions

Disruptive chemical reactions can occur in concrete that adversely affect durability. The chemical composition of the cemetitious material affects the rate of hydration and pozzolanic reaction, heat generation, and contributes to the formation of disruptive products. The fineness of the cementitious material affects the rate of reactions and water demand. At high water w/cm, permeability is increased and durability is reduced. The presence of certain chemicals in sufficient amounts in the cements contributes to the expansion. A further explanation follows.

3.3.1.2.5a Alkali-Aggregate Reaction (AAR)

Alkali-aggregate reactions are the reactions between the hydroxide ions in the pore fluid of concrete, usually associated with alkalis from the cement or from outside sources such as deicing salts, and the reactive constituents of the aggregates (ACI 221.1R, 1998). This reaction results in expansion and cracking.

3.3.1.2.5b Alkali-Silica Reaction (ASR)

A chemical reaction between aggregates containing reactive silica and the alkalis in concrete can produce an alkali-silica gel that swells when water is absorbed. The high pressures generated within the concrete lead to cracking (Figure 3.7). To prevent ASR, non-reactive aggregates, or pozzolanic materials or slag, lithium nitrate, low-permeability concrete and cements with low alkali contents are used.

Figure 3.7. ASR. (Courtesy Virginia DOT)

3.3.1.2.5c Alkali-Carbonate Reaction (ACR)

Some argillaceous, dolomitic aggregates can react with alkalis, causing the aggregates to expand. Figure 3.8 shows a joint closing due to ACR expansion. A common prevention for ACR is to avoid using reactive aggregate or to dilute the aggregate by blending with non-reactive aggregate.

Figure 3.8. Closing of joints due to ACR. (Courtesy Virginia DOT)

3.3.1.2.5d Carbonation

Carbon dioxide produced by plants penetrates concrete and reacts with the hydroxides, such as calcium hydroxide, to form carbonates. In this carbonation process the pH is reduced to less than nine, influences the protective layer over the steel (Neville 1995).

3.3.1.2.5e Chlorides

Chlorides which penetrate the concrete and reach the steel surface destroy the protective oxide layer making reinforcement prone to corrosion. Without the protective layer, steel will corrode rapidly in the presence of water and oxygen. The corrosion of steel is accompanied by expansive pressures, which lead to cracking (Figure 3.9). To stabilize the passive oxide layer on the reinforcement, corrosion inhibiting admixtures can be used or, viscosity modifying admixtures can be added to improve the stability of the mix.

Figure 3.9. Corrosion. (Courtesy Virginia DOT)

3.3.1.2.5f Sulfates

Sulfates react with the $Ca(OH)^2$ and the calcium aluminate hydrates causing expansive reactions that can affect the cement paste (Neville 1995). Figure 3.10 displays the loss of material due to sulfate attack. To increase the resistance of concrete to sulfate attack a low C_3A, tricalcium aluminate, content, low quantities of $Ca(OH)^2$, calcium hydroxide (lime), in the cement paste, and low permeability concrete is needed. These results can be obtained by using sulfate resistant cements and use of pozzolans.

Figure 3.10. Sulfate attack. (Courtesy Virginia DOT)

3.3.1.2.5g Acids

Pollutants cause acid rain that can cause deterioration (Neville 1995). In damp conditions, SO^2, CO^2, SO^3 and other acid forms that are present in the atmosphere may attack concrete by dissolving in water and removing parts of the cement paste. These acids will leave a soft and mushy mass behind (Eglinton 1975). To minimize acid attack, low-permeability concrete and barrier coatings may be used. A barrier material separates the concrete surface from the environment.

3.3.1.2.5h Salts

Chloride bearing deicing chemicals initiate and accelerate corrosion. Magnesium salts are very damaging to concrete causing crumbling (Lee et al. 2000). Calcium magnesium acetate has been suggested to prevent corrosion but was found to be very damaging to concrete. Deicing chemicals can also aggravate freeze/thaw deterioration. Osmotic pressure occurs since moisture tends to move towards zones with higher salt concentrations. In addition the salts increase the rate of cooling causing an increase in the potential for freeze/thaw deterioration at the concrete surface. Proper air entrainment and maturity are also needed to provide the necessary protection.

3.3.1.2.6 Functional Considerations

Functional considerations include vibrations, impact, concrete consolidation, concrete curing, and concrete placement.

3.3.1.2.6a Vibrations

Vibration of fresh concrete using vibrators may cause loss of air in mixtures with a high sand content and could result in freeze/thaw damage. In these mixtures, the frequency and duration of vibration should be reduced to prevent segregation (loss of stability) and loss of air. A recommended practice is to prepare mixtures with a large amount of coarse aggregate content. However, in some regions where D-cracking (Figure 3.11) is an issue the size and amount of coarse aggregate is reduced which leads to mixtures with an excessive sand content. In D-cracking, the aggregate has a pore structure that hinders the expulsion of water from the aggregate pores during freezing resulting in the cracking of the aggregate and concrete.

Figure 3.11. D-cracking. (Courtesy of PCA)

Concrete can exhibit fatigue behavior when subjected to cyclic loading of a given level, below its short-term static strength, and will eventually fail.

3.3.1.2.6b Impact

Concrete can be subjected to extreme loads due to impact of falling rocks, snow avalanches, landslides, or vehicle crashes. The impact resistance is related to the compressive strength and aggregate type (Kosmatka and Wilson 2011). Fiber reinforced concretes are used to minimize the effect of impact in certain applications.

3.3.1.2.6c Concrete Consolidation

Proper consolidation ensures that undesirable entrapped air voids are eliminated. Voids in concrete reduce strength and when interconnected or in large amounts can increase permeability. Currently, workable concretes are made using water reducing admixtures. It is also possible to make stable concretes that have high flow rates and are self consolidating.

3.3.1.2.6d Concrete Curing

Curing ensures that reactions occur and volumetric changes are minimized. Curing should continue until a certain level of the desired properties is achieved. Concrete should stay wet during the curing period and the temperature should be managed to eliminate large differentials. ACI 308R provides recommendations for adequate curing. ACI 305R and ACI 306R provide information on curing of concrete in hot and cold weather respectively.

3.3.1.2.6e Concrete Placement

Concrete should be placed without causing segregation and loss of moisture. Forms should be adequately set, clean, tight, and adequately braced. Forms should be oiled or treated with a form-release agent. Reinforcing steel should be clean and free of rust or mill scale. In cold weather, concrete should not be placed in contact with metal forms and embedments, such as steel structural members or reinforcement, that can freeze the concrete (ACI 306R, 2010). If the frozen concrete does not thaw before the bulk of the concrete sets, bond may be significantly reduced, and concrete quality adjacent to cold metal would be poor. Ideally, the adjacent metal should be heated to the temperature of the concrete immediately before concrete placement (ACI 306R, 2010). Concrete should be deposited continuously and as near as possible to its final position without objectionable segregation (Kosmatka and Wilson 2011). Concrete should be deposited in areas free of standing water. However, in some applications such as drilled shafts standing water may be present. In such applications, concrete should be placed in a manner that it displaces the water ahead of the concrete without mixing with the concrete. Pumps and tremies with ends buried in the fresh concrete can be used to maintain a seal below the rising surface. Pumps are widely used to place concrete in bridge structures. Care should be exercised in delivering with the pump since free fall within the pump hose can result in loss of slump and air.

3.3.2 Protection of Reinforcing Steel against Corrosion

Methods for protecting reinforcing steel elements from corrosion include the use of corrosion-resistant reinforcing steel and the use of non-intrinsic protection of the reinforcement (such as admixtures, cathodic protection systems, and electrochemical chloride extraction techniques) as described in the following. Chapter 5 also covers the cathodic protection system and electrochemical chloride extraction in additional detail.

3.3.2.1 Corrosion-Resistant Reinforcement

Corrosion-resistant reinforcement allows chlorides to penetrate around the reinforcement without causing significant damage to the reinforcing steel. These systems include epoxy-coated reinforcement, galvanized reinforcement, titanium reinforcement, stainless steel reinforcement, stainless steel clad reinforcement, nickel-clad reinforcement, and copper-clad reinforcement. Following is a description of the protection expected from these materials.

3.3.2.1.1 Epoxy-Coated Reinforcement (ECR)

Tests have shown that the diffusion rates of oxygen and chloride ions through a quality coating of adequate thickness 177 μm (7 mils) are extremely low, even in severe exposure conditions (Clifton et al. 1974; Pike 1973). However, epoxy-based coatings are not impermeable to water (Lee and Neville 1967).

ECR extends service life; however, experience in Florida and Virginia has shown that epoxy coating on ECR naturally degrades in the highly alkaline moist environment within concrete (Weyers et al. 2006). The estimated service life benefit of ECR may be as little as 3 to 5 years (Weyers et al. 2006). Today, structures are designed for service lives of 75 or 100 years, but the long-term protection provided by ECR is uncertain (Hartt et al. 2009). The improvements observed by some and attributed to ECR have coincided with other improvements such as reduced w/cm and increased cover.

3.3.2.1.2 Galvanized Reinforcement

Zinc coatings have a higher chloride (Cl^-) corrosion threshold (2 to 4 times) than that of uncoated steel, significantly extending the time until corrosion initiation (Yeomans 2004). Once corrosion of the zinc does occur, the properties of the corrosion products and their ability to migrate into the concrete matrix reduces stress generation in the surrounding concrete, further extending the life of the reinforced concrete structure.

One major drawback in the use of galvanized bar is the uncertainty that exists due to the effect of galvanizing on the brittleness of bars of different composition and with different degrees of work hardening. Some individuals believe it is preferable to bend cold-worked bars after galvanizing, even at the risk of damaging the zinc coating. It is generally easier and more economical to galvanize straight lengths of reinforcing bars completing all fabrication after galvanizing.

3.3.2.1.3 Titanium Reinforcing Bars

McDonald et al. (1995) found the corrosion rate of titanium bars to be 14,000-times less than that of black steel in a neutral solution and 135-times less than that of black steel in a high pH salt solution. The study estimated that titanium bars would extend the time-to-corrosion related cracking of concrete by about 130 times.

3.3.2.1.4 Stainless Steel Reinforcement

Chloride threshold values for stainless steel have been reported to be at least 10.4-times greater than for carbon steel (Clemeña 2003).

Solid stainless steel is preferred over stainless steel-clad carbon steel in Europe because the process of fusing the two types of metal together is not considered to be cost-effective. Another advantage of solid stainless steel bars is that they can be shipped, handled and bent without fear of damage to the coating (Smith and Tullmin 1999). In addition, the ends do not have to be coated after cutting (Russell 2004). Figure 3.12 shows disbondment after cutting.

Stainless steel is often used in areas in which sufficient cover cannot be obtained or at construction joints and critical gaps between columns and decks. Because of stainless steel cost, estimated to be four to six times more than black bar, it is not expected to be a standard for all reinforcement. Darwin et al. (2002) compared the costs of different types of reinforcement in a thick deck and reported that the initial cost of deck area for stainless steel was approximately 1.4-times more expensive than conventional reinforcement. However, based on total costs over 75 years, the stainless steel reinforcement was more economical.

The FHWA study *Corrosion Evaluation of Epoxy-Coated, Metallic-Clad, and Solid Metallic Reinforcing Bars in Concrete* (McDonald et al. 1998) examined two types of solid stainless steel in concrete exposure specimens, ASTM A955 Types 304 and 316. The results show that the lowest corrosion rates for Type 304 bars were obtained when the

stainless steel was used in both bridge deck mats. Cracks in the concrete did not appear to affect the performance of the stainless steel bars. Half of the bars from specimens that contained black steel in the bottom mat exhibited moderate to high corrosion currents and had red rust on them. When the stainless steel bars were used in both mats, the specimens did not exhibit any signs of chloride-induced corrosion, even when the slabs were pre-cracked. Both conditions (cracked and uncracked concrete) had about 1,500-times lower corrosion rates than black steel specimens. Corrosion currents on bars with black bottom mats were 20- and 700-times lower than for black steel specimens, for bent and straight bars, respectively.

All of the specimens containing Type 316 solid stainless steel showed good corrosion performance. There was no distinguishable difference between pre-cracked and uncracked slabs or between slabs with a black steel cathode or a stainless steel cathode. Measured corrosion for all conditions was about 800-times lower than that of the black steel specimens. During visual inspection of the slabs, only one of the bars exhibited corrosion and it was considered to be minor.

Figure 3.12. Stainless steel-clad reinforcement (disbondment after cutting). (Deshpande et al. 2000)

3.3.2.1.5 Nickel-Clad Reinforcing Bars

Nickel claddings for steel reinforcing bars were first used in the late 1960s. The corrosion resistance of nickel in alkaline chloride solutions is high, and although steel is less noble than nickel, the corrosion of the underlying steel is not substantially accelerated if breaks occur in the barrier (Tripler et al. 1966). Although research has been promising so far, steel bars with a nickel cladding of ample thickness to prevent corrosion damage are relatively expensive (Virmani and Clemeña 1998).

3.3.2.1.6 Copper-Clad Reinforcement

Copper-clad reinforcing bars were compared to black steel in concrete with and without calcium nitrite corrosion inhibitors and to non-specification epoxy-coated bars. The non-specification bars had been coated for an earlier study and stored outdoors for over two years; the bars used in the study all contained more than 25 holidays per foot and failed the ASTM bend test. Visible damage to the epoxy-coatings was estimated at less than 0.05% of the surface area. The copper-clad bars were not discussed in the FHWA report, but the results were published by McDonald et al. (1996). These results indicate that the copper-clad bars, with a coating thickness of about 0.5 mm (0.02 in.), exhibited much better corrosion resistance than the other types of reinforcement in the study, even the black steel with calcium nitrite corrosion inhibitor. Slabs containing copper-clad bars in the top mat only and in both mats were still in good condition after 13 years of outdoor exposure with no visible cracks. The average total chloride contents in the slabs were 8.50 to 10.32 kg/m^3 (14.33 to 17.40 lb/yd^3), which is well above the corrosion threshold level of steel. Copper is more noble than steel and could lead to corrosion of steel exposed in defect areas.

Tests up to this time (2012) have shown excellent corrosion resistance for copper-clad reinforcing bars in concrete and they could prove to be cost effective. However, before copper-clad reinforcement can be put to use in any structures, research needs to be performed on the structural effect of the retardation of cement hydration that is caused by these bars (McDonald et al. 1996; Virmani and Clemeña 1998).

3.3.2.2 Non-Intrinsic Corrosion Protection of Reinforcement

3.3.2.2.1 Admixtures for Corrosion Protection

Chemical admixtures that are added to concrete during batching to protect against corrosion of embedded steel reinforcement due to chlorides are available. There are two main types: corrosion inhibitors and physical-barrier admixtures. Some corrosion inhibitors also act as physical barrier admixtures (ACI 222.3R, 2003).

Corrosion inhibitors, although known for many years, are only now beginning to be actively marketed as a preventative treatment in a concrete repair program (Macdonald 2003). Calcium-nitrite admixtures are the most researched inorganic inhibitor and the most widely used (Berke and Rosenberg 1989). Inhibitors do not create a physical barrier to chloride ion ingress. Rather, they modify the steel surface, either electrochemically (anodic, cathodic, mixed-inhibitor) or chemically (chemical barrier) to inhibit chloride-induced corrosion above the accepted chloride-corrosion threshold level. They are added to the concrete at the time of batching (ACI 201.2R, 2008). Calcium nitrite is also an accelerating admixture. If the accelerating effect is undesirable, a retarding admixture can be added.

There are also admixtures with organic compounds which protect steel from chloride-induced corrosion. They include alkanolamines and an aqueous mixture of amines and fatty-acid esters (Nmai et al. 1992; Nmai and Krauss 1994). They are claimed to both reduce ingress of chlorides (physical barrier) and enhance the passivating layer on the steel surface (corrosion inhibitor). Other similar amine products are claimed to migrate through concrete in the vapor phase to provide protection to embedded steel. Corrosion inhibitors are attractive from a conservation point of view as they are almost invisible on application, although their long-term visual effect is unknown (Macdonald 2003).

Physical-barrier admixtures reduce the rate of ingress of corrosive agents (chlorides, oxygen, and water) into the concrete. These admixtures belong to one of two groups. One group comprises waterproofing and damp-proofing compounds. The second group consists of agents that create an organic film around the reinforcing steel, supplementing the passivating layer. They typically contain bitumen, silicates and water-based organic admixtures consisting of fatty acids, such as oleic acid; stearic acid; salts of calcium oleate; and esters, such as butyloleate (ACI 222R 2001). A liquid admixture containing a silicate copolymer in the form of a complex, inorganic, alkaline earth may also be effective in reducing the permeability of concrete and providing protection against corrosion of reinforcing steel (Miller 1995).

3.3.2.2.2 Cathodic Protection

Cathodic protection can effectively stop corrosion in contaminated reinforced concrete structures and can reduce the concentration of chloride ions at the steel surface of protected reinforcement (Kepler et al. 2000).

An advantage of using cathodic protection as a repair method for reinforced concrete bridges is that only spalls and detached concrete need to be repaired. Chloride-contaminated concrete that is still sound can remain in place because the cathodic protection system will prevent further corrosion and will, in fact, reduce the concentration of chloride ions adjacent to the protected reinforcing bars. This can significantly reduce repair costs (Polder 1998).

Both impressed current and sacrificial anode, or galvanic systems, have been used successfully on bridges in the United States. Impressed current systems are used most often on bridge decks, but there are some impressed current anodes that can be used on bridge substructure members as well. Historically, the use of galvanic anodes has been generally limited to substructure members (Kepler et al. 2000).

The wide use of cathodic protection has been hampered by the high cost and the maintenance of the power source or the protective material.

3.3.2.2.3 Electrochemical Chloride Extraction (ECE)

Electrochemical chloride extraction is applied to concrete structures containing reinforcement in order to extract chlorides from the concrete. ECE involves the application of current (1A/m^2 of steel) typically over a 4- to 8-week period. The steel acts as a cathode and is connected to the negative pole of the power source. The anode, which is either steel or a titanium mesh, is temporarily placed on the concrete cover and is connected to the positive pole of the power source. The electrolyte is placed on the concrete cover and allows the current flow. It is important to provide a buffer in order to keep a constant pH if titanium mesh is used. Due to the electric field, the negative ions, like chlorides, migrate from the rebar to the concrete cover. The electrochemical chloride extraction, using a current density of 1A/m^2 of steel for eight weeks (total charge 1344 A.h/m^2), removes about 40% of the initial free chloride on the average concrete

With ECE, the possibility of initiating the alkali-silica reaction as a side effect exists, as alkali metal ions are rearranged and hydroxyl ions are generated at the reinforcement. The pH necessary to initiate alkali-silica reaction is not known, but it can be assumed that if a structure is suffering from corrosion due to saltwater ingress, the concentration of sodium ions may be above the threshold for sustaining alkali-silica reaction (Kepler et al. 2000). Studies have indicated that the alkali-silica reaction can be mitigated by lithium (Velivasakis et al. 1997).

3.3.2.2.4 Other Protective Methods

Other protective methods include the use of drainage design, and stay-in-place metal forms. Posttensioning of members would also eliminate the formation of cracks.

3.4 FACTORS INFLUENCING SERVICE LIFE USING FAULT TREE

3.4.1 Service Life of Concrete

Concrete deterioration is caused by deficiencies in one or more factors listed as materials, design, and workmanship; the effect of external factors; and the occurrence of cracks. These factors are shown in the main fault tree (Figure 3.13) and are explained in additional detail in the fault trees for each factor, Figures 3.14 to 3.19.

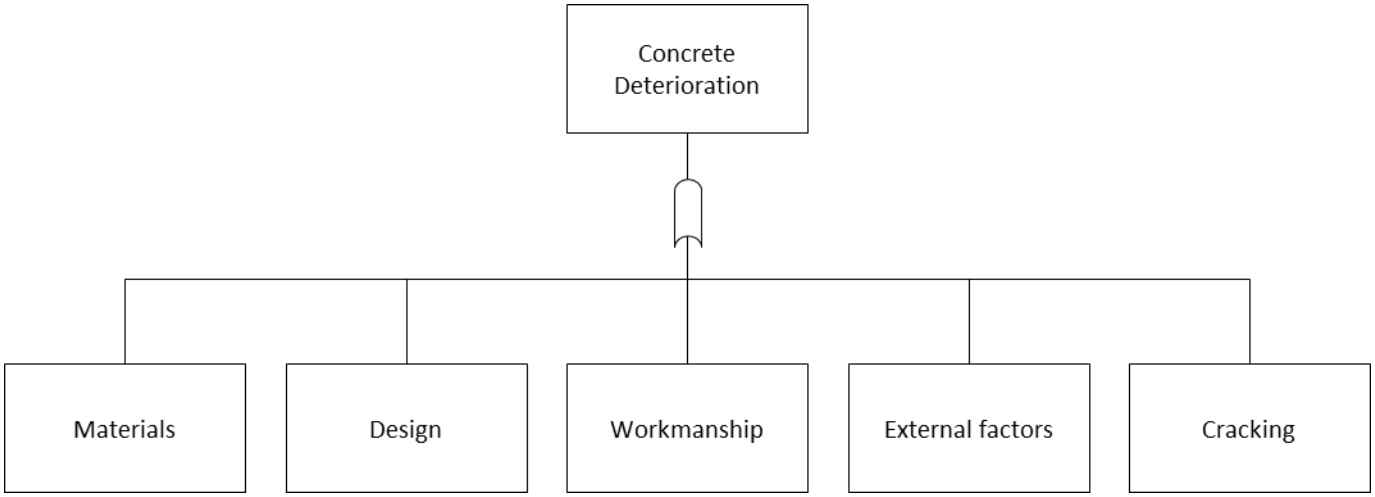


Figure 3.13. Concrete reduced service life main fault tree.

3.4.1.1 Material

Material-related factors are shown in Figure 3.14. They include the ingredients of the concrete—the cementitious material, aggregates, water, admixtures, and fibers. The water-cementitious materials ratio (w/cm) is also included in this module. The information on materials is given in Section 3.2 under description of material types. The cementitious materials have different chemical and physical properties that affect the hydration reaction, harmful chemical reactions, and volumetric changes that relates to the durability of concrete. The type, quality, grading, and texture of the aggregates affect the water content and durability. Aggregates may cause D-cracking, ASR, and ACR; therefore, precautions need to be taken in order to inhibit such occurrences. Water, whether it is potable or recycled, may have some excessive impurities that may cause durability problems due to expansive chemical reactions and increased w/cm. Small solid particles in water have large surface area; they increase the water

demand and the w/cm if the cement content is kept the same. Admixtures help in achieving workable, low w/cm, and low permeability concretes leading to improved durability. Fibers are used to control cracking.

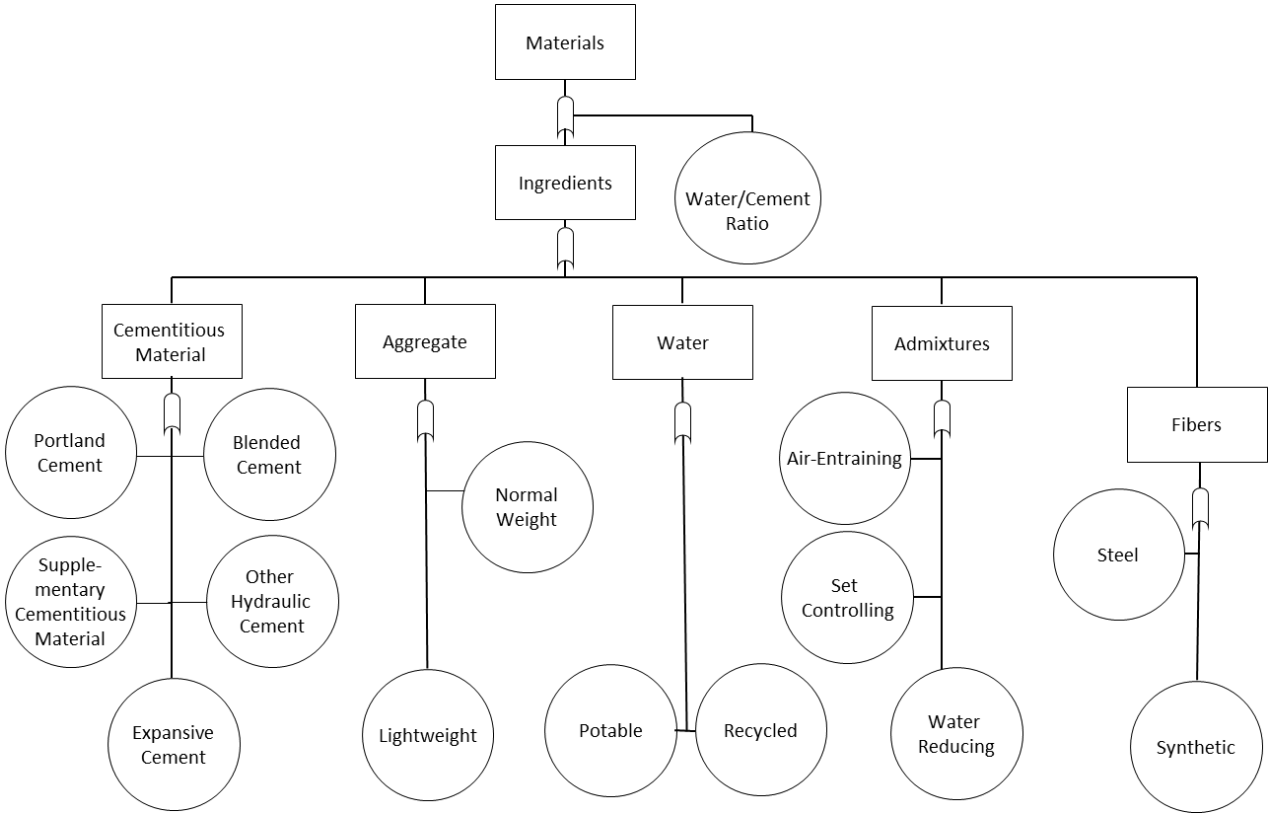


Figure 3.14. Materials fault tree.

3.4.1.2 Design

Design of the structure through the selection of the geometry, detailing, and flexibility affects the performance of the concrete. The design fault tree is shown in Figure 3.15. In the geometry, the cover depth over the reinforcement has a large influence on the salt intrusion to the level of steel. Thicker decks provide more rigidity and less cracking potential. Long-span length causes more flexibility, increasing the possibility of cracking.

Design details which minimize saturation with water would lead to improved durability. Eliminating joints is desirable since joints cause leakage problems.

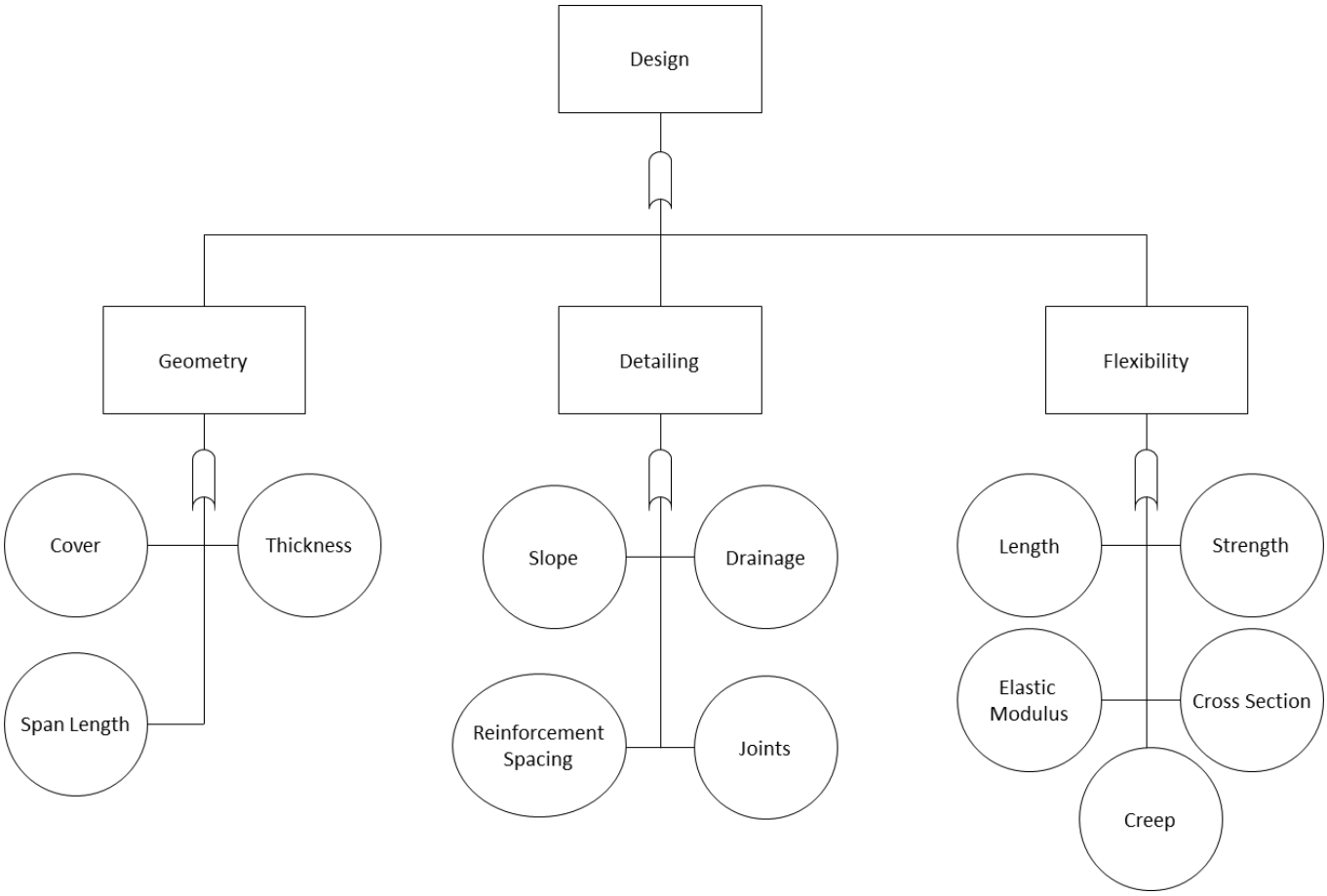


Figure 3.15. Design fault tree.

3.4.1.3 Workmanship

Workmanship is important in achieving the desired durability; the materials have to be fabricated properly under adequate inspection to achieve the desired performance. Frequent maintenance to correct deficiencies is also needed to eliminate premature failure of the elements. Fabrication of components includes mixing, consolidation, finishing, and curing as shown in the workmanship fault tree, Figure 3.16.

The concrete mixer should be in good working order, the capacity given by the manufacturer should not be exceeded, and enough mixing time at the specified mixing rate should be maintained. Following the proper mixing guidelines will ensure a uniform consistent concrete mixture. Consolidation, which is achieved by proper internal or external vibration, eliminates the large air voids that adversely affect the strength and durability of concrete.

Bridge decks require minimal finishing operations. The vibratory screed with proper speed, vibration, and set-up of the auger and the roller, can provide adequate vibration if the workability of the concrete is adequate. The sides and ends that the vibratory screed cannot reach are finished by hand. Extra hand finishing is detrimental and may

delay the curing operation and cause loss of entrained air voids near the top surface. Curing is essential for the continuation of the hydration reactions and the control of cracking due to volumetric changes. The best curing process is a water cure that enables moisture retention and temperature management. Curing compounds are also used to maintain the satisfactory moisture and temperature. Virginia DOT uses curing compounds after a seven-day wet curing of decks.

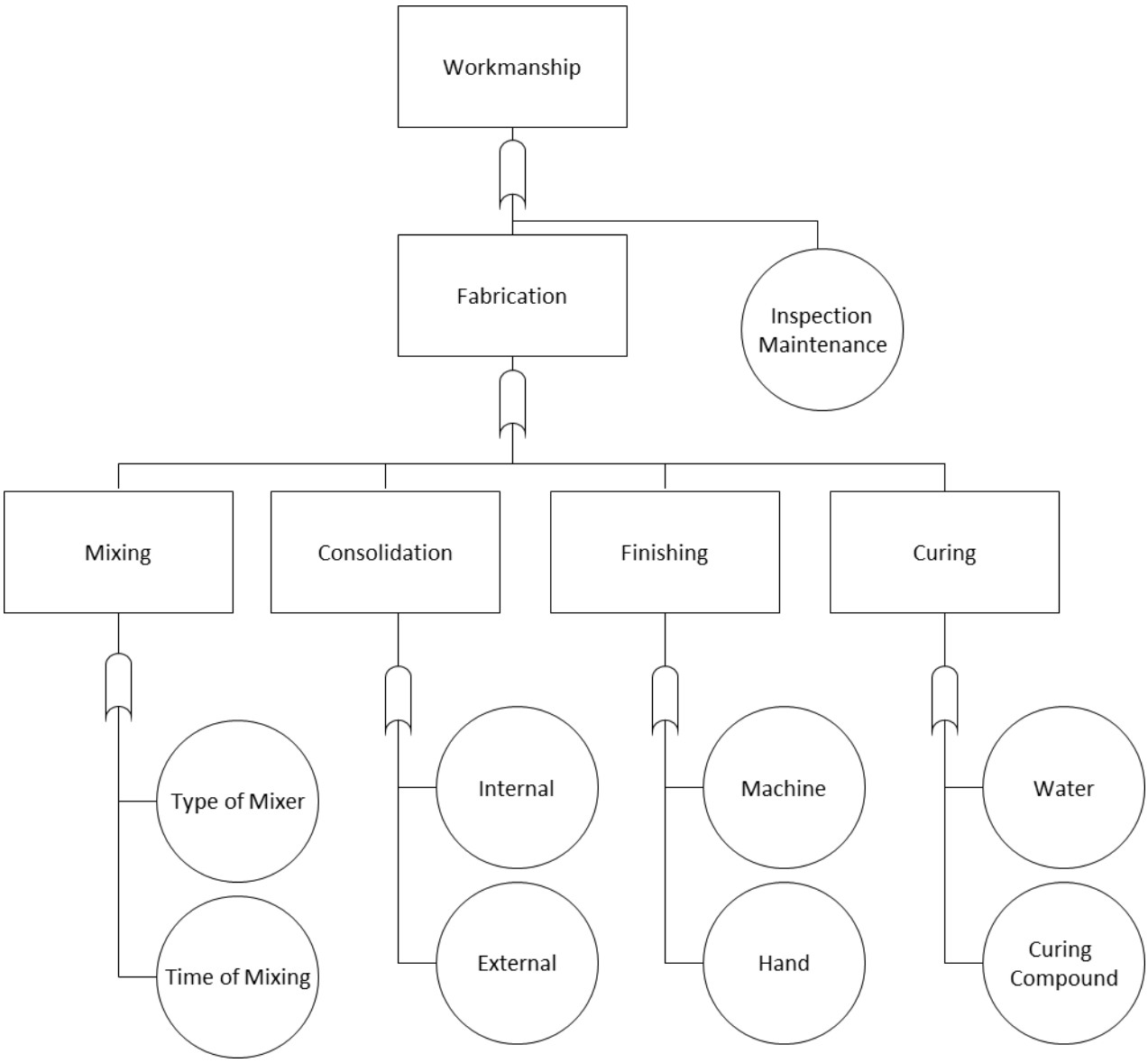


Figure 3.16. Workmanship fault tree.

3.4.1.4 External Factors

External factors that affect the service life of concrete are loads and the environment. Loads are illustrated in Figure 3.17, and the environment in Figure 3.18.

3.4.1.4.1 Loads

Loads can be traffic-induced, age-dependent, or system-dependent. Traffic-induced loads are a result of the vehicle loads and the frequency of traffic imparting fatigue stresses, overloads due to unexpected high loads. The adverse effect of loads are wide and frequent cracks that facilitate the intrusion of harmful solutions. The duration of loading has to be considered, as loads can be instantaneous or time-dependent. Time-dependent loads cause additional distress and result in additional cracks. System-dependent loads are a result of moisture and temperature variation and the available restraint. When restrained, the deformations lead to stresses that can exceed the strength of the material leading to cracking. When cement reacts with water, an exothermic reaction takes place, temperature rises and heat is given off, and concrete expands. As the reaction slows, cooling takes place and thermal contraction occurs subjecting the restrained concrete to tensile stresses. When the stresses exceed the strength of the material, cracks occur. This thermal effect is more pronounced in mass concrete since the dissipation of heat is difficult in a large mass. High heat can also result in delayed ettringite reaction that can cause cracks in hardened concrete. In hot weather, thermal effects can be detrimental; however, in a cold environment, the heat of hydration can provide the favorable temperature needed for the hydration reactions.

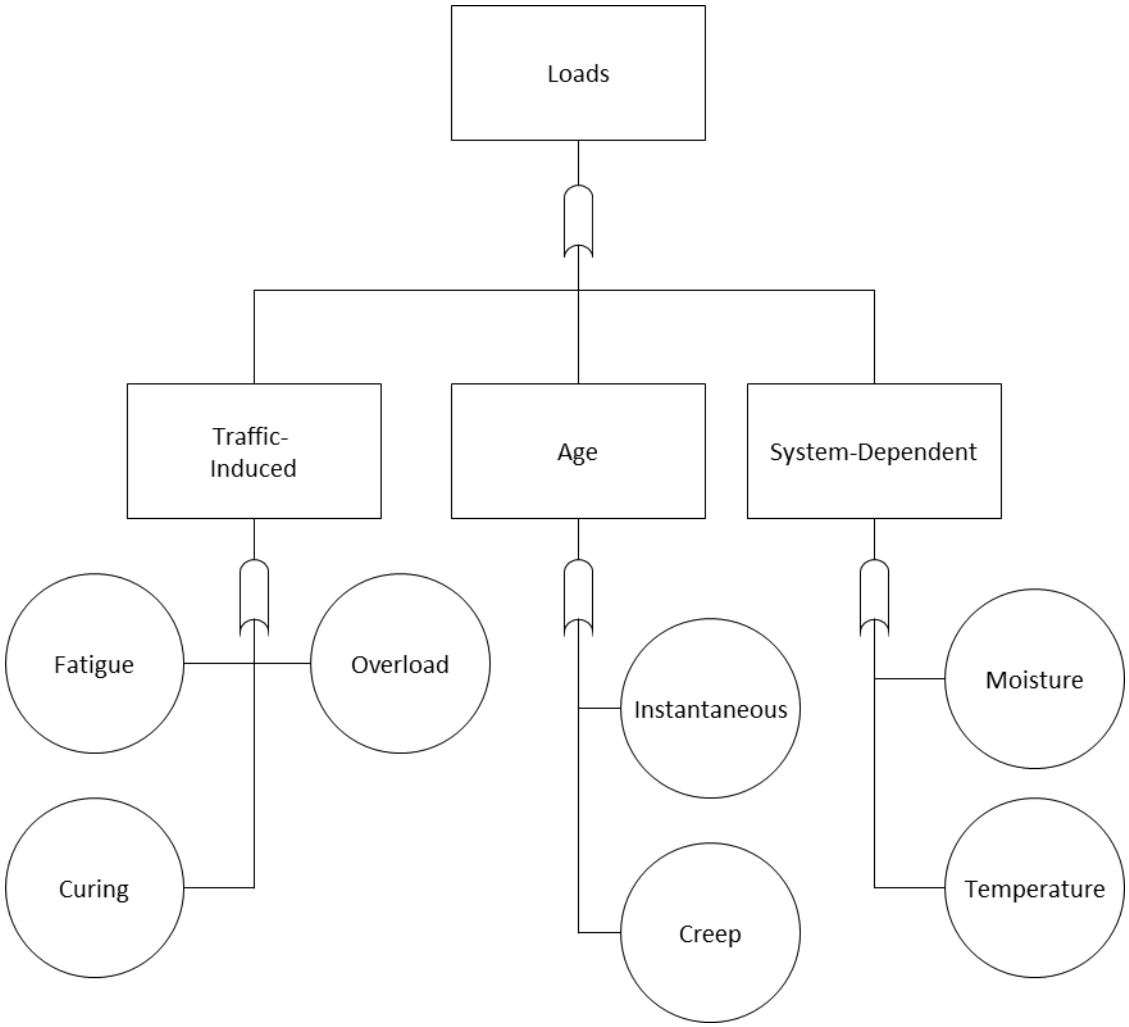


Figure 3.17. Load fault tree.

3.4.1.4.2 Environment

Environment can provoke damage in structures caused by physical and/or chemical factors. The physical factors are the result of freezing and thawing of critically saturated concrete, scaling of surfaces due to salt concentrations or excessive wear, seismic activity, settlement, or volumetric changes due to moisture and temperature variation, or stresses due to wind velocity. Chemical factors involve corrosion; carbonation, which reduces pH and makes steel vulnerable to corrosion; sulfate attack; or expansion due to alkali-aggregate or alkali silica reactions.

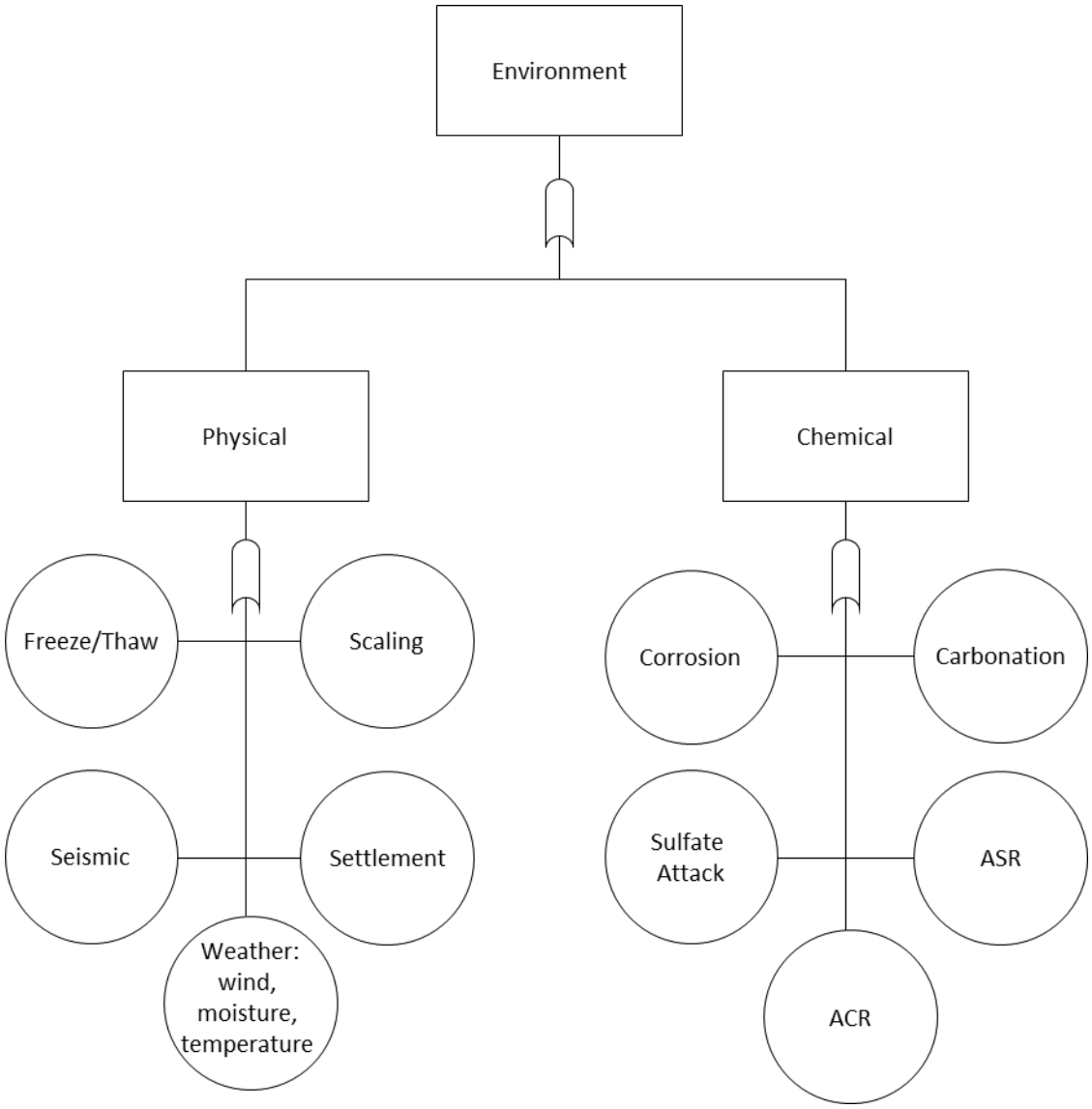


Figure 3.18. Environment fault tree.

3.4.1.5 Cracking

Cracking of concrete can cause serious and costly damage to concrete structures. The cracking fault tree is shown in Figure 3.19. The factors affecting cracking can be due to design, materials, and/or construction. In the case of design, the factors of interest include the restraint, span type, deck thickness, girder type, or the steel alignment and location. When concrete is restrained, deformations result in stresses. Flexible structures and low rigidity lead to additional cracks. For example, decks on steel beams exhibit additional cracking compared to decks on rigid concrete beams. High modulus and low creep that can be beneficial in reducing prestress losses and deflections in beams, can lead to additional cracking in decks. High modulus leads to brittle structures and low creep does not enable relaxation that reduces stresses. It can be expected that low water-cementitious material ratios, high paste

contents, and high heat of hydration can cause additional cracking. Concretes with low w/cm are brittle and more sensitive to curing. Concretes with high water, cement, and paste content exhibit additional shrinkage and additional temperature rise During construction and afterwards, the weather conditions, curing, time of setting, consolidation, and curing sequence and length affect the cracking pattern and severity.

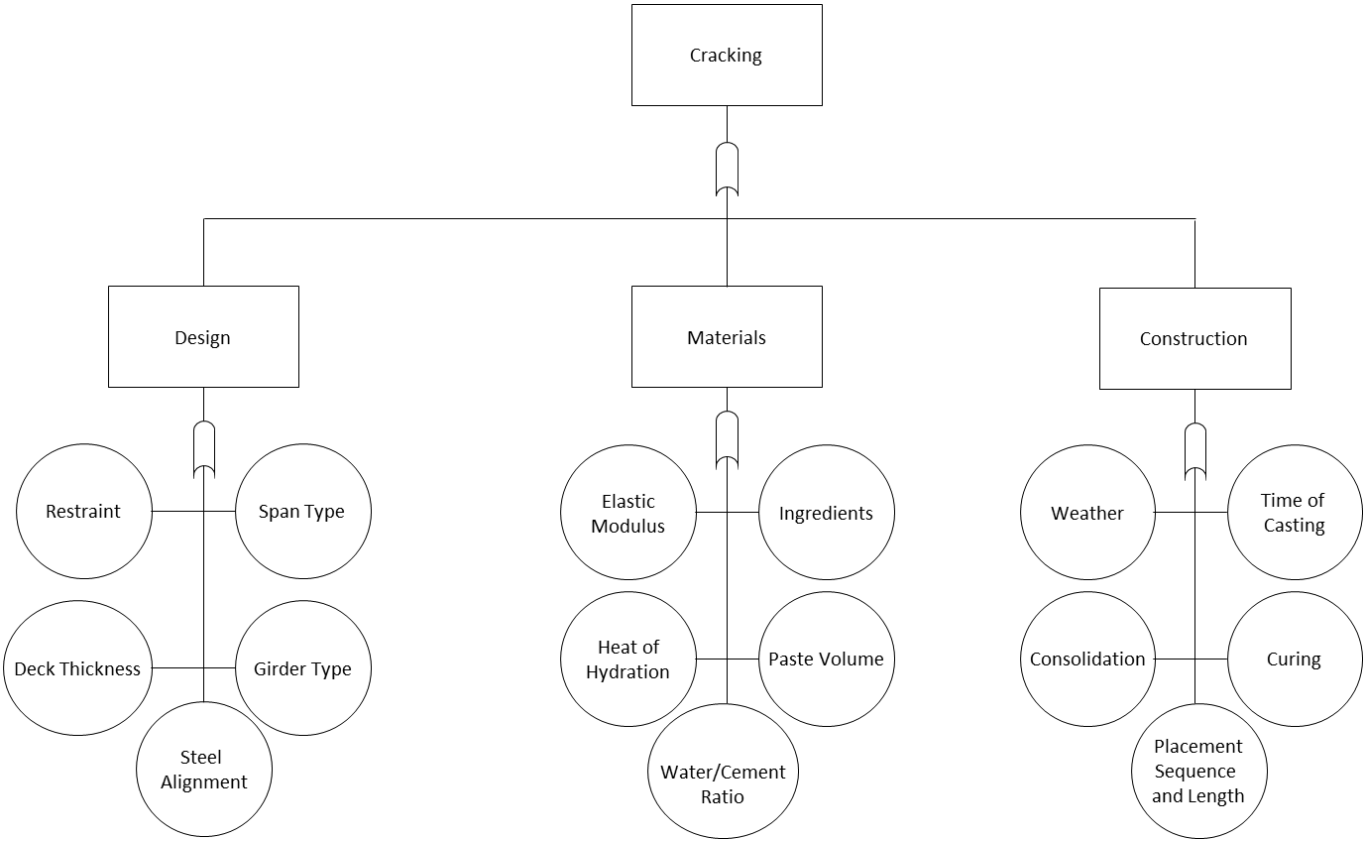


Figure 3.19. Cracking fault tree.

3.4.2 Service Life of Reinforcement

Reduced service life of reinforcement can be attributed to three causes: load-induced, man-made or natural hazards; causes resulting from production defects in construction processes and/or design details; or operational procedures. These deficiencies are illustrated in the main fault tree shown in Figure 3.20.

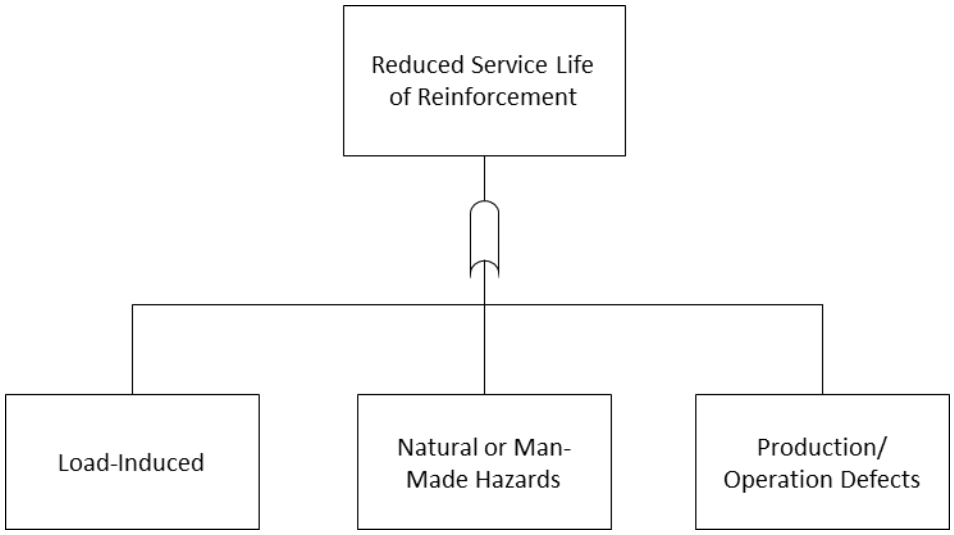


Figure 3.20. Reduced service life main fault tree.

3.4.2.1 Load-induced

Load-induced bridge deck deterioration can be attributed to fatigue, strength and brittleness, or thermal incompatibility. These load-induced factors are introduced in the fault tree provided in Figure 3.21.

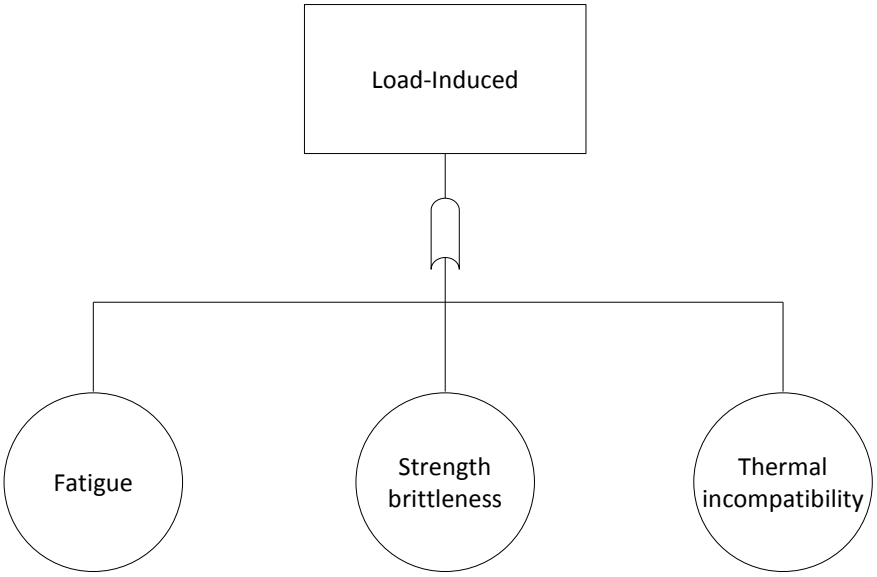


Figure 3.21. Load-induced deficiency fault tree.

Fatigue is caused by the repetition of applied loads that result in a degradation of the strength resistance of the reinforcement. Information on reinforcement material is summarized in Section 3.2.2. Corrosion-resistant reinforcement (CRR) has fatigue properties similar to that of carbon steel reinforcement when tested in the atmosphere. However, in a corrosive environment, CRR performs better than the carbon steel since carbon steel is

expected to corrode, lose material area, and develop corrosion pits. The fatigue limit is related to the tensile strength of the steel, hence CRR with increased strength has increased fatigue limit.

3.4.2.1.1 Strength and Brittleness

Strength and brittleness affect cracking and failure. In general, CRR has higher tensile strength and ductility than the carbon steels. Cold-formed austenitic reinforcement has a combination of high strength and good ductility; a yield stress level of 70 ksi or higher and elongation at maximum force higher than 15% is achieved (Bourgin et al. 2006). Duplex grades exhibit strengths exceeding 70 ksi in hot-rolled and 90 ksi in cold-rolled bars. At the higher strengths, ductility is at least as good as the carbon steels.

3.4.2.1.2 Thermal Compatibility

Thermal compatibility affects cracking potential. Temperature changes in a material result in deformations that can cause significant stress when restrained by the surrounding material. However, in reinforced concrete the carbon steel and concrete have similar coefficients of thermal expansion leading to negligible stresses due to temperature changes in the structure. Carbon steels have a coefficient of thermal expansion (CTE) of about 5.5 x 10^6/ºF; CRR with austenitic steels have a CTE of approximately 8.9 x 10^6/ºF; and the austenitic-ferritic duplex steels have a CTE of about 7.2 x 10^6/ºF (Markeset et al. 2006). There has been no reported problem due to these differences.

3.4.2.2 Natural or Man-Made Hazards

Natural or man-made hazards include effects from areas with adverse thermal climate, coastal climates and chemical climates, and fire. These natural and man-made hazards are introduced in the fault tree provided in Figure 3.22.

3.4.2.2.1 Thermal Climate

Thermal climate affects the corrosion activity. In cold climates, chloride-bearing deicing salts are commonly used to prevent ice buildup on roads. Chlorides destroy the protective iron oxide layer over the carbon steels exposing the reinforcement to corrosion. Corrosion of prestressing steel is generally a greater concern than corrosion of nonprestressed reinforcement because of the possibility that corrosion may cause a local reduction in cross section and failure of the prestressing steel (ACI 222R, 2001). The typical higher stresses in the prestressing steel also render it more vulnerable to stress-corrosion cracking and to corrosion fatigue. Because of the potentially greater

vulnerability and the consequences of corrosion of prestressing steel, chloride limits for prestressed concrete are lower than those for reinforced concrete (ACI 222R, 2001).

Corrosion rate is dependent on the temperature, humidity, and chloride content. An increase in temperature in dry environments results in reduced corrosion activity and an increase in expected life (Lopez et al. 1993). However, in environments with high humidity, increasing temperatures result in increased corrosion activity leading to reduced expected life.

Low temperature, cryogenic applications, can cause brittle failure. Carbon steel reinforcement exhibits brittle behavior below 0°F when exposed to sudden loading and seismic actions. Austenitic stainless steels do not present such a transition; therefore, can be used in cryogenic applications; their toughness remains very high at temperatures as low as -320°F. Duplex stainless steel may not be used below -60°F (Markeset et al. 2006).

3.4.2.2.2 Coastal Climate

Coastal climate introduces salt spray and high humidity, and salt and moisture accelerates the corrosion rate. Salt spray in coastal climates provides high chloride buildup that can destroy the protective iron oxide coating over the steel reinforcement.

Humidity affects corrosion activity. In the absence of chloride ions, little corrosion activity takes place when the relative humidity is under 60% (Jung et al. 2003). The corrosion activity increases as the RH is increased up to a fully saturated state (>95% relative humidity) and then begins to decrease again. When concrete is fully saturated, the corrosion rate is reduced because the oxygen level in the concrete pores is too low (Qian et al. 2002). When chlorides are present at relative humidity below 60%, corrosion activity may still develop.

3.4.2.2.3 Chemical Climate

Chemical climate influences the performance of reinforcement. The main effect can be attributed to corrosion-inducing chemicals. These influences both occur naturally and can be man-made. Chlorides are the main chemical substance that adversely affects the corrosion process.

3.4.2.2.4 Fire

Fire generates heat that affects mechanical properties. Cold-worked steel subjected to temperatures of less than 850°F typically recovers all of its yield strength after cooling (Suprenant 1996). Hot-rolled steel can be exposed to

temperatures as high as 1,100°F and recover its yield strength. Higher temperatures may cause rapid strength loss in reinforcing steel and lead to excessive deflections in reinforced members. The effect of fire is more critical on prestressing steel; at temperatures of 750°F, the strength of prestressing steel can be reduced by more than 50% (Suprenant 1996).

The austenitic stainless steels maintain their strengths at considerably higher temperatures than carbon steel. Therefore, such steels are more resistant and robust under fire loading than carbon steel (Markeset et al. 2006).

Heating also adversely affects the bond between concrete and reinforcement (Suprenant 1996). At 570°F, the bond strength is no greater than 85%, and at 930°F no greater than 50% than the strength at ambient temperatures.

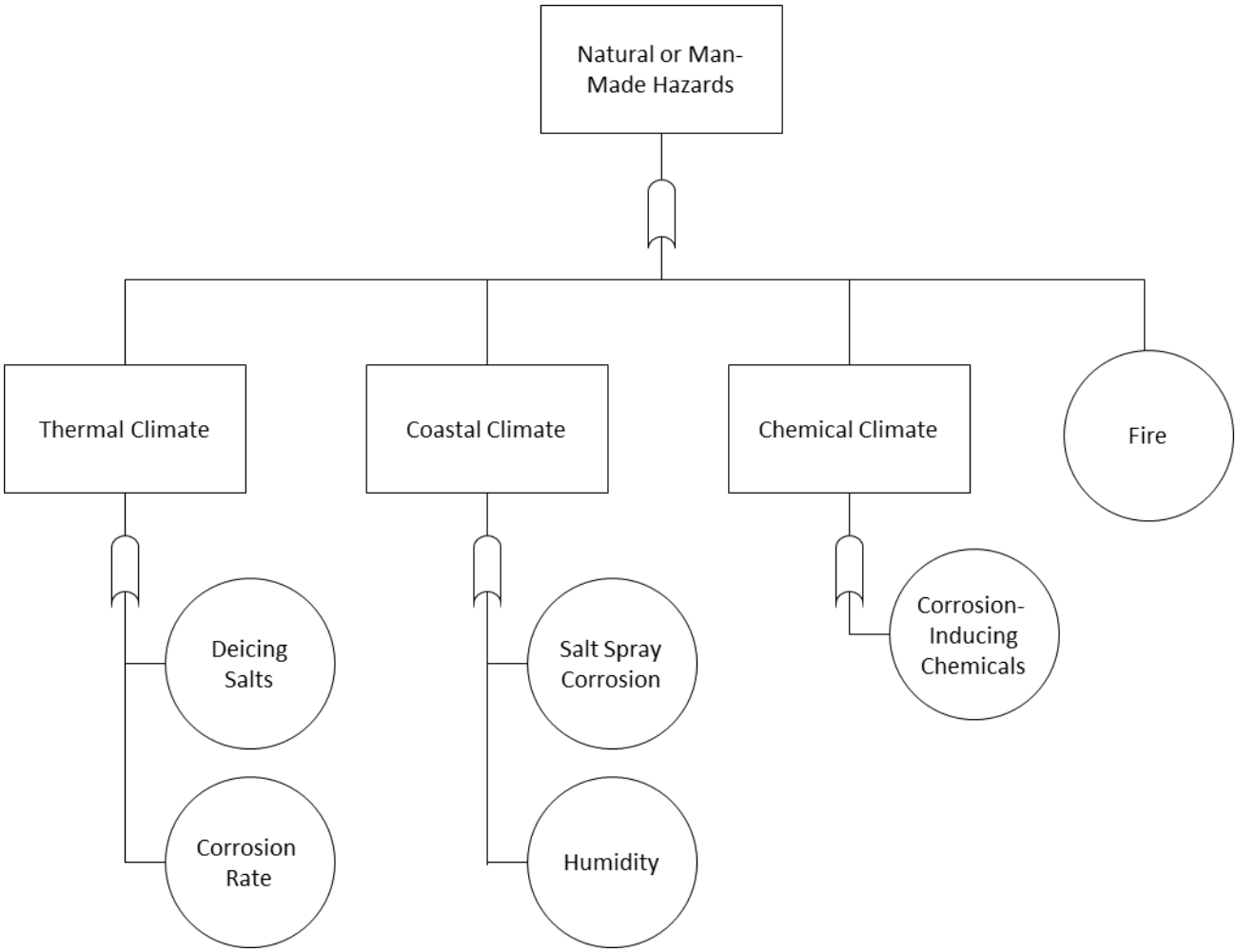


Figure 3.22. Natural or man-made hazard fault tree.

3.4.2.3 Production or Operational Defects

Production or operational defects are shown in the fault tree in Figure 3.23. They include design and detailing, construction, inspection, and maintenance issues.

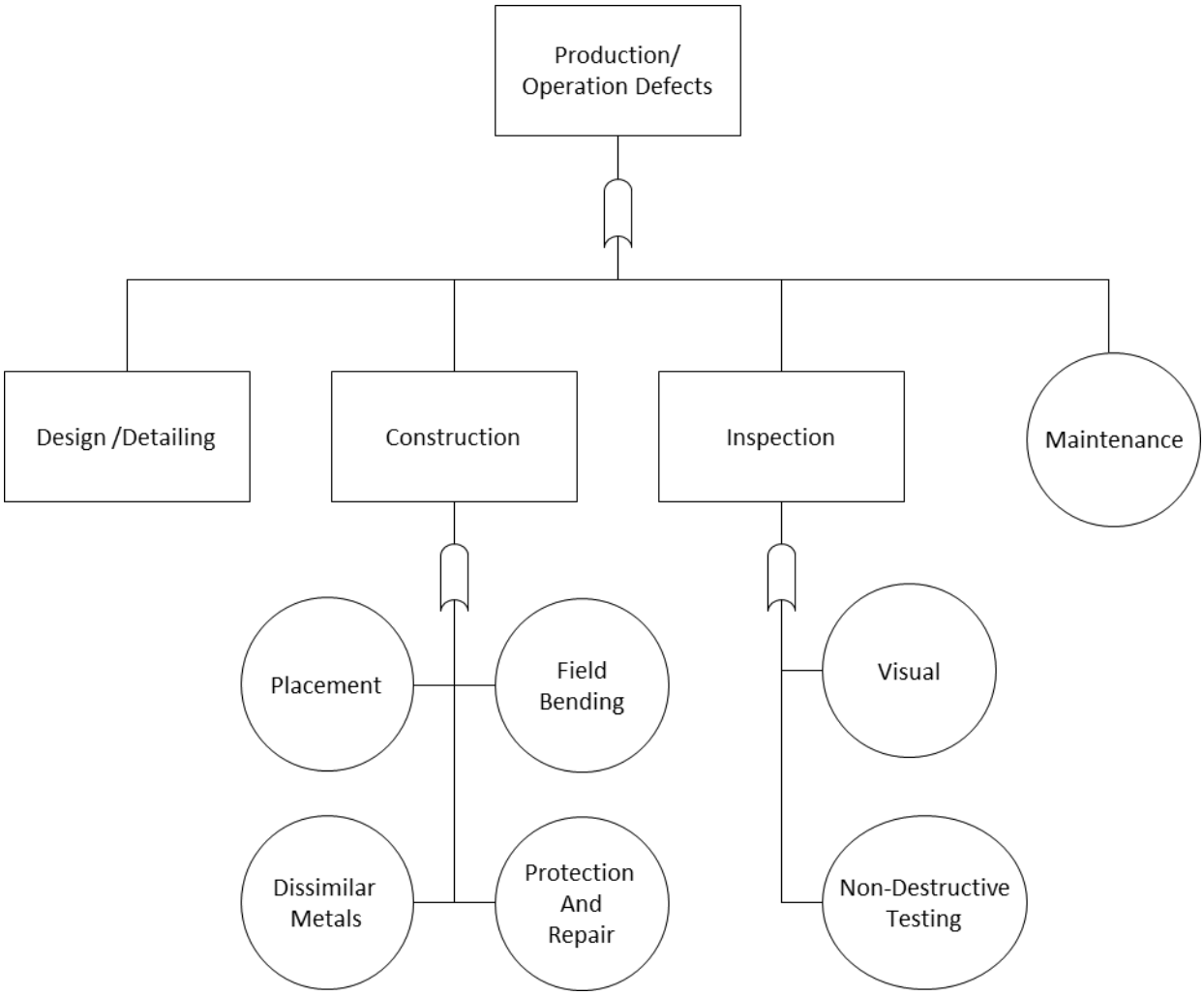


Figure 3.23. Production/operation defects fault tree.

3.4.2.3.1 Design and Detailing

Design and detailing factors are shown in the fault tree shown in Figure 3.24. They include the design philosophy, mix design, and drainage.

The design philosophy of providing proper concrete cover, eliminating joints, minimizing cracking through geometry (for example, large skews exhibit additional cracking), and the selection of corrosion resistant reinforcement affect the corrosion potential. The redundancy and ductility design aspects in structures should be improved to confine the damage to a small area in the event a major supporting element is damaged or an abnormal loading event has occurred (ACI 318-11). The following considerations and relationships should be adopted in the concrete mixture design:

- Low permeability concretes hinder the penetration of aggressive solutions to the level of reinforcement;

- High alkalinity of the concrete passivates the steel through its protective cover;
- Cracking resistance of concretes can be improved by optimizing the strength (high strength concrete exhibits brittle behavior), minimizing paste, and adding latex modifiers;
- The use of corrosion-inhibiting admixtures increases the passivation state of the reinforcement, extends the time to corrosion, reduces the corrosion rate of embedded metal;
- Increased creep and reduced elastic modulus and reduced shrinkage are helpful in reducing the cracking; and
- Cracks facilitate the intrusion of aggressive solutions into concrete; chlorides initiate and accelerate the corrosion process.

3.4.2.3.2 Construction

Construction-related parameters affect the performance of structures. It is critical that the correct amount of reinforcement is placed in the right location within the specified tolerances. Reinforcement during concrete placement should be free from mud, oil, or other nonmetallic coatings that decrease bond (ACI 318-11). A normal amount of rust or mill scale that is not loose on the reinforcement is not detrimental to the bond between the concrete and the bars.

In the field, bending to proper bend diameters is needed to ensure that there is no breakage and no crushing of the concrete inside the bend (ACI 318-11).

To minimize the corrosion potential, dissimilar metals should not be in contact. Further, reinforcement should be protected from the weather to minimize contamination and corrosion. If there is a coating over the reinforcement, special care is needed to avoid damage to the coating during handling and placement.

Visual inspection can indicate the condition of the reinforcement and determine if there are any gross mistakes in the reinforcement selection and placement. The availability of a large number of reinforcement types makes it difficult to identify the reinforcement visually; non-destructive evaluation (NDE) is beneficial in this respect.

Rebar corrosion in existing structures can be assessed by different methods such as (Song and Saraswathy 2007): open circuit potential (OCP) and surface potential, concrete resistivity, linear polarization resistance (LPR), tafel extrapolation, galvanostatic pulse transient method, electrochemical impedance spectroscopy (EIS), harmonic

analysis, noise analysis, embeddable corrosion monitoring sensor, cover thickness, ultrasonic pulse velocity technique, X-ray, gamma radiography, infrared thermograph, electrochemical method and visual inspection.

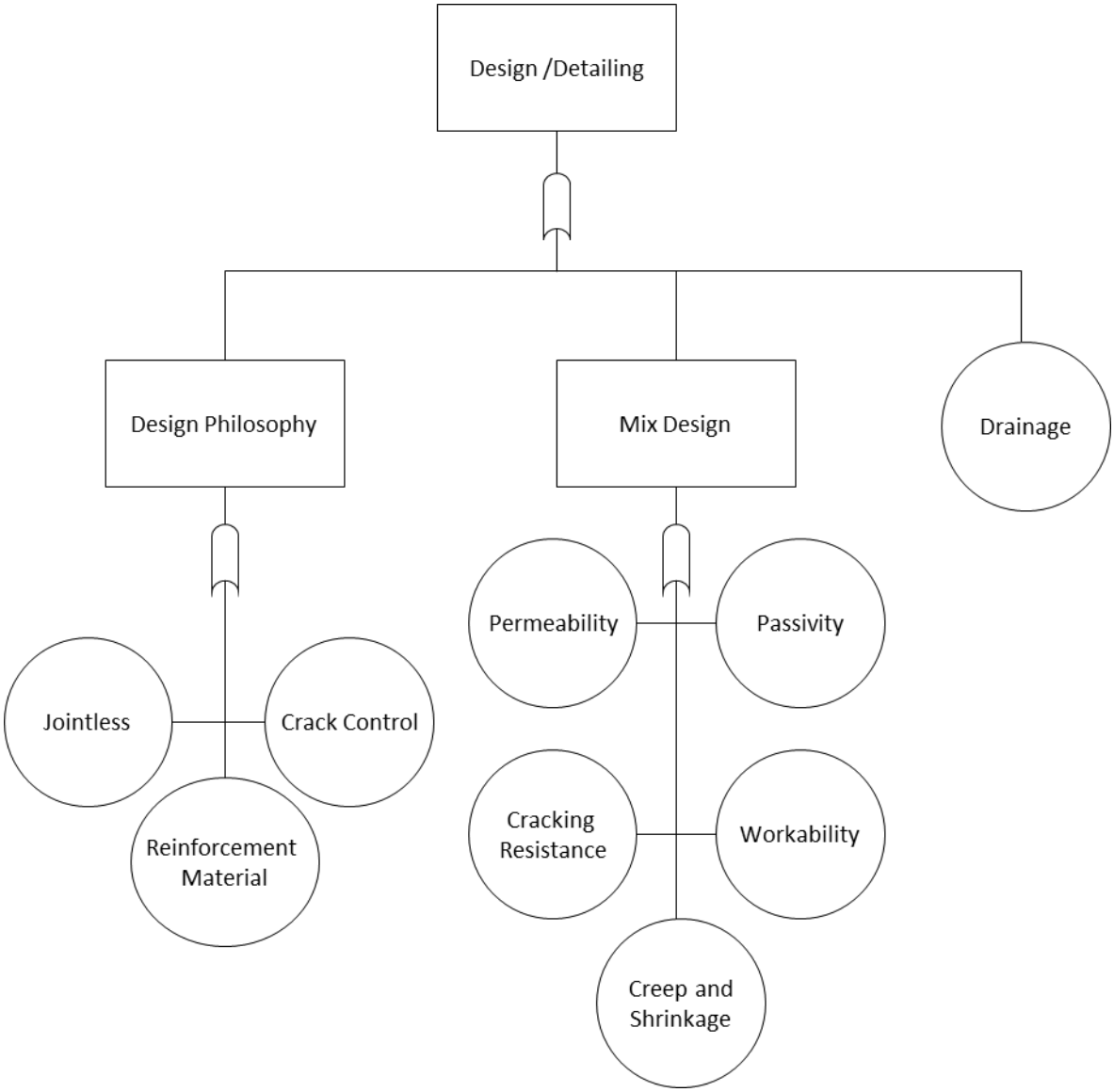


Figure 3.24. Design and detailing defects fault tree.

3.4.3 Service Life of Structural Steel

The primary factors affecting service life of structural steel are fatigue and fracture, and corrosion. These are covered in the following chapters:

- Chapter 7 - Fatigue and Fracture
- Chapter 6 - Corrosion Protection of Steel Bridges

3.5 INDIVIDUAL STRATEGIES TO MITIGATE FACTORS AFFECTING SERVICE LIFE

3.5.1 Concrete

The durability of concrete depends largely on its permeability. For longevity, concrete must be designed, proportioned, and constructed properly. In addition to the concrete material issues, the structural design must also be performed properly to avoid high stresses and load-related cracking. Section 3.3.1 provides additional information on distresses due to physical (volumetric changes, freezing and thawing), chemical (alkali-aggregate reaction [AAR], ASR, ACR, carbonation, chlorides, sulfates, acids, and salts), and functional (impact, concrete consolidation, curing, and placement) factors.

3.5.1.1 Technology Table for Concrete

The technology table that informs the designer of the most common types of service life issues related to concrete materials and is summarized in Table 3.9. For each service life issue, solutions are identified, along with their advantages and disadvantages. Technology table for concrete durability also includes concrete-related issues involved in resisting the corrosion of reinforcement such as low permeability, w/cm, aggregates, chemical admixtures, shrinkage, modulus of elasticity, cover, overlays, and corrosion inhibitors. The distresses commonly experienced are summarized in the following technology table.

Table 3.9. Technology Table for Concrete Durability.

Service Life Issue	Solutions	Advantage	Disadvantage
Freeze and thaw	Good air void system	High resistance to freezing and thawing	Reduction in strength due to extra air
	Sound aggregates	Durable aggregates	Availability
	Strength of 4,000 psi and up	Used to overcome stresses	Increased strength makes concrete more brittle
	Drainage design	Minimize saturation	Ingress of water
	Low w/cm	Reduce infiltration of water	Can produce high strength concrete that is brittle
Abrasion and Wear	Hard aggregates	Attain high concrete strengths and increased resistance to abrasion/wear	Hard to obtain in some areas
	High strength concrete	Reduce wearing	Concrete more brittle
	Add cover	Provides new surface	Extra weight

Table 3.9 Continued. Technology Table for Concrete Durability.

Service Life Issue	Solutions	Advantage	Disadvantage
Chemical reactions (ASR)	Non reactive siliceous aggregates	Reduce ASR	Hard to obtain in many areas
	Use of SCM	Reduce permeability, reduce ASR, limit alkalis from outside	Quality fly ash or slag missing in many areas
	Low w/cm	Reduce infiltration of solutions, limits alkalis from outside	Can produce high strength concrete that is brittle
	Chemical admixtures	Improved properties	Cost, incompatibility, side effects
	Lithium based admixtures	Inhibit ASR	Cost
	Limestone sweetening (blending with limestone)	Limit expansion	Reduced skid resistance
Chemical reactions (ACR)	Non reactive carbonate aggregates	Reduce ACR	Hard to obtain in some areas
	Reduce infiltration of solutions, limits alkalis from outside	Can produce high strength concrete that is brittle	Cracking
	Blend aggregate	Limit expansion	Hard to obtain in some areas
	Limit aggregate size to smallest practical	Limit expansion	Rich mixes with high paste content
Sulfate attack	Low C3A contents	Reduce sulfate attack	N/A
	Use of SCM	Reduce permeability, reduce sulfate attack, limit sulfates from outside	Quality fly ash or slag missing in many areas
	Low w/cm	Reduce infiltration of solutions, limits sulfates from outside	Can produce high strength concrete that is brittle

Table 3.9 Continued. Technology Table for Concrete Durability.

Service Life Issue	Solutions	Advantage	Disadvantage
Corrosion of reinforcement	Low permeability	Reduce infiltration of aggressive solutions	Can produce high strength concrete that is brittle
	membranes and coatings	Reduce infiltration of aggressive solutions	Difficult to apply in the field, wear of traffic
	Sealers for pore lining and blocking	Reduce infiltration of aggressive solutions	Difficult to apply in the field, concrete may be difficult to penetrate
	Use of low w/cm	High strength, low permeability	Excessive cracking, shrinkage
	Low shrinkage	Minimize cracking	Low water content may adversely affect workability
	Low modulus of elasticity	High deformation, minimize deck cracking	Reduce stiffness
	Use of SCM	Reduce permeability	Quality fly ash or slag missing in many areas
	Large max aggregate size	Less surface area, loess water, cement, and paste	Less bond
	Well graded aggregates	Less paste	Problem when good shape is missing
	Chemical admixtures	Reduced permeability	Cost, incompatibility, side effects
	Cover	More resistance to penetration of solutions	Wider cracks, extra weight and cost
	Overlays	Create a low permeability protective layer over the conventional concrete.	Difficult to place, expensive, and is prone to cracking, proper curing is critical.
	Corrosion Inhibitors	Stable protective layer on the steel	Cost

3.5.2 Steel Reinforcement

Corrosion of reinforcement is a major problem requiring costly repairs. There are several methods for protecting reinforcing steel elements from corrosion, such as the use of corrosion-resistant reinforcing steel, admixtures, cathodic protection systems and electrochemical chloride extraction techniques.

3.5.2.1 Technology Table for Steel Reinforcement

The technology table that summarizes the solutions to reinforcement corrosion is presented in Table 3.10. This table includes other protective methods mentioned above such as epoxy coated, Z-bar, low carbon chromium steel, and stainless steel.

Table 3.10. Technology Table for Corrosion of Reinforcement.

Service Life Issue	Solutions	Advantage	Disadvantage
Corrosion of reinforcement	Electrochemical chloride extraction	Extract chlorides from the concrete, or use in new structures to increase corrosion threshold	Extraction depends on the depth and location, risk of embrittlement (prestressed), difficult to predict service life
	Cathodic protection	Prevent corrosion from initiating, advantage as a repair method	High cost involved in maintaining the power source and sacrificial mesh anode. Embrittlement of strand and softening of concrete (prestressed structures)
	Sealers	Prevent solutions from penetrating the concrete, easy to apply either during or after construction	Difficult to ensure adequate coverage. Varying performance and cost. Short service life. Abrasion, sunlight and environment affect the sealer's efficiency
	Membrane	Prevent moisture infiltration	Varying performance. Difficult to install on curved or rough decks and to maintain quality and thickness during field installation.
Corrosion of reinforcement	Stay in place metal form for marines structures	Prevent infiltration of aggressive solutions	Cost
	Stainless steel	High resistance to corrosion	Initial cost
	FRP	High resistance to corrosion	FRP prone to degradation from environmental factors
	Z bars (galvanizing over epoxy coating)	High resistance to corrosion	
	Epoxy coated steel	Create protective layer over the steel and increase the electrical resistance	Epoxy coating can be damaged during handling, shipping and storage corrosion can initiate under the coating
	Low carbon chromium steel	High resistance to corrosion	High strength, no yield point
	Drainage design	Minimize saturation	Continuous maintenance
	Post Tension	Puts the concrete in compression minimizing cracks that facilitate the penetration of chlorides	Post tensioning ducts and grout are concerns in resisting corrosion

3.5.3 Structural Steel

Individual strategies to mitigate factors affecting service life of structural steel are discussed in Chapter 6.

3.6 OVERALL STRATEGIES FOR ENHANCED MATERIAL SERVICE LIFE

The introduction to this chapter described a process for developing a strategy selection to enhance material service life. This process is summarized in Figure 3.1.

Providing materials with enhanced service life requires a complete understanding of the potential deterioration mechanisms. These mechanisms, described in Section 3.3, are associated with load-induced conditions, local environmental hazards, production-created deficiencies, and lack of effective operational procedures. Mitigation of these deterioration mechanisms through the selection of enhancement techniques, described in Section 3.5, requires a thought process which combines the individual strategies to define a single family of symbiotic strategies. This will produce the best approach of providing materials with enhanced service life.

This chapter provides guidelines for selecting the most appropriate individual strategy to achieve the desired service life. While the individual strategies provide solutions to many of the material durability issues, the majority of the strategies must be developed in conjunction with its application, such as bridge decks. Subsequent chapters will reference this chapter where applicable.

3.6.1 Design Methodology

With limited funds available for bridge construction, cost is often an overriding factor in critical material selection decisions. However, to take advantage of the long-term advantages of durable materials, service life enhancement strategies must be applied to a cascading series of economic, design, construction, and maintenance measures. Success of the strategy selection process is dependent on the ability to predict service life and the incorporation of best practices to enhance service life.

3.6.2 Material Selection and Protection Strategies

The selection of the type of concrete for a particular application depends on many factors including the design of the structure (span length and slenderness of columns), availability of the type of concrete, subsurface conditions, and the environmental conditions (temperature, chemical exposures). Following are some examples. 1) If poor soil conditions exist and longer spans are planned or the substructure is to be kept but additional or wider lanes and shoulders are planned, lightweight concrete (LWC) would be the material of choice. 2) If there is severe exposure to salts or marine spray, high performance concretes with low permeability would be appropriate. 3) In areas with

congested reinforcement or intricate formwork, high performance concrete with high workability such as self-consolidating concrete (SCC) would be preferred. 4) In bridge decks, SCC can lead to difficulty in maintaining the grade or the cross slope due to high flow rates, then normal weight concrete (NWC) may be preferable unless durability or weight is of concern. 5) Ultra-high performance concrete (UHPC) can be used where very small cross sections or height restrictions exist or if high bond strengths and low permeabilities are needed as in connections.

Care should be exercised to select or specify only the necessary criteria for the subject application. Additional criteria can cause undesirable distresses that adversely affect performance and also increase the cost of construction. For example, for a bridge deck, if a low w/cm (less than 0.40) is specified to achieve lower permeability, high strengths will be obtained that would make the bridge deck concrete more prone to cracking. High strengths are accompanied by high stiffness (elastic modulus) and low creep that is instrumental in increased cracking potential. Cracks will facilitate the intrusion of chlorides negating the benefits obtained by low w/cm. A better approach would be to use moderate w/cm (0.40 to 0.45) with the pozzolanic material to reduce the permeability. In addition, to achieve a low w/cm, high cement factors are used that would increase the cementitious material and paste contents, thus making concrete more vulnerable to shrinkage and thermal problems.

Table 3.11 summarizes durability strategies for concrete materials. Selection of these strategies for each potential deterioration mode must be compared for conflicts in order to establish the overall strategy to be deployed. For example, a designer faced with a bridge deck having the potential for deterioration from wear and abrasion and differential shrinkage should not specify concrete with both high strength and a low modulus. In this case, using the overlay/membrane would be more appropriate.

The selection of appropriate material and protection strategies is highly dependent on the application of the materials. Therefore, the material selection and protection strategies considering the overall structure (not only materials) are provided in subsequent chapters.

Table 3.11. Concrete Durability Strategies.

Potential Deterioration Mode	Material Selection and Protective Measures Selection	Maintenance Modes	Life Cycle Costs	
			Initial	Long Term
Freeze and Thaw	Min. 6% Air Entrainment Sound Aggregates Strength > 3.5 ksi	None	Low	Low
	Proper Drainage and cover	None	Low	Low
	Membrane/Overlay	Continual Overlay Replacement every 20 years	Med	Med
ASR	Non-Reactive Aggregates	None	Med	Low
	Low Alkali Portland Cement	None	Med	Low
	Blended Aggregates Low Alkali Portland Cement SCMs (Fly Ash, Slag, etc.)	None	Med	Low
	Blended Aggregates Low Alkali Portland Cement Lithium Nitrate	None	Med	Low
	Proper Drainage	None	Low	Low
	Membrane/Overlay	Continual Overlay Replacement every 20 years	Med	Med
ACR	Non-Reactive Aggregates	None	Med	Low
	Blended Aggregates	None	Med	Low
	Proper Drainage	None	Low	Low
Sulfate Attack	Cement with low C3A content, early curing temperature <160°F	None	Low	Low
	Pozzolans, low w/cm, proper drainage	None	Low	Low
Delayed Ettringite Formation	Cement with low C3A content, early curing temperature <160°F	None	Low	Low
	Pozzolans, low w/cm, proper drainage	None	Low	Low

3.6.3 Construction Practice Specifications

Once the materials are selected, a proper set of specifications must be developed to ensure that the highest standard of care is used during construction. These specifications and procedures are fairly well established and documented by FHWA and the various state agencies.

3.6.4 Maintenance Plan

An effective maintenance plan should be developed to ensure the maintenance assumptions regarding upkeep made in the material selection process are properly identified for staff and budget requirements. If the bridge owner cannot commit to such a program, then strategies for low maintenance life-cycle costs should be recommended.

CHAPTER 4

BRIDGE DECKS

This chapter provides essential information and steps to be considered in developing a bridge deck system for a particular project in order to meet both strength and service life requirements. Section 4.1 provides a description of various deck systems and their known advantages and/or disadvantages, as summarized in Table 4.1.

Section 4.2 provides a summary of factors that affect service life of bridge decks using the fault tree format. Refer to Chapter 1 for a description of fault tree and how it is constructed.

Section 4.3 provides strategies that can be used to mitigate most of the factors affecting service life of bridge decks, as described in Section 4.2.

Section 4.4 provides a framework for systematically addressing the service life design of bridge decks designed for strength, based on design provisions stated in *AASHTO LRFD Bridge Design Specifications* (*LRFD Specifications*) (AASHTO 2012).

4.1 DESCRIPTION OF BRIDGE DECK TYPES

The primary function of a bridge deck is to provide a safe riding surface for traffic, ensuring direct structural support of wheel loads. Two principal superstructure types are considered as bridge decks in this section: 1) bridge decks cast on top of beams/stringers, acting either compositely or non-compositely with superstructure supporting elements, and 2) superstructure systems in which the top of the superstructure element forms the top of the riding surface.

By definition, numerous types of systems qualify as bridge decks including concrete deck systems, metal deck systems, timber deck systems, and fiber reinforced polymer (FRP) deck systems. The factors affecting service life of concrete deck systems, the main system utilized in the United States, are further described in this chapter. Metal, timber, and FRP bridge deck systems are not addressed. Major bridge deck systems are summarized in Table 4.1.

Table 4.1. Bridge Deck Systems

Bridge Deck Systems	Advantage	Disadvantage
Cast-In-Place Concrete Deck Systems	Readily available material. Accommodates tolerances. Low cost.	Susceptible to cracking and corrosion.
Precast Concrete Deck Systems	Readily available material. Typically prestressed, reducing cracking.	Requires construction joints between components. Higher initial cost.
Metal Deck Systems	Lightweight system. Prefabricated system.	Requires protective coatings. Difficult tolerance adjustments. High cost.
Timber Deck Systems	Lightweight system. Constructible with unskilled labor. Low cost.	Limited-span range. Susceptible to wear without overlays. Susceptible to moisture degradation.
FRP Deck Systems	Lightweight system. Noncorrosive system.	High cost. Limited history. Requires overlay for traction.

The following sections provide descriptions of the bridge deck systems listed in Table 4.1.

4.1.1 Concrete Bridge Deck Systems

Concrete bridge deck systems can consist of:

- Cast-in-place systems, and
- Precast systems.

The predominant bridge deck system in the United States consists of cast-in-place, reinforced concrete. Cast-in-place concrete systems are defined as concrete bridge decks that are cast in their final position. Typical cast-in-place systems include:

- Bridge decks on beams/stringers,
- Full depth concrete slab superstructure,
- Multi-cell box girders, and
- Cast-in-place segmental construction.

Precast concrete systems are defined as concrete bridge decks that are cast remotely, then brought to the bridge site for assembly into the final structure. Typical precast systems include:

- Adjacent member,
- Deck panel over beams/stringers, and
- Precast segmental construction.

4.1.1.1 Cast-in-Place Concrete Systems on Beams/Stringers

As shown in Figure 4.1, cast-in-place bridge decks on beams/stringers are typically reinforced with mild steel reinforcement. They are generally 7.5 in. to 9 in. in thickness and are cast on forms that span between beams/stringers. These forms can either be removable or stay-in-place.

Figure 4.1. Cast-in-place concrete deck over longitudinal beams/stringers. (Courtesy Atkins North America, Inc.)

4.1.1.2 Full-Depth Cast-in-Place Concrete Systems

Full-depth cast-in-place concrete deck slab superstructures are a classification of bridge decks that span between pier supports without the aid of supporting beams/stringers. These deck slabs can be either solid, can contain circular voids, as shown in Figure 4.2, or more trapezoidal shaped voids such as those used in cast-in-place multi-cell box structures and cast-in-place segmental structures. Voids are introduced to reduce the dead weight of the bridge. These bridge systems are usually conventionally reinforced with mild steel reinforcing, but can be posttensioned longitudinally and transversely to achieve longer span lengths.

Figure 4.2. Full depth cast-in-place concrete slab—posttensioned with voids. (Courtesy Atkins North America, Inc.)

4.1.1.3 Precast Adjacent Member Concrete Systems

One of the most commonly used superstructure systems is the adjacent member superstructure system that consists of prefabricated beam elements placed side-by-side in close proximity. This system has been used in various forms to expedite construction and minimize field forming and placing of concrete. These members are predominantly prestressed concrete beam elements in the form of prestressed solid and hollow-cored slab units, as shown in Figure 4.3, deck bulb-tees, double Ts, channels and adjacent box beams. These deck systems are typically built as simple spans, but can be constructed as continuous members. Typically, the members are tied together with a continuous longitudinal grout or concrete filled shear key that allows for the transverse distribution of applied vertical forces across the joint and prevents differential movement between adjacent members. The members may also be either transversely connected with conventional reinforcement or posttensioned together to develop the moments across the joint.

Figure 4.3. Examples of adjacent member slab unit superstructure system. (Courtesy Atkins North America, Inc.)

4.1.1.4 Precast Concrete Deck Panels

As shown in Figure 4.4, precast concrete deck panel systems employ a series of precast concrete panels that are usually full-depth in thickness and have a length and width determined by specific bridge geometry. The length of the panel along the roadway is approximately 8 to 12 ft, and is typically dictated by transportation limitations and crane capacity. Panels span across the supporting girders and are designed with conventional reinforcement or as prestressed concrete. The general preference of precasters/contractors is to use prestressed concrete to eliminate possible cracking from handling and shipping.

Precast concrete deck panel systems include both transverse and longitudinal slots for connections. The transverse slots are typically grout-filled keyways connected in a manner similar to the adjacent member bridge systems. The longitudinal slots may consist of grouted (or concreted) pockets or block-outs to accommodate the shear connections to the girder. The system may also require temporary support and forms along the girder to retain the grout and some type of overlay to improve pavement ride quality. Longitudinal posttensioning is typically included in the system to tie the panels together; however, systems without posttensioning have been used and at the time of writing (2012), new non-posttensioned connections are being developed.

Figure 4.4. Full-depth Precast Panel System. (Courtesy University of Nebraska, Omaha)

4.1.1.5 Precast Segmental Concrete Superstructure Systems

This structural system consists of numerous precast bridge elements that are posttensioned together to form either simple-span units or, more commonly, continuous spans. Segmental construction has gained favor in situations where access is challenging, such as in deep valleys, environmentally sensitive areas, across existing roadways, and where accelerated construction is warranted. The basic cross-section of a segmental bridge is usually a box shape with a top slab serving as the bridge deck-riding surface, as shown in Figure 4.5. The primary longitudinal reinforcement consists of either posttensioning tendons or bars that can be installed either internal to the web or externally inside the box section. The bridge deck is typically posttensioned transversely.

Figure 4.5. Segmental superstructure. (Courtesy Atkins North America, Inc.)

4.1.2 Metal Deck Systems

Metal deck systems are bridge deck systems that rely on a metal such as steel or aluminum to provide the structural resistance to vehicle wheel loads. Metal deck systems can consist of metal grid decks, orthotropic steel decks, or orthotropic aluminum decks.

4.1.2.1 Metal Grid Decks

This bridge deck system is a prefabricated module system consisting of main I- or T-shaped sections and secondary crossbars combined to form a rectangular or diagonal pattern. These members can be either steel or aluminum and the main elements span between beams, stringers, or other crossbeams. This system is typically used for movable bridges and for long-span structures where a reduced bridge deck weight is demonstrated to have an economic advantage. It has also been used in deck replacement projects. The system consists of open grid deck or can be combined with concrete to form a partially or fully filled grid deck. The partially- or fully-filled concrete is typically cast flush with the grid service, or it can be cast above the unfilled deck. This is known as the Exodermic™ bridge deck system. The addition of concrete in these systems reduces noise, improves fatigue performance, and improves the ability to channelize and collect storm water.

4.1.2.2 Steel Orthotropic Decks

Bridge structures can utilize the orthotropic steel plate as one of the key structural systems in the distribution of deck traffic loads and for stiffening the supporting slender plate elements in compression. Generally, the orthotropic system consists of a flat, thin steel plate, stiffened by a series of closely-spaced longitudinal ribs at right angles or orthogonal to intermediate floor beams. (See Figure 4.6.) The orthotropic deck is typically made integral with the supporting bridge superstructure as a common top flange to the floor beams and girders. This results in cost savings in the design of these other components. The defining characteristic of the orthotropic steel bridge is that it results in a nearly all-steel superstructure.

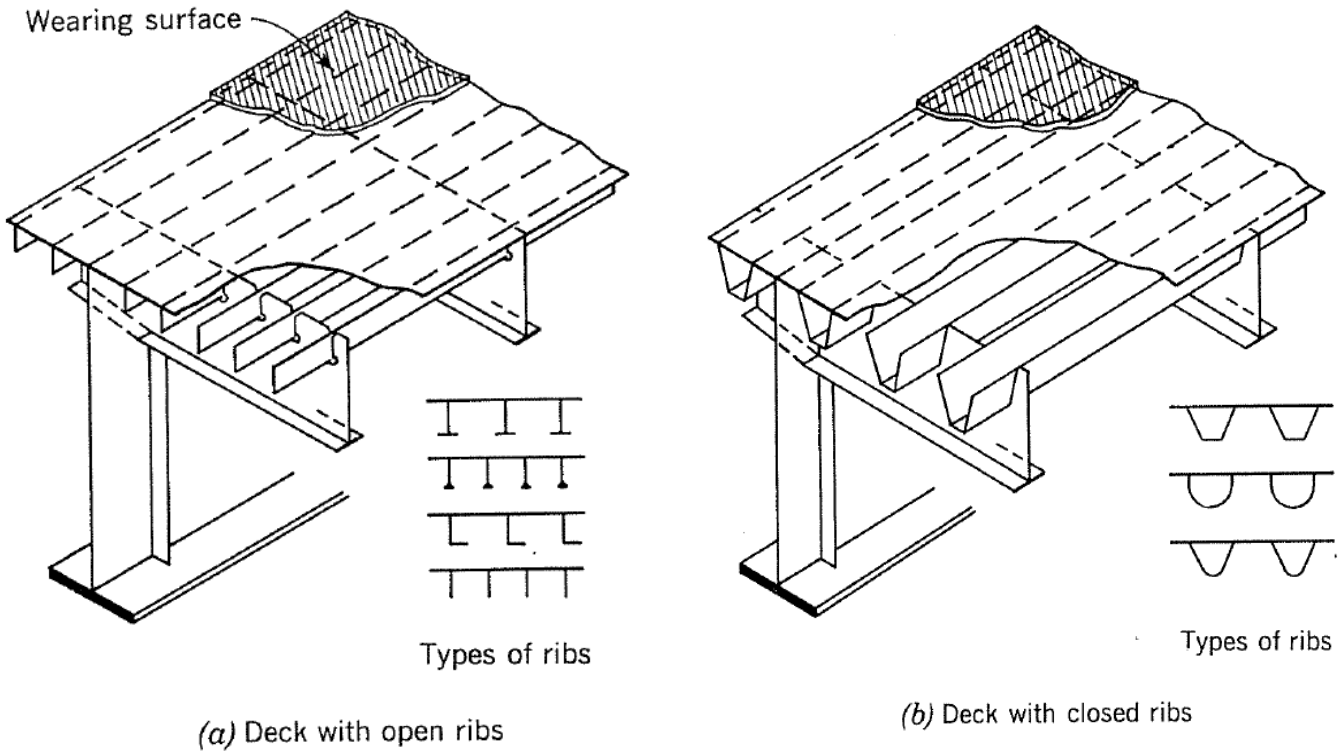


Figure 4.6. Orthotropic steel deck bridge. (Wolchuk 1963)

The orthotropic system has been utilized for many bridges worldwide, especially in Europe, Asia, the Far East, and South America. The United States has not yet fully embraced this technology and currently has fewer than 100 such bridges in inventory. The orthotropic deck has been most commonly used in the United States for long-span bridges in which the minimization of dead load is paramount and for re-decking bridges on urban arterials. Orthotropic construction has tremendous potential for use in short- to medium-span girder bridges. The system has not been used more extensively for economic reasons; however, its light weight makes it beneficial for increasing a

bridge's load rating during a deck replacement in instances where replacement of the bridge may have been the only other alternative.

4.1.2.3 Aluminum Orthotropic Decks

The aluminum orthotropic deck system configuration is similar to the steel orthotropic deck described in Section 4.1.2.2. The use of aluminum provides a corrosion resistance advantage that can result in lower maintenance costs, as it does not need periodic painting. While aluminum is lighter than steel, its additional cost has often deterred its use in the United States. Other factors to carefully consider that make it different from the steel orthotropic deck system include differences in thermal expansion coefficients, reactions with dissimilar materials, lower modulus of elasticity and lower fatigue strength of the material, particularly at weld locations.

4.1.3 Timber Bridge Decks

Timber bridge decks have been used for hundreds of years, but increases in vehicle loads have typically restricted their use to low volume roadways. The materials used for these bridge decks can be rough sawn timbers, glue-laminated panels. Their performance can be enhanced through the use of protective coatings that can minimize water absorption, which can be detrimental to the service life of the timber.

The timbers can be posttensioned together to form stress-laminated decks, or nailed together to form spike-laminated decks. Overlays are typically provided on these bridges to improve skid resistance; however, the overlay requires extensive maintenance due to the flexibility of the timbers and the numerous connections between members.

4.1.4 Fiber Reinforced Polymer (FRP) Bridge Decks

Fiber reinforced polymer bridge decks and superstructure systems are an emerging technology. FRP decks have been used for short-span bridges and for deck replacement on bridges. The principal advantages of FRP as a material are that it does not corrode under the same conditions as steel materials and it is lightweight. Its potential has shown promise for use in projects where deck replacement is needed, as shown in Figure 4.7, particularly if total load capacity is relatively low.

FRP bridge decks and superstructures have been constructed in many states. Comparisons with traditional cast-in-place concrete bridge deck systems have shown that they exhibit lower dead loads, higher live load fatigue ranges and lower dynamic allowance (impact). (Albers et al. 2007)

Figure 4.7. FRP bridge deck and superstructure applications. (Aboutaha 2001)

The surface of the FRP material has low skid resistance and the material itself is soft. Therefore, overlay systems are required to provide a safe riding surface that has adequate surface friction, and in order to withstand daily traffic wheel load abrasion. Failure of overlay adherence to the FRP material was evidenced in early applications of this technology. Connections for crashworthy barriers for FRP decks present additional challenges.

Further research is recommended to study the long-term behavior of this new material to demonstrate its acceptability to provide a sufficiently long service life.

4.2 FACTORS INFLUENCING BRIDGE DECK SERVICE LIFE

Bridge decks are one of the most costly maintenance items within a typical bridge system. The reduced service life of bridge decks can be attributed to two causes: 1) obsolescence, which is a functional planning issue and not a factor relating to durability issues, and 2) material service life performance deficiencies, which may be load-induced, caused by man-made or natural hazards, or result from production defects in construction processes and/or design details, or operational procedures. These deficiencies are illustrated in the fault tree shown in Figure 4.8.

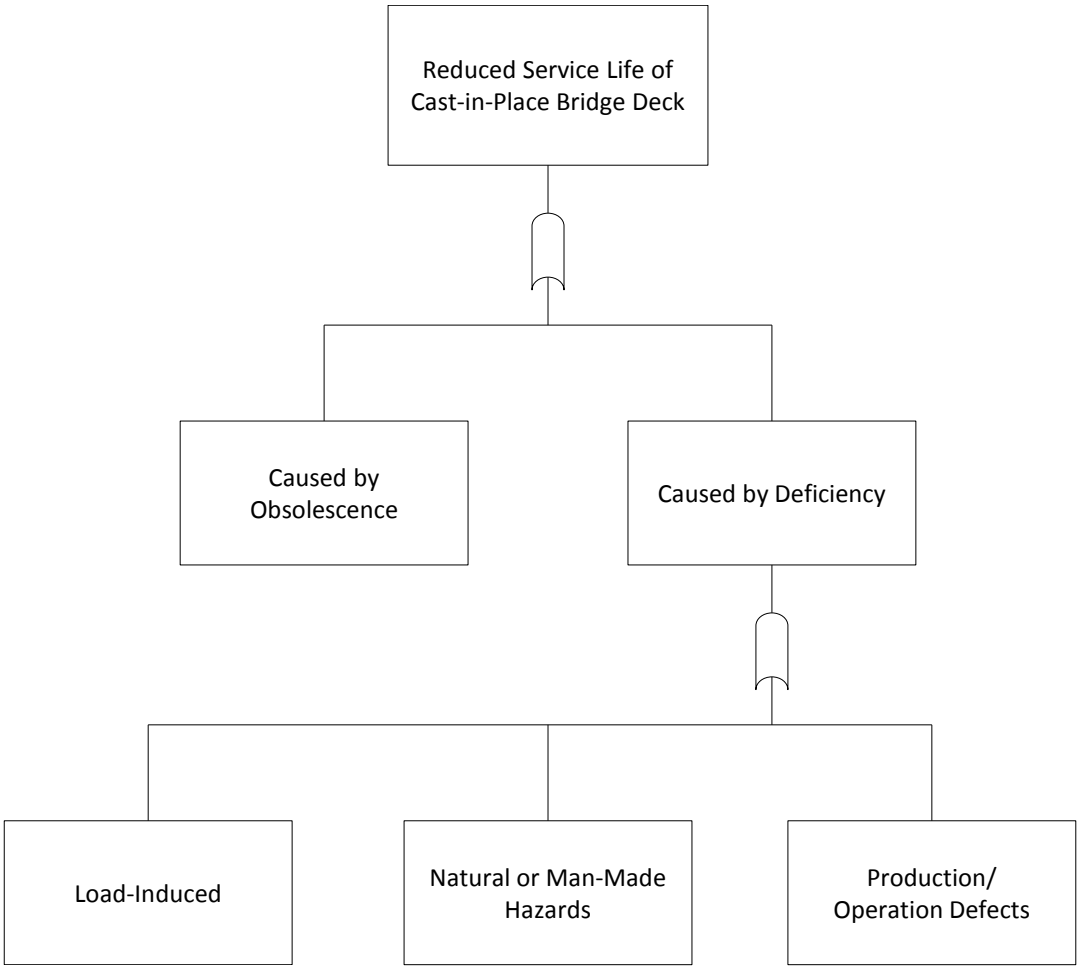


Figure 4.8. Bridge deck reduced service life fault tree.

Many interrelated factors during the design, construction, and management phases of a bridge deck's service life must be considered in developing long-lasting, cost-effective bridge decks. These factors vary depending on the bridge deck system utilized and can be arranged in four broad categories:

- Concrete bridge decks systems, including:
    - Cast-in-place concrete bridge deck systems, and
    - Precast concrete bridge deck systems;
- Metal deck systems;
- Timber deck systems; and
- FRP bridge deck systems.

The factors affecting the service life of cast-in-place concrete and precast concrete bridge deck systems are further described in this chapter. Metal, timber, and FRP bridge deck systems are not addressed in the *Guide*.

4.2.1 Cast–in–Place Concrete Bridge Decks

Cast-in-place bridge deck systems are systems in which the concrete for the bridge deck is cast in the field as an integral part of the final superstructure. This bridge deck system is one of the most common systems used in the United States today. These decks provide a major constructability advantage in that the casting process easily molds the bridge deck to meet geometric requirements, such as skews, lane tapering, and super-elevation transitions, and to match existing locations of supporting elements that are not precisely located in accordance with the plans. The main disadvantages of these decks include the quality of concrete produced as a result of workmanship and the curing processes.

Inspections of bridge decks have revealed numerous performance issues with cast-in-place concrete including cracking, corrosion of reinforcement, spalling, delamination, and concrete deterioration evidenced by scaling, wear, and abrasion. While concrete in compression is considered a very durable construction material, tension introduced through various loading and bridge restraint conditions can result in significant tension that can exceed the material's tension strength limits, resulting in cracking. Cracking of bridge deck concrete reduces the integrity of the passivated concrete layer that surrounds the reinforcing steel, significantly reducing the encased reinforcement's resistance to corrosion.

The following sections discuss factors affecting the service life of cast-in-place bridge deck.

4.2.1.1 Load-Induced Bridge Deck Considerations

Load-induced bridge deck deterioration can be attributed to either loads induced by the traffic, or by characteristics dependent on the overall bridge system. These load-induced factors are shown in the fault tree provided in Figure 4.9.

4.2.1.1.1 Traffic-Induced Load Considerations

Traffic-induced loads include the effects of truck and other vehicle traffic on the riding surface of the bridge. Bridge deck loading has a degree of uncertainty that must be addressed during design of the bridge, especially when

achieving long service life is an objective. Typically the service life of bridge decks will be affected by fatigue, overload, and wear and abrasion.

Fatigue. Cast-in-place concrete deck consists of two materials—steel and concrete, both of which can fail by fatigue. Design provisions for fatigue are addressed in the *LRFD Specifications*.

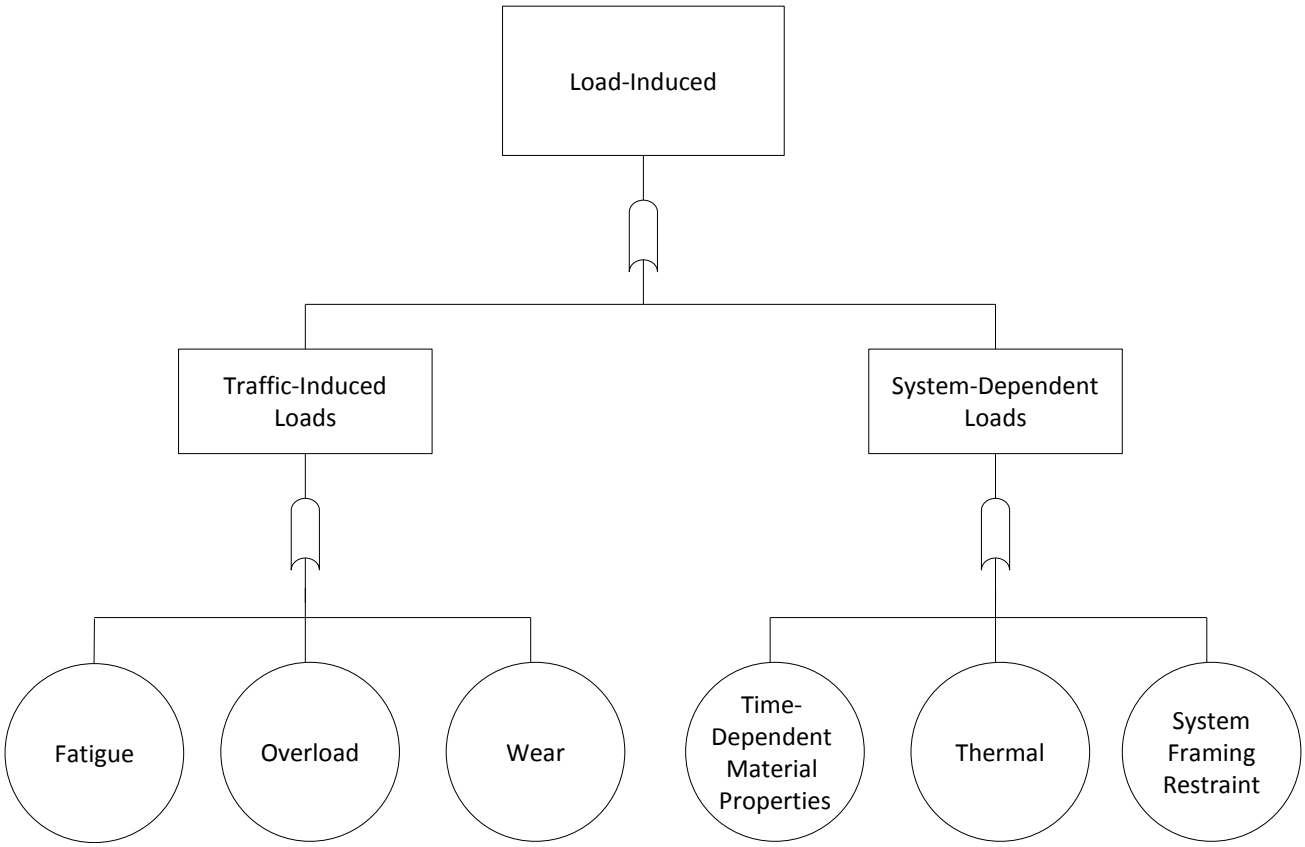


Figure 4.9. Cast-in-place bridge deck load-induced deficiency fault tree.

Overload. Despite weight limit regulations in most states that define load limits for permit and legal truck configurations, overloads exceeding these limits do occur. This is one of the main reasons for reduced service life of bridges.

Overloads result in additional flexural stresses in bridge decks that can cause excessive cracking not accommodated by the original design. Heavier tire loads may also affect the wear and abrasion on the structure, and multiple applications of these loads can affect the fatigue behavior of the deck.

Wear and Abrasion. Wear and abrasion is typically affected by high traffic volume, high tire loads, and the types of tires used on the facility. Tires in cold climates may have enhanced features to aid in traction, such as deep grooves, studs, and chains. These added tire features, while aiding traction, can abrade the surface of the bridge deck.

Wear and abrasion can result in reduced thickness of the bridge deck, which in turn reduces the concrete cover protecting the reinforcement from corrosion; can reduce the load resisting section resulting in higher stresses and cracking; and can change the deck stiffness assumed for distribution of loads between superstructure elements.

4.2.1.1.2 System-Dependent Loads

System-induced loads include the effects of the bridge system configuration on the behavior of the bridge deck, such as restraint of integral abutment systems.

Differential Shrinkage. Differential shrinkage occurs when bridge deck concrete is cast over previously cured concrete or over steel girders. The shrinkage of fresh concrete is restrained by the cured concrete or steel stringers resulting in a set of equal and opposite forces causing tension in the deck and compression in the girders. Heat of hydration also contributes to development of tensile forces in the freshly cast concrete deck.

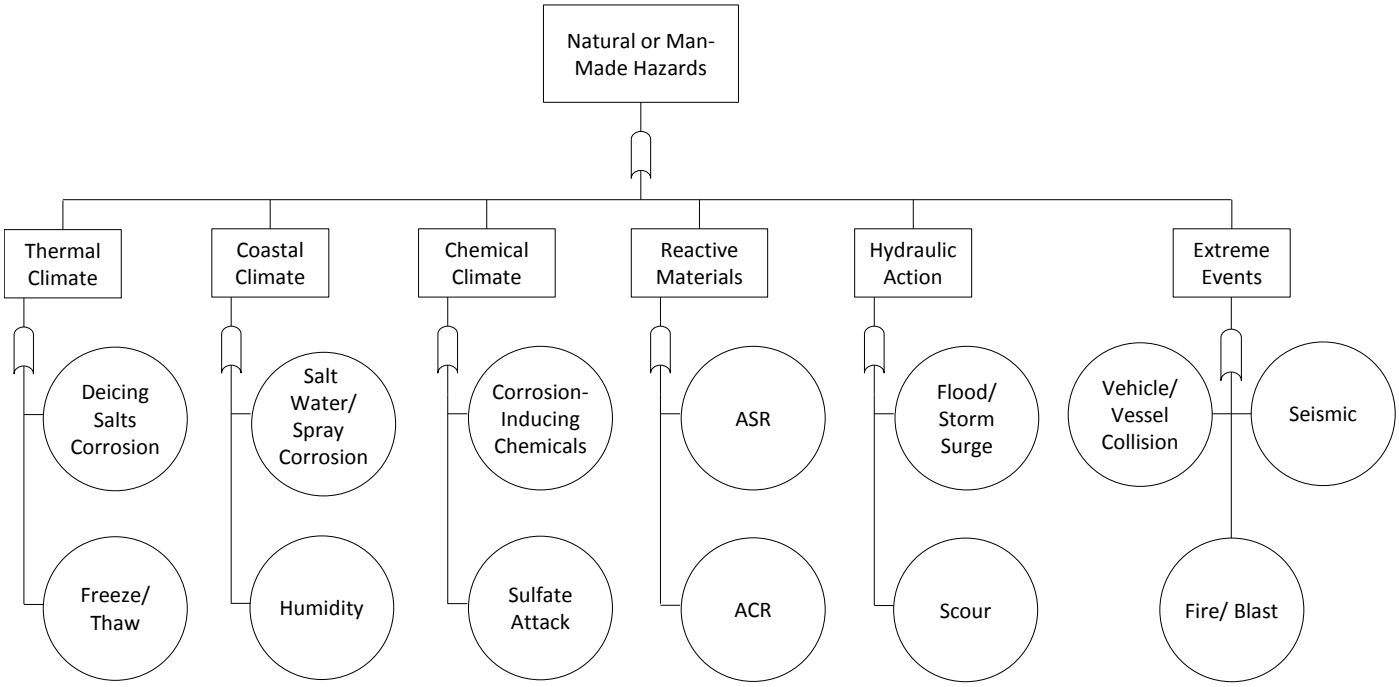
System Framing Restraint. Bridge decks can be subject to additional axial forces created by bridge boundary conditions. Bridge system boundary conditions are set at the design stage, during bridge system selection. For instance, in integral abutment jointless bridge systems, the elimination of expansion devices and reliance on flexibility of piles to resist the bridge expansion and contraction, subject the deck to additional axial forces. Another example of boundary conditions capable of creating axial forces in bridge deck includes choices for bearings and connections between superstructure and substructure made during design. These axial forces range from compression during system expansion to tensile forces during system contraction. Resistance to system contraction can create tensile forces in the bridge deck and cause cracking. Calculation of these additional axial forces is important and can be achieved through conducting proper analysis methods that correctly model the bridge boundary conditions.

Improper function, or seizing of the bearings, results in unintended movement restraint that can raise the force resisted by the substructure well above the intended design. This unintended restraint can cause unanticipated cracking with greater potential for corrosion. Proper bearing function is essential to the durability of substructure and is addressed in Chapter 10. Lack of maintenance may also result in bearings losing the movement capability intended by their design.

Thermal. Temperature changes can result in the development of axial forces in the bridge deck. These thermal forces are due to uniform internal temperature changes and temperature gradient. The level of these thermally induced axial forces is a function of bridge system boundary conditions.

4.2.1.2 Natural or Man-Made Hazard Bridge Deck Considerations

The environment to which the bridge deck is subjected can have a significant influence on the service life of bridge decks. These environmental influences include hazards from both natural and man-made sources, and include effects from areas with adverse thermal climate, coastal climates, and chemical climates, as well as from chemical properties of the materials and outside agents, such as fire. These natural and man-made hazards are introduced in the fault tree provided in Figure 4.10.



[LANDSCAPE IN FINAL]

Figure 4.10. Cast-in-Place Bridge Deck Natural or Man-made Hazard Fault Tree.

4.2.1.2.1 Thermal Climate

Thermal climate influences on bridge deck service life performance are primarily due to cold weather. These influences are both manmade, from the application of deicing salts, and natural, in the case of freeze/thaw.

Application of Deicing Salts Agencies in cold weather climates that deal with ice and snow on roadways and bridges have traditionally applied deicing salts to melt the ice and snow to facilitate tire traction. The application of these deicing salts is viewed as a safety enhancement for the traveling public; however, these chloride-laden

compounds tend to ingress into the concrete deck either through porosity in the concrete or through open deck cracks. The chloride ingress into the bridge deck continues to reduce the effectiveness of the passivating layer around the reinforcing steel, eventually initiating reinforcement corrosion. The reinforcement corrosion process causes the bar to expand, resulting in deck cracking, spalling, and/or delamination. The cross-slope built into bridge decks for drainage purposes causes the salt to wash down towards the bridge gutter adjacent to the traffic railing barriers bounding the bridge. Removal equipment that scrapes snow from the bridge deck also deposits residual snow laden with deicing salts at this location, resulting in a very high concentration of chlorides. Construction joints at this location are particularly susceptible to chloride intrusion, and in many cases this has led to corrosion of the barrier reinforcement.

Freeze/Thaw. Water absorbed into the concrete deck surface and contained in cracks can freeze in cold weather conditions. The frozen water tends to expand causing stresses within the concrete. Cyclic freezing and thawing of the water absorbed in the deck surface can result in bridge deck deterioration in the form of cracking, scaling, and spalling. Refer to Chapter 3 for additional information on freeze/thaw in concrete.

4.2.1.2.2 Coastal Climate

Coastal climate influences on bridge deck service life performance are primarily due to the introduction of chlorides through salt spray, and from the effects of high humidity. Both of these influences occur naturally.

Salt Spray. Coastal regions are subjected to a chloride-laden saltwater environment and a combination of wind and wave action that causes these chlorides to become airborne as salt spray. The susceptibility of the bridge deck to these environmental influences depends on the height of the bridge deck above the water elevation and the distance to coastal areas. The action of waves hitting substructure units and seawalls or abutments under the bridge tends to cause the salt spray to explode upwards, wetting the bottoms of lower level bridge decks. The salt spray can also deposit itself on the bridge deck surface, particularly on windy days. When the salt spray wets the surfaces it leaves a chloride residual that can absorb into the concrete, resulting in reinforcement corrosion.

Humidity. High humidity in coastal regions also results in cyclical wetting and drying of concrete surfaces. Concrete materials sensitive to repeated wetting, such as those where reactive aggregates are utilized, can have an adverse effect on the bridge deck service life.

4.2.1.2.3 Chemical Climate

Chemical climate influences on bridge deck service life performance can be attributed to corrosion-inducing chemicals and sulfate attack. These influences can occur naturally or can be man-made.

Corrosion-Inducing Chemicals. Corrosion-inducing chemicals can be introduced to the bridge deck from adjacent industries, where residuals from pollution can attribute to reduction in bridge deck service life. For example, oil and coal burning facilities release sulfur dioxide and nitrogen oxide into the air, which causes acid rain consisting of sulfuric and nitric acids. These acids can dissolve cement compounds in the cement paste and calcareous aggregates, and can leave crystallized salts on concrete surfaces that can lead to spalling and the corrosion of reinforcing bars.

Sulfate Attack. Exposure to sulfates can cause expansion of the concrete material and consequently result in spalling and cracking of bridge deck. Refer to Chapter 3 for additional information on sulfate attack in concrete.

4.2.1.2.4 Reactive Ingredients

Reactive ingredients within the mix used for bridge decks can affect service life performance as the reactive ingredients alter the volumetric stability of the concrete. These influences primarily occur naturally.

Alkali-Silica Reactivity. Alkali-silica reactivity (ASR) results in swelling within concrete that can lead to spalling, cracking, and general concrete deterioration. Refer to Chapter 3, for additional information on ASR in concrete.

Alkali-Carbonate Reactivity. Alkali-carbonate reactivity (ACR) results in aggregate expansion within concrete that can lead to spalling, cracking, and general concrete deterioration. Refer to Chapter 3 on materials, for additional information on ACR in concrete.

4.2.1.2.5 Fire

A key factor in the amount of damage that is caused to concrete is the duration of the fire and the heat levels generated. Because of the low thermal conductivity of concrete, it takes considerable time for the interior of concrete to reach damaging temperatures. When concrete is exposed to the extreme heat of a fire, the chemical bonds between the water molecules in the concrete break, resulting in dehydration and the destruction of the cement binder. The concrete loses its mechanical properties, exhibiting cracking and spalling, and exposes steel, leaving it unprotected

(ACI 216, 1989). Once the reinforcement has become exposed, it conducts heat and accelerates this action. Reinforcing steel in bridge decks subjected to temperatures above 550°C (1022°F) exhibits a rapid reduction of strength, which can lead to collapse. In addition, spalling can result from the rapid quenching of hot fires by fire hoses.

4.2.1.3 Design, Construction (Production), and Operation Bridge Deck Considerations

Decisions made for the design and construction of bridge decks and the activities that will occur during its operation can have a significant influence on the service life. These influences are introduced in the fault tree provided in Figure 4.11, and include decisions made during the design and detailing of the bridge deck, the quality of construction, the level of inspection, and the testing performed during operations and maintenance.

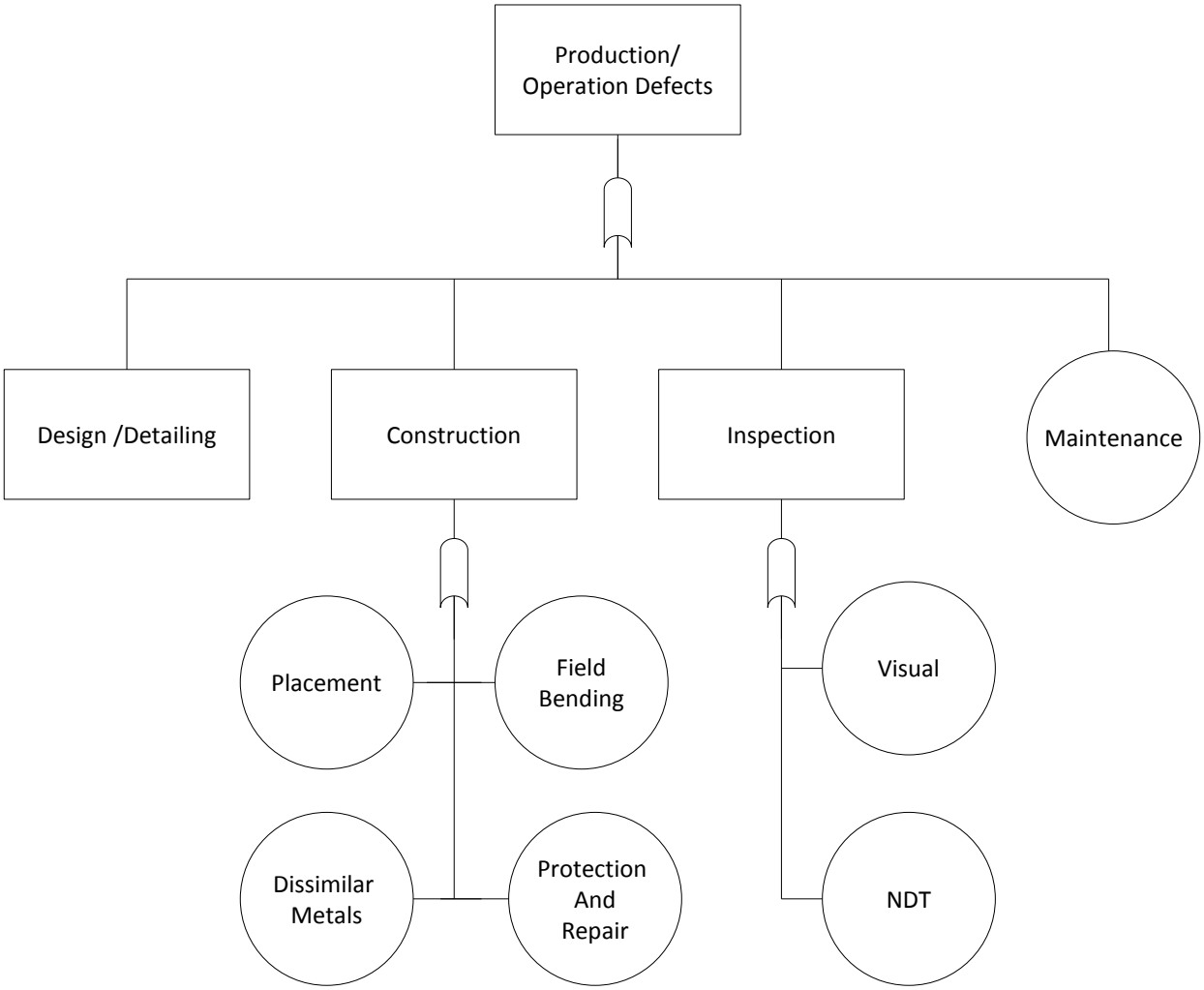


Figure 4.11. Cast-in-place bridge deck design, construction (production), and operation defects fault tree.

4.2.1.3.1 Design and Detailing Bridge Deck Considerations

Decisions made during the design and detailing phase of a bridge project can significantly impact the service life of the bridge. It is incumbent upon designers to understand the implications of these decisions in order to make rational choices that will improve the service life of bridge decks. These decisions are introduced in the fault tree provided in Figure 4.12, and include choices in design philosophy, expansion joints, construction joints, concrete mix design, and bridge deck drainage.

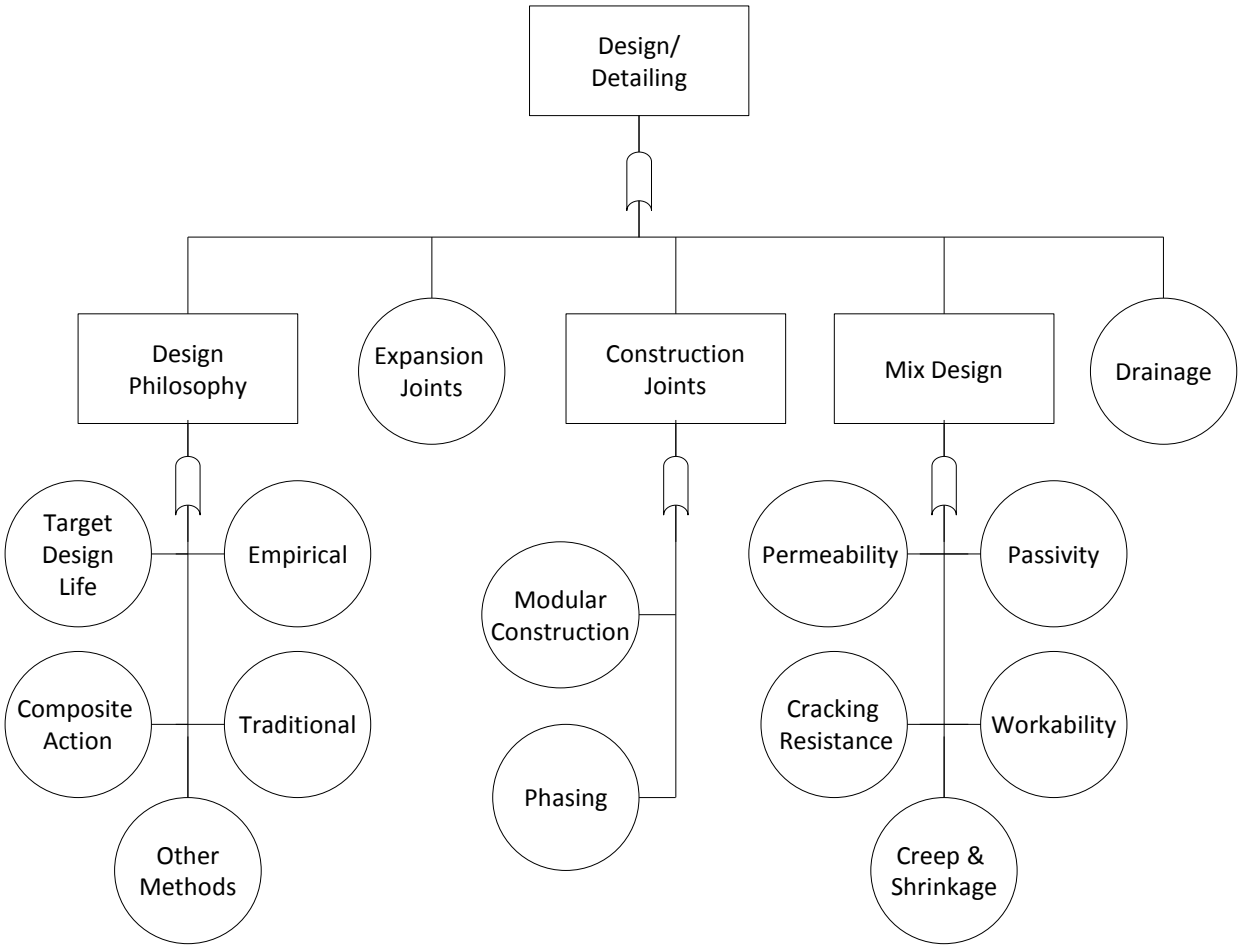


Figure 4.12. Cast-in-place bridge deck design/detailing deficiency fault tree.

4.2.1.3.1a Design Philosophy

There are two principal methods presented in the *LRFD Specifications* for the design of bridge decks: the traditional design method and the empirical design method.

Traditional Design Method. The traditional design method assumes flexural action to describe behavior of bridge deck spanning between supporting girders and ignores the axial forces created in the bridge deck as a result of

arching action. Under this assumption, the amount of reinforcement needed in the bridge deck will generally far exceed the demand, usually by more than a factor of two. Providing additional reinforcement in the bridge deck creates additional means for corrosion and deterioration of bridge deck.

Empirical Design Method. The empirical design method provides better estimation of bridge deck resistance to applied traffic loads than the traditional design method. Test results (Fang 1985; Holowka 1980) show that the principal mechanism for resisting the applied traffic loads in the bridge deck is the creation of axial compressive loads, commonly referred to as arching action. These axial compressive loads are resisted by supporting longitudinal beams. Consequently, the use of the empirical method is not applicable to cantilever portions of the deck. The axial compressive loads in the bridge deck significantly reduce the need for reinforcement in the bridge deck and reduction of reinforcement in the bridge deck significantly reduces the sources of corrosion.

Other Design Methods. Research in Canada in the past 15 years (Newhook and Mufti 1996) has focused on eliminating bridge deck reinforcement corrosion through the development of the concept of "steel-free" bridge decks. Similar to the methodology employed by the empirical design method, this concept provides the tension tie required to resist the compressive forces created in bridge deck by arching action. Under this concept, the tension ties are attached to top flanges of the supporting beams/stringers. These bridges have experienced some temperature and shrinkage cracking and in order to control this cracking to acceptable levels, recent recommendations suggest supplementing the steel tension ties below the deck with a mat of FRP reinforcing bars (Menon and Mufti 2004).

4.2.1.3.1b Expansion Joints

Expansion joints are provided to relieve system framing restraints that can cause a build-up of tension stresses in the superstructure and the bridge deck. Refer to Section 4.2.1.1.2 for additional information on system framing restraints. Refer to Chapter 9, for additional information on expansion joints.

Expansion devices provide a location within the bridge deck where dirt and other deleterious material, such as de-icing salts, can collect and produce an adverse effect on the service life of bridge decks. Impact from vehicles and from snow removal equipment can cause spalling that reduces the protective concrete cover over reinforcing and/or exposes the reinforcement, which leads to the corrosion.

4.2.1.3.1c Construction Joints

Construction joints are surface discontinuities where successive concrete placement regions meet and they are generally specified by the designers or construction contractors.. The following are some typical construction methods where construction joints are required:

Modular Construction/Adjacent Members. Construction joints within adjacent member superstructure systems are requirements of the modular type of construction utilized for accelerate bridge construction. Typically these joints are designed to transfer shear and moment across the interface, and if not properly designed and detailed may lead to cracking along the adjacent member interface.

Phasing of Construction. Public pressure and demand for uninterrupted traffic flow requires some bridges to be constructed using a phased construction approach. This approach is used for both the widening of existing bridges and the construction of new bridges. In the case of new construction, a portion of the new bridge is constructed (Phase 1) while the existing bridge carries the traffic. Traffic is then transferred to the new bridge (Phase 1), the existing bridge is demolished, and the new bridge is completed (Phase 2). Finally, the two phases are typically joined together using a closure pour. The phases will experience different deflections at the time of placing the closure pour and this differential deflection can result in major construction problems.

One of the characteristics of phased constructed bridges is that transverse and longitudinal cracks form near the points where Phase 1 and 2 are joined. Formation of these cracks is a well-known feature of these bridge types. Further, closure pour regions need to be water proofed to prevent deterioration of the deck in these regions as a result of chloride ingress and initiation of reinforcement corrosion. It should also be mentioned that placing additional reinforcement in the deck will not prevent formation of these cracks; it will only make the crack width smaller.

A well-constructed phase bridge can perform very satisfactorily (Azizinamini et al. 2003). Nevertheless, several factors as described in the following need to be taken into consideration.

Composite sections experience long-term displacement because of creep and shrinkage. Figure 4.13 shows an exaggerated difference between elevations of Phase 1 and Phase 2 for newly constructed bridges. The same phenomenon also exists for widening projects.

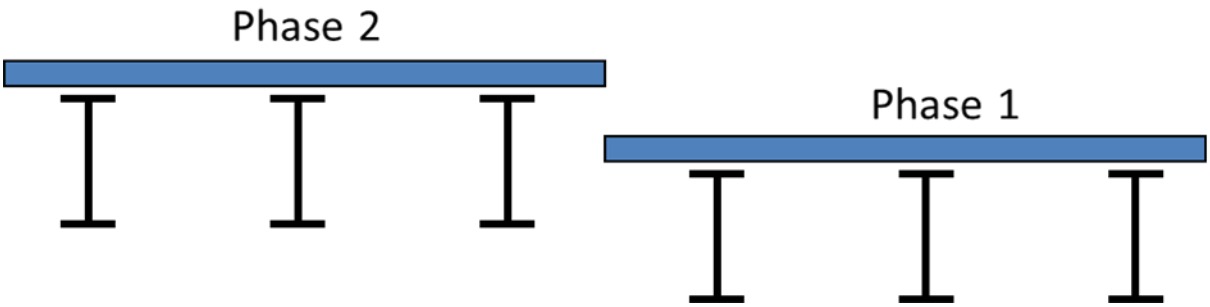


Figure 4.13. Exaggerated differential displacement between phases. (Azizinamini et al. 2003)

The reason for this observed differential displacement is illustrated in Figure 4.14, which shows the displacement of the Phase 1 girders due to creep and shrinkage. In about 90 days after completion of Phase 1, the girders experience maximum creep and shrinkage displacement. As shown along the horizontal axis, construction of Phase 2 starts after Phase 1 is completed. Phase 2 also experiences the creep and shrinkage displacements and depending on the time of casting the closure pour region, differential displacement will exist between Phase 1 and Phase 2 girders. In the case of steel bridges, this differential deflection between the two phases has resulted in major fit up problems for the cross frame in the bay containing the closure pour. Once the closure pour region is cast, the two systems are locked in, while Phase 2 continues to experience additional displacements that subject the deck in the closure pour region to additional stresses.

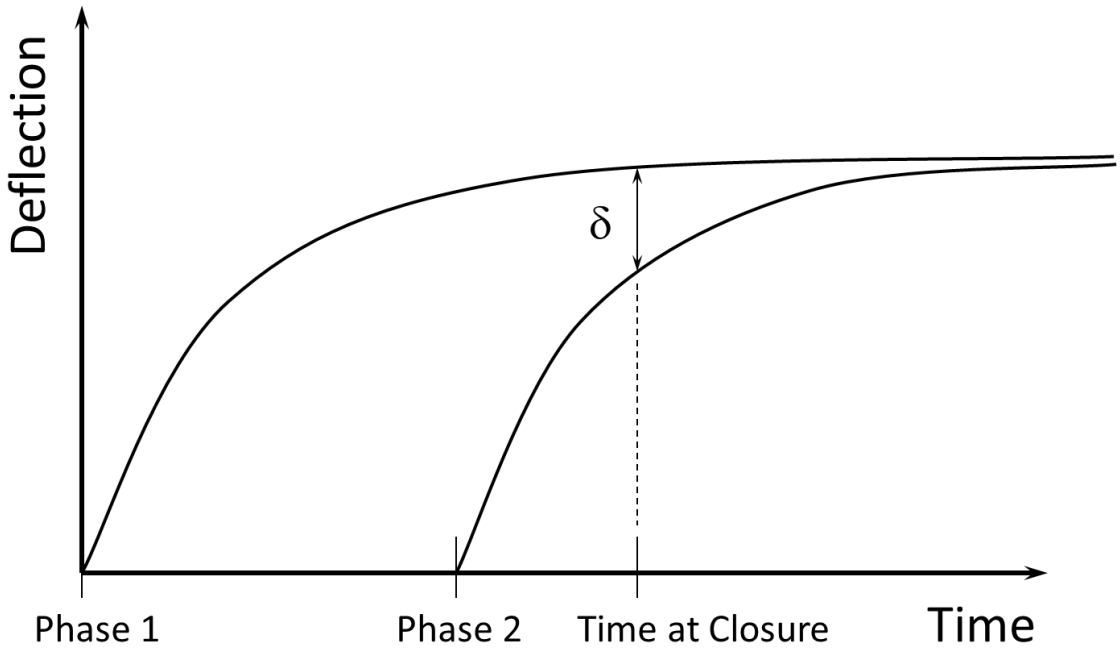


Figure 4.14. Displacement of Phase 1 and Phase 2 portion of the bridge. (Azizinamini et al. 2003)

When the deck elevations of the two phases do not match, the contractor may attempt to force the two separate superstructure portions together. This practice subjects the deck in the closure pour region to additional stresses, which are difficult to estimate and may also jeopardize the service life of deck concrete.

There is debate and conflicting beliefs among bridge engineers on conditions under which the closure pour region should be cast. Some contractors prefer to close traffic completely until the concrete in closure pour region is set, while others believe that vibration caused by traffic is helpful for better consolidation of concrete in the closure pour region. Generally, traffic closure is not an option, due to public demand that the traffic interruptions be minimized.

Another major condition that could facilitate the construction of phased bridges is the elimination of cross frames in the bay containing the closure pour, if possible. However, the pros and cons of such action need to be investigated.

4.2.1.3.1d Mix Design

Chapter 3 provides detailed information concerning factors which affect the service life of concrete. The following provides a brief description of mix design factors which affect the service life of bridge decks.

Permeability. Concrete is a durable material that largely depends on its ability to resist the infiltration of water and aggressive solutions. Concretes with high permeability provide less resistance to aggressive solutions or water penetrating the concrete and possibly causing expansive forces due to physical (freeze/thaw) or chemical (corrosion, ASR, sulfate attack) factors.

Passivity around Reinforcement. The loss of passivity of the outer layer of the reinforcing steel initiates a corrosion process that deteriorates the steel. This corrosion process begins by diffusing chloride ions to the depth of the reinforcing steel, and/or carbonation reducing the pH of the concrete to the passivating layer surrounding the concrete.

Crack Resistance. Mix design can affect the extent of cracking for all cast-in-place concrete bridge decks and slab superstructures. Mixtures with high water and paste content are prone to shrinkage cracks that occur over time. The use of large aggregate sizes and well-graded aggregates reduces the water and paste content and minimizes shrinkage. In fresh concrete, when the rate of evaporation exceeds the rate of bleeding, plastic shrinkage occurs.

Concrete with low bleed water, stiff consistency, and low water/cement ratio is prone to plastic shrinkage cracking. Prevention of plastic shrinkage cracking depends on prompt, effective curing.

Workability. Concrete mix designs must include good quality aggregates and appropriate admixtures to facilitate construction. Mix designs with poor workability can cause uncontrolled field adjustments to the mix through the addition of water in the field, resulting in higher water-cement ratios and over-vibration that can cause aggregate segregation. Proper workability must be ensured for the integrity of the mix design to provide concrete with the intended properties.

Creep and Shrinkage. The creep and shrinkage properties of concrete mixes can affect the service life performance of bridge decks. The adverse effects of concrete's shrinkage characteristics have been discussed in conjunction with system-dependent loads in Section 4.2.1.1.2. High levels of creep, on the other hand, can be either beneficial or detrimental depending on the application: 1) *Beneficial* when differential shrinkage occurs, particularly within bridge decks. In instances of higher creep, the restraining force between the bridge deck and the supporting superstructure will reduce significantly, consequently reducing the potential for cracking. 2) *Detrimental* when the creep results in unrestrained volumetric changes that result in significant, unintended movements of the structure, such as in posttensioned structures.

4.2.1.3.1e Drainage

In design, poor drainage details can result in ponding and prolonged exposure of bridge components to moisture and aggressive solutions, causing corrosion and other environmental distress. When concrete gains moisture, it expands slightly or swells. When concrete loses moisture, the concrete contracts or shrinks. As drying occurs, the portion of concrete near the surface dries and shrinks faster than the inner portion of the concrete. This results in differential moisture condition in which tensile stresses that can cause cracks may occur on the surface.

The degree of frost damage to concrete is also highly dependent on the degree of saturation. Ponding of water on bridge decks can cause critical saturation resulting in bridge damage. High moisture is also detrimental to concrete susceptible to alkali-silica reactivity expansion that can cause spalling and cracking.

4.2.1.3.2 Construction

Attention to good practices during construction is crucial to the long-term durability of reinforced concrete. A work force that is well-qualified, well-trained, and work which is well-executed increases productivity, reduces material waste, and provides expected service life. The proper use of appropriate equipment provides better workability by increasing efficiency, well-planned construction schedules reduce the overall cost by providing set times for equipment rental and reducing downtime. The correct implementation of test methods ensures quality concrete.

4.2.1.3.2a Placement and Curing of Concrete

Good construction practices, which ensure the proper location of reinforcing steel for proper cover depth, consolidation, and curing, are essential for longevity. Proper consolidation minimizes entrapped air voids that can reduce strength and durability.

Proper curing is necessary for formation of the binder and control of volumetric changes and includes both moisture and temperature control. In bridge structures, the deck surfaces require special attention due to their large surface areas where loss of moisture is a concern.

Handling of concrete affects the final product. Delay in placement, particularly on hot days, should be avoided as it can lead to stiffening of the concrete that can cause tearing of the deck surface during finishing, resulting in a poor surface finish and reduced durability.

4.2.1.3.2b Formwork

The type of concrete formwork can affect the surface finish of the concrete. Impermeable forms can allow surface voids to occur resulting in increased surface permeability, reduced strength, and an overall decrease in durability.

Cast-in-place bridge decks on beams/stringers are cast on forms that span between the beams/stringers. These forms can be either removable or stay-in-place. Stay-in-place forms are typically made of galvanized steel, as shown in Figure 4.1, precast concrete panels (either conventionally reinforced or prestressed), and FRP. Many owners do not allow the use of stay-in-place forms, citing the inability to inspect the bottom surface of the concrete deck and the potential for the collection of water intruding through the cracks.

The use of precast concrete stay-in-place panels designed to be composite or noncomposite with the deck pour above, has resulted in reflective cracking over the panel joints in past applications, and has raised questions about their long-term durability. Research is needed to develop acceptable details for control or elimination of reflective cracks

4.2.1.3.2c System Vibration during Construction

Excessive traffic-induced vibrations during construction can occur during the widening of a new bridge deck that is being cast against existing bridge deck subjected to active traffic. The effect of these vibrations on the quality of finished deck, especially in the closure pour regions, is not well understood and requires further investigation.

4.2.1.3.2d Casting Schedule

The casting schedule for bridge decks should carefully consider weather conditions, particularly hot days. The rise of heat from hydration in concrete can be exacerbated by a concurrent rise in the ambient temperature, resulting in a greater cooling differential that can cause bridge deck cracks. It takes approximately 18 hours for heat of hydration to reach its peak value. Following casting, the concrete at the surface is always at ambient temperature, while within the deck, the temperature varies due to the development of heat of hydration. The center of deck cools last. As a result, the maximum temperature differential takes place between the center of the deck and the deck surface. When the maximum temperature differential exceeds a certain limit, which most DOTs limit to about 30°F, the deck can crack. Therefore, this maximum temperature differential needs to be controlled. When casting is performed at night, the peak ambient temperature generally occurs 12 to 18 hours later, while the center of the deck is at its highest temperature. This results in a minimum temperature differential that can be achieved between the center and outside surfaces of deck concrete. On the other hand, when the deck is cast in the morning, the temperature differential between the center of the deck and the surface of deck is much higher. In conclusion, although casting deck at night is ideal, the common practice is to cast the concrete deck in the morning, the most undesirable time.

4.2.1.3.2e Casting Sequence

Construction joints within bridge decks control the sequence of casting bridge decks, due either to concrete volume placement constraints, or, in the case of continuous steel girders, in order to minimize bridge deck tension in

the negative moment area over substructure elements. These locations form a discontinuity that can open as cracks within the bridge deck allow ingress of moisture.

4.2.1.3.3 Visual Inspection

Although inspections are valuable tools for identifying deficiencies in bridge decks they are typically visual, making them subject to the ability, training, and disposition of the individual inspector. Often a deficiency is not easily detectable and may show only subtle signs that can easily be missed by cursory inspections or by inexperienced inspectors. Deficiencies can also be located below the undamaged surface or in inaccessible areas. The inability to see the deficiency leads to inadequate identification of repair methods, scope, and material selection and could cause failure of the structure without visible signs, since surficial repairs may cover the damaged area.

4.2.1.3.4 Maintenance

Lack of preventive maintenance reduces the service life of bridge decks. Sometimes simple maintenance tasks are delayed until a problem becomes a safety issue, at which time the required repairs may be either significantly more extensive or ultimately irreparable.

4.2.2 Precast Concrete Bridge Deck Systems

Precast concrete bridge deck systems are systems in which the concrete components for the bridge deck are produced in a more controlled environment, minimizing variability in concrete uniformity of both material behavior and construction personnel performance. Use of these systems can minimize traffic disruption caused by prolonged concrete casting operations over active roadway facilities and is a key consideration for accelerated bridge construction.

Inspections of precast concrete bridge decks have revealed many of the same performance issues experienced with cast-in-place concrete as described in Section 4.2.1 including cracking, corrosion of reinforcement, spalling, delamination, and concrete deterioration evidenced by scaling, wear, and abrasion. Precast concrete bridge deck components introduce numerous joints in the superstructure that are usually the source of many bridge deck service life issues, particularly in cases where the material provided to seal these construction joints breaks down causing cracking, leakage, and eventually reinforcement corrosion.

Rather than repeat the discussion of the many performance-related service life issues inherent with concrete systems described in Section 4.2.1, this section describes the factors affecting service life specific only to precast concrete bridge deck systems. The user of this *Guide* should also become familiar with the other factors described in Section 4.2.1 when considering precast concrete bridge deck systems.

Load-induced (Section 4.2.1.1) and natural or man-made hazard (Section 4.2.1.2) bridge deck considerations for precast bridge deck systems are the same as those for cast-in-place bridge deck systems. The following sections discuss the factors affecting precast concrete bridge deck service life in production and operation.

4.2.2.1 Design, Construction (Production), and Operation Bridge Deck Considerations

Decisions regarding the production of bridge decks and the activities which will occur during its operation can have a significant influence on service life. These production and operation influences are introduced in the fault tree provided in Figure 4.15. They include decisions made during the design and detailing of the bridge deck, fabrication and manufacturing requirement, quality of construction, and decisions concerning the level of inspection and testing performed during future operation and maintenance.

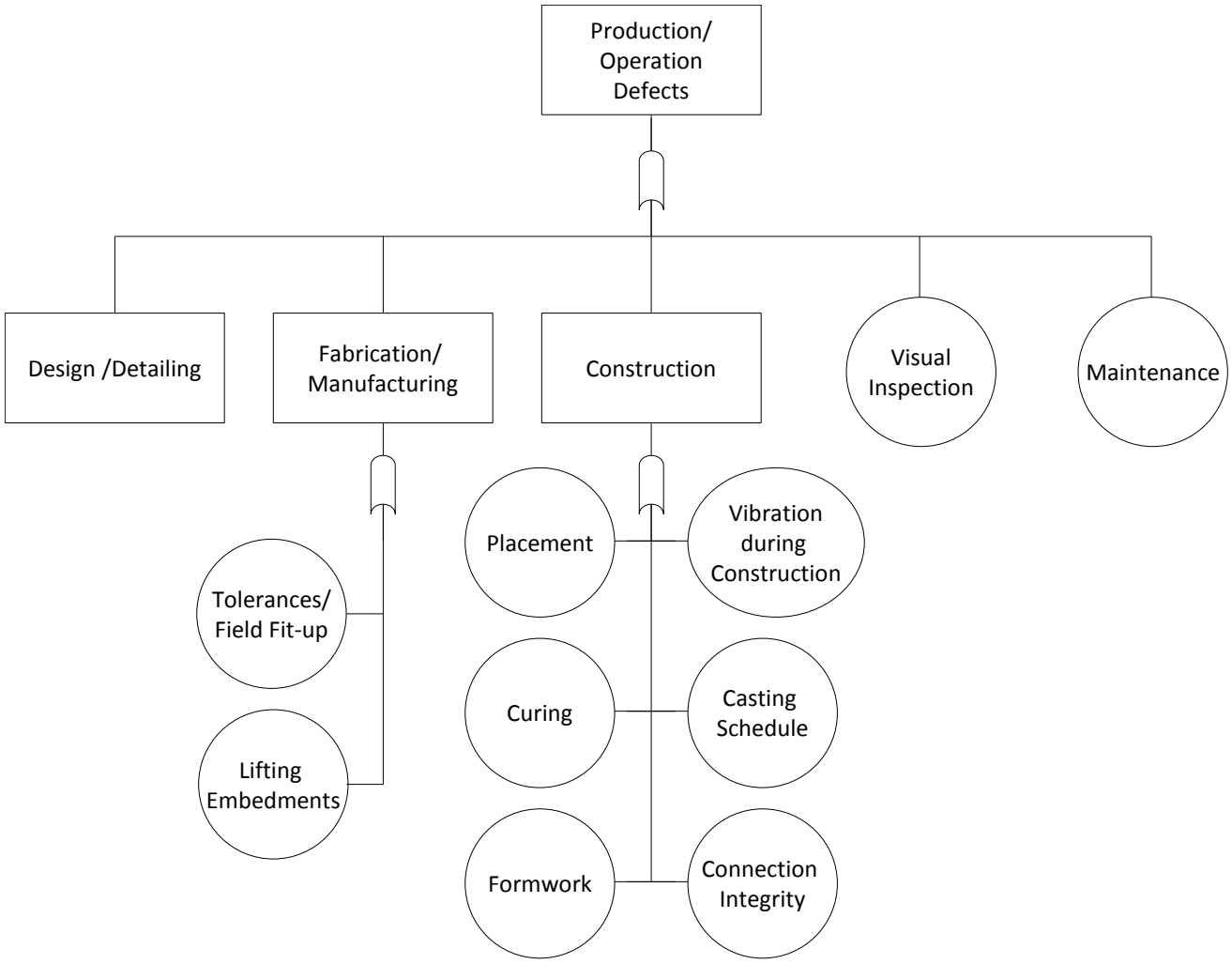


Figure 4.15. Precast concrete bridge deck design, construction (production) and operation defects fault tree.

4.2.2.1.1 Design and Detailing Bridge Deck Considerations

Decisions made during the design and detailing phase of a bridge project can significantly influence the service life of the precast bridge deck. It is incumbent upon designers to understand the implications of these decisions in order to help make rational choices that will improve service life. These influences are introduced in the fault tree provided in Figure 4.16, and include choices in regard to design philosophy, expansion joints, construction joints, concrete mix design, and bridge deck drainage. Again, many of these influences are similar to those for a cast-in-place concrete deck, except for the addition of composite action considerations for precast decks.

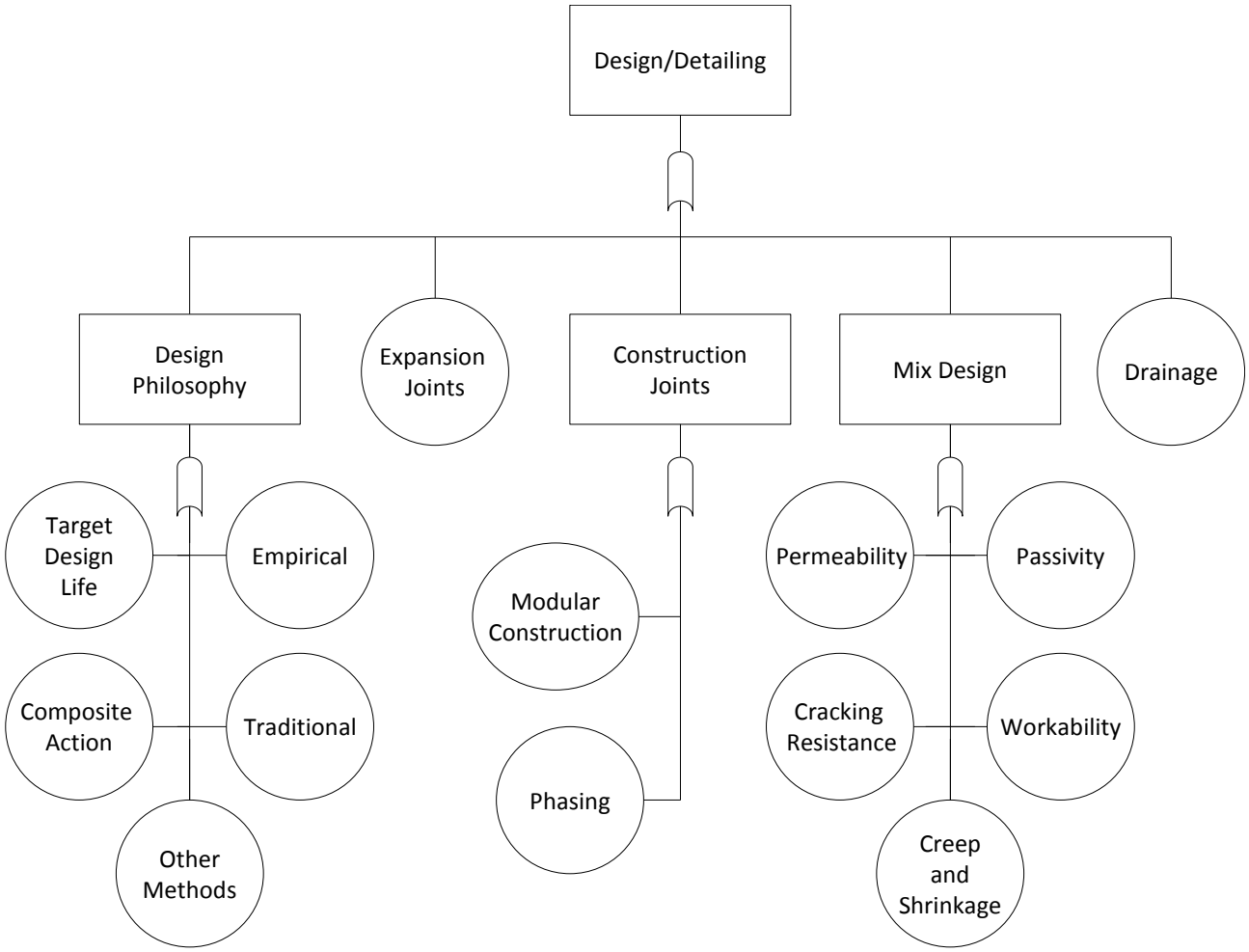


Figure 4.16. Precast concrete bridge deck design/detailing deficiency fault tree.

4.2.2.1.1a Composite Action

For precast systems such as full-depth deck panels, the design philosophy addresses either a composite or a non-composite connection of the deck panels to the supporting superstructure element.

Composite systems. Composite bridge deck systems add to the stiffness of the overall bridge system, reducing deflection and vibration and improving bridge deck performance. The connection requirements for composite systems are developed through field casting of concrete or grout around shear connectors or studs accessed through continuous full depth open pockets as shown in Figure 4.4, through localized, full-depth open pockets, or through continuous or localized embedded channels in the panels under the deck surface. Open pocket systems introduce construction joints, forming a discontinuity that can open as cracks. Embedded channel systems require pressure grouting to fill the void. Improper grout installation can lead to entrapped air voids that can fill with bleed water and

water intruding through deck cracks, leading to freeze/thaw issues and increased potential for reinforcement corrosion.

Non-composite systems In non-composite systems, excessive flexibility and inconsistent friction between the deck panels and the superstructure along the length of the supporting stringer could result in localized stress that can cause cracking and delamination in the concrete. Excessive vibrations can also lead to fatigue issues.

4.2.2.1.2 Modular Joint Construction

In precast systems consisting of adjacent members or segmental construction, the design and detailing of the joint is essential to its proper performance. These joints can open as cracks within the bridge deck if not properly considered, leading to leakage, spalling, and reinforcement corrosion.

4.2.2.1.3 Precast Component Fabrication and Manufacturing Considerations

Decisions relating to the fabrication and manufacturing of precast components for bridge decks can significantly impact the service life of the bridge. It is incumbent upon the fabricators to understand the implications of these decisions in order to help make rational choices for improving the service life of bridge decks. These decisions are introduced in the fault tree provided in Figure 4.15, and include choices in field fit-up and casting tolerances, as well as methods for lifting the precast elements in the precast yard and for erection in the field.

4.2.2.1.3a Tolerances and Field Fit-Up

Careful planning must be utilized to incorporate precast components as bridge decks on bridges. These components must be aligned in the field fairly accurately to ensure a smooth, safe ride for the travelling public. Often the construction sequence and schedule must be assessed to establish casting dimensions for the precast components. Liberal tolerances and insufficient control in the precasting facility can result in ill-fitting pieces that require unintended adjustments in the field leading to spalling and other structural failures due to localized, non-uniform contact surfaces that were not anticipated during design.

4.2.2.1.3b Transportation and Lifting Methods

Precast components must be moved from the casting facility to their final position in the bridge, a transport that may consist of multiple lifts depending on storage requirements and transportation schedules. These components must also be transported between the casting bed, storage facilities, and the project site. Improper consideration of

the stresses imposed on the precast component during these critical events can result in cracking, spalling, and sometimes failure.

Lifting precast components also requires devices for attaching slings and/or cables, usually consisting of steel embedded in the finished surface of the concrete. Improper removal and treatment of these embedments once the precast component is set in the field can result in localized spalls due to steel corrosion.

4.2.2.1.4 Construction

Attention to good practices during construction is crucial to the long-term service life of reinforced concrete. Well-qualified, well-trained, and well-executed workmanship increases productivity, reduces material waste, and provides expected service life. Proper use of adequate equipment provides better workability by increasing efficiency, and a well-planned construction schedule reduces the overall cost of the project by providing set times for equipment rental and reducing downtime. Proper use of test methods is needed to ensure that quality concrete is achieved. These practices are introduced in the fault tree provided in Figure 4.15, and include workmanship related to the connectivity of the precast components.

Connection of precast components is performed in several ways. Match cast components are typically joined with epoxy and prestressed across the joint. Improper epoxy material for the temperature of application, inadequate epoxy set time, and inconsistent, non-uniform application can cause spalling of the joint as the prestressing compresses the joint across a non-uniform contact surface.

Components that are not match cast are typically detailed with open joints to be filled with concrete or grout. Improper surface preparation and incomplete filling of these joints can result in early breakdown of the joint filler material, resulting in cracks, leakage, reinforcement corrosion, and a reduction in load distribution characteristics that can be detrimental to the carrying capacity of the bridge.

4.3 INDIVIDUAL STRATEGIES TO MITIGATE FACTORS AFFECTING SERVICE LIFE

Section 4.2 defined numerous factors affecting the service life of bridge decks. This section provides individual strategies to mitigate those factors and improve service life of bridge decks. Table 4.2 summarizes the areas in which strategies are provided.

Table 4.2. Mitigation Categories.

Section	Mitigation Category
4.3.1	Strategies to Mitigate Load-Induced Effects
4.3.2	Strategies to Mitigate System-Dependent Loads
4.3.3	Strategies to Mitigate Natural or Man-Made Environment Deterioration
4.3.4	Strategies to Improve Production and Operations

4.3.1 Strategies to Mitigate Load-Induced Effects

This section addresses concrete bridge decks. Load-induced effects are created from the traffic using the bridge and from system dependent framing restraints. Strategies for mitigating deterioration from these effects are provided in the following.

4.3.1.1 Strategies to Mitigate Traffic-Induced Loads

A complete understanding of the traffic characteristics utilized on the structure is required to define the strategies required for enhancing the service life of bridge decks. These characteristics include vehicle configuration, such as axle and wheel spacing and individual wheel weights; type of wheel or tire; potential for overloads; type of suspension system; traffic volumes and frequency of truck and overload application; and vehicle location on the deck.

In order to establish criteria to adequately address fatigue response, overload, wear, and abrasion, these characteristics must be understood. Table 4.3 identifies the strategies for these service life issues.

Table 4.3. Traffic-Induced Load Mitigating Strategies.

Service Life Issue		Mitigating Strategy	Advantage	Disadvantage
Traffic-Induced Loads	Fatigue	Design per *LRFD Specifications*	Minimizes the possibility of reinforcement failure	May increase the area of steel
	Overload	Increase deck thickness	Minimizes cracking	Adds weight to bridge structure, increases cost
	Wear and Abrasion	Implement concrete mix design strategies	See Chapter 3 - Materials	See Chapter 3 - Materials
		Implement membranes and overlays	Protects surface from direct contact with tires	Requires periodic rehabilitation every 10 to 20 years

Bridge deck systems can be adequately designed for fatigue based on individual wheel loads, dynamic impact considerations, and the frequency of load application developed from the volume of truck traffic to which the bridge deck will be subjected. The fatigue design of reinforcing steel within the concrete deck is adequately addressed by the threshold design methods provided in the *LRFD Specifications*.

Bridge decks can also be designed for overload conditions with adequate determination of the potential for overload and the frequency of its application. Strategies for addressing the additional strength requirements for overloads can be addressed by increasing the thickness of the deck.

Additional information on wear and abrasion can be found in Chapter 3. Membranes and overlays can be utilized to separate the wheel contact surface from the deck surface.

4.3.2 Strategies to Mitigate System-Dependent Loads

Bridge deck performance can be enhanced by the proper selection of a system to accommodate bridge movements, whether caused by differential shrinkage or from system framing restraints of movements from thermal expansion and contraction, and creep and shrinkage. Table 4.4 identifies the strategies for these service life issues.

Table 4.4. System-Dependent Load-Mitigating Strategies.

Service Life Issue		Mitigating Strategy	Advantage	Disadvantage
System-Dependent Loads	Differential Shrinkage	Use low modulus concrete mix designed for composite decks	Allows additional strain to be accommodated up to cracking stress	Typically lower in strength and may be subject to wear and abrasion
		Use high creep concrete mix designed for composite decks	Reduces locked-in stresses	Uncommon mix design. Difficult to assess stress relief.
		Develop composite action after concrete has hardened	Allows slippage between deck and supporting members, minimizing locked-in stresses	Very limited knowledge on available systems capable of developing composite action after hardening of bridge deck
		Use precast deck panels	Allows slippage between deck and supporting members, minimizing locked-in stresses	Introduces numerous construction joints
	Thermal Restraint	Develop an accurate system model for analysis purposes	Identifies design criteria for establishing stresses	Analysis is time consuming
	System Framing Restraint	Develop an accurate system model for analysis purposes	Identifies design criteria for establishing stresses	Analysis is time consuming

These strategies are further expanded in the following sections.

4.3.2.1 Differential Shrinkage

Several enhancements are viable for addressing the restraint forces at the interface of the bridge deck and supporting beam/stringer superstructure elements. Potential enhancements include:

- Using low modulus concrete mix design, which allows the deck to accommodate the shrinkage strain with less tension force, which can reduce cracking. Refer to Chapter 3 for additional discussion of low modulus concrete mix designs.

- Using a high creep concrete mix in the supporting superstructure element, which continues to reduce the locked-in tension force in the bridge. Refer to Chapter 3 for additional discussion of high creep concrete mix designs.

- Using delayed composite action systems where the interface of the bridge deck and the supporting beam/stringer superstructure elements is not made composite until a majority of the deck shrinkage has occurred.

4.3.2.2 System Framing Restraint

Superstructure and substructure systems must be designed to provide either movement or restraint of the structure, with proper consideration of internally-induced forces. For additional information on the proper system selection, see Chapter 2 - System Selection.

4.3.2.2.1 Fully-Integral Deck Systems

Eliminating expansion joints at abutments and over piers can enhance bridge deck performance. This bridge system is commonly referred to as a jointless bridge, and is addressed in Chapter 8.

4.3.2.2.2 Semi-Integral Deck Systems

A significant number of states utilize a semi-integral approach to bridge deck systems. This system provides expansion joints at the beginning and end bridge abutments, and no joints (or limited joints) in the remainder of the bridge. Bridge deck performance is improved by eliminating joints. Separating the bridge deck from substructure at the abutment locations reduces the tensile forces that could otherwise be generated in the bridge deck during deck contraction, as a result of traffic and thermal loads.

4.3.3 Strategies to Mitigate Natural or Man-Made Environment Deterioration

Proper studies for identifying environmental exposures detrimental to bridge deck performance should be performed. Understanding of the causes of deterioration leads to proper consideration during design.

The following tables describe the strategies developed for natural and man-made environment deterioration.

Table 4.5. Thermal Climate Environmental Deterioration.

Service Life Issue		Mitigating Strategy	Advantage	Disadvantage
Natural or Man-Made Environment Deterioration	Thermal Climate—Deicing Salts	Use impermeable concrete.	Increases passivity around reinforcement. Refer to Chapter 6 - Corrosion Protection of Steel Bridges.	High initial shrinkage, which can result in cracking.
		Use corrosion resistant reinforcement.	Eliminates deck spalls, delaminations, and cracking from reinforcement corrosion.	High cost. Limited availability. Some performance issues as noted in Chapter 3 - Materials.
		Use waterproof membranes/overlays.	Minimizes intrusion of dissolved chlorides into deck. Easily rehabilitated.	Requires periodic rehabilitation to replace riding surface every 5 to 20 years.
		Use external protection methods, such as cathodic protection.	Reduces corrosion. Refer to Chapter 5 - Corrosion Protection of Reinforced Concrete.	High cost. Requires extensive maintenance and anode/battery replacement. Could have limited effectiveness.
		Use effective drainage to keep surfaces dry and minimize ponding.	Minimizes intrusion of dissolved chlorides into deck.	Requires maintenance of drainage.
		Use periodic pressure washing to remove contaminants.	Minimizes intrusion of dissolved chlorides into the deck. Low cost.	Requires dedicated maintenance staff and appropriate budget.
		Use non-chloride based deicing solution.	Eliminates corrosion from chlorides.	High cost.
	Thermal Climate—Freeze-Thaw	Refer to Chapter 3 - Materials, for strategies relating to freeze thaw deterioration.		

Table 4.6. Technology Tables for Coastal Climate Environmental Deterioration.

Service Life Issue		Mitigating Strategy	Advantage	Disadvantage
Natural or Man-Made Environment Deterioration	Coastal Climate—Salt Spray	Use impermeable concrete.	Increases passivity around reinforcement. Refer to Chapter 3 - Materials.	Higher cost. Not effective at transverse cracking locations.
		Use corrosion resistant reinforcement.	Eliminates deck spalls, delaminations, and cracking from reinforcement corrosion. Refer to Chapter 3 - Materials.	High cost. Limited availability. Some performance issues as noted in Chapter 3 - Materials.
		Use waterproof membranes/overlays on travel services of bridge deck.	Minimizes intrusion of dissolved chlorides into the deck	Requires periodic rehabilitation every 5 to 20 years.
		Use external protection methods, such as cathodic protection.	Reduces corrosion. Refer to Chapter 5 - Corrosion Protection of Reinforced Concrete.	High cost. Requires extensive maintenance and anode/battery replacement.
		Use sealers on non-travel surfaces of bridge deck.	Minimizes intrusion of dissolved chlorides into deck.	Requires periodic rehabilitation every 5 to 10 years.
		Use corrosion resistant stay in place forms on bottom of bridge deck.	Minimizes intrusion of dissolved chlorides into deck.	Difficult to inspect.
		Use effective drainage to keep surface dry.	Minimizes intrusion of dissolved chlorides into deck.	Requires maintenance of drainage and periodic cleaning.
		Use periodic pressure washing to remove contaminants.	Minimizes intrusion of dissolved chlorides into deck.	Requires dedicated maintenance staff and appropriate budget.
	Coastal Climate—Humidity	Use materials that are not sensitive to moisture content.	Refer to Chapter 3 - Materials.	

Table 4.7. Chemical Climate, Reactive Ingredient, and Fire Deterioration.

Service Life Issue		Mitigating Strategy	Advantage	Disadvantage
Natural or Man-Made Environment Deterioration	Corrosion Inducing Chemicals and Sulfate	Use materials and mix design that are not sensitive to chemical attack.	Refer to Chapter 3 - Materials.	
	Reactive Ingredients—ASR and ACR	Use materials and mix designs that are not sensitive to aggregate reactivity.	Refer to Chapter 3 - Materials.	
	Extreme Events—Fire	Incorporate fire rating.	The height of the concrete structure and concrete cover can protect reinforcement from softening significantly, lessening collapse risk.	Can Result in High Cost. Increases weight with increased fire rating.
		Provide fire protective coatings.	Increases fire rating, reduces collapse risk.	Aesthetically unappealing. Subject to deterioration in an exposed environment.

Tables 4.4 through 4.7 present strategies for addressing the various factors affecting service life presented in Section 4.2. Proper incorporation of design features and materials is important for enhancing the service life of cast-in-place and precast bridge decks. Likewise, there are numerous protection strategies for enhancing the service life of concrete bridge decks. These include providing adequate concrete cover, proper concrete mix design, proper reinforcement selection and protection, proper drainage, application of prestressing, and the use of external protection systems. These enhancement strategies are described in the following.

4.3.3.1 Concrete Cover

Concrete cover is recognized as an effective method for protecting steel from corrosion. A minimum top concrete cover of two inches is required by *LRFD Specifications*. Generally, 2.5 in. of cover is used to allow for .5 in. of wear

over the life of the deck. Cracking of the deck, however, can allow chlorides to quickly penetrate to the level of the reinforcement, initiating the corrosion process.

4.3.3.2 Concrete Mix Design

The impermeability of concrete enhances the protection of bridge deck reinforcement. Concrete mix design is addressed earlier in this section and is further discussed in Chapter 3.

Enhanced service life of bridge decks can be achieved by implementing a mix design to obtain desirable properties for mitigating the potential for deficiencies. The desirable properties for enhanced performance include crack resistance through improved tension capacity, low permeability to delay chloride intrusion, low modulus of elasticity to allow deck strain with lower tension force, and high creep to allow reduce locked-in stresses over time.

Bridge deck concrete can also be enhanced by incorporating proper materials and admixtures, among them:

- Proper cement selection. In areas where sulfate attack may be a concern, Type II or Type V cements may be utilized to provide added resistance to its detrimental effects. Heat of hydration, which adds to the differential shrinkage strain, may be reduced by using a Type IV cement.

- Proper aggregate selection. Some readily available aggregates may be reactive to the internal concrete chemistry and be more susceptible to ASR and ACR. Admixtures and/or proper blending with non-reactive aggregate can minimize these effects. Using high quality aggregates also enhances abrasion resistance.

- Proper air entrainment agent. Air entrainment increases workability in the field and also enhances concrete performance when subjected to freeze/thaw cycles.

- Proper admixture selection. Numerous admixtures are available to enhance the properties of concrete and improve concrete durability substantially. In particular, admixtures for concrete decks can inhibit corrosion, improve workability providing a proper durable concrete finish, delay initial set to provide time for concrete placement and finishing, and reduce water requirements to improve concrete strength and density.

It should be noted that some of these enhancements may conflict, and therefore a mix design must incorporate desired features with the understanding that all enhancement strategies cannot be achieved. For more information on mix designs, refer to Chapter 3.

4.3.3.3 Reinforcement Selection

The selection of bridge deck reinforcement can enhance the service life of the bridge deck and increase resistance from corrosion and section loss, particularly in marine environments or in areas where deicing salts are used. Enhanced reinforcing steel includes:

- Corrosion-resistant reinforcing, such as FRP, stainless steel, and titanium bars;
- Reinforcement protection systems, such as epoxy coating and galvanizing; and
- Multiple posttensioning protection strategies, such as those defined by FHWA in the *Posttensioning Tendon Installation and Grouting Manual* (Corven and Moreton 2004).

Refer to Chapter 3, for more information on reinforcing materials.

4.3.3.4 Bridge Deck Drainage

Eliminating prolonged exposure to moisture, allowing the bridge deck to be maintained in a dry condition, can enhance the performance of bridge decks. Proper deck slopes, both transversely and longitudinally, should be provided to channel water to appropriate collection points. Construction joints at these collection points, as shown on the left in Figure 4.17, should be eliminated or moved away from the collection point, as shown on the right in Figure 4.17, to minimize contaminant intrusion, which can lead to the deterioration of reinforcement. Bridge drains, drain grates, and piping should be sized appropriately to be self-flushing, minimizing maintenance requirements.

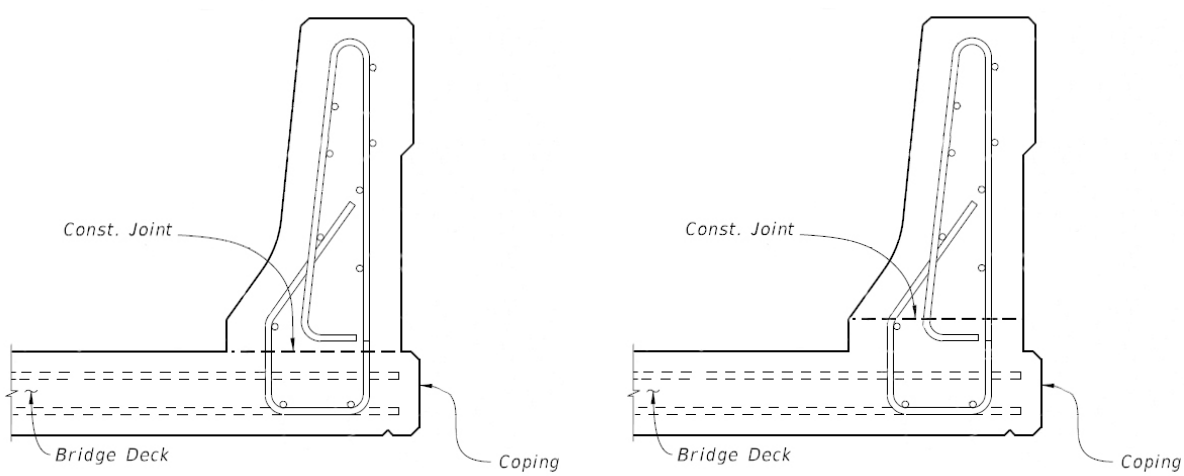


Figure 4.17. Construction joint at barrier—bridge deck interface.

4.3.3.5 Application of Compression to Relieve Tension

The elimination of tension in a cast-in-place bridge deck enhances the performance of the bridge deck by eliminating or significantly reducing deck cracking. Compression is typically introduced by posttensioning the concrete; however, the durability of the posttensioning is contingent on the proper incorporation of durability enhancements as well. Refer to Chapter 3 for additional information on the durability concerns of posttension systems.

Compression can also be introduced into the bridge deck of continuous girder bridge systems through the use of a self-stressing bridge deck system developed by *SHRP 2 R19A Project* (Silva 2011). The compression in this system, typically applicable to a two-span steel girder bridge unit, is introduced by casting the bridge deck with the intermediate support higher than required. After the deck has cured, the intermediate support is lowered to its final position, thereby introducing compression into the area of the deck usually subjected to tension forces from negative moments. The system, however, requires additional research in order to establish a history of satisfactory performance. Refer to Appendix A for more information on this system.

4.3.3.6 Membranes

Membranes are placed on top of the concrete and are protected by an asphalt layer that also functions as a riding surface. Effective waterproofing enhances service life of the membrane system, and in turn, the bridge deck.

4.3.3.7 Overlays

The purpose of concrete overlays is to create a low permeability protective layer over the conventional concrete on bridge decks. An overlay serves as a barrier to chloride ions, therefore increasing the time required for the concentration of the ions at the level of the reinforcement to reach the threshold for corrosion. Low permeability overlays also decrease water penetration into a structure allowing it to dry out, which reduces chloride ion mobility. Overlays can be applied to new decks, or as a rehabilitation method to existing decks. However, overlays are not as effective when applied to existing decks because if chloride ions are already present in the deck when the overlay is placed, then the only protection that the overlay can offer is a decrease in moisture infiltration.

The most common type of overlay has been a low-slump dense concrete overlay, which has been effective in extending the service-life of damaged bridge decks in some states. It requires special equipment to handle the very

stiff concrete, special attention to placement and consolidation are needed, and the overlays are prone to rapid loss of moisture necessitating extra care in curing. Recently, silica-fume concrete and latex-modified concrete overlays have been successfully used in extending the service life of contaminated structures and have improved workability compared to low-slump concrete overlays. Polymer concrete overlays are also available, generally as a temporary repair method on damaged bridge decks. Refer to Chapter 3, Materials, for more information on overlays.

4.3.3.8 Sealers

Sealers are expected to minimize the intrusion of aggressive solutions into concrete. The primary purposes of sealers are to prevent water and chloride ions from penetrating the concrete and thereby reduce the corrosion of reinforcement or the deterioration of concrete.

An important property of a sealer is its vapor transmission characteristics. Moisture within the concrete needs to pass through the sealer and escape in order to prevent high vapor pressures from building up in the concrete during drying periods, which could cause the sealer to blister and peel.

Sealers can be either pore blockers or water repellents. Pore blocker sealers work by forming a microscopically thin (up to 2 mm) impermeable layer on the concrete surface. Most are not appropriate for use on bridge decks because they do not offer good skid resistance and do not hold up under traffic wear. Water repellent sealers, on the other hand, work by penetrating slightly into the concrete and acting as hydrophobic agents. Hydrophobic sealers for bridge decks include silanes and siloxanes.

Sealers can protect all of the exposed concrete surfaces of the structure, including bridge decks, superstructure members, substructure members, and deck undersides. Proper surface preparation and consideration of application rates are key factors to be considered during installation of the sealer. Abrasion, sunlight, and the environment can affect the effective life of sealers, and resealing of the bridge deck could be expected every two to five years.

4.3.3.9 External Protection Systems

Several external protection systems are available to enhance the service life of cast-in-place bridge decks, including electrochemical chloride extraction and cathodic protection systems.

- Electrochemical chloride extraction (ECE). This system involves the application of a direct current to an existing bridge deck for a four to eight-week period. ECE extracts chlorides from concrete and enhances the

passivated zone around the reinforcement. An anode is provided by a titanium mesh or steel anode, which is temporarily placed on the concrete cover. This removes an average of 40% to 90% of the initial free chlorides. The extraction depends on the depth and location of the reinforcement. The pH of the concrete is increased, and the remaining chloride contents are typically below threshold levels near the reinforcement and increase with distance from the rebar. Prior application of this technology has resulted in more than 20 years of a passive/non-corroding condition. Typically applied to existing bridge decks, this system also has potential to pre-treat a new concrete bridge deck, enhancing the passivated zone around the reinforcement.

- Cathodic Protection. Cathodic protection is utilized to prevent corrosion from initiating, thereby reducing the concentration of chloride ions. Mainly used to prevent further corrosion after repair of damaged structures, this method has recently been used to prevent corrosion from initiating in new structures. The most common impressed-current anode uses a titanium mesh anode in conjunction with a concrete overlay or titanium ribbon. The current must be uniformly distributed and the system must be regularly monitored and inspected to ensure that polarization is in the desired range.

4.3.4 Strategies to Improve Production and Operations

Improving the performance of bridge decks relies on following proper methods and procedures during construction. The strategies to improve production and operations are included in Table 4.8.

Table 4.8. Mitigation of Production and Operation Defects.

Service Life Issue		Mitigating Strategy	Advantage	Disadvantage
Production/Operation Defects	Design Philosophy	Use empirical design.	Uses less reinforcement.	Limited application for future bridge widening.
	Expansion Joints	Eliminate expansion joints.	See Chapter 8 - Jointless Bridges	
	Construction Joints	Minimize construction joints.	Reduces corrosion potential. Minimal cost.	Requires larger casting volumes.
		Locate joint away from areas of ponding.	Minimizes saturation at joint location. Reduces corrosion potential. Minimal cost.	Could require additional formwork.
	Phased Construction	Use minimum width closure pour with UHPC.	Minimizes cracking.	High cost. Difficult to finish. Tight tolerances.
		Use wider closure pour, conventional concrete with waterproofing, and overlay.	Minimizes water intrusion and closure pour cracks. Shortens construction time. Accommodates residual differential deflection.	Slower construction, more cracking from differential shrinkage and restraint.
		Allow sufficient time for creep deformation to stabilize before casting closure pour.	Minimizes differential elevation between adjacent bridge sections.	More construction time and higher cost.

Mitigation of production and operation defects utilizes methods and procedures include proper construction joints selection, proper choice of formwork and bar supports, specification of enhanced placement procedures, and other maintenance considerations.

4.3.4.1 Construction Joint Selection

4.3.4.1.1 Sequence of Casting Deck

Bridge deck performance can be enhanced through proper selection of deck casting sequence. Concrete mix designs that delay initial set until the weight and vibration of casting and finishing machinery has passed, reduce concrete cracking potential from movement of the supporting structure below.

If the bridge deck cannot be cast in one operation, the appropriate location of construction joints and the proper sequencing of the deck pour can improve performance. Performance is enhanced by delaying the casting of those sections of the bridge susceptible to tension from adjacent casting operations. This is typical of casting sequences for continuous steel girders in which the positive moment areas are cast first, followed by the negative moment areas. The casting sequence should be specified by the designer and noted on the design plans.

4.3.4.1.2 Adjacent Members

Proper sealing, making the construction joint waterproof and preventing water intrusion between the bridge deck elements, enhances construction joints between adjacent cast-in-place members, such as transverse construction joints in cast-in-place segmental structures. Waterproofing strategies are addressed in the FHWA *Posttensioning Tendon Installation and Grouting Manual* (Corven and Moreton 2004) and in Chapter 3 on materials.

Numerous details have been utilized by many states for these types of structures. Many details transfer only the vertical shear across the construction joint, causing the adjacent members to act together vertically, but allowing the bridge system to flex at these joints. This at times has resulted in a breakdown of the material in the field construction joint. In general, details in which the connection between these adjacent members is designed to transmit the applied vertical shears and transverse moments, have performed significantly better over time.

Enhanced service life of filled construction joints between adjacent cast-in-place members can be achieved through the use of ultra-high performance concrete (UHPC). UHPC, described in Chapter 3, is a product that provides high strength and stiffness with negligible permeability and improved durability. It is expected to reduce maintenance requirements and extended service life. When used in bridge deck construction joints, consideration should be given to two issues. a) grinding the surface due to higher strength of UHPC and b) dissimilarities in color.

4.3.4.1.3 Staged/Phases Construction

Construction joints at the interface between adjacent phases of construction, such as in bridge widening, can be enhanced by a combination of the following:

- Properly locating the construction joint away from areas where water and waterborne contaminants can collect, such as at the construction joint between traffic railing barriers and the bridge deck, as shown in Figure 4.17;

- Ensuring proper reinforcement through the construction joint to control cracking;
- Applying epoxy to bond the surfaces together in order to prevent water intrusion, such as at the construction joint between traffic railing barriers and the bridge deck, as shown in Figure 4.17;
- Limiting live load influence near the joint to prevent vibration and joint flexing until concrete has attained the appropriate resistance to tension;
- Using admixtures in the concrete design to increase the time to initial set until all construction activities affecting the deflection of adjacent supporting members has been completed;
- Delaying the casting of the deck between adjacent phases of construction by adding a closure pour to be completed after the casting of the deck on the supporting members for both phases; and
- Addressing differential shrinkage between the phase-constructed closure pours and the adjacent completed bridge phases using procedures identified in Section 4.3.2.1.

4.3.4.2 Formwork

Formwork for bridge decks can be either removable or stay-in-place. Removable forms can be made of numerous materials, but usually consist of some type of plywood. Stay-in-place forms can consist of steel panels supporting a full-depth, cast-in-place deck, or precast panels that can be either composite or non-composite with the cast-in-place deck above.

The use of improved formwork technologies can improve the quality of the concrete surface, increasing its impermeability. Permeable formwork is a special class of lined formwork intended to produce improvements in the strength and durability of the surface of concrete. The bracing and the liner in the formwork are engineered to resist the pressure of plastic (or fresh) concrete, while allowing trapped air and excess water to pass through and be removed during concrete placement and consolidation. The objective in using permeable formwork is to eliminate voids (bug holes) on the surface of the concrete and increase the strength and durability of the concrete surface immediately behind the formwork.

Cast-in-place concrete with metal stay-in-place forms has gained popularity nationwide. However, several states are reluctant to adopt it because the underside cannot be easily inspected. The steel forms are susceptible to corrosion from salt spray and should be limited to areas where this type of corrosion is not an issue.

4.3.4.3 Bar Supports

Bar supports typically rest on concrete surfaces that are exposed to natural and manmade environmental hazards. Enhanced service life can be achieved through the use of non-corroding materials or non-corroding coatings on chair legs.

4.3.4.4 Bridge Deck Construction Procedures

Concrete placement procedures are fairly well established. Improvements in concrete mix design, placement, and curing specifications appear to have adequately addressed many of these service life issues, except for cracking and corrosion of reinforcement.

Service life is enhanced through proper planning of the bridge deck casting process. Concrete placement is enhanced by ensuring that sufficient vibration equipment is utilized; that vibration is effective at areas of congestion; such as at expansion joints; and that concrete is not dropped from excessive heights.

Curing is among the most important factors in developing durable deck concrete (Darwin et al. 2010) and is essential for the continuation of hydration reactions and the control of cracking due to volumetric changes. Curing of concrete is enhanced by ensuring that moisture is not lost. This can be accomplished through the use of curing compounds, and maintaining a wet curing environment, such as under a moist burlap covering, for seven to ten days. Performance enhancements are maximized with longer wet cure periods. The wet curing process also helps maintain thermal control of the bridge deck in its critical early stages of hydration. Refer to Chapter 3 on materials for more information on concrete curing.

4.3.4.5 Maintenance Considerations for Existing Bridge Decks

To extend concrete bridge service life in existing structures, preventive maintenance should be emphasized in a Maintenance Plan and proper repairs should be performed before extensive damage occurs and costly rehabilitation is required. The scope of repair or rehabilitation work can vary significantly from sealing cracks, to applying overlays,

to replacing large components such as bridge decks. Preventive maintenance may also include tasks as simple as washing the structure to eliminate chloride build up.

4.4 OVERALL STRATEGIES FOR ENHANCED BRIDGE DECK SERVICE LIFE

This section provides tools for selecting the most appropriate individual strategy to achieve the desired bridge deck service life and can be viewed as providing a template to the designer for selecting the optimum solution available, and quantitatively predicting the service life, where applicable.

Figure 4.18 identifies the flowchart for the bridge deck system component selection process. Figure 4.19 identifies the flowchart for the service life factor mitigation process.

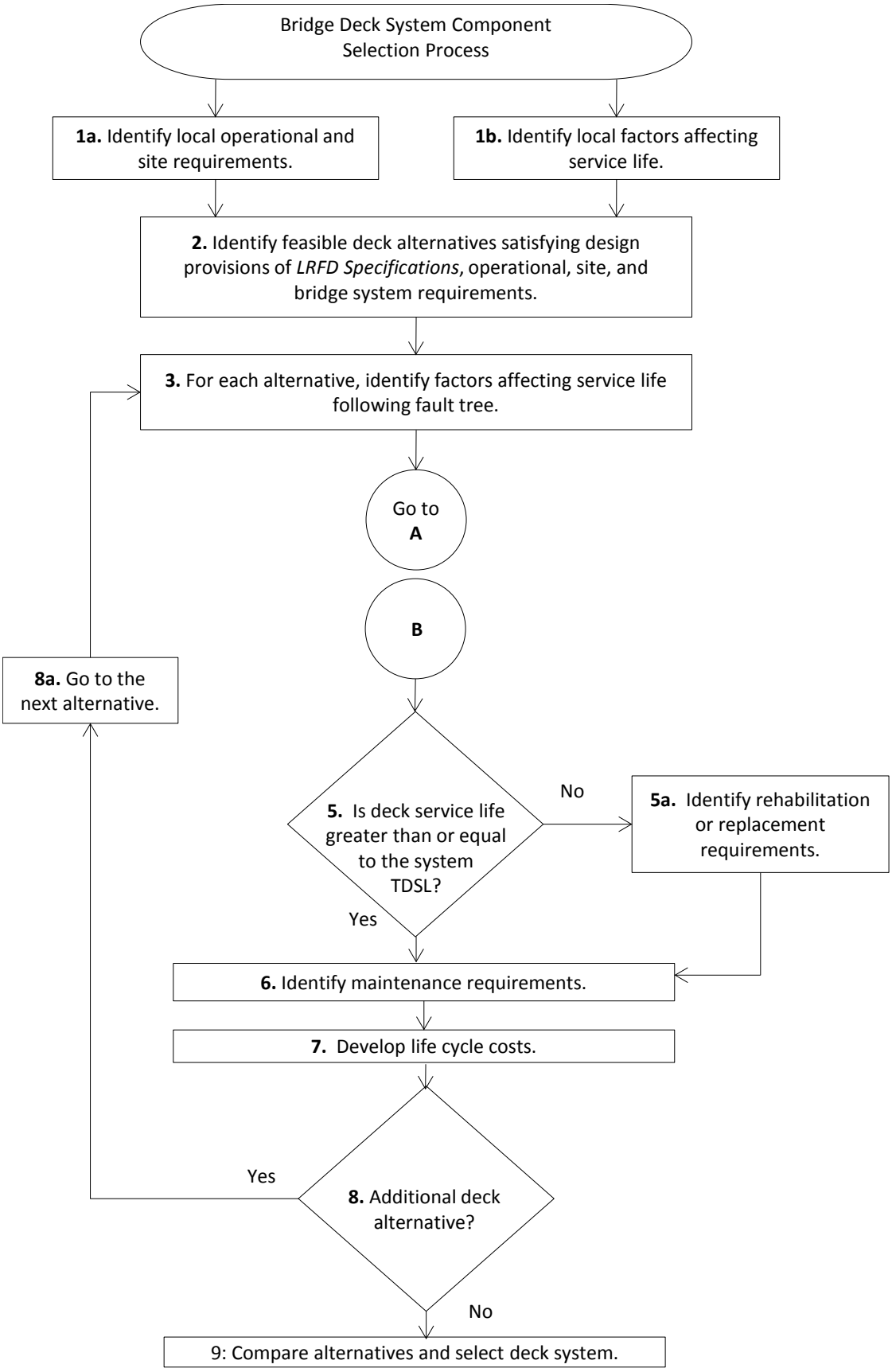


Figure 4.18. Bridge deck system component selection process.

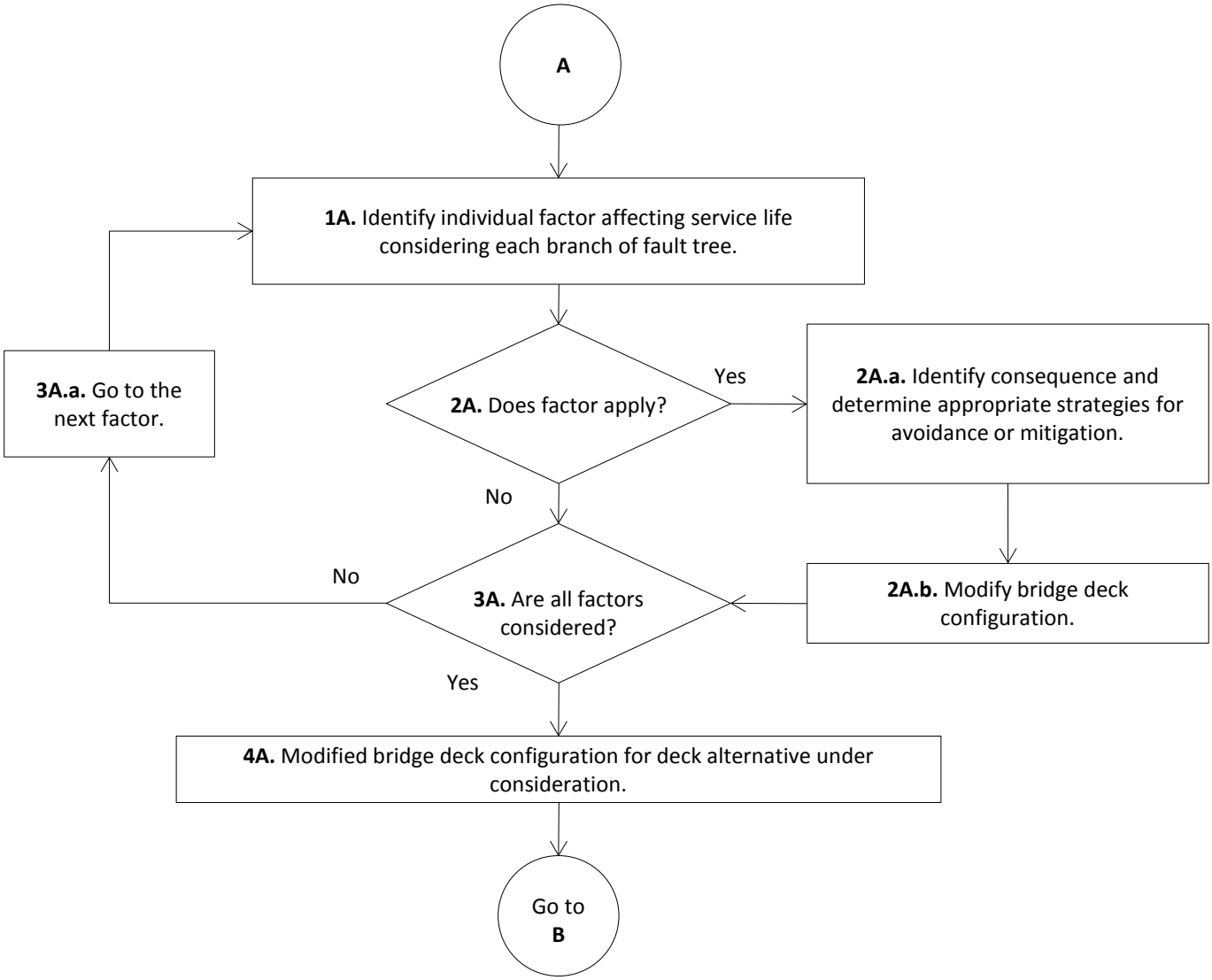


Figure 4.19. Mitigation of Factors Affecting Service Life Process.

An identifying number in each step designates each activity within the flowchart. These identifying numbers are used in the following discussion of the various elements of the flowcharts.

Steps 1a and 1b. Development of Design Criteria

These activities identify the project local operational site requirements and the local factors affecting service life. They are crucial to the development of design criteria used to identify and evaluate bridge deck alternatives. Examples of the information to be gathered during these activities are provided in Table 4.9

Table 4.9. Bridge Deck Demands.

Identify Demand—Local Operational, Site and Service Life Requirements	
Demand	**Examples**
Identify Owner Requirements	Legal and permit loads Commitments made during the NEPA process Noise Access Limitations Environmental / Biological Other design directives Acceptable risk Bridge deck target design service life Contingency planning for future expansion
Identify Traffic Load Demands	Potential for overloads Construction loads Impact considerations from suspension systems Frequency of load application Tire type for wear and abrasion
Identify System-Dependent Demands	Coordinate with bridge system selection Differential shrinkage effects Boundary conditions for system framing restraint Thermal exposure
Identify Environment/Man-made Hazards Demands	Deicing requirements Freeze/thaw potential Local aggregate reactivity Susceptibility to fire Susceptibility to collision. Chloride concentrations (natural) Chloride concentrations (applied) Sulfate concentrations Humidity levels
Identify Other General Demands	Traffic maintenance requirements Construction phasing requirements Need for accelerated construction Drainage and storm water requirements Identify local construction practice expertise

Step 2. Identification of Feasible Bridge Deck Systems.

The next step in the process is the selection of various bridge deck type alternatives that can meet the project requirements and the design provisions stated in the *LRFD Specifications*. For example, cast-in-place and precast deck systems could be identified as potential alternatives. Table 4.1 in Section 4.1 provides the advantages and disadvantages of these various decks. The selection of potential deck alternatives should consider the requirements of

other bridge subsystems, components and elements, and their interaction. Refer to Chapter 2, for more information on the overall bridge system requirements.

The most prominently-used bridge deck system is the cast-in-place or precast concrete bridge deck. This bridge deck can either be self-supporting as part of the overall superstructure system, e.g., voided slabs and segmental structures, or supported on beam/stringer superstructure elements. All of these concrete deck systems are subject to cracking, which typically results in reduced life from corrosion.

Selection of the overall concrete bridge deck system may be affected by the following:

- Need for accelerated construction to shorten overall user impacts;
- Maintenance of traffic requirements that may dictate construction staging;
- Commitments made during the NEPA process, such as acceptable noise levels, access limitations or environmental/biological limitations that may dictate a precast system;
- Availability of special mix designs to provide a more durable concrete;
- Availability and construction expertise to incorporate prestressing to compress the concrete, minimizing or eliminating tension in the concrete;
- Availability and the construction expertise to incorporate internal and/or ancillary protective systems for the bridge deck; and
- Ability to provide an alignment and/or a bridge drainage system to prevent ponding of water and soluble pollutants on the bridge deck.

Step 3. Identification of Factors Affecting Service Life

At this step in the process, feasible bridge deck alternatives are selected and designed based on applicable provisions in the *LRFD Specifications*. For each feasible deck alternative, factors affecting service life are identified using the fault tree described in Section 4.2.

Process A. Refinement of Alternatives—Mitigation of Factors Affecting Service Life

The process continues with the development of mitigating design features for each factor affecting service life identified under Step 3, as shown in Figure 4.19. An alternative selection process would be to evaluate these strategies by examining material selection and protection strategies, construction practice specification requirements, and maintenance requirements. A summary example of this alternative process is provided in Table 4.10. Further explanation of some for these elements is provided in the text following the table.

Table 4.10. Alternative Bridge Deck System Development Process.

Demand	Examples
Determine Function Strength Design	Accommodate span requirements and constraints Accommodate curvature and skew requirements Accommodate proposed deck joint layout and its effect on system restraint
Identify Deck Deterioration Modes	Concrete cracking Reinforcement corrosion Wear and abrasion Aggregate reactivity Freeze/thaw
Assess Risk, Deterioration Consequences	Loss of required strength Safety concerns from pot holing (large concrete spalls) Loss of skid resistance
Develop Mitigation Strategies (Section 4.3)	Traffic-induced loads System-dependent loads Natural or man-made environment/hazard deterioration
Develop Design Requirements	Concrete cover Concrete mix design Reinforcement selection Bridge deck drainage Introduction of compression to relieve tension Application of Membrane, overlay, and sealers External protection systems
Identify Strategies to Improve Construction	Sequence of deck casting Construction joint location and detailing Construction phasing and staging details Formwork selection Bar support selection Specification of construction procedures Quantification of maintenance requirements

The steps of this process include the following:

Step 1A. Identify the individual factor affecting service life by considering each branch of the fault tree defined in Section 4.2.

Step 2A. Evaluate whether this factor has an effect on the service life of the bridge deck, based on the design criteria. If the factor identified in Step 2a has an effect on the service life of the bridge deck,

Step 2A.a. Identify the consequences of the factor, and determine appropriate strategies to mitigate or avoid the effects of deterioration. Refer to Section 4.3 for mitigating/avoidance strategies. There may be more than one strategy alternative to consider for each factor. For example, bridge decks subjected to wear and abrasion can be mitigated through concrete mix design alternatives and/or the use of waterproofing membranes and overlays. It should be noted that the Applicable Factors identified in the "circle" symbols are basic factors and demand development of strategies to mitigate them.

Step 2A.b. Modify the bridge deck alternative under consideration as needed to address the incorporation of the chosen strategy(ies).

Step 3A: Continue with the evaluation process until all factors have been considered, then continue to the next factor within the fault tree (Step 3A.a) until all factors have been considered.

Step 4A: Finalize the modifications to the bridge deck configuration for the bridge deck alternative under consideration. Verify that strength and service performance have not been affected.

A sample series of strategies for a bridge with cast-in-place concrete deck supported on stringers is shown in the next three tables. Table 4.11 provides strategies for cast-in-place bridge deck systems, and Table 4.12 provides strategies for precast bridge deck systems. The identified systems can achieve long service life with the proper inspection and maintenance as indicated. Table 4.13 provides strategies for rehabilitation of existing bridge decks, which may be used in the case of a bridge widening. Note that deterioration modes addressed through proper design, such as fatigue and overload potential, are not addressed with a strategy in these tables. Other material deterioration modes, such as freeze/thaw and sulfate attack, are addressed in Chapter 3.

The selection of appropriate material and protection strategies is fairly consistent for all of the concrete bridge deck systems included in this chapter, with only minor differences in the durability performance selection process between cast-in-place and precast deck systems. Selecting the appropriate strategy for each potential deterioration mode must be compared for conflicts in order to establish the overall strategy to be deployed for a specific project. For example, a bridge deck with the potential for deterioration from wear and abrasion and differential shrinkage

cannot easily use concrete with both high strength and a low modulus. In this case, the wear and abrasion strategy using the overlay/membrane is more appropriate.

Table 4.11. Bridge Deck Selection Strategies—Cast-in-Place Systems.

Potential Deterioration Mode	Material Selection and Protective Measures Selection	Maintenance Modes	Life-Cycle Costs	
			Initial	Long-Term
Differential Shrinkage and Thermal Restraint	Low modulus concrete Proper bearing design	None	Low	Low
Wear/Abrasion	Overlay/membrane	Continual overlay replacement 5 to 20 years	Med	Med
Wear/Abrasion	High concrete strength Hard aggregates	None	Med	Low
Wear/Abrasion	Sacrificial thickness and overlay	Continual overlay replacement 5 to 20 years	Low	Med
Reinforcement Corrosion	Overlay/membrane	Continual overlay replacement 5 to 20 years	Med	Med
Reinforcement Corrosion	Corrosion resistant rebar	None	High	Low
Reinforcement Corrosion	External protection systems	Continual inspection and system maintenance	High	High
ASR	Refer to Chapter 3 - Materials			
ASR	Blended aggregates, proper drainage	None	Med	Low
ASR	Blended aggregates, waterproof membrane, proper drainage	Continual overlay replacement 5 to 20 years	Med	Med
ACR	Refer to Chapter 3 - Materials			

Table 4.12. Bridge Deck Selection Strategies—Precast Systems.

Potential Deterioration Mode	Material Selection and Protective Measures Selection	Maintenance Modes	Life-Cycle Costs	
			Initial	Long-Term
Differential Shrinkage and Thermal Restraint	Low modulus concrete, proper bearing design	None	Low	Low
Wear/Abrasion	Overlay/membrane	Continual overlay replacement 5 to 20 years	Med	Med
Wear/Abrasion	High concrete strength / Hard aggregates	None	Low	Low
Wear/Abrasion	Sacrificial thickness and overlay	Continual overlay replacement 5 to 20 years	Low	Med
Reinforcement Corrosion	Overlay/membrane	Continual overlay replacement 5 to 20 years	Med	High
Reinforcement Corrosion	Corrosion resistant rebar	None	High	Low
Reinforcement Corrosion	External protection systems	Continual inspection and system maintenance	High	High
Reinforcement Corrosion	Application of compression by design to eliminate tension	None	Med	Low
Reinforcement Corrosion	Application of compression through posttensioning	Continual inspection, supplemental posttensioning	Med	Med
ASR	Refer to Chapter 3 for material component based solutions			
ASR	Blended aggregates, proper drainage	None	Med	Low
ASR	Blended aggregates, waterproof membrane, proper drainage	Continual overlay replacement 5 to 20 years	Med	Med
ACR	Refer to Chapter 3 for material component based solutions			

Table 4.13. Bridge Deck Selection Strategies—Existing Bridge Decks.

Potential Deterioration Mode	Material Selection and Protective Measures Selection	Maintenance Modes	Life-Cycle Costs	
			Rehab	Long-Term
Wear/Abrasion	Overlay/membrane	Continual overlay replacement 5 to 20 years	Med	Med
Reinforcement Corrosion	Overlay/membrane	Continual overlay replacement 5 to 20 years	Med	High
Reinforcement Corrosion	External protection systems	Continual inspection and system maintenance	High	High
Reinforcement Corrosion	Epoxy injection of deck cracks	Continual inspection and system maintenance	Med	Med
ASR	Waterproof membrane, proper drainage	Continual overlay replacement 5 to 20 years	Med	Med

Step 5. Check Service Life

At this step in the process, the service life for the bridge deck alternative is determined either through the use of deterioration models or through empirical evidence based on past performance. Refer to Chapter 1 for methods available for predicting service life. If the bridge deck service life does not equal or exceed the bridge system design service life, then rehabilitation and/or replacement requirements (Step 5.a) should be added to increase the longevity of the bridge.

Step 6. Identify Maintenance Requirements

At this step in the process, maintenance requirements for the proposed alternative and their associated costs should be identified. Table 4.14 identifies example maintenance issues. The cost for developing a Maintenance Plan should also be included in the bridge Owner's Manual. Refer to Chapter 1 for a detailed description of the bridge Owner's Manual.

Table 4.14. Maintenance Requirements.

Maintenance Requirements	
Demand	**Examples**
Maintenance Issues	Inspection requirements and intervals Drainage system maintenance Membrane/overlay/sealer maintenance Expansion joint maintenance Health monitoring Scheduled maintenance
Examples of Items to be included in the bridge Owner's Manual	Describe how bridge was designed, constructed, and intended to function from an operational perspective. Include: Design loads Expected movements at expansion joints Relevant as-built data, including, but not limited to: - Concrete mix design - Slump test results - Chemical content of materials - Curing methods utilized - Compression cylinder test results - Reinforcing steel material certifications - Coating tests on reinforcement - Formwork materials - Actual construction procedures - Temperature of concrete - Ambient temperature and time of casting - Timing of casting sequence and concrete delivery - Concrete cover measurements For each component, describe what is needed to achieve design service life for specific elements. Include: Required maintenance Expected rehabilitation and/or replacement of bridge elements with service life less than overall bridge system design service life Areas for inspection and types of adverse behavior to watch for

Step 7. Life-Cycle Cost Analysis (LCCA)

At this step in the process, the bridge deck alternative is evaluated to establish its life-cycle cost. LCCA provides an assessment of the overall long-term cost of a strategy throughout the target service life of the structure. LCCA is discussed in Chapter 11 - Life Cycle Cost Analysis.

Step 8. Consideration for Additional Bridge Deck Alternatives

Once the life-cycle cost is established, the bridge deck alternative under investigation is complete and the process continues cycling back to Step 3 until all feasible bridge deck alternatives are addressed.

Step 9. Alternatives Comparison and Deck System Selection

The final step in the process is the comparison of bridge deck system alternatives. The life-cycle cost of these alternatives is combined with the analysis performed for each bridge subsystem, component, and element for a specific bridge system. Refer to Chapter 2, for the overall bridge system selection process. In general, the most cost-effective bridge deck system should be utilized; however, the cost-effectiveness has to be weighed against the requirements of the overall bridge system. The selection should also be presented to the bridge owner for acceptance.

CHAPTER 5

CORROSION OF STEEL IN REINFORCED CONCRETE BRIDGES

5.1 INTRODUCTION

This chapter of the *Guide* provides essential information for addressing corrosion of reinforcing steel in conventionally reinforced concrete structures. The focus is on controlling and mitigating corrosion for extended durability and service life. Corrosion in prestressed or posttensioned concrete structures is not discussed.

The description of corrosion in Section 5.2 covers the diffusion process that enables the penetration of chlorides through concrete and the creation of corrosion cells once the chlorides infiltrate. It also addresses the patch-accelerated corrosion commonly referred as "ring anode corrosion" in repairs. Section 5.3 describes factors influencing corrosion including chloride contamination and carbonation. Section 5.4 provides a summary of strategies for addressing corrosion in new and existing structures; different levels of corrosion protection is considered, such as the corrosion prevention, corrosion control, corrosion passivation, and the electrochemical treatments. Section 5.5 summarizes case studies that address corrosion in existing structures.

Faced with rising maintenance costs, many engineers and owners recognize the need to protect existing structures from future corrosion damage. As a result, the use of corrosion mitigation systems to delay the need for future concrete rehabilitation is increasing. Selecting the appropriate corrosion mitigation approach is based on many factors, including the amount and depth of contamination (chloride ingress or carbonation), amount of concrete cracking and concrete damage, severity and location of corrosion activity (localized or widespread), expected environmental exposure, use and service life of the structure, and the cost and design life of the corrosion protection system.

Deicing salts applied during winter months generally contain chlorides. Chloride solutions penetrate existing cracks and diffuse through the concrete cover to the reinforcing steel, initiating corrosion. Corrosion products exert stresses that can crack the concrete and cause delaminations and spalling.

One approach to mitigating the problem is to prevent or minimize chloride penetration of chlorides by minimizing cracking using low permeability concretes, adequate concrete cover over the steel, membranes, sealers,

or overlays. Another approach is to prevent the steel from corroding or to minimize the rate of corrosion through means such as the use of corrosion-resistant reinforcement or cathodic protection. Depending on the specifics of a project, one or a combination of both of these approaches may be desirable.

5.2 DESCRIPTION OF CORROSION

5.2.1 The Corrosion Process

The corrosion of steel in concrete is an electrochemical reaction similar to that of a battery. The corrosion rate is influenced by various factors including chloride-ion content, pH level, concrete permeability, and availability of moisture to conduct ions within the concrete. For corrosion to initiate in reinforced concrete, four elements are required to complete the corrosion cell: an anode, a cathode, ionic continuity between the anode and cathode through an electrolyte, and a metallic (electrical) connection between the anode and cathode.

The anodic site becomes the site of visible oxidation (corrosion) and the cathode is the location of the reduction reaction which is driven by the activity at the anode. In reinforced concrete, the metallic path can be provided by the mild steel reinforcing or embedded prestressing strands. The ionic path is provided by the concrete matrix with sufficient moisture due to the permeability of concrete.

At the anode, iron is oxidized to ferrous ions:

$$Fe \rightarrow Fe^{2+} + 2e^{-}$$

At the cathode, a reduction reaction takes place. In an alkaline environment, the reduction reaction is typically:

$$2H_2O + O_2 + 4e^{-} \rightarrow 4OH^{-}$$

For the corrosion process to be initiated, the passive oxide film on the reinforcing steel must be broken. In most cases this is due to the presence of sufficient quantities of chloride ions in the concrete matrix at the level of the steel. Chloride-induced corrosion is most commonly observed in structures exposed to roadway deicing salts or in marine environments with direct exposure to salt water or wind-borne sea spray. Chlorides can also be introduced to the concrete during the original construction by the use of contaminated aggregates, water, or chloride-containing admixtures.

Over time, the corroding area (anode) will become more acidic as hydroxyl ions (OH^-) are consumed from the concrete in contact with the corroding area, and the cathode will become more alkaline by the generation of hydroxyl ions (OH^-).

5.2.2 The Diffusion Process

To the casual observer, uncracked concrete is a solid and impenetrable material. Viewed under a microscope, however, concrete is a labyrinth of fine capillaries, pores and voids between the individual cement and aggregate particles. The degree of porosity depends on the quality and density of the concrete mix.

Due to this porosity, liquids can soak into the exposed surfaces of the concrete and carry contaminants such as chloride ions with them. Over time, the concentration of chloride ions within the concrete will tend to equalize as governed by Fick's Law.

In a one-dimensional case, Fick's Law can be expressed and illustrated as follows;

$$C_{(x,t)} = C_o\left(1 - erf\,\frac{x}{2\sqrt{D_c t}}\right) \qquad \textbf{EQ 5.1}$$

Where:

$C_{(x,t)}$ = chloride concentration at depth x and time t
C_0 = surface chloride concentration (kg/m^3 or lb/yd^3)
D_c = chloride diffusion constant (cm^2/yr or in^2/yr)
erf = error function (from standard mathematical tables)

This expression indicates that the chloride concentration within the concrete will tend to equalize with the chloride concentration exposed to the surface over time. As expected, the chloride concentration within the concrete is greater near the exposed surface and increases with time at any point within the concrete. Concrete with a lower chloride diffusion constant (D_c) will provide longer term protection to reinforcing steel located at depth *x* from the surface of the concrete.

The diffusion constant for a particular point in a concrete structure may be determined in which chloride data is available for one location at two points in time or in which a complete chloride profile is available some time after the structure has been constructed. With current chloride data and an estimate of the diffusion coefficient, future chloride profiles can be predicted using the formula in Equation 5.1. Figure 5.1 displays the chloride concentration within concrete over time.

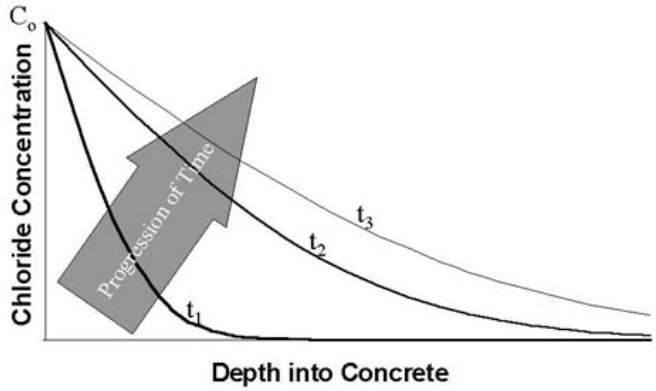


Figure 5.1. Chloride concentration within concrete over time.

Figure 5.2 shows the chloride contents with depth. Based on the chloride profile, the calculated (best fit) diffusion coefficient D(c) is 4.38 x 10^{-13} cm^2/sec.

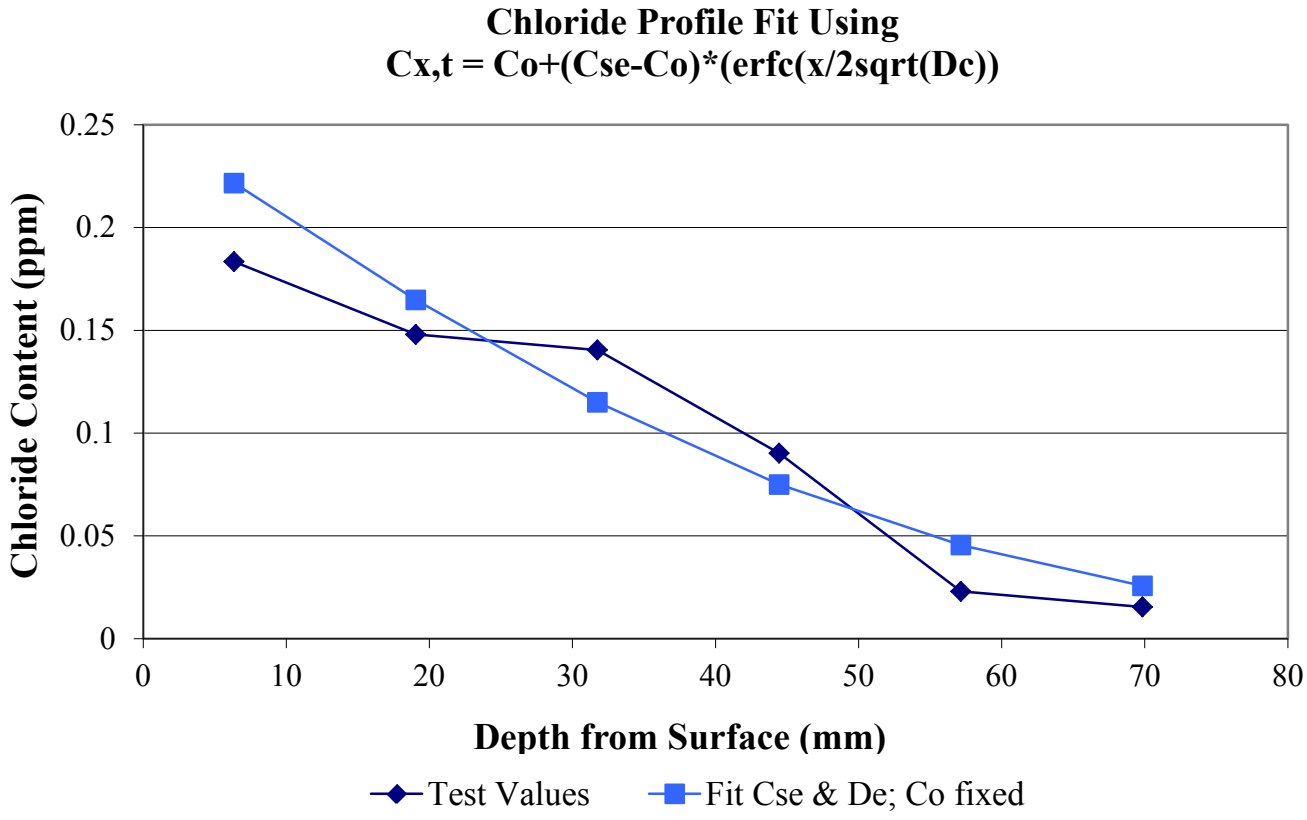


Figure 5.2. Chloride contents with depth.

If the concrete element is cracked, chloride penetration at crack locations may greatly exceed chloride levels in the surrounding concrete. This can lead to corrosion initiation at crack locations long before general corrosion may otherwise occur.

5.2.3 Corrosion Cells

Once the chloride concentration at the depth of the reinforcing steel exceeds threshold levels, the passive oxide film will begin to degrade and corrosion may be initiated. Chlorides act similarly to a catalyst in the corrosion process—the chlorides are involved in the corrosion reaction, but are generally not consumed by the corrosion reaction itself, such that a single chloride ion can be responsible for the corrosion of many atoms of iron.

On a localized basis, corrosion cells can be formed due to differences in chloride concentration at various locations along a single bar. These variations can result in localized pitting-type corrosion. Similarly, if entire sections of a reinforced concrete structure become contaminated relative to other adjacent areas, an overall corrosion cell or "macro-cell" can be created as illustrated in Figure 5.3. Macro-cell corrosion can be very aggressive and is responsible for much of the severe structural damage seen on bridges and other structures. Both pitting-type corrosion and general corrosion result from corrosion cells.

The corrosion products (rust) occurring as a result of macro-cell corrosion occupy a large volume and cause cracking, concrete delamination, and spalls.

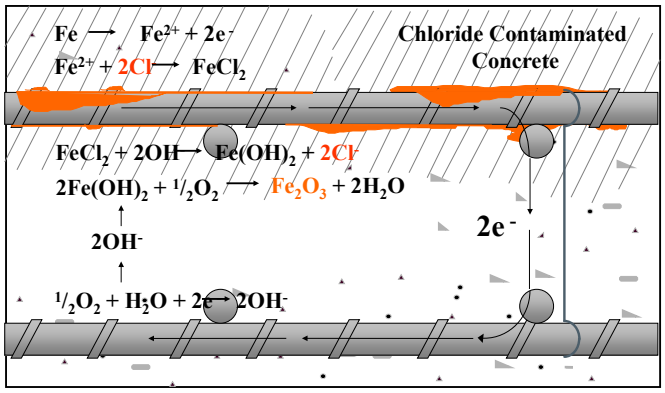


Figure 5.3. Corrosion macro-cell in a concrete deck.

5.2.4 Patch Accelerated Corrosion

Commonly referred to as “ring anode corrosion” or “halo effect,” patch accelerated corrosion is a phenomenon specific to concrete restoration projects (Figure 5.4). When repairs are completed on corrosion-damaged structures, abrupt changes in the concrete surrounding the reinforcing steel are created. Typical concrete repair procedures call for the removal of the concrete around the full circumference of the reinforcing steel within the repair area, cleaning corrosion by-products from the steel, and refilling the cavity with new chloride-free, high pH concrete. These procedures leave the reinforcing steel embedded in adjacent environments with abruptly different corrosion

potentials. This difference in corrosion potential (voltage) is the driving force behind new corrosion sites forming in the surrounding contaminated concrete. The evidence of this activity is the presence of new concrete spalling adjacent to previously completed patch repairs.

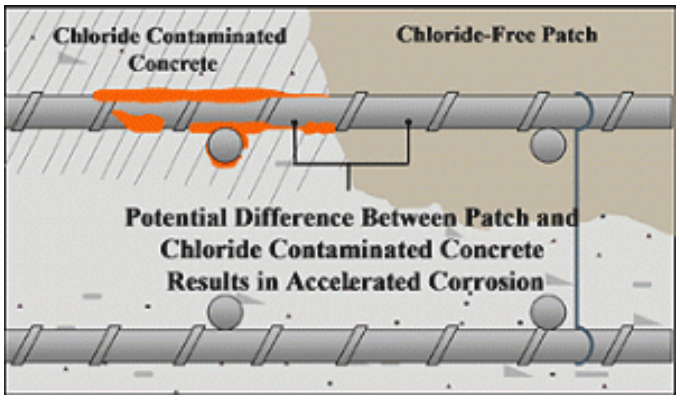


Figure 5.4. Patch accelerated corrosion.

5.3 FACTORS INFLUENCING CORROSION

One of the leading causes of concrete rehabilitation is corrosion-induced concrete damage and spalling in reinforced concrete structures (Figure 5.5). In steel reinforced concrete, the concrete matrix must be sufficiently strong to resist applied forces from a structural standpoint, and to serve as a corrosion protection mechanism for the embedded reinforcing steel. The ability of concrete structures to resist corrosion attack is not related to the mechanical strength of the concrete alone. Instead, two important factors limit the ability of concrete structures to resist corrosion: the presence of cracks and the porosity of the concrete.

The presence of cracks and the ability of chlorides to permeate the concrete allow chlorides to get to the reinforcing steel, thus compromising the corrosion resistance provided by concrete's naturally high alkalinity.

Figure 5.5: Corrosion-induced delamination on a bridge pier.

Numerous factors may influence the durability of concrete including water-cement ratio, permeability, curing, shrinkage and cracking, ingredients including admixtures, and the severity of environmental exposure. Due to the high alkalinity of the concrete pore water solution, a thin passive oxide layer is formed and maintained on the surface of the embedded steel thus, protecting it from corrosion activity. Until this passive film is destroyed by the intrusion of aggressive elements or a reduction in the alkalinity of the concrete, the reinforcement will remain in a passive, non-corroding state.

The causes of corrosion are summarized in Figure 5.6. Chloride contamination and carbonation are explained in the sections that follow.

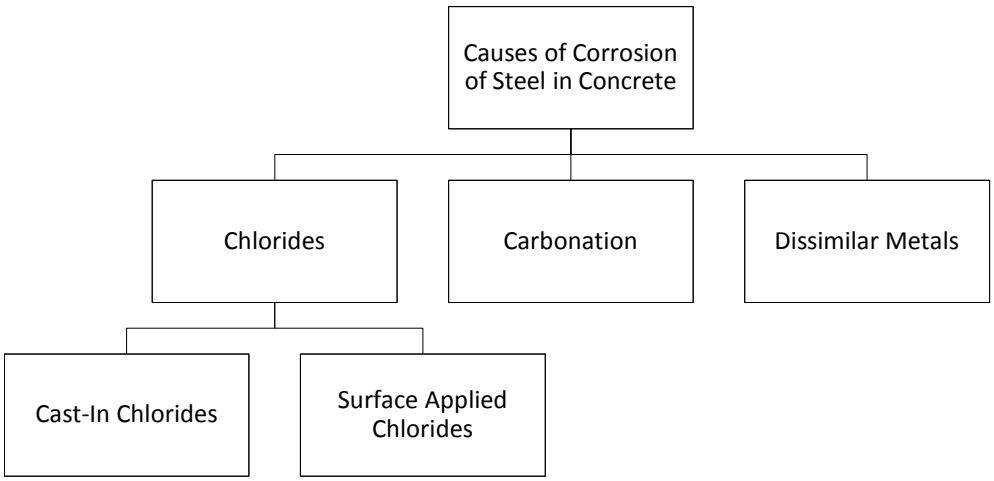


Figure 5.6. Causes of corrosion of steel in concrete.

5.3.1 Chloride Contamination

Destruction of the protective oxide film on reinforcing steel is most often caused by the presence of elevated levels of chloride ions. The chloride threshold that initiates corrosion is generally considered to be around 1.0 to 1.4 lbs of water soluble Cl^- per cubic yard of concrete (at the level of the steel). This chloride threshold varies depending on the pH of the concrete. For example, concrete that has experienced a loss of alkalinity requires less chloride to initiate corrosion. Chloride-induced corrosion, shown in Figure 5.7, is common in structures exposed to deicing salts, marine environments, or certain industrial processes. In some cases, sufficient amounts of chlorides capable of causing corrosion have been introduced during construction by the use of chloride-containing admixtures or contaminated aggregates.

Non-chloride-bearing salts including calcium magnesium acetate, magnesium acetate, and calcium acetate, lower the freezing temperature of water and can be used for ice control. However, magnesium-bearing solutions cause severe paste deterioration by forming brucite and non-cementitious magnesium silicate hydrate (Lee et al. 2000). The detrimental effects caused by calcium acetate were much less severe than those containing magnesium, but the use of these non-chloride-bearing salts has not gained wide acceptance due to cost and the distress they cause.

Figure 5.7. Chloride-induced corrosion of reinforcing steel.

5.3.2 Carbonation

The passive condition of the reinforcing can also be disrupted by the loss of alkalinity in the concrete matrix surrounding the reinforcing steel. It is generally accepted that a pH greater than 10 is sufficient to provide corrosion protection in chloride-free concrete. The reduction in alkalinity is generally caused by carbonation, a reaction

between atmospheric carbon dioxide and calcium hydroxide in the cement paste in the presence of water. The result is a reversion of the calcium hydroxide to calcium carbonate (approx. pH 8.5) which has insufficient alkalinity to maintain the passive oxide layer.

The zone of carbonation begins on the surface of atmospherically exposed concrete. The amount of time for the carbonation zone to reach the level of the reinforcing is a function of the thickness of concrete cover, presence and extent of cracks, concrete porosity, humidity levels, and the level of exposure to carbon dioxide. Carbonation-induced corrosion is more likely to be observed in structures situated in industrial environments where airborne pollutants are commonplace, or in old or historic structures, structures with a high degree of concrete porosity, or structures with low concrete cover over the reinforcement.

In bridge structures, there is generally good quality concrete cover of sufficient thickness (about 2.5 in. in decks) to resist carbonation for up to 100 years.

5.4 STRATEGIES FOR ADDRESSING CORROSION

Because of the magnitude of the corrosion problem, both the public and private sectors have ongoing activities aimed at reducing or eliminating corrosion damage to concrete structures. While many technologies and materials have been developed for prevention and repair of corrosion-induced damage, the challenge is to select durable, cost-effective technologies and materials from the numerous choices available.

Durability and desired service life must be considered during design. In the case of new structures, it is desirable to avoid, prevent, or delay the initiation of corrosion through the use of low permeability concretes, proper precautions against cracking (see Chapter 3), and other corrosion-prevention techniques. For existing structures, the condition of the structure should be evaluated to determine if it is corroding or not. If the structure is corroding and the chloride content is high in the concrete, the contaminated concrete is removed; if rust is forming on the surface of the reinforcement, it is cleaned or removed and overlaid with a low permeability concrete overlay. If the existing structure is not corroding, some of the techniques used on new structures, such as applying sealers and membranes, may also be used on the existing structure.

An overview of the available options is provided in Figures 5.8 through 5.11.

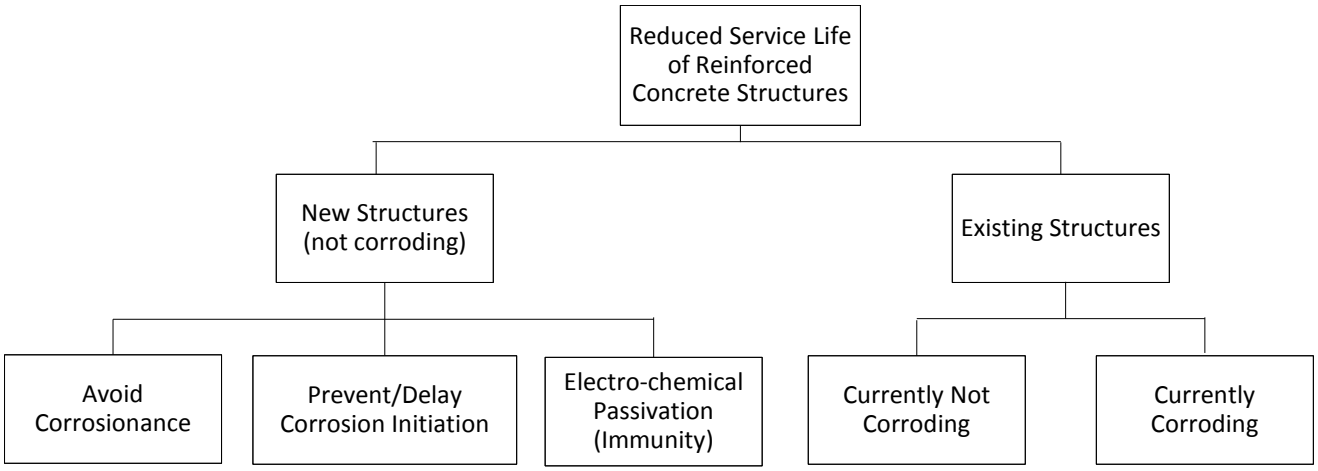


Figure 5.8. Reduced service life of reinforced concrete.

Many strategies have been used successfully to improve the corrosion resistance and durability of new structures. These include:

- The use of low permeability concrete,
- The use of increased concrete cover,
- The use of improved construction methods such as curing to minimize cracking,
- The use of corrosion-resistant reinforcement,
- The use of corrosion inhibitors to increase the corrosion initiation threshold,
- The use of membranes, coatings, and sealers, and
- The use of improved design details to keep elements dry and to prevent exposure to chlorides.

In the United States, epoxy-coated reinforcement (ECR) has been widely used as a corrosion protection system for concrete bridges. However, recent work and observations in the field have shown that the longevity desired (75 years and beyond) may not be achievable and therefore, other corrosion resistant reinforcements are being considered (see Chapter 3 on materials).

Each of these methods can be effective if it significantly extends the time for corrosion to initiate. In many cases it is preferable to employ more than one technique as this will generally reduce the overall risk of corrosion.

Additional information on these topics is provided in Chapter 3 on materials.

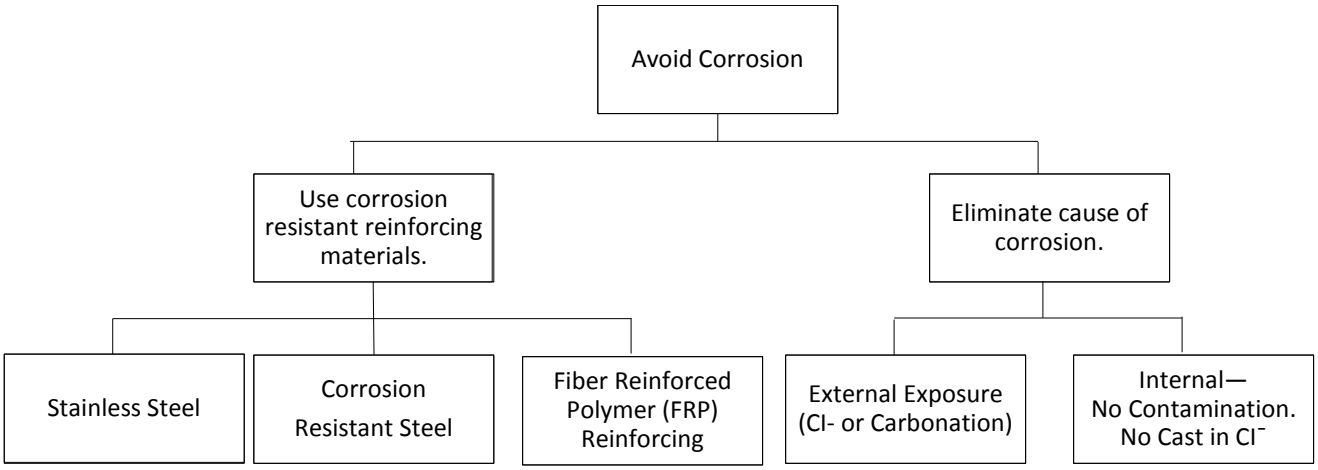


Figure 5.9. Options for avoiding corrosion.

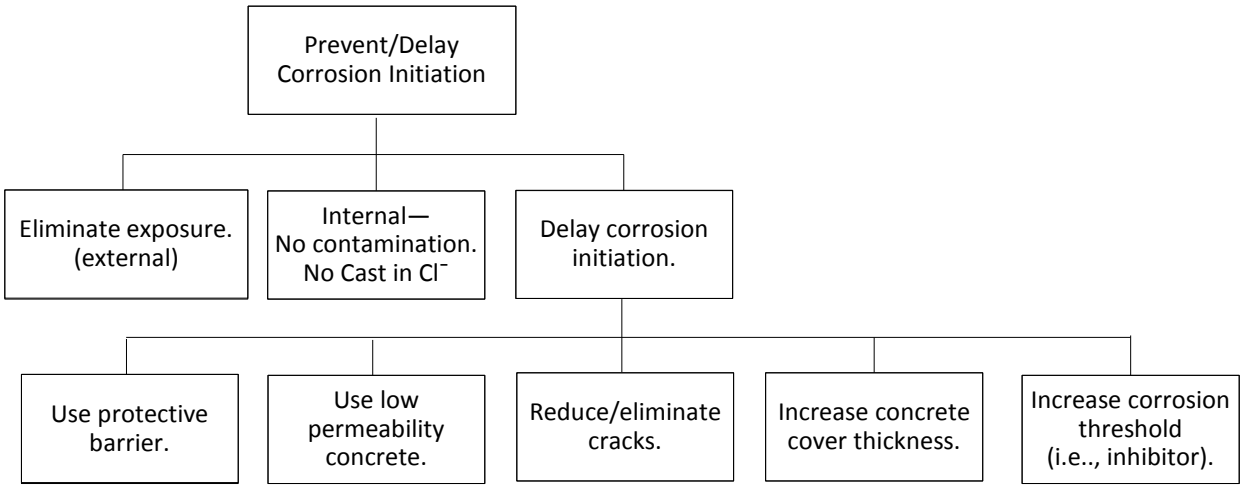


Figure 5.10. Options for preventing or delaying corrosion initiation.

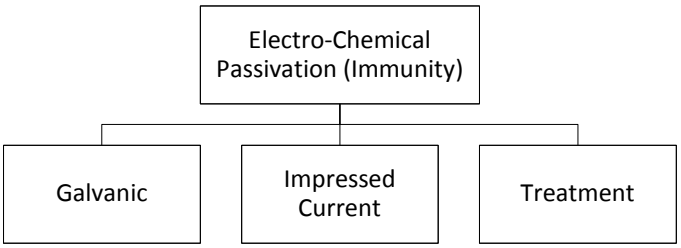


Figure 5.11. Electro-chemical passivation.

5.4.1 Existing Structures

Options for protecting structures from corrosion and extending their service life are much more limited when dealing with existing structures, as many of the physical parameters are already defined and therefore cannot be changed or easily altered. In particular, the concrete and reinforcing steel of existing structures are already in place

and the characteristics of these materials, including type, quality, cover thickness, permeability, resistance to corrosion initiation, presence of cracks, etc., are already fixed. Due to these limitations, many of the options that are viable for new (non-corroding) construction, as shown in Figure 5.12, are not possible or may not be economically practical for use with existing (corroding) structures. Figure 5.13 shows the options for corroding structures.

Despite the reduced number of options available for existing structures, it is still beneficial to be as proactive as possible. Preventing or delaying corrosion is generally preferable to managing corrosion after it has initiated. Additional options (and often more economical options) exist to prevent or delay corrosion activity if the structure is not chloride contaminated or has not already started to corrode.

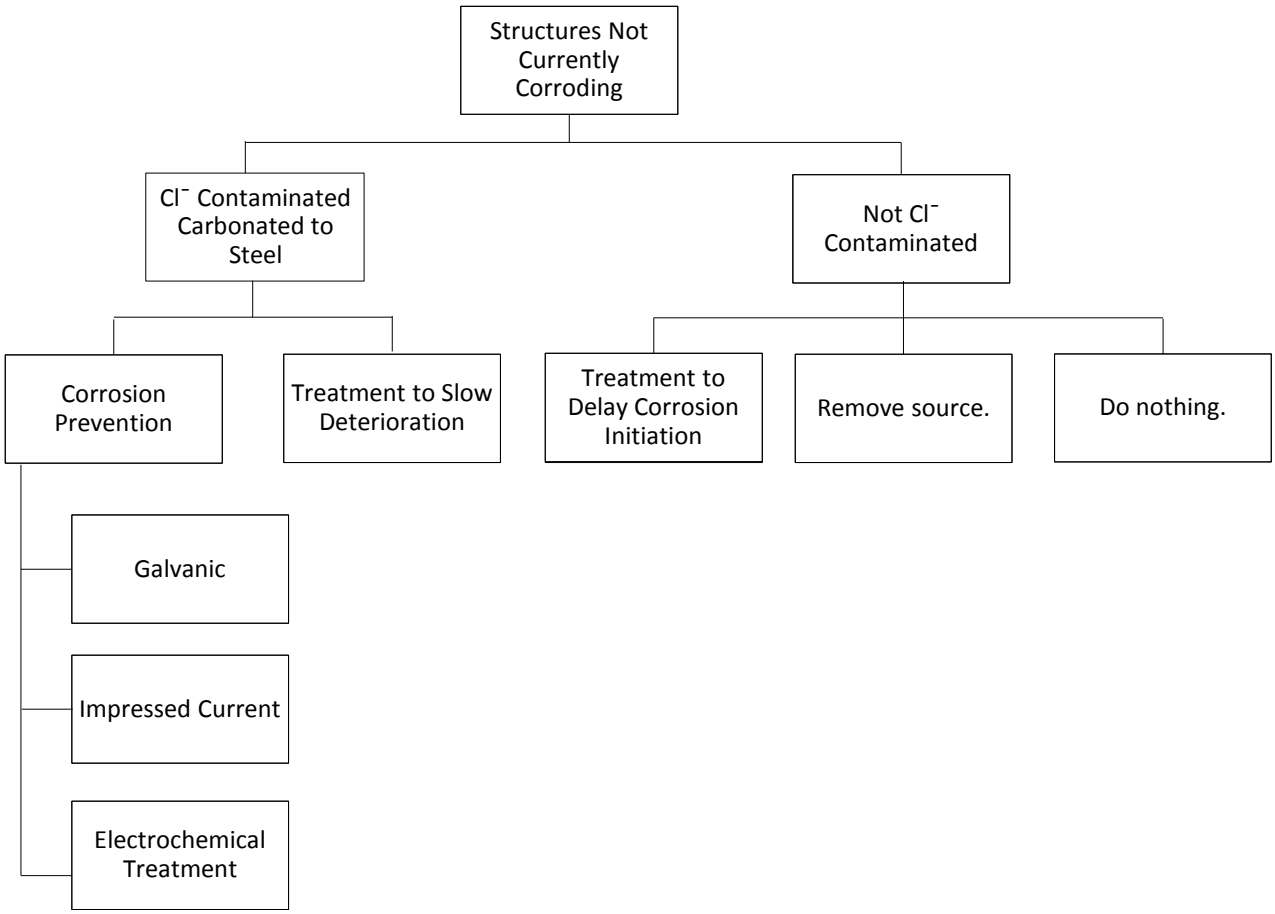


Figure 5.12. Non-corroding structures.

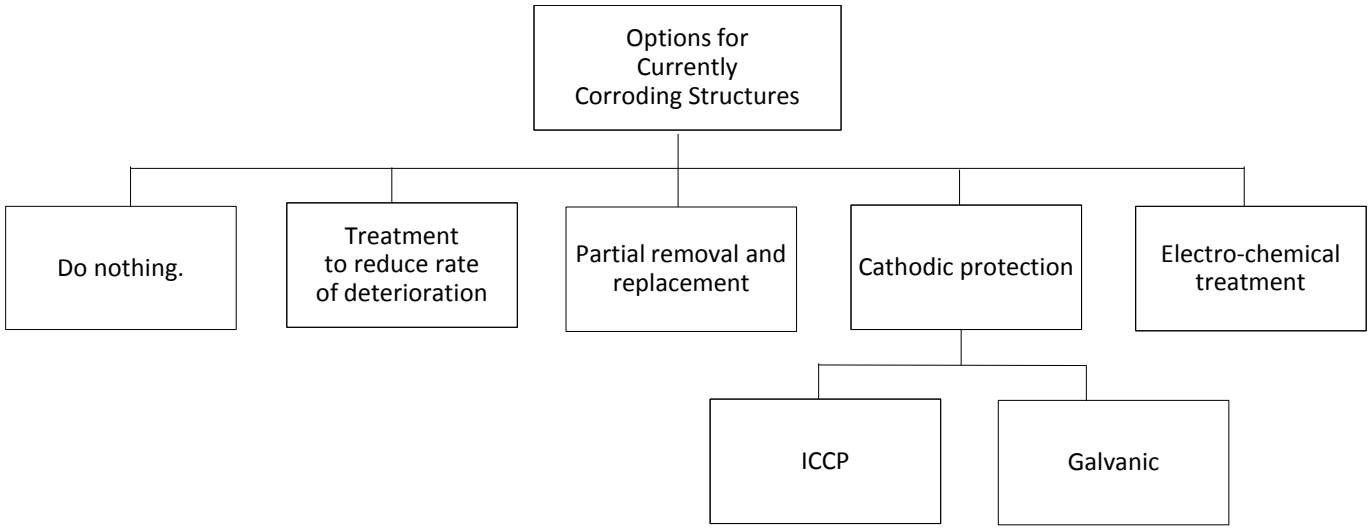


Figure 5.13. Options for corroding structures.

5.4.2 Levels of Corrosion Protection for Existing Structures

5.4.2.1 Selecting an Active Corrosion Protection Strategy for Reinforced Concrete Structures

The selection of the appropriate level of corrosion protection is based on many factors such as the level of chloride contamination and carbonation, amount of concrete damage, location of corrosion activity (localized or widespread), the cost and design life of the corrosion protection system, and the expected service life of the structure.

The levels of corrosion protection described in this section are summarized in Table 5.1.

Table 5.1. Summary of the Levels of Corrosion Protection for Electrochemical Corrosion Mitigation Systems.

Level of Protection	Description
Cathodic Protection	Stopping active corrosion by applying ongoing electrical current
Corrosion Prevention	Preventing new corrosion activity from initiating
Corrosion Control	Significantly reducing active corrosion
Corrosion Passivation/ Electrochemical Treatment	Stopping active corrosion by changing the chemistry of the concrete around the steel

5.4.2.2 Cathodic Protection

Cathodic protection provides proven corrosion protection and is intended to effectively stop ongoing corrosion activity. It should be selected when the highest level of protection is necessary and the cost is economically justified. Cathodic protection systems are grouped into two general categories: impressed current (ICCP) and galvanic (GCP).

Impressed current systems may use titanium or zinc-based anodes and utilize an outside power source. For long-term performance, these systems should be monitored and maintained. Discrete anodes are ideal to protect heavily reinforced concrete, thick structural sections such as columns or beams, or steel framed masonry buildings while titanium ribbon or mesh anodes are placed into slots cut into the concrete surface or cast into a concrete overlay.

Galvanic systems may be designed to provide corrosion control or cathodic protection. These galvanic systems are self-powered and typically require less monitoring and maintenance than ICCP systems. Galvanic jackets are used to protect marine pilings and other structures. Galvanic anodes may be arc-sprayed zinc or otherwise applied to the concrete surface, or they may be cast into a concrete overlay, jacket, or encasement to provide galvanic cathodic protection over a desired area. If galvanic anode systems are cast into concrete or are not directly exposed to a marine environment, they should be activated to ensure that sufficient current is supplied to the reinforcing steel to provide long lasting corrosion protection.

Cathodic protection systems are generally designed to meet National Association of Corrosion Engineers (NACE) cathodic protection standards and typically use 100mV depolarization as the acceptance criteria (NACE 2000). The current density required to achieve cathodic protection is higher than the current required for corrosion prevention or corrosion control applications. Typical cathodic protection systems operate in the range of 2 to 20 mA/m^2 of steel surface area. At these current densities and polarization levels, cathodic protection has demonstrated a very high level of corrosion protection.

Galvanic cathodic protection systems were evaluated in the research phase of the *SHRP 2 R19A Project,* the report for which is forthcoming. In this study, different galvanic anodes were evaluated for the purpose of minimizing corrosion in reinforced concrete members; a commonly used anode (OA), an anode with 4-times larger zinc surface area than ordinary anodes (OA4), and two high voltage anodes with varying degrees of output voltage (HVAH, higher level; and HVAL, lower level). Concrete test slabs were cast in two layers; the concrete in the upper layer was contaminated with salt to accelerate the corrosion activity and the lower layer was not modified. Salt was not added to the concrete in the lower layer. The test results indicated that there was no corrosion in any of the specimens in the given time period. The testing further indicated that specimens with HVAH provided increased corrosion protection by having higher current and generating more negative potential values than the HVAL. Anodes having 4-times surface area (OA4) provided additional current and corrosion protection than OA anodes.

HVAL anodes exhibited current and potential values similar to the OA4 anodes. Both high voltage anodes and OA4 anodes provided higher current and generated more negative potential values, indicating better corrosion protection than OA anodes. Due to time constraints, the tests were terminated without observing corrosion in the specimens. Further research with an extended time frame is recommended.

5.4.2.3 Corrosion Prevention

Corrosion prevention is used to keep corrosion activity from initiating in contaminated concrete. In concrete repair projects, the removal and replacement of damaged concrete, if completed in accordance with industry guidelines (ICRI 2008), will address the areas with the highest levels of corrosion. However, new corrosion sites are likely to form in the surrounding contaminated concrete which was passive before the repairs. To mitigate new corrosion activity from occurring around concrete repairs or at other interfaces between new and old concrete, such as bridge widening, joint repairs, and slab replacements, a localized corrosion prevention strategy may be employed utilizing embedded galvanic anodes to extend the life of the concrete repairs. Size and spacing of embedded galvanic anodes should be adjusted to suit site conditions, including quantity and existing condition of reinforcing steel, level of chloride contamination, and environmental conditions.

There has been a significant amount of research in the area of corrosion prevention, some of which has indicated that applied current densities as low as 0.5 to 2.0 mA/m^2 of steel surface area have been shown to be effective at preventing the initiation of corrosion for concrete with chloride concentrations up to at least ten times the chloride threshold (Pedeferri 1996). Other research has shown beneficial effects of applied currents between 0.25 and 1.0 mA/m^2 (Bertolini et al. 1996).

5.4.2.4 Corrosion Control

Corrosion control systems are utilized in instances in which corrosion has initiated but has not yet progressed to the point of causing concrete damage. The use of corrosion control systems will either stop ongoing corrosion activity or provide a significant reduction in the corrosion rate and increased service life for the rehabilitated structure. In many cases, this level of protection can be provided with low incremental cost, as the protection can be targeted at specific areas of contamination or corrosion activity. Galvanic anodes embedded in drilled holes or installed on a grid pattern in a concrete repair or overlay can be used to provide targeted galvanic corrosion control to

columns, beams, decks, posttensioned anchorages, and other areas where ongoing corrosion activity threatens the service life or serviceability of the structure.

The applied current necessary to control active corrosion is significantly higher than the current required for corrosion prevention. Research has indicated that the typical current density to control active corrosion is in the range of 1 to 7 mA/m^2 (Davison et al. 2003).

In the study by Davison et al., corrosion control of specimens with very high initial corrosion rates was achieved. Some polarization of the reinforcing steel will typically occur at these current densities although the level of polarization may be significantly less than the NACE 100mV depolarization criteria for cathodic protection.

There are many situations where the corrosion activity is moderate or is localized in nature such that corrosion control is an appropriate approach since large scale cathodic protection may not be economically justifiable. Examples of this include localized areas beneath leaking expansion joints, or decks with isolated areas of high corrosion potentials. In these cases, targeting the protection to address the specific contaminated zone, or "hot spot," rather than the entire structure may make sense from a cost/benefit point of view.

5.4.2.5 Corrosion Passivation/Electrochemical Treatments

Corrosion passivation is provided by electrochemical treatments that are aimed at directly addressing the cause of the corrosion activity. Electrochemical chloride extraction (ECE) is used to address corrosion caused by chlorides in chloride-contaminated structures. Electrochemical re-alkalization is used to address corrosion resulting from carbonation of the concrete. These systems are installed on the structure, operated for a short duration, then dismantled and removed leaving the structure in a passive condition. Electrochemical treatments provide many of the long-term corrosion mitigation benefits of cathodic protection systems, but without the need for long-term system maintenance and monitoring. Additional information about the implementation and evaluation of the two systems follows.

Electrochemical chloride extraction is an electrochemical treatment in which an electric field is applied between the reinforcement in the concrete and an externally mounted mesh (Figure 5.14). The mesh is embedded in a conductive media, generally a sprayed-on mixture of lime, water, and cellulose fiber.

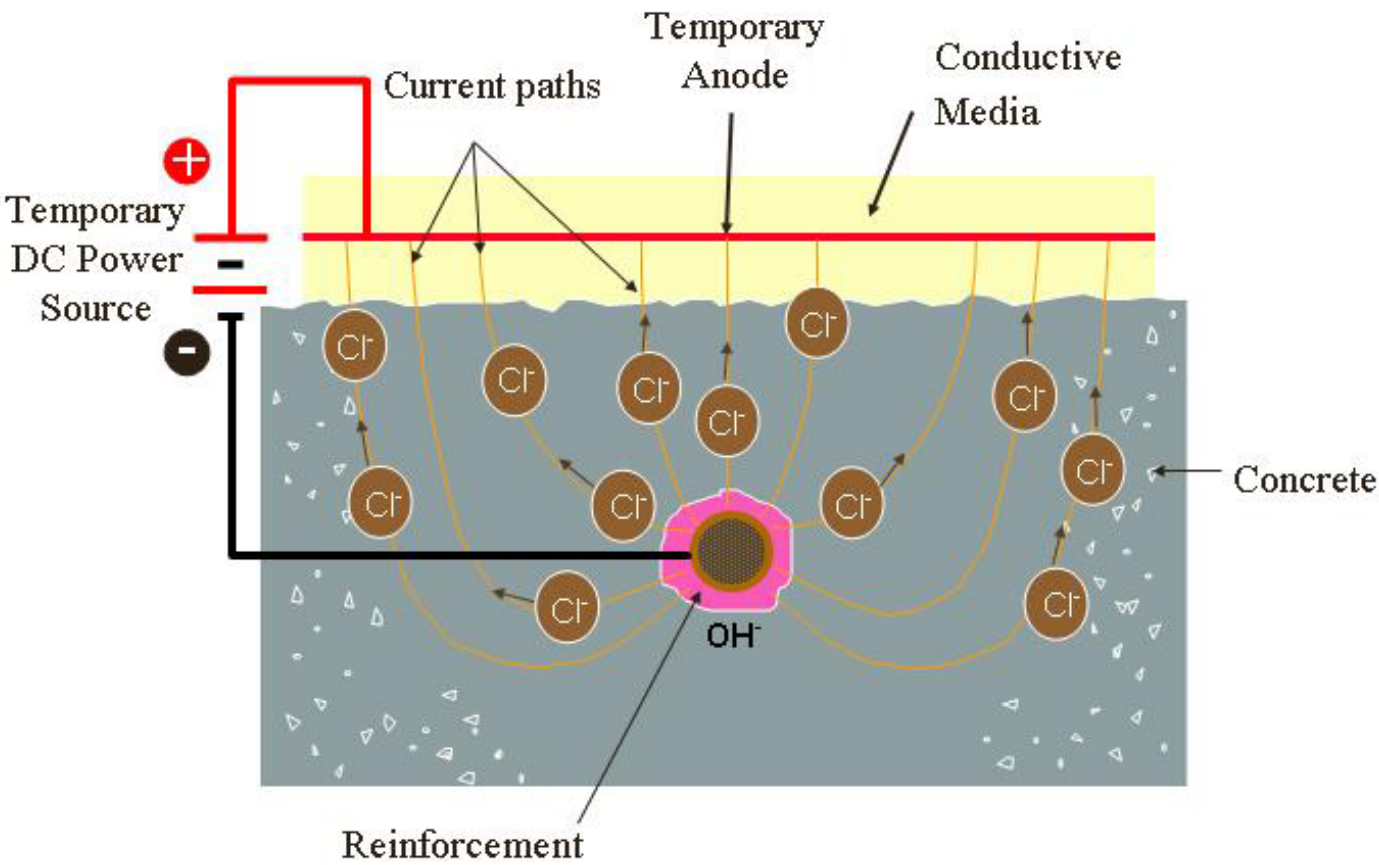


Figure 5.14. Schematic diagram showing ECE treatment process.

During treatment, the concrete is kept saturated allowing chlorides to go into solution within the pores of the concrete. The negatively charged chloride ions (Cl^-) are repelled from the negatively charged rebar and attracted toward the positively charged external electrode mesh as a result of the applied electric field. This process lowers the amount of chloride in the concrete and particularly adjacent to the steel. An ECE treatment generally takes 4 to 8 weeks to complete.

In instances of structures with carbonation-induced corrosion, a different electrochemical treatment process, re-alkalization, can be utilized to increase the pH of the concrete cover. The installation for re-alkalization is essentially the same as for ECE except the conductive media is saturated with an alkaline potassium carbonate solution. The potassium (K+) ions in the alkaline solution are transported into the concrete by the application of the electric field. A re-alkalization treatment generally takes four to seven days to complete and will not re-carbonate.

Electrochemical treatment was evaluated in the research phase of the *SHRP 2 R19A Project* with detailed results to be presented in the forthcoming final report. In the SHRP 2 study, corrosion resistant reinforcement and electrochemically treated black bars were evaluated to determine if the chloride threshold levels increased to a level

to resist corrosion. Both low and high levels of electrochemical treatment were used. The test matrix included black bars, electrochemically-treated black bars, stainless steel bars, and titanium bars, each subjected to salt solution. Due to time constraints, testing was terminated after a total of 26 cycles consisting of four days of wet cycle and 10 days of dry cycle; a total duration of one year. At termination, only one specimen from a set of three with black bars and electrochemically-treated black bars showed an increase in current or potential values indicative of uncertain corrosion activity. The remaining specimens indicated no corrosion activity. Thus, initial observations indicate that electrochemically-treated black bars may not provide the protection expected of stainless steel or titanium; however, whether or not they provide benefits over the black bars without treatment cannot be concluded from this study due to time constraints. Therefore, further research with an extended time frame is recommended.

5.5 CASE STUDIES ADDRESSING CORROSION IN EXISTING STRUCTURES

5.5.1 Project Overview: Impressed Current Cathodic Protection, Corrosion Control, and Electrochemical Chloride Extraction

During the summer of 1989 and continuing until 1994, the Ontario Ministry of Transportation (MTO) completed a number of restoration and protection projects on the reinforced concrete piers of the Burlington Skyway, a major viaduct located between Toronto and Niagara Falls, Ontario. This work was monitored by MTO's Materials and Research Branch. Some of this research was conducted under the original SHRP project and is documented in SHRP-C-620, "Evaluation of Norcure Process for Electrochemical Chloride Removal from Steel-Reinforced Concrete Bridge Components" (Bennet and Schue 1993).

An impressed current cathodic protection (ICCP) system was installed on over 200,000ft^2 of reinforced concrete substructure. The system was designed to operate as an ICCP system with an applied current density of 10 mA/m2 (1mA/ft2). After an initial period of operation at the design current density, and due to operational issues, the system was run at an average current density of 1mA/m2 (0.1mA/ft2). As such, instead of being operated at cathodic protection current density (2 – 20mA/m2), the system was effectively operated at a corrosion control current density (1 – 7mA/m2). Due to the low applied current density, much of the area did not meet the 100mV NACE criteria for cathodic protection. Despite operating at approximately 10% of the intended cathodic protection current density, the system operation did fall within typical corrosion control current densities and experienced a significant reduction in

concrete deterioration and damage. Over the study period, concrete delamination within the protected area was reduced by 96% compared to the rate of deterioration of unprotected concrete piers.

Also in 1989, an electrochemical chloride extraction trial was completed by MTO on a section of the same structure. The treated portion was comprised of rectangular piers and bents. The piers were contaminated with chlorides due to long-term leakage of the deck joints above.

Electrochemical chloride extraction was used to reduce the chloride content of the concrete to below the threshold level of corrosion at the rebar. This process eliminated high corrosion potential readings and corrosion potentials in the treated area were shifted into the passive range as shown in Table 5.2; in this table ECE treated section exhibited a high percentage of area with a low risk of corrosion. Corrosion current as measured by linear polarization (3-LP) was greatly reduced as shown in Table 5.3; in this table ECE treated sections exhibited a high percentage of passive area. Thus, corrosion potentials and corrosion currents were reduced to the passive, non-corroding range as a result of the electrochemical chloride extraction treatment for a 20-year duration as shown in Tables 5.2 and 5.3. These readings have remained stable and show no significant changes over the period.

Additional piers were treated during the fall of 1997 as part of the next phase of work on this project. The long-term results from these more recent tests are expected to be similar to the electrochemical chloride extraction trial completed in 1989.

Table 5.2. Corrosion Potential Measurements.

	Untreated (Control)			**ECE Treated**		
	% Area with Low Risk of Corrosion <200mV	**% Area with Moderate Risk of Corrosion 200 – 350mV**	**% Area with High Risk of Corrosion >350mV**	**% Area with Low Risk of Corrosion <200mV**	**% Area with Moderate Risk of Corrosion 200 - 350mV**	**% Area with High Risk of Corrosion >350mV**
Pre-Treatment	0	85	15	0	96	4
1 Yr. After	41	59	0	98	2	0
2 Yr. After	41	59	0	100	0	0
3 Yr. After	26	74	0	96	4	0
4 Yr. After	26	70	4	98	2	0
6 Yr. After	26	59	15	96	4	0
8 Yr. After	11	78	11	96	4	0
10 Yr. After	15	78	7	96	4	0
15 Yr. After	20	70	10	98	2	0
20 Yr. After	15	70	15	96	4	0

Corrosion potentials measured in -mV vs Cu-$CuSO_4$
Note: Values represent percentage of readings within range

Table 5.3. Corrosion Current Measurements.

Corrosion Current (mA/cm^2)	Untreated (Control)			ECE Treated		
	% Area with Passive			% Area with Low Corrosion		
	% Area with Passive 0<0.22	% Area with Low Corrosion 0.22 - 1.08	% Area with High Corrosion >1.08	% Area with Passive <0.22	% Area with Low Corrosion 0.22 - 1.08	% Area with High Corrosion >1.08
Pre-Treatment	0	46	54	0	87	13
1 Yr. After	0	52	48	87	13	0
2 Yr. After	0	84	16	78	20	2
3 Yr. After	4	88	8	89	11	0
4 Yr. After	8	86	6	90	10	0
6 Yr. After	0	40	60	65	35	0
8 Yr. After	0	29	71	63	37	0
15 Yr. After	0	62	38	65	34	1
20 Yr. After	15	70	15	96	4	0

Note: Values represent percentage of readings within range

5.5.2 Project Overview: Galvanic Cathodic Protection: Galvanic Encasement of Severely Corroded Elements

Galvanic anodes have also been developed for more global corrosion control. One such configuration is the system installed to repair and protect severely damaged and corroding bridge abutments in the Midwest (Figure 5.15). The abutments had been contaminated with chlorides causing corrosion of the reinforcing steel and significant concrete damage.

Figure 5.15. A distributed anode system used for corrosion control bridge abutment in the Midwest.

As part of the rehabilitation, which also included enlargement and strengthening of the abutment, the cracked and spalled concrete was removed. Figure 5.16 shows the cross-sectional detail of the rehabilitation system. Elongated anodes were connected to the existing reinforcing steel and encased in a new layer of concrete to reface the abutment wall. The purpose of the anodes was to protect the existing steel from chloride-induced corrosion. This allowed un-cracked chloride-contaminated concrete to remain in place, and thereby reduce concrete breakout and the need for structural shoring. The cross-sectional configuration of the repaired abutment wall and adjoining structural elements is shown in Figure 5.16.

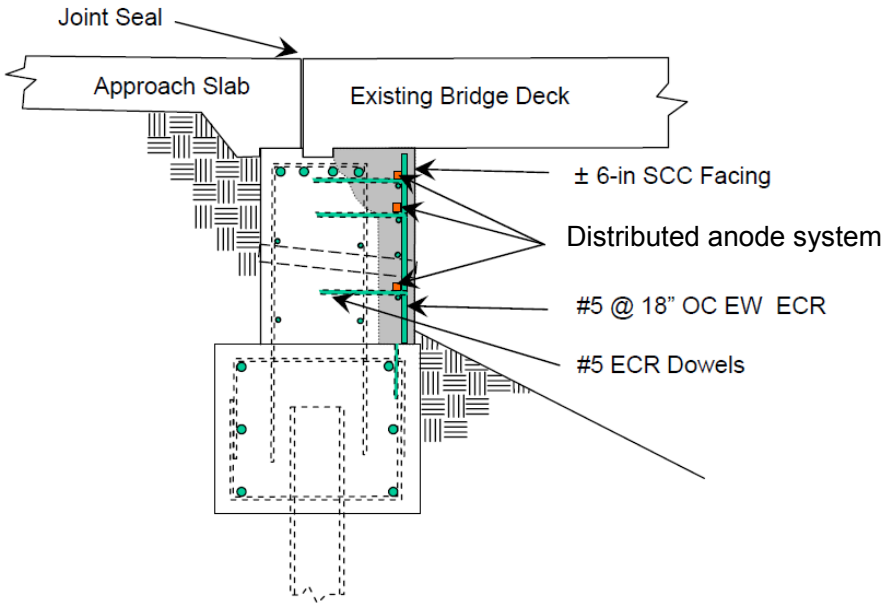


Figure 5.16. Cross-sectional detail of the abutment rehabilitation system.

The current output, shown in Figure 5.17, appears to be strongly related to temperature. Its magnitude varied considerably with temperature on an annual basis, with the mean current density gradually reducing year by year. After supplying an initial current of over 35 mA/m^2 of steel area in the first few days, it averaged over 8 mA/m^2 during the first year, lowering gradually to around 5 mA/m^2 in the fourth year. These levels of current density are within the design limits of 2 to 20 mA/m^2 of steel area for cathodic protection as specified in BS EN 12696:2012 (BS 2012). Current densities in impressed current cathodic protection systems are also normally reduced with age as the steel becomes easier to polarize. Depolarization levels were measured to be well in excess of 100mV as specified in the same standard, suggesting that the galvanic system was deemed to satisfy the criteria for cathodic protection of steel reinforcement. Depolarization and corrosion protection status are given in Table 5.4.

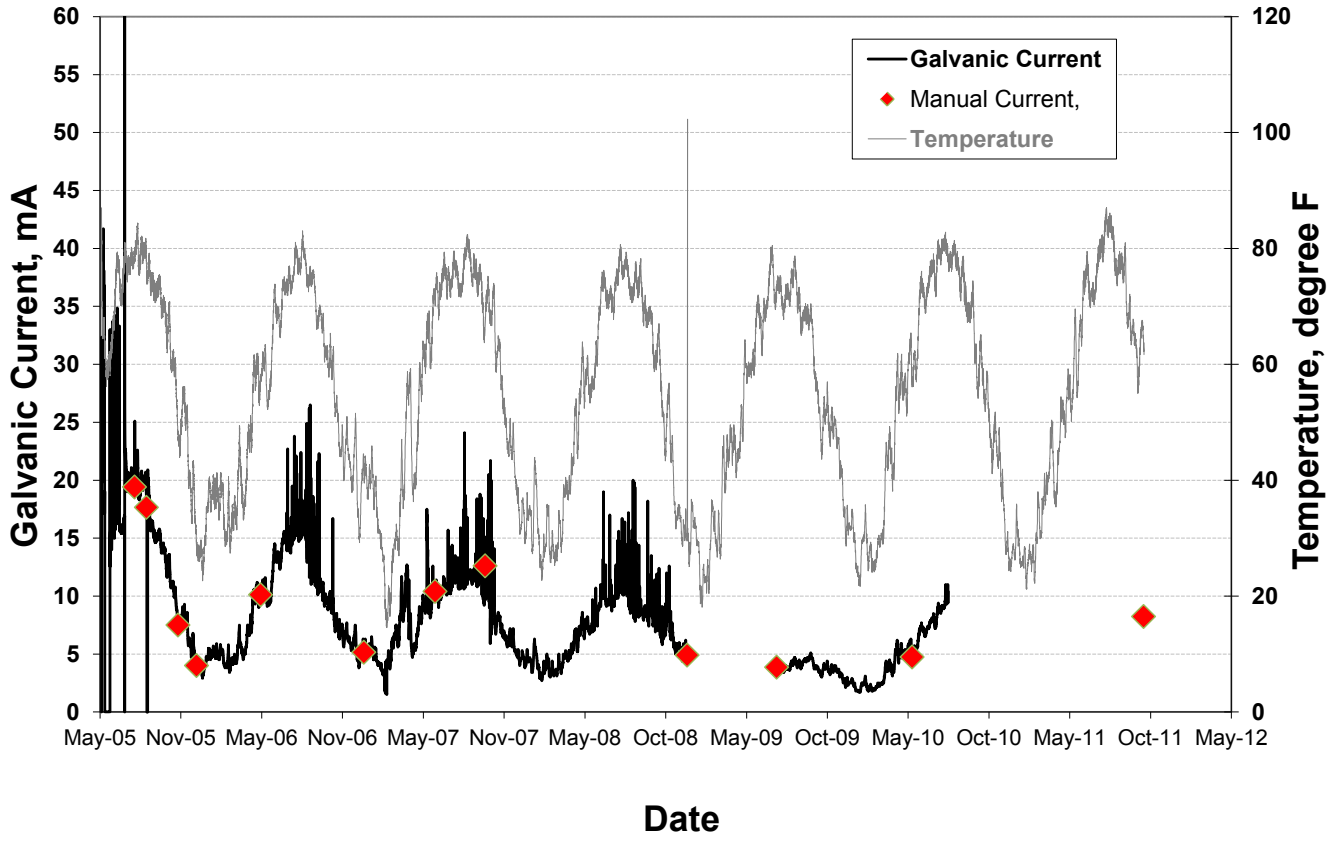


Figure 5.17. Current output of anode system and its relationship to temperature.

Table 5.4. Depolarization and Corrosion Protection Status.

Date	Temperature	Current Density	Depolarization	Status
	(F)	(mA/ft^2)	(mV)	
6-May-05	88	>3.5	-[1]	Corrosion-Protected
16-Aug-05	87	1.23	344	Corrosion-Protected
26-Oct-05	54	0.52	368	Corrosion-Protected
7-Dec-05	51	0.28	310	Corrosion-Protected
1-May-06	57	0.70	313	Corrosion-Protected
20-Dec-06	40	0.36	459	Corrosion-Protected
30-May-07	79	0.72	449	Corrosion-Protected
20-Sep-07	75	0.88	482	Corrosion-Protected
19-Dec-08	40	0.34	450	Corrosion-Protected
9-Jul-09	74	0.27	471	Corrosion-Protected
11-May-10	54	0.33	485	Corrosion-Protected
16-Oct-11	72	0.57	488	Corrosion-Protected

[1]Not Applicable

5.5.3 Project Overview: Corrosion Prevention Using Galvanic Anodes

The oldest site trial of discrete galvanic anodes is over 10 years old (Figure 5.18). To verify the performance of the anodes, 12 anodes were installed in an otherwise conventional patch repair on the soffit of a bridge beam (Figure 5.19). The performance of these anodes was monitored with time.

Figure 5.18. North side of bridge.

Figure 5.19. Installation of anodes within the repaired area of a beam also showing control box and wiring.

The anodes were inserted around the perimeter of the repair area between 600mm and 700mm centers (Figure 5.19). The anodes were installed to enable monitoring by connecting a single wire from each anode to a control box such that the anodes could be monitored via the box. Monitoring of the anodes consisted of measuring current output for each installed anode, and, on occasion, performing a depolarization test over a 4-hour or 24-hour period after disconnection of the anodes. Monitoring started in April 1999.

5.5.4 Results and Discussion

The 10-year results of the current output of each anode are presented in Figure 5.20. They indicate a variable current depending on the ambient temperature and moisture content in the concrete. For example, the same anode could generate up to 400-600μA of current during hot periods and less than 100μA during cold spells. Corrosion of the steel is expected to have similarly-varying corrosion rates so that the current output of the anodes is thought to be self-regulating, producing higher levels when the steel is corroding most.

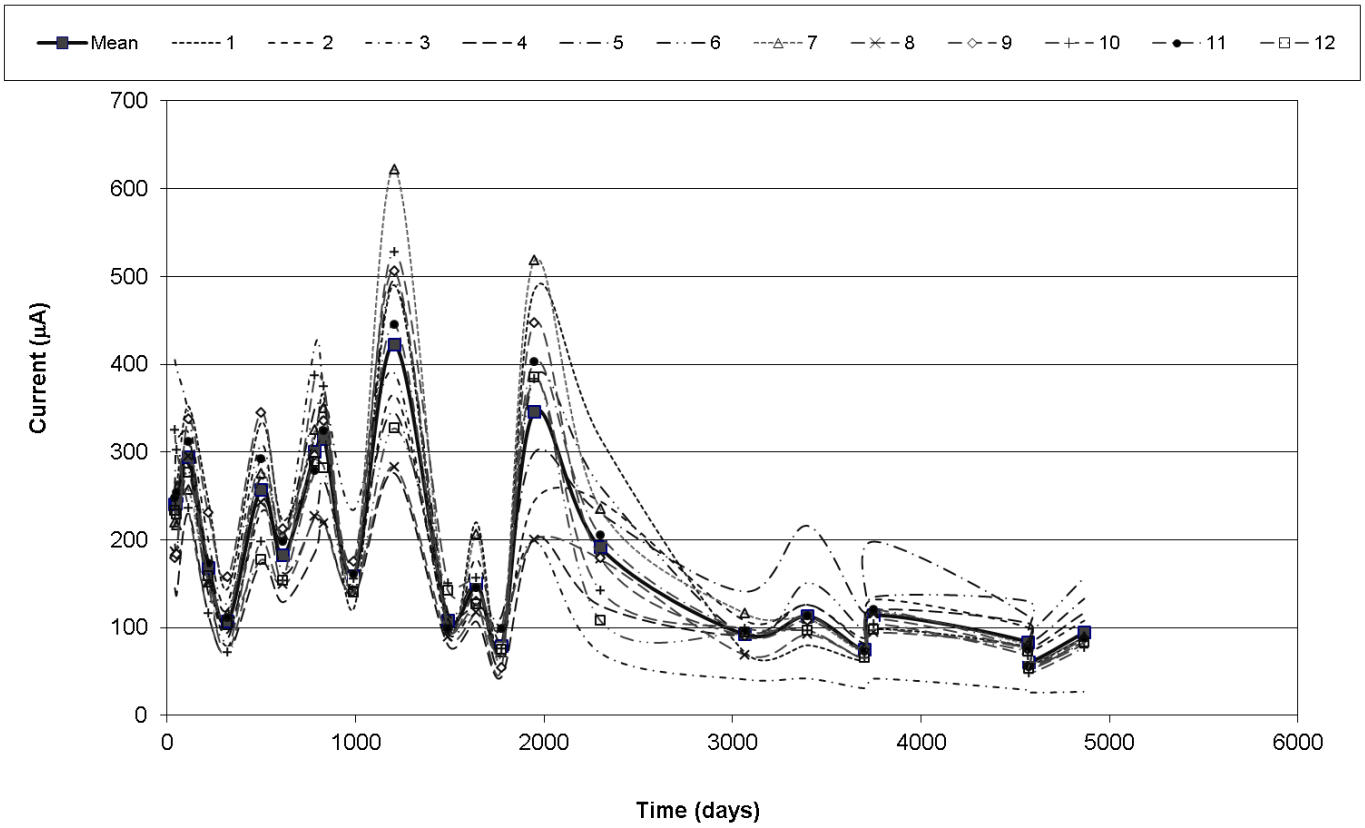


Figure 5.20. Protective current from galvanic anodes over a 10-year period.

Systems such as this are designed to prevent the onset of corrosion of the reinforcement. The current density required for corrosion prevention (referred to as cathodic prevention in Europe) is 0.2-2mA/m^2, as reported by Bertolini et al. (1993) and Pedeferri (1996), and adapted in the European Standard BS EN 12696:2012 (BS 2012). Based on the steel surface area within the repaired and affected adjacent area, the mean current density ranged between 0.6 mA/m^2 and 3.0 mA/m^2 through the duration of this trial, with a mean current density of around 1.4 mA/m^2 over the 10-year period. This current density is within the suggested range of 0.2 to 2.0mA/m2 for corrosion prevention (cathodic prevention).

Monitoring the depolarized potential of the steel in the vicinity of the repair with time may be another way of determining the effectiveness of the system. Figure 5.21, which shows the mean depolarized potential with time both within and outside of the repaired area, indicates that the mean potential is moving to a more noble level with time. This indicates increasing passivation of the steel over time.

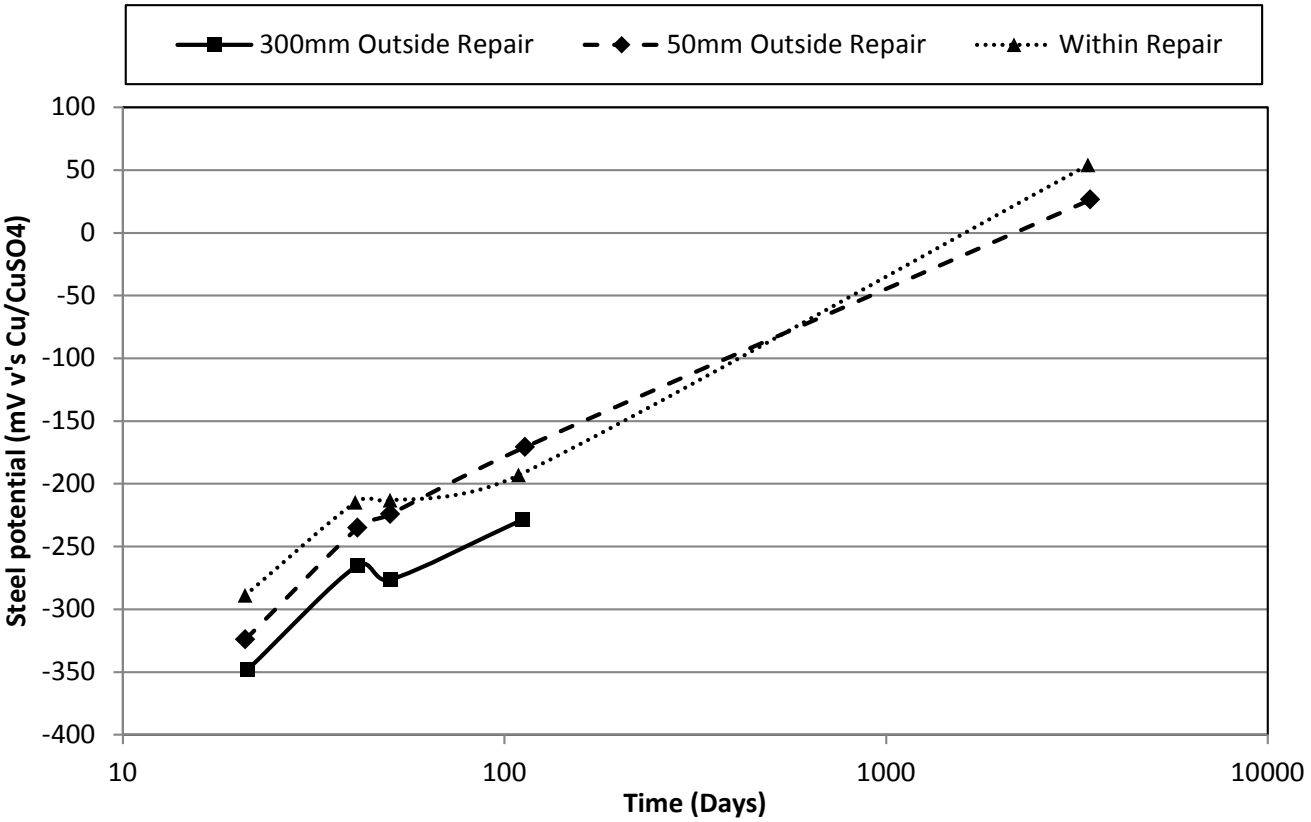


Figure 5.21. Mean depolarized steel potentials with time (4 hours or 24 hours after disconnection of the anodes).

CHAPTER 6

CORROSION PREVENTION OF STEEL BRIDGES

6.1 INTRODUCTION

This chapter is a "Best Practices" guide and discussion for preventing corrosion of exposed structural steel for bridges, and includes factors to be considered for design through installation, inspection, and maintenance. Various types of coatings including painting, galvanizing, and metalizing are discussed along with other methods of corrosion prevention that include use of steels with higher resistance to corrosion, such as weathering steel.

Figure 6.1 shows the structural steel elements susceptible to corrosion. The focus of this chapter is on superstructure elements; however, much of the discussion presented is also applicable to deck and substructure elements.

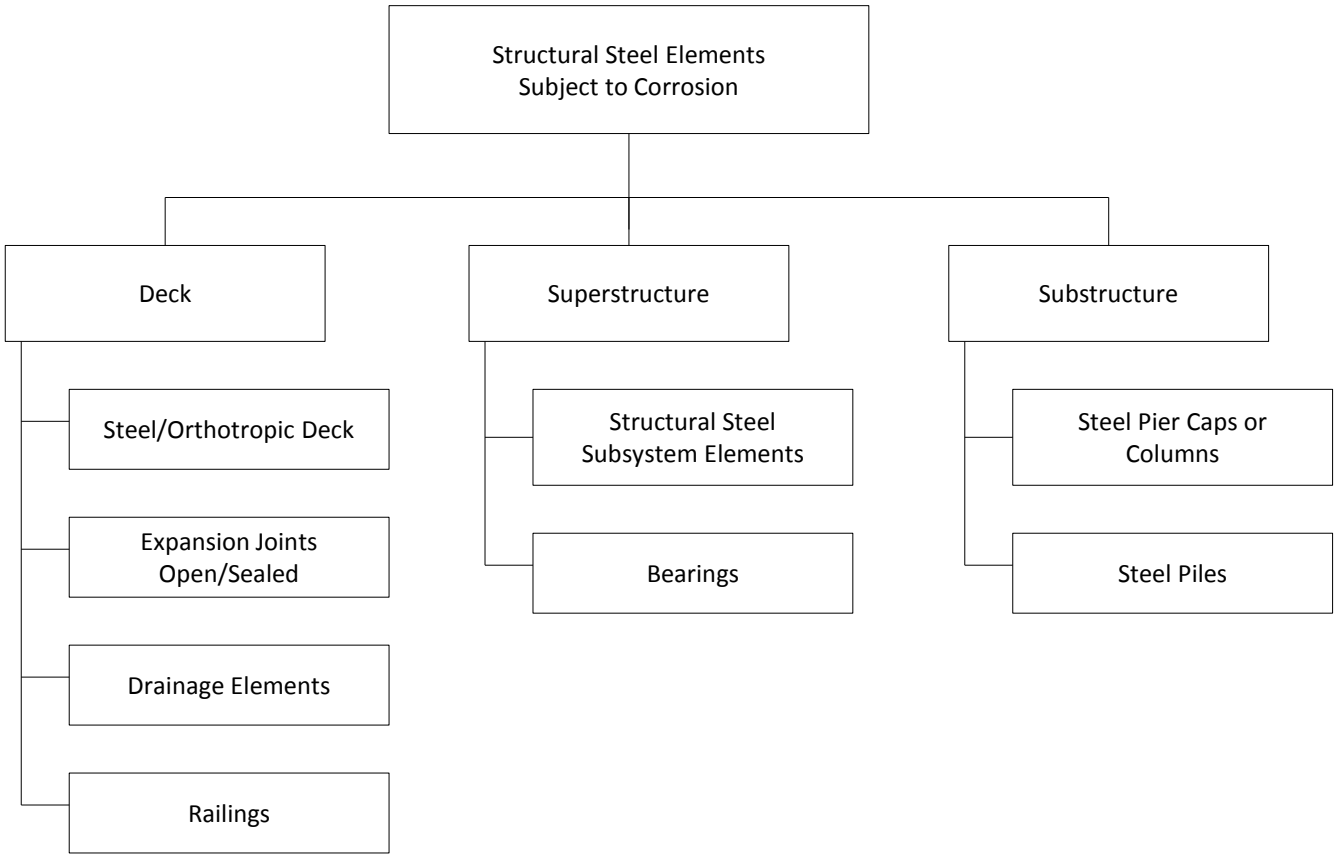


Figure 6.1. Structural steel elements subject to corrosion.

Within the superstructure component, structural steel subsystem elements include all configurations of steel shapes and plates which, alone or in combination, comprise members utilized as supporting steel on various types of

structures including trusses, beams, haunch parallel flange welded plate girders, multiple web and single bottom flange "tub" girders, and square or rectangular cross-section box girders. Also included are all angles, channels, fasteners, sole plates, diaphragms, shims, and bearings.

There are three primary methods for preventing corrosion of structural steel:

1. Use of coating systems,
2. Use of corrosion-resistant steel (weathering steel) or non-corrosive steel, and
3. Avoiding corrosive environments or corrosion-prone details.

Each of these methods is discussed. The use of partially painted weathering steel is also addressed.

6.2 DESCRIPTION OF METHODS FOR CORROSION PREVENTION

This section provides a general discussion of the corrosion process and a description of three main methods used to prevent corrosion of steel bridges.

6.2.1 Corrosion of Steel: General Discussion

In its simplest form, the corrosion of steel results from exposure to oxygen and moisture. Corrosion is accelerated in the presence of salt from roadway deicing, salt water, or perhaps salt deposited from other sources. The fact that steel corrodes is one of the few fundamental limitations of steel as a material of construction.

As noted, while steel corrodes readily in the presence of oxygen and moisture, the rate of corrosion is accelerated in the presence of chloride ions or other corrosive chemicals. Chloride ions result mainly from the use of deicing agents comprised of materials with readily-soluble chloride ions. These ions create an atmosphere in which unprotected steel corrodes very quickly. In order to ameliorate corrosion issues, engineers have used protective coatings as one means of protecting steel from the impact of the environment.

6.2.2 Description of Coating: Painting

It is generally recognized that in a cost-effective, multi-coat paint system, the primary purpose of the coating layer closest to the steel surface is to provide corrosion protection for the steel surface. Any special aesthetic considerations are accommodated in the subsequent coating layers, principally the topcoat. Aesthetics, while important for some applications, is not the focus of this chapter.

The performance of protective coatings is dependent on the environment in which they are exposed. In some dry-climate areas of the country where corrosion is not an issue, aesthetic considerations can play a more compelling role. The survey of state Departments of Transportation (DOT) conducted as a part of the *SHRP 2 R19A Project* (final report forthcoming) confirms that relatively dry states such as Arizona can expect 50+ years of service life for the same system expected to last 20 to 30 years in states with a moister climate.

It follows, therefore, that a sure way to protect steel from corrosion is to keep it from getting wet and a key means of accomplishing this is by using coating to provide a barrier to the elements, protect the steel from moisture, and keep it dry. The ability of the topcoat, or outmost layer, to shed water is the key to using coating as corrosion protection.

When water penetrates the outer coating layer(s) and comes in contact with the steel substrate, the primer acts to inhibit corrosion as the steel surface is subjected to repeated wet/dry cycles.

From the earliest years in the steel bridge era in the United States, beginning around 1874 with the Eads Bridge in St. Louis, Missouri, lead and chromium rust-inhibitive pigments were added to paint to supplement the barrier protection offered by a coating film. For almost 100 years, the use of lead/chromium pigmented, multi-layer coatings was the norm during new bridge construction, maintenance overcoating, and maintenance repainting.

Figure 6.2. Eads Bridge. (Courtesy KTA-Tator, Inc.)

After about 1965, bridge coating engineers began to turn away from coatings containing these toxic "heavy metal" pigments and instead started using coatings containing metallic zinc as the corrosion-inhibitive pigment. The DOT survey conducted as part of the *SHRP 2 R19A Project* (final report forthcoming) indicated that all states responding use a system consisting of a zinc-rich primer. As long as the zinc pigment in the coating is in close metal-to-metal contact with the steel substrate, the coating provides galvanic protection to the steel. Galvanic protection is provided when zinc and steel (iron) are connected (i.e., have a conductive pathway between them) with one another

in the presence of air (oxygen) and moisture. In this coupling of materials, zinc (the less noble metal) will oxidize (corrode) in preference to the iron (steel). The preferential oxidation of zinc provides protection for the steel as long as there is nearby zinc left to be consumed in the chemical reaction which takes place at the anode. When the zinc is consumed, the steel beneath will be subject to corrosion (oxidation). The method used to resist corrosion attack since about the mid-1960s has been to utilize a multi-coat, "belt and suspenders," approach. In a multi-coat system, the outer layer(s) resist the effects of weather and protect the zinc from being consumed in the atmosphere, while the zinc-rich primer inhibits corrosion from occurring at the steel beneath in locations where the coating is breached.

Even in instances in which steel is painted with a coating system utilizing a zinc-rich primer, when the protected steel surface is bathed in salt water and is subjected to many wet/dry cycles, discontinuities in the coating inevitably provide a pathway through the coating for moisture to reach the zinc-coated steel surface beneath. As a result, the zinc begins to react to protect the steel from corroding. Eventually, the metallic zinc in the zinc-rich primer is consumed, and corrosion in the form of red rust (iron oxide) results. The corrosion protection offered by zinc may take many years to be consumed before evidence of corrosion of the steel; therefore, the rate of corrosion is dictated by the local factors surrounding the steel (wet/dry cycles, chloride contamination, humidity, etc.).

The following general discussion of coatings serves as an introduction to discourse on the use of protective coatings in the prevention of corrosion of structural steel. Figure 6.3 identifies the various items discussed.

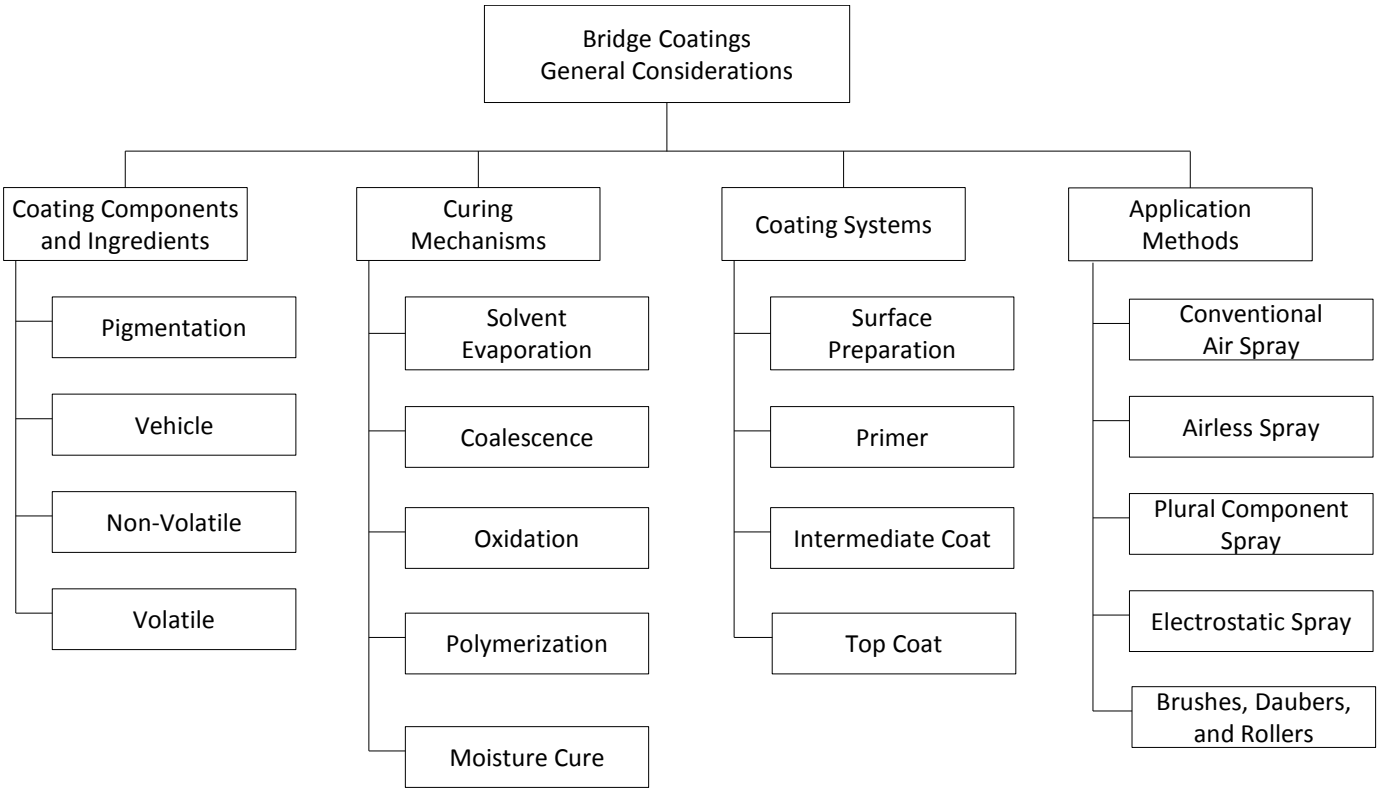


Figure 6.3. Paint system general considerations.

6.2.2.1 Composition of a Paint Coating

Figure 6.4 illustrates the basic ingredients of an industrial protective coating. The chart divides a coating into two major components: *pigmentation* and *vehicle*. The pigmentation typically consists of corrosion inhibitors, colorants, and extenders, although other raw materials may also be included. The vehicle typically consists of the resin or binder, solvents, and any additives that may be included in the formulation. It may also contain other raw materials to provide additional or different performance characteristics. The vehicle "carries" the pigmentation to the surface and binds it into the coating film.

The ingredients can also be categorized as non-volatile components and volatile components, indicated on the chart by (NV) and (V). Non-volatile components remain in the coating and on the surface once applied. Conversely, the volatile components evaporate from the coating into the air once the coating is applied to the surface. The non-volatile components typically include the resin or binder, the pigmentation, and any additives that may be incorporated into the formulation. The volatile component is the solvent system used in the formulation that is a component of the wet film, but not the dry film of the coating.

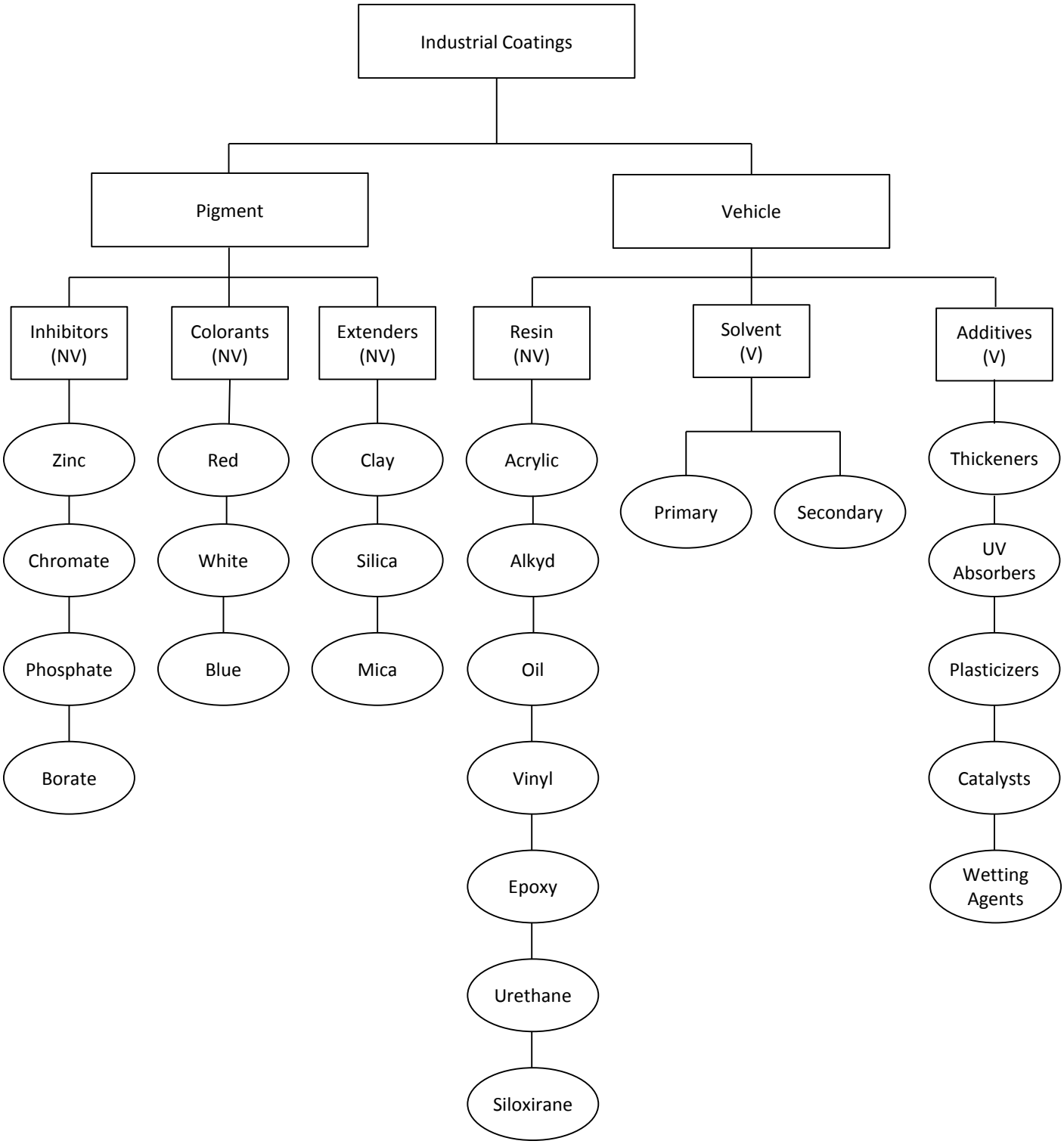


Figure 6.4. Industrial coating elements. (Courtesy KTA-Tator, Inc.)

6.2.2.1.1 Vehicle Resin

The vehicle resin (or binder) portion of the coating vehicle is comprised of both volatile a non-volatile component. That is, it is both part of the wet film and the dry film. Often a coating is identified generically by the

type of resin used in the formulation. For example, a two-coat epoxy is a commonly specified coating system. In this case, "epoxy" is used to describe both the coating type and the raw material resin system used to formulate the coating. The resin system is the film-forming component of a coating. It cohesively bonds the pigmentation together and adhesively bonds the coating to the underlying substrate or coating layer. It is essentially the "glue" of the coating. In many cases, the resin system dictates the performance properties of a coating.

Pigmentation

The pigment is also a non-volatile component of the coating formulation and is essentially an insoluble raw material. It suspends in the resin and solvent rather than dissolving. While some feel that the pigment merely gives the coating its color that is only one of several potential functions.

The pigment in a coating may also provide corrosion protection. If so used, the pigmentation must be formulated into the primer layer (the layer adjacent to the steel substrate). Inhibitors like barium, phosphorous, and others formulated into a primer inhibit the corrosion process. Zinc powder added to a primer in sufficient quantities galvanically protects the underlying steel. Because of the shape, certain pigments even provide barrier protection; in other words, their inherent shape and the way in which they orient themselves in the dry film, create a barrier to moisture penetration through the coating. Examples include micaceous iron oxide and leafing aluminum pigments. These raw materials are lamellar, meaning they are plate-like, tend to lie flat in the coating film, and cause any moisture that penetrates the coating film to take a considerably longer pathway to the substrate. Extenders such as silica, mica, and clay may be incorporated into the formulation to improve film build; increase the solids content of the coating, and/or provide added barrier protection.

Additives

Additives formulated into the coating also become part of the dry film. Various quantities of additives are used by the formulator to adjust the consistency, flow-out, surface wetting, color, ultraviolet (UV) light (sunlight) resistance and flexibility, or to prevent settling in the can (suspending agents). For example, an alkyd coating that typically chalks and fades upon exposure to sunlight can be formulated with silicone (minimum 30%) to provide better color and gloss retention characteristics. In this case, the silicone is an additive. Polyurethane coatings are

formulated with hindered amine light stabilizers (HALS) to help preserve gloss and color upon exposure to sunlight, and plasticizers formulated into a coating provide film flexibility.

Solvents

The solvent system in a coating is the volatile component. While the solvent system is part of the wet film during application, it is not intended to be part of the dry film once the coating dries or cures. This component is referred to as a solvent system because it is uncommon for a coating to be formulated with but one type of solvent. Typically, a blend of solvents is used, and each type of solvent in the blend may perform a different function. As a general rule, primary solvents are formulated into the coating to reduce the viscosity of the resin, pigment, and additives, so that the coating can be properly atomized through a spray gun, or applied by brush and roller. Secondary solvents typically stay in the wet coating film a little longer than the primary solvents, as they are more slowly evaporating solvents, and help the coating to flow-out to form a uniform, continuous film.

Volatile Organic Compounds (VOCs)

Solvents have been used in coatings for many decades, as they have been useful and affordable. Many solvent systems in a coating (and thinners added to a coating by the applicator) are categorized as volatile organic compounds by the Environmental Protection Agency (EPA). Therefore, the type and amount of solvent(s) used in an industrial coating may be regulated by the EPA because as they evaporate from the coating film into the air, they can photochemically react with sunlight and become a precursor to ozone (a component of smog). Federal and state environmental agencies have developed regulations to control ozone-producing operations as part of the Clean Air Act. The amount of VOCs that can be legally emitted into the atmosphere varies considerably from location to location. For example, densely populated areas like Southern California and Houston, Texas, have very strict VOC regulations, while less populated areas typically comply with the federal limit, which represents a considerably higher threshold. California has led the nation in the march toward coatings with ever-lower amounts of VOCs. Coatings suppliers have been reformulating and retesting their coatings as VOC regulations have tightened, and eventually, the use of such materials in coatings may diminish to the point that they are not a significant part of coatings used on bridges.

The VOC content of a coating is expressed in pounds per gallon (or grams per liter), and is reported on the manufacturer's product data sheet (PDS). Many manufacturers also recalculate the VOC content of a coating after the addition of thinner and this information is also commonly referenced on the PDS.

When painting a structure in the field, the VOC limit is typically dictated by the specification or the local air pollution agency for the project. Conversely, fixed facilities such as painting shops are sometimes required to log the number of gallons of paint used over a specific period (say 90 days) and the VOC content of each type.

6.2.2.2 Curing Mechanisms

The method in which a coating converts from a liquid to a solid state is known as the curing mechanism. Many liquid-applied coatings dry by solvent evaporation, but cure by employing a separate reaction.

6.2.2.2.1 Solvent Evaporation

Coatings that cure by solvent evaporation actually only dry. That is, the resin, pigment, and additives are suspended in a solvent system. When the solvent evaporates from the applied film into the air, the resin, pigment, and additives remain on the surface. Because there is no subsequent curing reaction, the resin can be re-dissolved by the solvent system that evaporated from the coating film. That is why a coating that cures by solvent evaporation should not be over-coated with a coating containing strong solvents, as such solvents may re-dissolve the underlying coating film. A vinyl coating is an example of a coating that cures by solvent evaporation.

6.2.2.2.2 Coalescence

Waterborne acrylic coatings cure by solvent evaporation and form a coating film by a process known as coalescence. Water, the primary solvent in these coatings, first evaporates from the acrylic-containing emulsion coating film. As the water evaporates, a special coalescing solvent (e.g., propylene glycol) aids in fusing the acrylic emulsion particles together to form a solid film. The coalescing solvent then evaporates from the coating film. Without this coalescing solvent, the acrylic particles will not impinge and fuse together; this can result in a poor performing coating film. Note that the coalescing process typically requires a minimum 50°F air temperature. Should the air temperature fall below 50°F before the coalescing process is complete, curing may stop and may not start again once the temperature recovers. This major concern with industrial waterborne acrylic coatings should be carefully considered by the would-be specifier.

6.2.2.2.3 Oxidation

Coatings that cure by oxidation react with oxygen (air) to form a film. This oxidation process never stops as long as the coating is exposed to oxygen. For example, long-used alkyd coatings, which typically contain unsaturated oils, pigments, and driers, cure by oxidation. Many aged alkyd systems, even those that have been in service for decades, become very brittle, as the resin continues to oxidize long after the coating is fully cured.

6.2.2.2.4 Polymerization

The prefix "poly" means "many." Many monomers or "mers" are used to create a polymer. These monomers are formulated into components, and the components are packaged by the coating manufacturer separately. It is only when these components are blended together in the correct proportions that a chemical reaction known as polymerization occurs, generating a very resilient coating layer. Coatings that cure by polymerization are multi-component, typically two or three containers. Prior to application, these components are blended together in the correct ratio. Generally, only complete, pre-packaged kits are blended. Once blended, the chemical reaction begins. Coatings that cure by polymerization have a limited pot life. That is, the blended components must be applied before that pot life expires. The pot life will vary from a few minutes to several hours, depending on the formulation and temperature of the coating. Many coatings cure by polymerization; epoxy coatings and aliphatic acrylic or polyester polyurethane coatings are a few of the more common types used on bridges.

6.2.2.2.5 Moisture Cure

Hydrolysis is the reaction of a coating with moisture in order to cure. Only a few industrial coatings hydrolyze in the curing process. These include moisture-cure urethanes and ethyl silicate-type inorganic zinc-rich primers, which require a minimum amount of moisture to cure. In this process, moisture-cure urethanes release carbon dioxide (CO_2), and inorganic zinc-rich primers release ethyl alcohol. The moisture cure process results in a very resilient coating layer, similar to that achieved by polymerization. (Zinc coatings are further discussed in a subsequent section.)

6.2.2.3 Coating Systems Defined

In many cases, coatings can be combined to create a coating system, which includes both the surface preparation and the application of one or more coating layers. If multiple coats of the same product are specified, contrasting colors are sometimes used to help ensure the coverage of the applicator.

Although coating layers usually consist of a primer and topcoat, in many instances an intermediate coat may also be specified. When multiple coatings are used to create a system, they must be compatible. In addition, each coating layer has a function that is performed at a given thickness. Accordingly, adding extra thickness of an epoxy intermediate coat cannot make up for an inadequate zinc-rich primer thickness. Each layer should be applied at the optimum thickness (i.e., neither too thick nor too thin) and verified for proper thickness prior to the application of subsequent layers.

6.2.2.3.1 Surface Preparation

The Society for Protective Coatings (SSPC) and NACE International (NACE) have developed standard requirements that include the end condition of the surface and materials and procedures necessary to achieve and verify the end condition.

The level of surface preparation to be performed is an integral component to the coating system. For example, there would be little point in applying a zinc-rich primer to a marginally-prepared surface, since the zinc must maintain intimate contact with clean steel to provide galvanic protection. If zinc were to be applied over an old coating, the desired galvanic protection cannot develop. Conversely, applying a surface-tolerant coating to a surface prepared to SSPC-SP 5/NACE No. 1, White Metal Blast Cleaning, is overkill, as equivalent performance could be achieved over a much lesser degree of cleaning, usually at a much lower cost.

6.2.2.3.2 Primer Function

The function of the primer is to bond the coating system to the substrate. It also provides corrosion protection for the steel substrate using one or more methods of barrier, inhibitive, or galvanic protection. The primer must also be tolerant of the level of surface preparation performed and must be compatible with the next layer applied. If the primer is the only layer, as with a single coat system, it must be resistant to the service environment and provide corrosion protection to the steel beneath.

6.2.2.3.3 Intermediate Coat Function

An intermediate coat is typically incorporated into a coating system for the purpose of adding barrier protection. It must be compatible with both the primer and the topcoat.

6.2.2.3.4 Topcoat Function

The topcoat, or finish coat, is the first line of defense in a corrosion protection system. It must also be aesthetically compatible with the specifier's priorities and is often required to maintain color and gloss levels for long periods of time. Naturally, the topcoat must be resistant to the service environment, and must be compatible with the underlying layer (i.e., primer or intermediate coat, as appropriate). In addition, the topcoat must be able to accept a maintenance overcoat.

6.2.2.4 Coating Application Methods

6.2.2.4.1 Conventional (Air) Spray

Conventional or "air-atomized" spray uses compressed air to transport the paint from a pressurized pot to the spray gun, in order to atomize the coating into a fine spray and then propel the atomized coating to the surface. As the ratio of air to paint is quite high (~600:1), compressed air cleanliness is critical and transfer efficiency is relatively low. The primary reason conventional (air) spray is used to apply industrial coatings is the ability to precisely control the amount of paint that exits the spray gun and to control the shape of the spray pattern. The apparatus used for the application of metalizing spray is a special variation of a conventional spray gun.

Conventional air spray equipment consists of a pressure pot equipped with two regulators. The first is used to control the amount of pressure in the pot itself, known as pot pressure, and the other is used to control the volume of atomization air that is used to break up the stream of paint exiting the spray tip. Coating manufacturers provide recommended pot and atomization pressures, which often have to be adjusted slightly based on project conditions (e.g., amount of thinner addition, temperature, etc.). Two hoses connect the spray gun to the pot; one containing the paint and the other contains the atomization air. The spray gun has two controls. The lower control regulates the amount of paint that comes out of the spray tip, and essentially adjusts how far the operator can pull back the spray gun trigger, which regulates the amount of paint that exits the spray tip. The upper control regulates the shape of the fan pattern from a small circle for striping of corners, etc. to a larger oval for spraying flat surfaces. A conventional spray gun can be half-triggered—that is, the trigger can be pulled back part way—so that the atomization air, without

paint, exits the spray nozzle. This compressed air can be used to perform a final blow-down of the surface immediately prior to coating application.

The conventional spray gun is held 6-10 in. from the surface, with variations in distance dependent on the type of coating and prevailing spraying conditions, such as air temperature and wind speed.

6.2.2.4.2 Airless Spray

Airless spray has long been the most common method used for applying bridge coatings. If the equipment is operating properly and the applicator employs good spray technique, the finish quality can approach that which is created by conventional spray, but at much higher production levels. Because airless spray does not use compressed air to atomize the coating, the transfer efficiency is relatively higher than conventional spray, reducing airborne emissions. Further, because airless spray does not incorporate compressed air into the paint stream, the cleanliness of the compressed air is not critical.

Airless spray equipment consists of a paint pump that is operated using an air compressor equipped with a regulator. Coating manufacturers provide recommended airless spray pressures, which frequently have to be adjusted slightly based on project conditions such as amount of thinner addition, temperature, etc. A single hose containing the paint connects the spray gun to the pump.

The airless spray gun is held 18-24 in. from the surface is, with variations in distance dependent on the type of coating and prevailing spraying conditions.

6.2.2.4.3 Plural Component Spray

Plural component spray is used for coating materials with a relatively short pot life or coating materials that do not have viscosity-reducing solvents in them (e.g., 100% solids materials). Plural component spray does not require pre-mixing the coating components. Rather, the individual components are pumped to a mixer/manifold at the correct ratio, then are mixed and delivered to the spray gun using a short material hose that can be flushed with solvent using a solvent purge system. This is known as an internal mix process. There are also external mix plural component systems that send each component to the spray gun in separate material hoses. The components blend as they exit the spray tip. It is common for the material hoses to be heated for plural component spray, in order to reduce the viscosity of the components and allow for easier transport and improved atomization.

Plural component spray is available in two basic designs: fixed ratio and variable ratio proportioning pumps. Fixed ratio pumps can only proportion the components in a set ratio (e.g., 1:1), while a variable ratio pump can proportion the materials according to the required ratio (e.g., 2:1, 4:1, 8:1, 16:1). Plural component spray equipment can be complex and typically requires a technician to set up the equipment and monitor the mix ratio, so that the coating materials are not applied off-ratio. This equipment is set up to spray similar to airless.

6.2.2.4.4 Electrostatic Spray

Electrostatic spray is sometimes used to apply bridge coatings. Because of its potentially high transfer efficiency rate, and the resulting reduction in material usage, it can be an attractive application method. In principal, the paint particles are energized (+) as they exit the spray gun. The electrical charge is imparted by a small wire protruding from the spray nozzle. The surface to be coated is grounded (-). The particles are "attracted" to the component or part having the opposing electrical charge, significantly reducing overspray and material usage. The coating, however, must be able to accept an electrical charge, and the addition of a polar solvent is sometimes required to inhibit the coating's natural resistivity. Electrostatic spray can also result in a wrapping of the coating on the opposite side of the direction of spray, making it an attractive alternative for coating inaccessible areas. Because of the difficulty in achieving a uniform ground on large structures, electrostatic spray has not been widely used to coat bridges. Electrostatic spray equipment is typically set up to spray as an airless operation, and electrostatic spray can be used to coat smaller members and hard-to-reach areas.

6.2.2.4.5 Brushes, Rollers, and Daubers

The use of brushes for bridge projects is typically limited to striping—the application of a layer of coating to surfaces where it is difficult to achieve a normal film build. For the same reason, brushes are also used on pitted or rough surfaces around rivet heads, welds, and bolt and nut assemblies, and to cut-in inside and outside corners. Daubers are often used to coat surfaces within crevices like back-to-back angles. Rollers have high coating transfer efficiency and can be used to coat large flat surfaces with limitations: roller nap can become embedded in the dry coating film and act like a wick to pull moisture into the coatings and film thickness is hard to control with a roller.

Choosing the method of coating application depends on many factors including the size and configuration of the surfaces to be coated, the proximity to other structures, environmental regulations, and the specification and the

coating manufacturer's recommendations. While airless spray is no doubt the most frequently utilized method, there are other issues for the contractor/applicator to consider when choosing an application method, including speed and control.

6.2.3 Description of Coating: Hot-Dip Galvanizing (HDG)

Hot-dip galvanizing is the process in which fabricated steel, structural steel, castings, or small parts, including fasteners, are immersed in a kettle or vat of molten zinc, resulting in a metallurgically-bonded alloy coating that protects the steel from corrosion. HDG is often referred to as simply "galvanizing." The term is often used incorrectly to describe steel coated with zinc by other methods such as paint or plating. All of these methods of applying zinc to steel for corrosion protection are very different from HDG.

HDG forms a metallurgical bond between the zinc and the underlying steel or iron, creating a barrier that is part of the metal itself. During galvanizing, the molten zinc reacts with the surface of the steel or iron article to form a series of zinc/iron alloy layers. Figure 6.5 is a photomicrograph of a galvanized steel coating cross-section and shows a typical coating microstructure.

The HDG coating consists of four separate layers. The first three layers have a mixture of iron and zinc, and the external top layer is typically composed of 100% zinc.

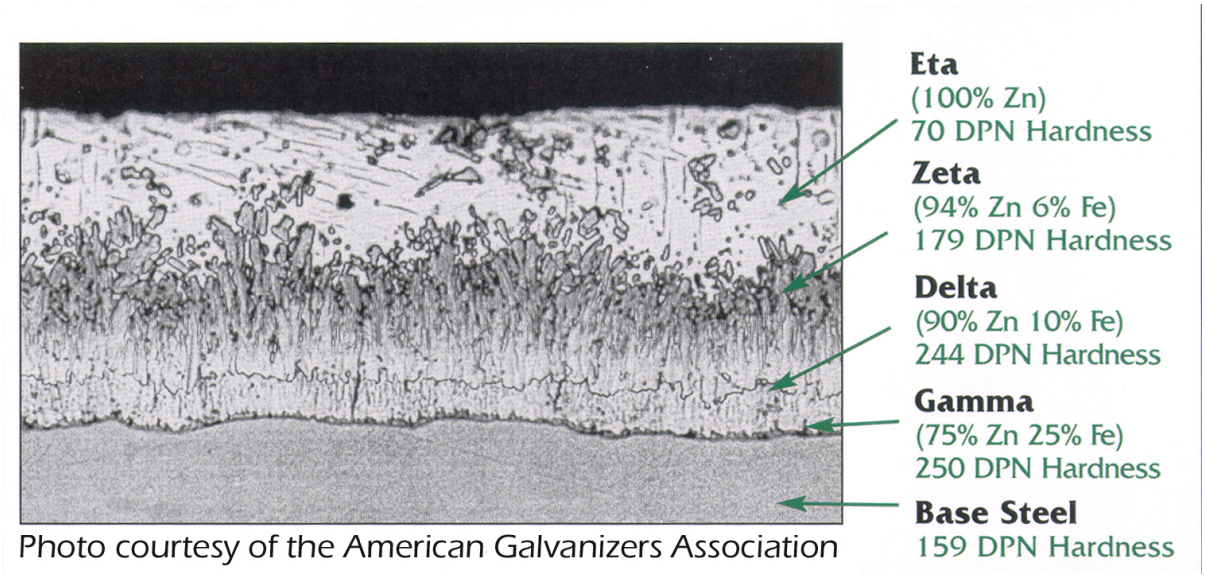


Figure 6.5. Magnified cross-section of galvanized coating.

Below the name of each layer in Figure 6.5 is its respective hardness, expressed by a Diamond Pyramid Number (DPN). The DPN is a progressive measure of hardness; the higher the number, the greater the hardness. Typically,

the Gamma, Delta, and Zeta layers are harder than the underlying steel. The hardness of these inner layers provides unusual protection against coating damage by abrasion. The Eta layer is quite ductile, providing the coating with some impact resistance. The galvanized coating is adherent to the underlying steel on the order of several thousand pounds per square in. (psi). Hardness, ductility, and adherence combine to provide the galvanized coating with unmatched protection against damage caused by rough handling during transportation to the project site, and/or at the project site, as well as while in service. The toughness of the galvanized coating is extremely important since barrier protection is dependent upon the integrity of the coating.

6.2.3.1 The HDG Process

Though the process may vary slightly from plant to plant, the fundamental steps in the galvanizing process are surface preparation, galvanizing, and finishing.

Surface Preparation

Degreasing/Caustic Cleaning. A hot alkaline solution removes dirt, oil, grease, shop oil, and soluble markings.

Pickling. Dilute solutions of either hydrochloric or sulfuric acid remove surface rust and mill scale to provide a chemically clean metallic surface.

Fluxing. Steel is immersed in liquid flux, usually a zinc ammonium chloride solution, to remove oxides and prevent oxidation prior to dipping into the bath of molten zinc. In the dry galvanizing process, the item is separately dipped in a liquid flux bath, removed, allowed to dry, and then galvanized. In the wet galvanizing process, the flux floats on top of the molten zinc and the item passes through the flux immediately prior to galvanizing.

Galvanizing

The article is immersed in a bath of molten zinc between 815°F to 850°F (435°C to 455°C). During galvanizing, the zinc metallurgically bonds to the steel, creating a series of highly abrasion-resistant zinc–iron alloy layers, commonly topped by a layer of impact-resistant pure zinc.

Finishing

After the steel is withdrawn from the galvanizing bath, excess zinc is removed by draining, vibrating, or, in the case of small items, centrifuging. The galvanized item is then air-cooled or quenched in liquid.

6.2.3.2 Coating Uniformity

The galvanizing process produces coatings that are at least as thick at the corners and edges as the coating on the rest of the article. As coating damage is most likely to occur at the edges, this is where added protection is needed most. Figure 6.6 is a photomicrograph showing a cross-section of a corner of a galvanized piece of steel.

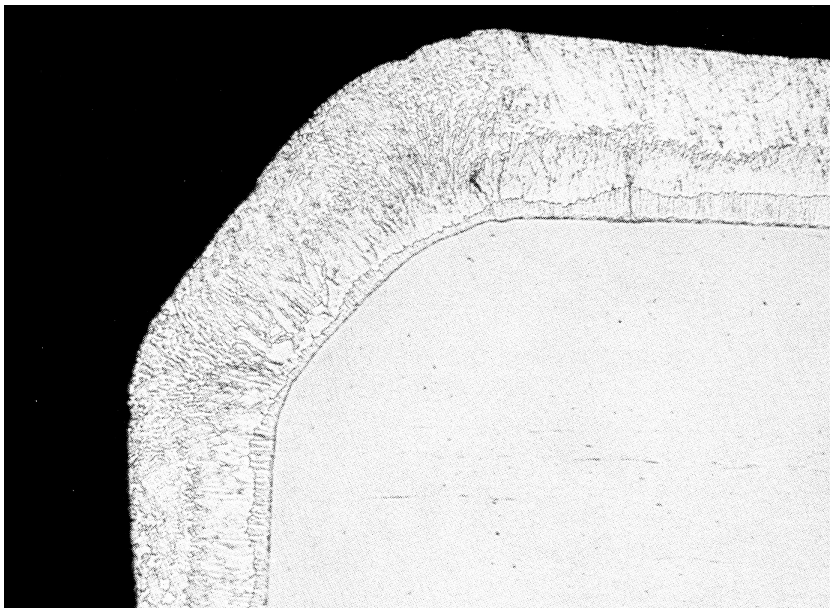


Figure 6.6. Cross-section of corner of galvanized steel section. (Courtesy American Galvanizers Association)

Because the galvanizing process involves total immersion of the material, all surfaces are coated. Galvanizing provides protection on both exterior and interior surfaces of hollow structures, which must be detailed in a way that allows zinc to drain from the interior when the item is removed from the kettle.

The inspection process for galvanized items is straightforward and requires minimal labor. Galvanizing takes place in a factory regardless of weather or humidity conditions. The galvanizer's ability to work in any type of weather provides a higher degree of assurance of on-time delivery and a turnaround time of two or three days is common.

The American Society of Testing and Materials International (ASTM), the Canadian Specification Association (CSA), and the American Association of State Highway and Transportation Officials (AASHTO) specifications establish minimum standards for thickness of galvanized coatings on various categories of items. These minimum standards are routinely exceeded by galvanizers due to the nature of the galvanizing process, which includes factors that influence the thickness and appearance of the galvanized coating such as chemical composition of the steel, steel

surface condition, cold-working of steel prior to galvanizing, bath temperature, bath immersion time, bath withdrawal rate, and steel cooling rate.

6.2.3.3 Effect of Amount of Silicon in Steel on Galvanize Coating

The chemical composition of the steel being galvanized is very important and the amount of silicon and phosphorus in the steel strongly influences the thickness and appearance of the galvanized coating. Silicon, phosphorus, or combinations of the two elements can cause thick, brittle galvanized coatings. The coating thickness curve shown in Figure 6.7 relates the effect of silicon in the base steel to the thickness of the zinc coating. The carbon, sulfur, and manganese content of the steel also may have a minor effect on the galvanized coating thickness.

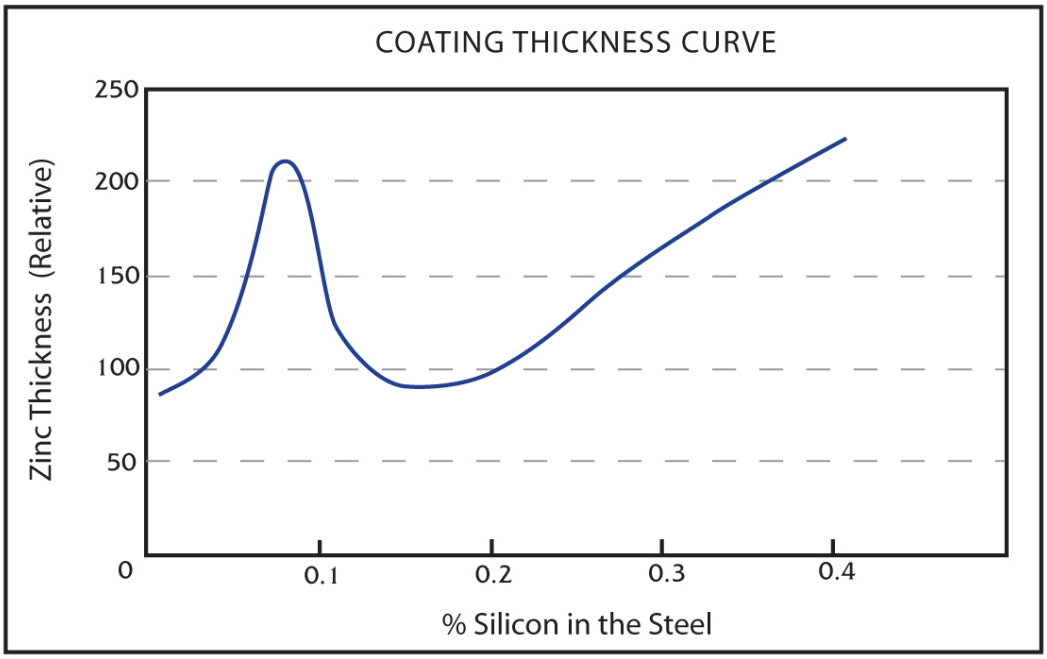


Figure 6.7. Galvanize coating thickness curve. (Courtesy American Galvanizers Association)

The combination of elements mentioned, known as "reactive steel" in the galvanizing industry, tends to accelerate the growth of zinc-iron alloy layers. This may result in a finished galvanized coating consisting entirely of zinc-iron alloy. Instead of a shiny appearance, the galvanized coating has a dark gray, matte finish that will provide as much corrosion protection as a galvanized coating having a bright appearance.

It is difficult to provide precise guidance in the area of steel selection for galvanizing; however, the guidelines discussed below usually result in the selection of steels that provide good galvanized coatings.

- Carbon levels less than 0.25%, phosphorus less than 0/04%, or manganese less than 1.35% are beneficial,

- Silicon levels less than 0.03% or between 0.15% and 0.25% are desirable.

Although it is not part of the controlled composition of the steel, silicon may be present in many steels commonly galvanized. This occurs primarily because silicon is used in the de-oxidization process in making steel and is found in continuously-cast steel. The phosphorus content should never be greater than 0.04% for steel that is intended for galvanizing. Phosphorus acts as a catalyst during galvanizing, resulting in rapid growth of the zinc-iron alloy layers. This growth is virtually uncontrollable during the galvanizing process.

As the galvanizing reaction is a diffusion process, higher zinc bath temperatures and longer immersion times will generally produce somewhat heavier alloy layers. Like all diffusion processes, the reaction proceeds rapidly at first and then slows as layers grow and become thicker. However, continued immersion beyond a certain time will have little effect on further coating growth. When galvanizing reactive steels, the diffusion process proceeds at a faster rate, producing thicker coatings.

The thickness of the outer pure zinc layer is largely dependent on the rate of withdrawal from the zinc bath. A rapid rate of withdrawal causes an article to carry out additional zinc and generally results in a thicker coating.

Figure 6.8. Coating thickness measuring. (Courtesy American Galvanizers Association)

ASTM, CSA, and AASHTO specifications and inspection standards for galvanizing recognize that variations occur in both coating thickness and compositions. Thickness specifications are stated in average terms. Further coating thickness measurements must be taken at several points on each inspected article to comply with ASTM A

123/A 123M for structural steel and A 153/A 153M for hardware. Figure 6.8 shows thickness measurement being taken.

Fortunately, many grades of steel commonly used in steel bridges meet the chemical requirements and are readily galvanized. When in doubt the owners/engineers should be queried and/or the galvanizer's advice should be sought.

6.2.3.4 Inspection

Inspections for coating thickness and surface condition complete the process. Inspection of structural steel will normally fall under ASTM A123/A123M—12 Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products or ASTM A153/A153M—09 Standard Specification for Zinc Coating (Hot-Dip) on Iron and Steel Hardware.

6.2.4 Description of Coating: Thermally Sprayed Metal Coating (TSMC) or Metalizing

When sizes and shapes of steel members will not fit in a galvanizing kettle or when the schedule simply does not allow time to transport items to a galvanizer's plant, there is the option to metalize the item.

Thermally sprayed metal coating, referred to as metalizing, is the process of applying metallic zinc in wire form to clean steel by feeding it into a heated gun, where it is heated, melted, and spray applied by using combustion gages or auxiliary compressed air to provide ample velocity. Metalizing may be used on any size steel object, thereby eliminating limitations due to vat size and awkward shapes. Applying a consistent coating in recesses, hollows, and cavities adds a measure of complexity. Pure zinc can be used, but often zinc is alloyed with 15% aluminum to provide a smoother abrasion resistant film.

6.2.4.1 TSMC Processes

Two similar processes are used to apply the metallic zinc to the steel surface. These are differentiated by the manner in which the zinc metal is melted as described in the following sections.

Flame Spray Process.

The flame spray process can be used to apply a wide variety of feedstock materials, in addition to metal wires including ceramic rods, and metallic and nonmetallic powders. In flame spraying, the feedstock material is fed continuously into the tip of the spray gun or torch, where it is then heated and melted in a fuel gas/oxygen flame and accelerated toward the substrate being coated in a stream of atomizing gas. Common fuel gases used include

acetylene, propane, and methyl acetylene-propadiene (MAPP). Oxyacetylene flames are used extensively for wire-flame spraying because of the degree of control and the higher temperatures attainable with these gases. The lower-temperature oxygen/propane flame can be used for melting metals such as aluminum and zinc. The basic components of a flame spray system include the flame spray gun or torch, the feedstock material and a feeding mechanism, oxygen and fuel gases with flow meters and pressure regulators, and an air compressor and regulator.

In wire-flame spraying, the wire-flame spray gun or torch, shown in Figure 6.9, consists of a drive unit with motor and drive rollers for feeding the wire, and a gas head with valves, gas nozzle, and an air cap that controls the flame and atomization air. Compared with wire-arc spraying, wire-flame spraying is generally simpler and less expensive. Both flame spraying and wire-arc spraying systems are field portable and may be used to apply quality metal coatings for corrosion protection.

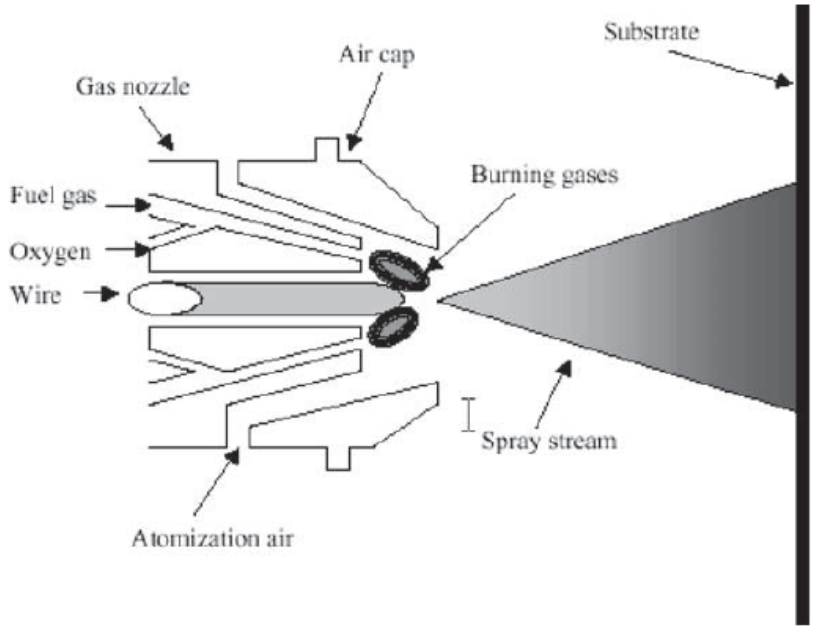


Figure 6.9. Schematic of a typical flame-wire spray gun. (Figure reprinted with permission from *NCHRP Report 528: Thermally Sprayed Metal Coatings to Protect Steel Pilings: Final Report and Guide*. [Ellor et al., 2004])

Wire-Arc Process

Due to its high deposition rates, excellent adhesion, and cost-effectiveness, wire-arc spray is the preferred process for applying TSMCs to steel bridges. In the wire-arc spray process, two consumable wire electrodes of the metal being sprayed are fed into a gun such that they meet at a point located within an atomizing air, or other gas, stream. An applied DC potential difference between the wires establishes an electric arc between the wires that melts

their tips. The atomizing air flow subsequently shears and atomizes the molten droplets to generate a spray pattern of molten metal directed toward the substrate being coated. Wire-arc spray is the only thermal spray process that directly heats the material being sprayed, a factor that contributes to its high energy efficiency.

The wire-arc spray system consists of a wire-arc spray gun or torch (shown in Figure 6.10), atomizing gas, flow meter or pressure gauge, a compressed air supply, DC power supply, wire guides/hoses, and a wire feed control unit. Operation of this equipment must be in strict compliance with the manufacturer's instructions and guidelines.

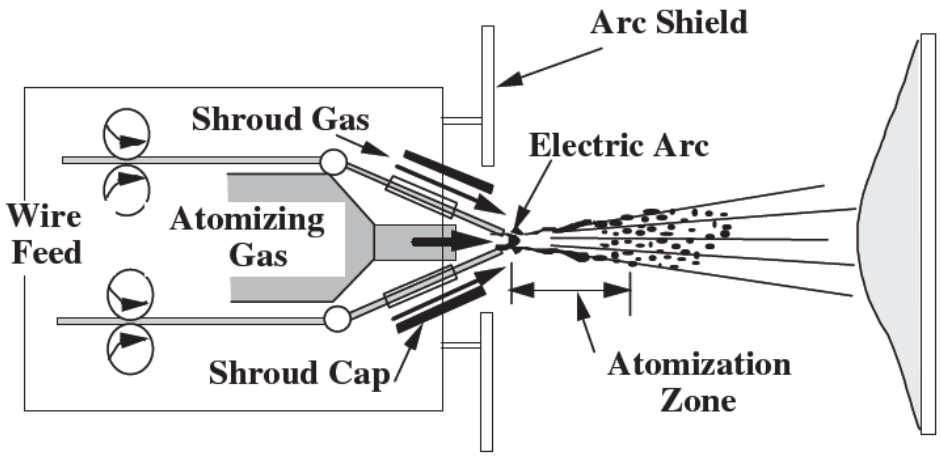


Figure 6.10. Schematic of a typical wire-arc spray gun. (Figure reprinted with permission from *NCHRP Report 528: Thermally Sprayed Metal Coatings to Protect Steel Pilings: Final Report and Guide*. [Ellor et al., 2004])

6.2.4.2 TSMC Guidelines

Table 6.1 provides a thermal sprayed metal coating selection guide for 20- to 40-year life, and shows the TSMC thickness typically applied under various environmental conditions.

Table 6.1. TSMC Coating Guide. (National Cooperative Highway Research Program, Report 528 [Ellor et al. 2004])

Environment	Coating	Thickness mils [μm]	Sealer
Atmospheric			
Rural	Zinc or zinc-aluminum	6-8 [150-200]	No
Industrial	Zinc or zinc-aluminum	12-15 [305-308]	Yes
Marine	Aluminum or zinc-aluminum	12-15 [305-308]	No
Immersion			
Freshwater	Zinc-aluminum	12-15 [305-308]	Yes
Brackish Water	Aluminum	12-15 [305-308]	No
Seawater	Aluminum	12-15 [305-308]	No
Alternate Wet-Dry			
Fresh water	Zinc-aluminum	10-12 [250-305]	Yes
Seawater	Aluminum	12-15 [305-308]	Yes
Abrasion	Zinc-aluminum	14-16 [355-405]	Yes
Condensation	Zinc or zinc-aluminum	10-12 [250-305]	Yes

TSMCs should always be applied to "white" metal (SSPC-SP 5/NACE No. 1, White Metal Blast Cleaning). It is common practice in fieldwork to apply the TSMC during the same work shift in which the final blast cleaning is performed. The logical end point of the holding period is when the surface cleanliness degrades or a change on performance, a bend or tensile test, for example, occurs. If the holding period is exceeded, the surface must be re-blast cleaned in order to establish the correct surface cleanliness and profile.

Thermal spraying should be started as soon as possible after the final anchor-tooth or brush blasting, and completed within six hours for steel substrates that are subject to variations in dew point temperature or holding period. In high-humidity and damp environments, shorter holding periods should be used.

In environments with low humidity, or in controlled environments in which enclosed structures use industrial dehumidification equipment, it may be possible to retard the oxidation of the steel and hold the near-white-metal finish for more than six hours. With the concurrence of the purchaser, a holding period of greater than six hours can be validated by determining the acceptable temperature-humidity envelope for the work enclosure by spraying and analyzing bend test coupons, tensile adhesion coupons, or both. Should the sample fail the bend test, the work must be re-blasted and re-tested.

When specified, a flash coat of TSMC equal to or greater than 1 mil (25 um) may be applied within six hours of completing the surface preparation in order to extend the holding period for up to four hours beyond the application of the flash coat. The final TSMC thickness, however, should be sprayed within four hours of applying the flash coat. This procedure should be validated using a tensile adhesion test, bend test, or both, by spraying a flash coat and waiting through the delay period before applying the final coating thickness.

When dealing with small and movable parts, if more than 15 minutes is expected to lapse between surface preparation and the start of thermal spraying, or if the part is moved to another location, the prepared surface should be protected from moisture, contamination, and finger/hand marks. Wrapping the part with clean, print-free paper is generally adequate.

If rust bloom, blistering, or a degraded coating appears at any time during the application of the TSMC, the following procedure should be performed:

1. Stop spraying;
2. Mark off the satisfactorily sprayed area;
3. Repair the unsatisfactory coating (i.e., remove the degraded coating and re-establish the minimum "white metal" finish and anchor-tooth profile depth as per the maintenance and repair procedure);
4. Record the actions taken to resume the project in the project documentation; and
5. Contact the coating inspector to observe and report the remedial action to the purchaser.

The materials and application costs of a TSMC system is higher than the cost of conventional liquid-applied coatings; however, the dominant factor may not be either material or application-labor cost. Instead, the cost of taking the facility out of service, contractor mobilization, environmental constraints, and monitoring costs are often the major considerations of the total cost. In some complex coating work, the actual cost of materials and application is less than 20% of the total process. Therefore, if service life is increased by the use of TSMC, the process can pay for itself. Nevertheless, TSMC is used most effectively on broad flat surfaces, and complexities occur when the gun/hoses are maneuvered around elements that are mounted at an angle to the main flat surface. For example, TSMC is most efficient when used on a girder web or flange, but progress may be slower when connection plates or stiffeners are encountered. The current premium for TSMC can be as little as 40% as compared to other protective

treatments. In many cases, this is a small price to pay for a material which has proven performance and an estimated service life measured in decades.

Impact and abrasion are significant environmental stresses for any coating system. Abrasion is primarily a wear-induced failure caused by contact of a solid material with the coating, for example foot and vehicular traffic on floor coatings, ropes attached to mooring bitts, sand particles suspended in water, and floating ice. When objects of significant mass and velocity move in a direction normal to the surface as opposed to parallel, as in the case of abrasion, the stress is considered to be an impact. Abrasion damage occurs over a period of time, whereas impact damage is typically immediate and discrete. Many coating properties are important to the resistance of impact and abrasion including adhesion to the substrate, cohesion within the coating layers, toughness, ductility, and hardness. Thermally sprayed coatings of zinc, aluminum, and their alloys are very impact resistant. Zinc metalizing has only fair abrasion resistance in immersion applications because the coating forms a weakly adherent layer of zinc oxide. This layer is readily abraded, which exposes more zinc, which in turn oxidizes and is abraded; 85:15 wt% zinc/aluminum is more impact/abrasion resistant than pure zinc or pure aluminum.

6.2.4.3 Concerns Related to Performance of the TSMC

Coating selection may be limited by the degree or type of surface preparation that can be achieved on a particular structure or structural component. Because of physical configuration or proximity to other sensitive equipment or machinery, it may not always be possible to abrasive blast a steel substrate. In such cases, other types of surface preparation, such as hand tool or power tool cleaning, may be necessary, which, in turn, may place limits on the type of coatings that may be used. In some cases, it may be necessary to remove the old coating by means other than abrasive blasting, such as using power tools, high-pressure water jetting, or chemical strippers. These surface preparation methods do not impart the surface profile that is needed for some types of coatings to perform well. In the case of thermally sprayed coatings, a high degree of surface preparation is essential. This kind of preparation can only be achieved by abrasive blast cleaning using a good-quality, properly-sized angular blast media. Thermal spraying should never be selected for applications in which it is not possible to provide the highest quality surface preparation.

An angular blast media must always be used. Rounded media such as steel shot, or mixtures of round and angular media, will not produce the appropriate degree of angularity and roughness in the blast profile. The adhesion

of TSMCs can vary by an order of magnitude as a function of surface roughness profile shape and depth. TSMCs adhere poorly to substrates prepared with rounded media and may fail in service by spontaneous delamination. Hard, dense, angular blast media such as aluminum oxide, silicon carbide, iron oxide, and angular steel grit are needed to achieve the depth and shape of blast profile necessary for good TSMC adhesion. Steel grit should be manufactured from crushed steel shot conforming to SAEJ827. Steel grit media composed of irregularly shaped particles or mixtures of irregular and angular particles should never be used. Steel grit having a classification of "very angular," "angular," or "subangular," as classified by the American Geological Institute, should be used (Hansink 1994). Also see: Joint Standard SSPC-CS 23.00/AWS C2.23M/NACE No. 12 *Specification for the Application of Thermal Spray Coatings (Metallizing) of Aluminum, Zinc, and Their Alloys and Composites for the Corrosion Protection of Steel* (2003).

6.2.5 Description of Corrosion Resistant Steels—ASTM A1010

ASTM A1010 steel is a 10.5% to 12.5% chromium structural steel with superior corrosion resistance. A1010 is widely used in thicknesses from 1/8 in. to 1/2 in. for structures subjected to aggressive service conditions, such as coal railcars and coal processing equipment. Because of the A1010 superior corrosion resistance, it is also a candidate for challenging bridge applications. The steel can meet the strength and impact properties of AASHTO M270 Grades 50W and HPS 50W up to the thickness of 4 in., making it attractive steel for traditional plate girder bridges.

6.2.6 Description of Weathering Steels (WS)

Uncoated weathering grade steels have a chemical composition containing small amounts of copper, phosphorus, chromium, nickel, and silicon to attain their weathering or corrosion resistant properties.

Weathering grade steels are currently supplied under AASHTO Specification M270, ASTM A709 Grade 50W, and in high performance steel grades HPS 50W, 70W, and 100W. When used in the right environment, these steels are very cost-effective in both the short and long term as they eliminate the need for shop and field painting.

Weather steels have been successfully utilized on coal hopper cars, buildings, and electric transmission towers and began appearing in bridges on a large scale in the mid-1960s. Currently, thousands of weather steel bridges are providing trouble-free service across the United States.

Unpainted weather steel, properly designed and detailed, can realize bridge life cycles up to 120 years with minimal maintenance. This high-strength, low-alloy steel forms a tightly adhering "patina" during its initial exposure to the elements. The patina is essentially an oxide film of corrosion by-products about the same thickness as a heavy coat of paint.

The initial corrosion of weather steel depends on the presence of moisture and oxygen. As corrosion continues, a protective barrier layer forms that greatly reduces further access to oxygen, moisture, and contaminants. This stable barrier layer greatly resists further corrosion, reducing it to a low value. Under appropriate conditions, weather steel will generally corrode at a rate of less than 0.3 mils per year. Corrosion of conventional steels, on the other hand, forms rust layers that eventually disengage from the surface, exposing fresh metal below, thereby continuing the corrosion cycle.

Weathering steel bridges initially look orange-brown in color; however, the color will darken as the patina forms. In two to five years, depending on the climate, the steel will attain a dark, rich, purple-brown color that many think is attractive.

The protective patina will start to form during construction. Workers should avoid damaging the steel while it is being stored or handled, otherwise, the weather steel will appear mottled until the patina reforms to match the undamaged areas. If the beam ends are unpainted, constructors should wrap piers and abutments to protect them from staining.

Bridges constructed of weathering steel in suitable environments and with proper detailing have all the qualities of those using conventional steel and offer the following benefits:

- Initial cost savings compared to conventional painted alternatives;
- Low maintenance consisting of periodic inspection and cleaning, which reduces direct operating costs;
- Minimal indirect costs from traffic delays for major maintenance operations;
- Faster construction resulting from elimination of shop and field painting;
- Good aesthetics since weathering steel bridges eventually achieve an attractive dark brown color that blends well with the environment and improves with age;

- Low impact on the environment compared to painted alternatives that emit undesirable volatile organic compounds;
- Minimal health and safety issues relating to initial and future painting; and
- A good track record for long-term performance based on various state and federal studies.

Several factors can impact the satisfactory performance of weather steel. Experience has shown, for example, that weather steel requires alternating cycles of wet and dry conditions to form a properly adhering protective patina. This would generally rule out its use in areas of high rainfall and humidity or persistent fog. Extreme marine conditions, the presence of roadway deicing salts, pollution, surrounding vegetation, and tunnel-like conditions can also lead to unsatisfactory performance, as can poor detailing and maintenance.

Bridge engineers should avoid specifying weather steel that will be exposed to sea water spray, salt fogs, and immediate coastal salt environments, as salt film deposited on the metal surface, being hygroscopic, tends to maintain continuously damp conditions, preventing the formation of a proper patina.

Heavy use of deicing salts over and under weather steel bridges may cause problems. For example, salt-laden runoff that flows through leaking expansion joints and directly over the steel has been identified as a cause of poor weathering steel performance.

In all probability, bridge expansion joints eventually leak, so many states recommend painting the steel beam ends to a length 1.5 times the girder depth. Painting the ends also eliminates staining of the concrete piers below the joints. The inclusion on the uphill end of the girder of a drip bar detail should be considered to prevent leakage from travelling down the girder flange. Fabricators and erectors should avoid marking blast-cleaned steel with paint, crayons, or wax, which interferes with the forming of the desired patina. Painted ends, plus the requirement to shop blast mill scale from the weather steel to promote a uniform patina, tend to reduce the initial cost savings as compared to painted steel. Tunnel-like conditions result from a combination of a narrow road with minimum shoulders between vertical retaining walls or a wide bridge with minimum headroom and full-height abutments. Such situations may be encountered at urban or suburban grade separations, in which case the lack of air currents to dispel roadway spray leads to excessive salt deposits on the bridge girders.

In addition, weather steel bridges should not be located where ambient atmospheres contain high concentrations of pollution and industrial fumes, especially those caused by sulfur dioxide. However, moderate industrial environments typically speed the weathering process and more quickly achieve the mature dark color.

Detailing of weather steel bridges must promote the wet/dry cycles necessary to form the protective patina and to avoid salt deposits on the girders. In general, the detailing should allow all parts of the steelwork to dry, avoid moisture and debris retention, and promote adequate ventilation. For example, designers should avoid closely-spaced girders that inhibit ventilation, and should avoid or seal overlaps and crevices that may attract moisture via capillary action. Drainage below an overpass should prevent ponding of water, which results in continuous traffic spray.

What follows are recommendations on detailing based on those outlined in the FHWA *Technical Advisory T-5140.22, Uncoated Weathering Steel in Structures* (FHWA 1989).

- Eliminate bridge joints where possible through use of continuous girders and integral abutments.
- Control water on the deck near the expansion joints deck. Consider the use of a trough under the deck joint to divert water away from vulnerable elements.
- Paint all superstructure steel within a distance of 1.5 times the depth of girder from bridge joints.
- Locate welded drip bars in areas of low stress.
- Minimize the number of bridge deck scuppers (holes cut near the edge of a deck to drain water below). The use of fewer scuppers results in a higher amount of flow-through per scupper, minimizing the chances for blockage.
- Eliminate geometries that serve as water and debris traps.
- Hermetically seal box members when possible, or provide weep holes to allow proper drainage and air circulation.
- Cover or screen all openings in boxes that are not sealed.
- Consider protecting pier caps and abutment walls with drip pans and plates to minimize staining.

Designers should specify weathering grade bolts, nuts, and washers for joining weathering steel components. Bolted connections inevitably result in crevices that can trap moisture. If detailing is such that close fit ups are difficult, the joint could be sealed.

While the patina surface generally represents a good sign of performance, laminations and flaking are bad signs. However, a fine-grained adherent layer indicates good expected weathering performance. Inspectors should specifically look for leaking expansion joints, blocked drains, buildups of debris and other moisture traps, sealant failure, and bulging joints and overlaps. If inspectors uncover any of these conditions, appropriate maintenance should follow. In addition, research performed by Texas DOT has resulted in the following recommended measures:

- Flush debris, dirt, and bird and bat droppings from the bridge structure;
- Clear vegetation from pier and abutment areas to enhance air circulation;
- Reseal deteriorating joints; and
- Unblock drains and troughs.

6.3 FACTORS ADVERSELY AFFECTING SERVICE LIFE

6.3.1 General

One of the most important tasks for developing corrosion prevention systems is properly identifying the prevailing service environment, for existing structures, or the projected service environment, for new structures. Among the questions to be answered are, to what will the system and the bridge be subjected? In the case of a new structure or existing structure, this can be a challenge. Service environments can be both predictable (e.g., deicing salt exposure on a bridge in the winter) and unpredictable (e.g., hurricanes and other like storms may bring unexpected conditions). Figure 6.11 shows some of the factors that can influence the service life of steel bridge elements related to corrosion.

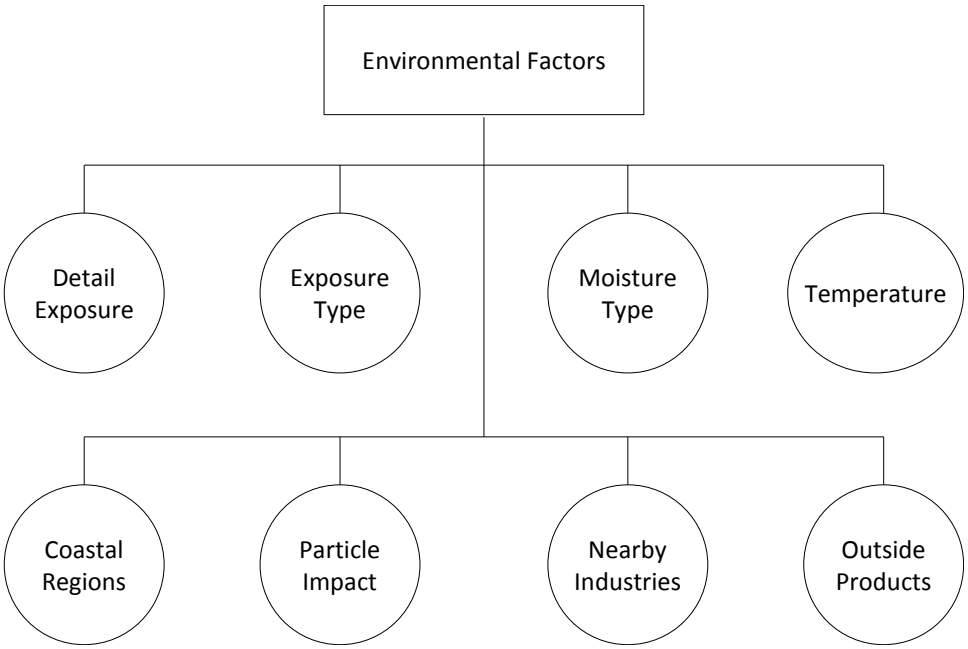


Figure 6.11. Factors that can influence service life of coatings.

The following is a brief description of environmental effects shown in Figure 6.11.

Detail Exposure. Effects will vary for details having interior versus exterior exposure. In cases of interior exposure, entire structures or parts of structures are sheltered and exposed to a less aggressive environment. Coatings on the interior beams or the interior of box members are examples.

Exposure Type. Effects will vary for atmospheric versus immersion-like exposure. In cases of immersion-like exposure, it must be determined whether the exposure is constant or intermittent (i.e., splash), or condensation.

Moisture Type. If exposure is immersion-like, the medium must be considered (i.e., fresh water, salt water).

Temperature. Normal operating and extreme conditions must be considered.

Coastal Regions. Prevailing environment (i.e., coastal airborne sea saltwater mist) must be considered.

Particle Impact. The likelihood and type of physical damage (i.e., impact damage from traffic or traffic-propelled debris) must be considered.

Nearby Industries. Surrounding operations (e.g., adjacent chemical plant) must be considered.

Outside Products. The type and concentration of product that will be stored/transported in tank cars or vessels over or beneath the structure must be considered.

The specifier should consider these and other likely potential environments before selecting a coating system. Any coating manufacturer will almost certainly request this type of information before recommending a coating system. Note also that there may be multiple service environments for a given structure, and interviewing nearby facility owners and plant maintenance personnel may provide added insight into the actual service environment that may be less than obvious.

6.3.2 Factors Affecting Service Life of Steel Bridge Elements, Specific to Paint Coating

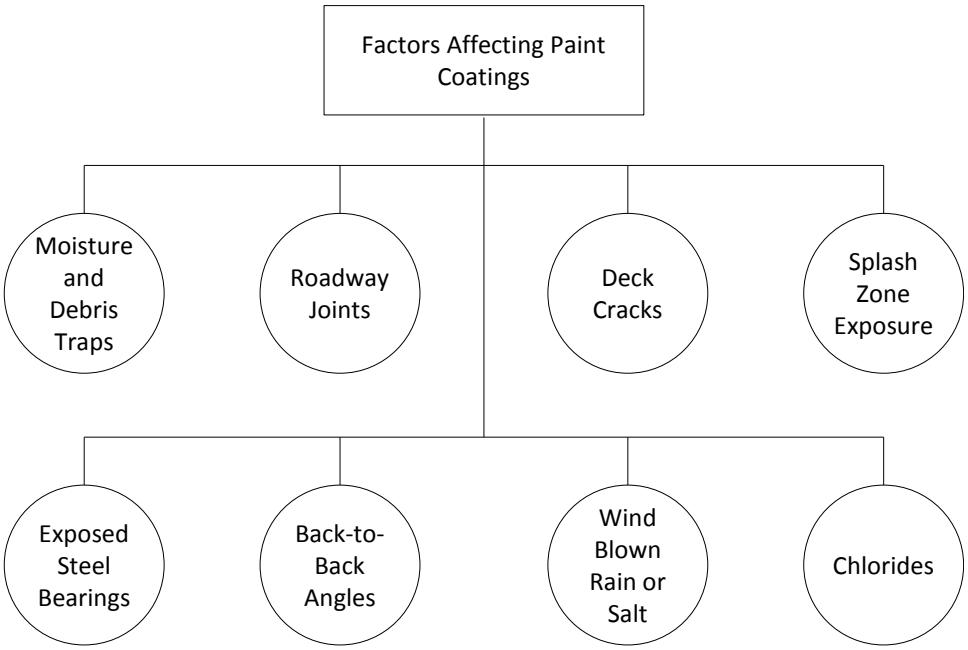


Figure 6.12. Factors affecting service life of paint coatings.

Figure 6.12 shows factors that affect service life of steel bridge elements specific to paint coating. The following sections describe these factors.

6.3.2.1 Moisture and Debris Traps

The creation of moisture and debris traps in new structures is an area of obvious concern, as the presence of such areas will certainly shorten the service life of any organic coating system. The design of new structures should focus on eliminating the creation of these corrosion-prone conditions. If such areas are absolutely essential to the design, corrosion mitigation strategies must be developed.

During maintenance recoating or overcoating projects on existing structures, the consideration of debris and moisture traps is even more critical. If residual contaminants remain on the surface and are overcoated or recoated in any area, the service life of virtually any system will be shortened. This effect is especially exacerbated by the

presence of chloride-laden residue from seawater or snow/ice removal activities. The service life of any coating will be extended by extra cleaning efforts in these moisture trap locations. In addition, owners should consider the use of zinc spray metalizing in these areas, making use of the best protection in the most aggressive corrosion locations.

As a result of this type of exposure, a surface that is normally expected to be dry is in effect an area of severe exposure. These areas are categorized by the SSPC as: SSPC Category 2A (frequently wet by fresh water), 2B (frequently wet by salt water), 2C (fresh water immersion), or 2D (saltwater immersion). Coatings for these areas should be chosen with care. SSPC currently recommends that zinc-rich, primer-based materials be used.

6.3.2.2 Roadway Joints

The design of roadway joints is discussed in Chapter 9. In the past, one of the principle reasons that steel below the deck of bridges got wet and corroded was because of leaky expansion joint seals. These leaks allow water from the bridge deck to cascade from the deck to pour onto the steel members beneath the deck. These leaks change the exposure conditions in such areas from an exposure zone that is designed to be dry (exposure Zone 1B exterior, normally dry) to one of the following: Zone 2A (frequently wet by fresh water), 2B (frequently wet by salt water), 2C (fresh water immersion), or 2D (saltwater immersion). The type of corrosion protection used in these areas is described in Section 6.2. See also the discussion about composite protection in Section 6.5.1.6.

6.3.2.3 Deck Cracks (Cracks in the Concrete and/or the Wearing Surface)

Cracks in the deck and/or in the wearing surface allow water, especially salt water, to penetrate through the deck and pour salt water onto steel surfaces below. When the steel becomes wet, corrosion almost always follows.

6.3.2.4 Splash Zone Exposure

When traffic travels beneath a steel overpass or through a steel truss or like structure, an area of the steel above and often beside the traffic is bathed in water from the roadway. This water is deposited on the steel surfaces. Splash from automobiles and trucks can travel vertically as high as about 20 ft and horizontally 10 ft or more. Painted steel surfaces within that envelope will be in an environmental Zone 2A, 2B, 2C, or 2D as described above, and perhaps in a zone requiring special treatment with zinc spray metalizing where possible, galvanizing any steel that is replaced and/or any steel which can be removed, galvanized, and returned to service.

Figure 6.13. Effect of roadway splash on coated railing.

6.3.2.5 Exposed Steel Bearings

Steel bearings are often sheltered and isolated from water or salt water by the steel and roadway deck directly above as shown in Figure 6.14. On some structures, steel bearings can be the target of corrosion. At times bridges must be closed to traffic and entire bearings must be replaced, which can require extensive shoring as shown in Figure 6.15. Steel bearings can be exposed to an immersion-like environment as a result of leaks from joint areas above.

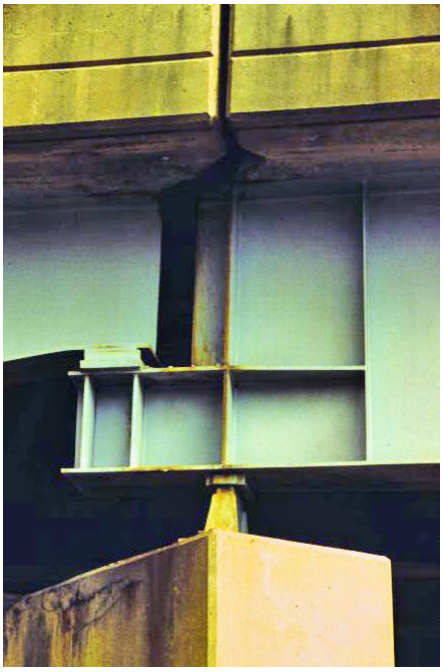


Figure 6.14. Bearings below roadway joints are difficult to protect. (Courtesy KTA-Tator, Inc.)

(Courtesy KTA-Tator, Inc.) **(Courtesy Pennsylvania DOT, District 11-0)**

Figure 6.15. Shoring to support bridge during bearing replacement operation. Note heavy corrosion on bearings in the photo on the right.

6.3.2.6 Back-to-Back Angles

The use of back-to-back angles should be discontinued when new or replacement steel configurations are encountered.

In rehabilitation maintenance, the use of back-to-back angles presents a configuration that is very difficult to clean or coat effectively, and the use of great care in cleaning and coating such areas is recommended. Special tools, often developed by the contractor for these special situations, are needed for effective protection and even then, effective protection is usually unattainable.

6.3.2.7 Wind Blown Rain or Salt Spray

During rain, water can be blown onto steel surfaces even in those instances where there is a bridge sidewalk above. The ability of a coating to withstand this occasional exposure to fresh, non-brackish water should not present a problem for zinc-rich based coating systems.

On the other hand, storms can provide a benefit in that rain water can cleanse exposed surfaces. During storms, salt water from nearby brackish water or from ice- or snow-melt can be blown by storms onto steel surfaces. Repeated exposure to salt water is very corrosive, and the time of wetness and the number and length of wet/dry

cycles affects the degree and extent of corrosion. In areas with a high time-of-wetness, for example frequent wet/dry cycles, HDG or zinc spray metalizing should be considered for primary corrosion protection.

6.3.2.8 Chlorides

In a landmark literature survey compiled by Albias, von Loden, Onderzock and Advies, which was published in the *Journal of Protective Coatings and Linings* (JPCL) in February, 1997, the researchers stated that "It has been clearly established that soluble salts on the surface of steel will increase the rate of corrosion and paint breakdown for many [coating] systems now in use." This conclusion is as true in 2012 as it was in 1997. The authors offered several conclusions:

- From available data, it is not possible to establish a definitive allowable level of chloride contaminants.
- In relation to the durability of the paint system, a maximum chloride level of 10 to 50 $\mu g/cm^2$ is thought to be permissible, depending on the use and exposure conditions. This is only a rough guideline.
- Under specific conditions, higher maximum levels of chloride (up to hundreds of $\mu g/cm^2$) are allowed for special, durable paint systems (e.g., zinc silicate).
- Exposure to marine conditions and/or industrial environments considerably increases the chloride contamination on steel.
- Abrasive blast cleaning does not remove all the chloride.
- Results of detection methods for soluble chlorides are affected by temperature, mechanical forces and the chemicals and type of analytical method used.
- The effect on steel of the hydrochloric acid generated as a consequence of the corrosion reaction is notable. Therefore, the removal of as much chloride as possible during blast cleaning and other surface preparation efforts is crucial. While there is still not complete agreement as to the precise level of chloride residue that is acceptable, SSPC-SP COM " Surface Preparation Commentary for Steel and Concrete Surfaces"(2009), Subsection 4.3.6 "Soluble Salts" identifies three levels of chloride removal:
    - Zero $\mu g/cm^2$

- Less than 7 μg/cm^2
- Less than 50 μg/cm^2

- A level commonly specified for conventional mild steel is 7 μg/cm^2.

An FHWA-sponsored study by Appleman in 1995 concluded that the maximum safe level of chloride to remain when cleaning weathering steel was 50 μg/cm^2.

In accordance with SSPC-SP 10, abrasive blast cleaning on non-corroded areas will reduce chloride levels to an acceptable <7 μg/cm². It will not always do so on heavily rusted, rust scale covered, or pitted or pack rust affected areas.

In some areas, it is believed that complete removal of all chlorides via only dry abrasive blast cleaning is at best unlikely, and at worst, provides a false sense of security, even if white metal blast cleaning (SSPC-SP 5, White Metal Blast Cleaning) is specified. Cleaning efforts beyond abrasive blast cleaning are usually needed. The use of high-pressure water cleaning (HP WC), 5,000 to 10,000 psi, has been found to significantly reduce residual chlorides to a very low level. High-pressure waterjetting (HP WJ), 10,000 to 30,000 psi, has also been used to reduce residual chloride levels. As noted, the effect of chlorides on the corrosion rate of steel has been studied and well documented.

6.3.3 Factors Affecting Service Life of Galvanized/Painted Steel Bridge Elements Specific to a Galvanizing Coating

The corrosion protection of unpainted galvanizing comes from the formation of a thin, invisible layer of insoluble corrosion products. Zinc, as an active metal, reacts with oxygen in the air, and zinc oxide starts forming within about 24 to 48 hours after galvanizing and takes about a year for the zinc oxide to cover the entire galvanized surface. The zinc oxide converts to zinc hydroxide upon exposure to moisture in the form of rain, dew, or high humidity. The final step is the reaction of zinc oxide and zinc hydroxide with carbon dioxide in the air to form zinc carbonate. This reaction requires free-flowing air. Zinc carbonate is the dense insoluble material that forms the protective layer, sometimes called the patina. Zinc oxide and zinc hydroxide are water-soluble and not very dense. They adhere loosely to the surface, so painting over zinc oxide or zinc hydroxide does not provide good adhesion of the coating to the surface. The practical problem is that zinc oxide, zinc hydroxide, and zinc carbonate are all white and cause the galvanized surface to appear a dull, matte grey, allowing no way to visually determine what form of

zinc compound is present. Knowing the compound is important because some forms are not corrosion resistant and are unsuitable for painting over.

6.3.3.1 Reactivity of Zinc

The reactivity of zinc is well-known to galvanizers. For instance, they know that if pieces are closely stacked together for shipment, there will be no access to carbon dioxide in free-flowing air to form the zinc carbonate. In such a case, only loose zinc oxide and zinc hydroxide will form, causing rapid consumption of the zinc. For this reason, closely-spaced galvanized pieces should be unpacked. Upon doing so, the loose white deposit on the surface should be noted. If this reaction process is allowed to continue, it can totally consume all of the zinc by reaction with the moisture caught between the pieces. While rare, rusting of the unprotected steel may then occur, resulting in the presence of rust beneath the deposit.

6.3.3.2 Prevention/Passivation of White Storage Stain

This white deposit is called "wet storage stain." Galvanizers apply a light coating of oil to prevent the stain. The oil forms a barrier to keep moisture from reaching the zinc, thus preventing the zinc from being converted to oxide and hydroxide forms. However, as paints do not stick to oil, painting the surface without first removing the oil is unacceptable. This is true no matter what type of coating is applied. Another process used to prevent wet storage stain is quenching or passivating with chromates or phosphates. Quenching (i.e., cooling and water bath) is not harmful in and of itself. However the quenching bath may contain small amounts of oil and grease on the surface of the water that are picked up when pieces are removed. Coatings also do not stick to chromate-quenched galvanizing, but the phosphate improves adhesion. While wet storage stain can damage galvanizing, the methods used to prevent it can affect painting results. It is always recommended to consult the galvanizers as to the process employed, especially if the galvanized items are to be painted.

6.3.3.3 Repair of Defects in the Galvanized Surface

The next step in surface preparation is to repair any defects or handling damage. Galvanizing can leave high spots and zinc droplets, which occur when a galvanized piece is withdrawn from the bath and excess zinc runs down the edges onto a protrusion or irregular edge. Droplets form at edges where zinc drains from the piece and can be removed with hand tools. High spots are usually ground down with power tools. Care is required to avoid removing

so much zinc that the remaining thickness is below the specified minimum. *SSPC-Guide 14, Guide for Repair of Imperfections in Galvanized or Inorganic Zinc-Rich Coated Steel Using Organic Zinc Rich Coating*, (2004) should be consulted.

Unstable zinc oxide or zinc hydroxide may not have been entirely removed during the initial cleaning process. There is no simple method for identifying the presence of either, so the surface must be further treated.

Galvanizing can be eroded if exposed to very strong acids or alkali, which may cause the zinc to dissolve as metallic zinc is soluble in very strong acid or alkali environments. In these unusual circumstances, if re-galvanizing is not possible, repairs can be made with coatings in accordance with *SSPC-Guide 14* described above. After restoring the zinc protection, a decision can be made as to whether painting is desired.

6.3.3.4 Preparing Weathered Galvanizing for Painting

Fully weathered galvanizing (i.e., galvanizing that has been outdoors for at least one year and preferably about two) should have a fully-formed layer of protective zinc carbonate. Nothing is required to prepare the surface for normal atmospheric exposure and its service life will not be limited in the normal course of exposure events. Bare galvanized surfaces will be subjected to the vicissitudes of the local weather environment and this unknown, complex exposure may mean that it makes sense to combine galvanizing and painting. A so-called duplex system (i.e., galvanizing and paint) should be considered. Such systems are said to provide 1.5 to 2.5 times the service life of the sum of both galvanize and paint if each is considered separately, and an aesthetically-pleasing palette of colors is available.

If a surface of weathered galvanizing is to be painted, the surface must normally be power washed with clean water at about 1450 psi. Spot repairs of any damage in accordance with *SSPC-Guide 14* are all that is necessary.

6.3.3.5 Surface Preparation of New Galvanizing for Painting

The often-made statement that galvanized surfaces "…cannot be painted" is incorrect. In fact, galvanized surfaces are routinely painted successfully. The following are several steps in which errors of omission or commission are routinely made, with remedies for each. There is no reason why so-called duplex systems cannot perform for decades.

New galvanizing means galvanized steel that is between one or two days new and up to about two years old.

Wet storage stain, if present, must be removed before surface preparation. This can be done by brushing the stain with a 1-2% ammonia solution such as diluted household ammonia. After treatment, ammonia should be removed by rinsing with warm water.

The first step in the surface preparation is to wash off oil, grease, and dirt. This is performed in accordance with SSPC-SP 1 "Solvent Cleaning." Water-based emulsifiers or alkaline cleaners work best. A mildly alkaline cleaner should be used. The cleaning solution should be applied by dipping, spraying, or brushing with soft bristle brushes. A temperature range of 140°F to 185°F works well. Afterwards, the surface should be thoroughly rinsed with hot water and allowed to dry. One helpful tip for determining if oil was applied to the galvanized surface to prevent wet storage stain is to contact the galvanizer; another way is to perform a water bead test in which a drop of water is placed on the surface. If it beads, oil will probably be present. The best advice is that when in doubt, the entire surface should be washed as described above. After the surface is washed, it should be examined for zinc ash, a residue that consists of particles of oxidized zinc that float on the surface of the galvanizing bath. The ash can be removed by washing the surface with a 1% to 2 % ammonia solution.

Common methods for treating the surface in the field prior to painting are phosphating by the use of wash primers, or sweep blast cleaning.

6.3.3.5.1 Phosphating Preparatory to Painting

Phosphating is often accomplished by using a wash primer, a coating that neutralizes the surface oxides or hydroxides and etches the galvanized surface. The most common wash primer is polyvinyl butyral, for example SSPC-Paint 27. These materials are very thinly applied (0.3-0.5 mils) by brush or spray. The galvanized surface should shadow through the coating at this thickness. If the galvanized surface is completely hidden, the wash primer is too thick. Wash primers have poor cohesive strength and will split apart if they are too thick, resulting in paint disbondment.

Phosphating is not recommended if a zinc-rich primer is going to be applied. Zinc-rich primers require intimate contact between the zinc particles in the paint and the zinc metal on the galvanized surface. The zinc phosphate acts as an insulator in the same way that iron oxide (i.e. rust) acts as an insulator on steel surfaces.

6.3.3.5.2 Sweep Blast Cleaning Preparatory to Painting

Sweep blasting is a method of lightly blast cleaning that can remove zinc oxides on the surface and roughen the surface without significantly removing the galvanizing. Sweep blast cleaning should be performed with abrasives that are softer than the galvanized surface. The use of materials within Mohs scale hardness of five or less is suitable. Sweep blast cleaning should be performed in accordance with SSPC-SP 7 "Brush-Off Blast Cleaning."

6.3.4 Factors Affecting Service Life of Steel Bridge Elements, Specific to Metalizing Coating

Metalized coatings provide corrosion protection to steel by both sacrificial and barrier protection. The coating itself provides a barrier between the environment and the steel surface, especially when applied in combination with conventional sealer coatings (e.g., epoxies, polyurethanes, acrylics, etc.) as topcoats. Due to the electrochemical reaction between steel and zinc or aluminum in an aqueous or salt-contaminated environment, these coatings sacrifice themselves to protect the steel at the site of any damage, or holes, in the coating. This sacrificial protection is akin to the protection provided by zinc-rich primers or galvanizing.

6.3.4.1 Metalized Coatings

Metalized coatings can be applied in the shop or in the field using a variety of techniques and equipment. The metal or metal alloy is applied in wire form and is fed through a source and liquefied. The source may be either flame (i.e., oxygen-acetylene) or electric arc. The liquefied metal is immediately propelled onto the prepared steel surface using airspray in a manner similar to that used in painting. Once on the surface, the liquid metal cools and dries very quickly to form a continuous protective coating over the steel surface.

6.3.4.2 Cost of Metalizing

Recent (2012) cost estimates place metalizing as 2 to 3 times per square ft. the cost of conventional painting. A recent project at a large fabricator queried indicates that a significant differential currently exists. On a best-case basis, it is estimated that metalizing costs at least 40% to 50% more than painting.

6.3.4.3 Salt-Contaminated Areas

Metalized coatings consist of spray droplets which have solidified and overlapped providing a somewhat coarse matrix. This matrix is a barrier coating as well as a chemically active one due to the anode/cathode relationship between zinc and iron. The primary benefit of metalizing over other coating technologies is its durability and

corrosion resistance especially in salt-rich environments. For this reason, the application of metalizing should be considered as an option for bridge structures in salt-rich environments or for areas or components of bridge structures which receive considerable exposure to salt and moisture from drainage and runoff. While there are cost differences between metalizing and painting, in many cases metalizing should be specified. Based on the performance of metalizing over a long period of time, repairs and renovations on steel bridges would benefit by its use.

Metalized coatings have been shown to perform very well in studies when applied over steel that has been blast-cleaned in accordance with SSPC-SP 5, "White Metal Blast Cleaning," or SSPC-SP 10, "Near-White Blast Cleaning." These coatings have a dull, grey appearance with a rough texture as applied, but may be sealed and topcoated with most conventional paints. Sealing is recommended by many existing guidelines as it tends to increase coating life, reduce the deleterious effects of metalized coating porosity, and improve aesthetics.

Metalized coatings provide the benefit of defect tolerance. The sacrificial nature of these coatings provides corrosion protection to the underlying steel at the site of breaches in the coating film. Metalized coatings, particularly aluminum and aluminum alloys, also tend to be quite abrasion resistant.

6.3.4.4 Bond Strength

The bond between the metalized coating and the steel surface is mechanical in nature. As such, the bond is sensitive to surface contaminants and to the shape of the surface profile. Surface preparation should be specified as above (SSPC-SP 10 or SSPC-SP 5 with an angular 2-4 mil anchor (tooth) profile.

Blast cleaning with rounded steel shot has produced deficient adhesion results and as such steel shot abrasives should not be used on surfaces which will or may be metalized.

As a "solventless" coating application method, metalizing is less forgiving than conventional paint application. Applicators must be properly trained and experienced with the specific equipment and metals or alloys to be used.

Because metalized coatings are inherently porous, achieving an adequate coating thickness (6-8 mils min.) in an overlapping spray pattern is critical to coating life.

6.3.4.5 Field vs. Shop Application

Metalizing technology may also be applicable to field maintenance coating operations where a long-term, durable corrosion protection coating system is required. Applications of metalized coatings in the shop, and particularly in the field require technically sound specifications and practices.

6.3.5 Factors Affecting Service Life of Steel Bridge Elements, Specific to Weathering and Non-corrosive Steels

6.3.5.1 Corrosion Resistant Weathering Steel

With the introduction of steel as a material of construction for bridges in the late 1800s, the industry has sought to find a form of steel that can overcome its most basic limitations—corrosion. It was believed that an answer had been found in the 1970s with the introduction of weathering steel. Since the 1970s, the search for the ideal material has evolved through improved higher strength weathering steel—high performance steel (HPS). Each step has produced incremental improvements in the performance of weathering steel in a normal weathering environment. Unfortunately, there has been even wider application of the use of weathering steel in bridges at locations which are not recommended for the best use of weathering steel. These areas are often in heavily salted areas, or are in areas where the steel is sheltered or exposed to other conditions in which the very corrosion-resistant patina simply does not form. These areas are discussed elsewhere in this chapter.

6.3.5.2 A1010 Structural Stainless Steel

Although initial corrosion studies performed on A1010 steel have been very favorable, the use of A1010 steel is currently inhibited somewhat by its premium cost. It is believed that with sufficient production, volume costs will come down. While testing has produced promising results, its performance in aggressive, salt-laden areas is not completely known. Even with these unknowns, however, it is hopeful that a solution to the 125+ year-old problem of dealing with the corrosion characteristics of steel will be determined. As additional bridges are constructed using A1010 steel, time will tell. Currently (2012), one bridge has been completed in California and two others are under construction in Oregon.

6.4 OPTIONS FOR ENHANCING SERVICE LIFE WITH RESPECT TO CORROSION PERFORMANCE OF STEEL

6.4.1 General Categories of Solutions for Preventing Corrosion of Steel Bridge Elements

In general, there are three options for developing corrosion prevention systems for steel bridges:

1. The use of a coating system, which can consist of paint, galvanizing, or metalizing systems;
2. The use of corrosion resistant steel (weathering steel) or non-corrosive steel; and
3. Avoidance of corrosive environments or corrosion-prone details.

6.4.2 General Strategies Capable of Producing Effective Corrosion Protection System

Regardless of the option selected from the list provided in Section 6.4.1, there are five major strategies that can result in an effective corrosion protection system. These are described in the following sections.

- Design review to assure that the best protection is designed into the structure,
- Use composite protection,
- Use corrosion-proof materials,
- Employ super-durable coatings, and
- Use on-going engineered maintenance painting.

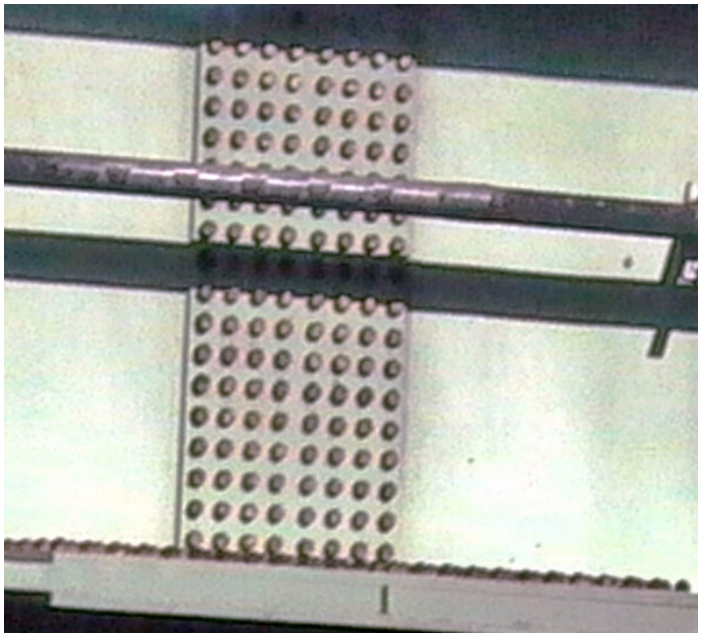
6.4.2.1 Designing Corrosion Protection into the Structure from the Beginning

Through the first 125 years of the steel bridge era, steel bridges have benefitted from the corrosion control foresight utilized by their designers. The elimination or minimization of corrosion on such structures has resulted in a knowledge base which, if systematically applied to every structure, can benefit each one. As these lessons-learned are applied, the corrosion resistance features, principles, experiences, and insights should be designed into every new and rehabilitated steel bridge. Actions taken during the earliest project design stages can cause a dramatic lengthening of the coatings part of the maintenance/repair/replace cycle by the elimination of areas likely to corrode early in the service life of a structure. It is known that if corrosion resistance is designed into a structure, by carefully managing the configuration and details of bridge design and detailing, while using the current coatings systems, bridge corrosion resistance should improve dramatically. This design review should be considered a design hold-point. In

this instance, hold point means that further progress on the design would depend on having a corrosion review performed and a corrosion resistance control plan initiated.

This design phase review is a major means for creating a 100-year life for a new steel bridge. In order to attain 100 years of service life, it will be necessary to develop and utilize preferred details, which will serve as a way to lengthen the time before any maintenance painting is needed during the structure's expected 100-year service life.

Lack of stiffener clearance to back wall results in an almost unpaintable detail.



Coating failure at splice, especially around fasteners.

Figure 6.16. Details difficult to paint. (Courtesy KTA-Tator, Inc.)

6.4.2.2 Composite Protection

Currently the use of zinc to protect steel from corrosion is the gold standard of care. The use of a composite protection strategy is based on the premise that there is an order of efficacy in terms of corrosion protection provided by zinc as delivered in its various forms.

Hot-Dip Galvanizing (HDG) is considered the most efficacious protection because of the iron/zinc alloy that is formed on the steel surface closest to the outside of the HDG part. Thereafter, even if the HDG surface is nicked, the alloy layer will afford substantial protection from corrosion. Many smaller bridge elements, steel bearings, cross frames, bolts, and expansion devices, etc. can be protected with HDG.

Metalizing, as noted, has been tested for decades and also found to be excellent means of protecting steel from corrosion. The spray–applied zinc does not form an alloy layer like HDG, but does provide zinc in intimate contact with steel (iron) in order to provide effective galvanic protection. Metalizing has been tested repeatedly in both the laboratory and field and found to provide a very high level of corrosive protection.

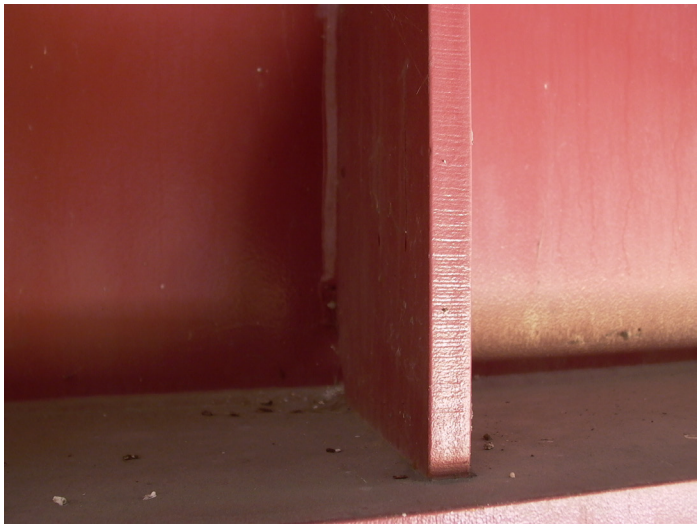
Zinc-rich, primer-based coatings systems have been the workhorse of the steel bridge industry for over 40 years and coatings systems based on zinc-rich coatings have a successful track record on countless bridges.

Uncoated weathering steel also has a 40-plus year history of providing successful corrosion protection in certain exposure areas.

Structures and parts of structures can be protected using combinations of protective steps, for example steel bearings or cross frames can be hot-dip galvanized or metalized and then painted. Some fasteners (ASTM A-325 bolts) are available with either an HDG or mechanically-galvanized coating and either can be coated or not as required.

A composite approach is often adopted on weathering steel bridges when girder ends are cleaned and coated.

Welded cross frame could be galvanized.



Mill to bear stiffener leaves crack in the coating design.

Large trunnion girder for a lift bridge-a unique opportunity for composite protection is presented by its configuration.

Figure 6.17. Details suitable for composite protection. (All photos courtesy KTA-Tator, Inc.)

In the future, ASTM A1010 or a similar material may conceivably be routinely employed in coastal areas or where salt usage is a certainty. It may prove feasible to utilize A1010 in combination with weathering steel. In such a case, perhaps girder ends could be made of A1010 steel, while the remainder of the girder is comprised of weathering steel or painted regular mild steel.

Table 6.2 is a summary of the comparative functionality of galvanizing/metalizing and zinc-rich paint in terms of cost, protection, and durability.

Table 6.2. Three Ways to Apply Zinc to Steel. (KTA-Tator, Inc.)

	Efficacy	Relative Cost	Durability	Duplexable?
Galvanize	Best	Best ($ 1.76/ sf)*	Best	Y G/P
Metalize	Better	Better ($ 4.10 sf)*	Better	Y M/P
Zinc-rich Paint System	Good	Good ($ 2.27 sf)*	Good	Y P/P/P

Strongest Performer "Best"
Second Best "Better"
Third Best "Good"

* Galvanizing costs from American Galvanizing Association. Other costs from experience and research among fabricators.

In the table, the heading term "Duplexable" refers to the particular coating's ability to be combined with other types to provide a composite coating system. In the column, the term Y signifies yes, G represents Galvanize, M represents Metalize, and P represents a paint layer. For example, galvanizing can be combined with a top paint coat, and zinc-rich paint as a primer can be combined with multiple additional paint layers.

6.4.2.3 Use of Corrosion-Resistant or Corrosion-Proof Materials

6.4.2.3.1a Corrosion-Resistant Steel

Weathering steel (coated or uncoated) has been the subject of much research and discussion since its initial use on bridges in about 1970. Weathering steel's roots lay in the improvement in the corrosion resistance of steel when small amounts of copper, chromium, nickel, phosphorous, silicon, manganese, or combinations thereof, are added to carbon steel. When weathering steel is properly exposed, a rusty red-orange to brown or purple-tinted patina forms. When the patina is formed, the corrosion rate of the steel stabilizes within about three to five years. The formation of the protective patina requires a series of wet and dry periods. In certain situations, the protective patina does not form completely or not at all. For example, when the steel is sheltered from the rain, the dark patina cannot form. In areas with high concentrations of corrosive industrial or chemical fumes, weathering steel may exhibit a much higher corrosion rate. In a saltwater marine environment or in areas heavily exposed to chloride-containing deicing materials, the protective patina does not form. The use of uncoated weathering steel in such locations is not recommended.

When weathering steel is used in locations where regular wet/dry cycles occur, the steel is corrosion resistant to the point that no coating is necessary. In some locations, weathering steel enjoys vastly enhanced corrosion resistance which can render it relatively impervious to corrosion. The exact degree of corrosion resistance afforded is dependent upon a number of variables, including climatic conditions, pollution levels, and the degree of sheltering from the atmosphere, as well as the composition of the steel itself. These variables influence the areas in which the use of weathering steel is appropriate.

In a survey conducted as a part of the *SHRP 2 R19A Project* (final report forthcoming), about one third of the 16 DOTs responding reported the use of weathering steel on over 50% of their steel bridges.

Many states use the guidance document published by the Federal Highway Administration, FHWA T5140.22 Technical Advisory, *Uncoated Weathering Steel in Structures* (FHWA 1989). Note that the Technical Advisory for weathering steel use is in the process (as of 2012) of being revised and updated by the FHWA.

In order to shield the weathering steel in areas likely to experience chloride-laden water exposure, many states paint the end of the weathering steel members for a distance of 1.5 to 2 times the depth of the web. The same zinc-rich, primer-based coating systems used by the various DOTs for non-weathering steel are employed. In other

locations, weathering steel is used for its other characteristics and is coated in its entirety for protective and/or aesthetic reasons.

6.4.2.3.1b Non-Corrosive Steel

The "perfect" weathering steel would be a material that is corrosion proof in all environmental conditions. Such a material would present a desirable solution to addressing the main limitation of steel as a construction material, that is to say, steel, rusts. And steel rusts even more in the presence of chlorides. Therefore, the "perfect" material would be a true "unpainted, corrosion resistant solution" for any environment.

A steel product believed by some to meet these stringent requirements has now been developed. The corrosion resistance has been accomplished by chemically augmenting the steel's metallurgical composition. This new grade of weathering steel, ASTM A1010, contains 10.5% to 12.5% chromium and is said by its vendors to be immune to corrosion based on testing performed in Kure Beach, NC in the 25-meter test site. (See Figure 6.18.)

This "corrosion proof" steel is just beginning to be used on bridges. One small structure was constructed in 2004 in Colusa County, California. This project was the subject of a report presented at the 2004 Prefabricated Bridge Elements and Systems Conference. The bridge design was a prefabricated lightweight section referred to as a multi--cell box girder (MCBG). Less than 23 tons of A1010 were needed to form the structure of this short span bridge with overall dimensions of 72 ft long by 32 ft wide.

Construction of two bridges using A1010 steel is underway in Oregon, and the possibility of constructing other bridges has been mentioned in other states.

Although A1010 steel remains a somewhat costly alternate material choice, it does demonstrate that it may be possible to anticipate the use of such a material in the future.

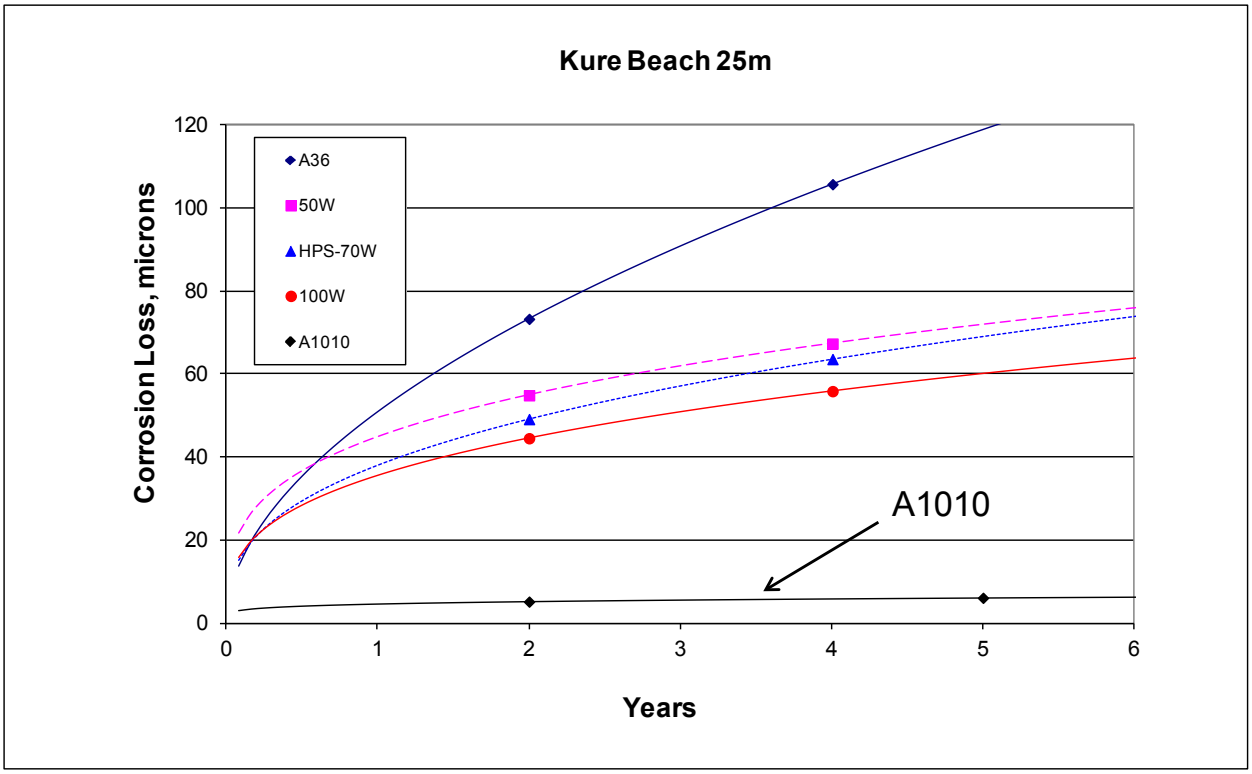


Figure 6.18. Corrosion resistance of A36, A50W, HPS 70W, 100W and A1010 grades of steel exposed at Kure Beach, North Carolina. (Fletcher et al., 2003)

NOTE 25.4 MICRONS=1 MIL (.001")

6.4.2.4 Use of Super-Durable Coatings

The search for the "Superman" of coatings continues:

- FHWA has an active coatings RD&T Program, which recently has focused research on two-coat systems and their ability to perform as well as the traditional three-coat system that has been in use since around1965. Testing of one-coat system candidate materials for steel bridges has also been underway. FHWA has released its final report on that testing as FHWA Report HRT11-046 in June. 2011. While there were some promising prospects among the materials tested, none of them approached the performance of the three-coat control system in the testing.

- On-going efforts to identify new resins and pigments for improved coatings are underway in the private sector.

- Developing new super-durable coating materials via both basic and applied research efforts, including industry-to-industry technology transfer, is underway. The use of nano particles in coatings has begun but

has not yet spread to the bridge industry. The potential use of nano-sized (a billionth of an in.) pigment particles that can dramatically alter the performance of a coating is much anticipated.

- In new construction, the coating systems and procedures are basically unchanged since the late 1960s. For new bridges the steel is coated using a system consisting of a zinc-rich primer, usually an epoxy mid-coat and usually a urethane topcoat, applied over steel cleaned in accordance with SSPC-SP 10 Near-White Metal Blast Cleaning. Initial cleaning and priming is normally done in the fabrication shop.

6.4.2.5 Maintenance Painting

As in many other areas of the construction industry, quality in bridge painting must be built-in and cannot simply be added after the fact. Properly selected and applied coatings can often last for many decades with periodic planned maintenance painting. A comprehensive approach to maintenance painting requires considerations of surface preparations, inspection, and proper planning.

6.4.2.5.1 Surface Preparation

The initial condition of the surface to be cleaned will determine the amount of work, time, and money required to achieve any particular degree of surface cleanliness. It is more difficult to remove contaminants from rusty steel and to remove mill scale from new steel than it is to wash surface film off of steel in good condition. Therefore, it is necessary to consider the surface condition prior to selecting the method of cleaning. The method of cleaning is an integral part of how the coating system may be expected to perform in any given environment. The initial condition of the steel may determine the choice of abrasives. Steel shot is an economical and effective choice for removing intact mill scale. Although their use in the field is not unknown, steel abrasives are usually recycled and therefore find their most common usage in the shop. However, if the steel is rusted or pitted, an angular abrasive, such a steel grit or a non-metallic mineral abrasive, will more effectively scour out the rust.

The initial conditions encountered can be broadly divided into three categories as follows:

1. New construction—steel not previously coated;
2. Maintenance—repainting of previously coated, painted, metalized or galvanized steel; and
3. Contaminated surfaces—common to both new construction and maintenance.

Typical contaminants that should be removed during surface preparation are rust, corrosion products, mill scale, grease, oil, dirt, dust, moisture, soluble salts (i.e., chlorides, sulfates, etc.), paint chalk, and loose, cracked, or peeling paint. Discussions of each of these follow.

Rust, Rust Scale, and Pack Rust. Rust consists primarily of iron oxides, the corrosion products of steel. Whether loose or relatively tightly adherent, rust must be removed for satisfactory coating performance. Rust resulting from the corrosion of steel is not a good base for applying coatings because it expands and becomes porous.

Ideally, rust and rust scale should be removed, even when using the lowest degrees of hand and power tool cleaning (SSPC-SP 2, Hand Tool Cleaning, and SSPC-SP 3, Power Tool Cleaning. Judgment should be used on an individual project basis whether the cost and effort required to remove the stratified rust, rust scale, and to a greater or lesser extent, pack rust, can be justified by the expected increase in the life of the coating system.

To effectively repair pack-rusted joints, it may be necessary to remove rivets, separate the plies of steel, clean, paint, and refasten with bolts.

On riveted and bolted connections, bridge management practices are required that cause surfaces to be repaired long before such inefficient, costly repairs are necessary. It is obvious that many square feet of steel can be cleaned and recoated before the cost of disassembly and reassembly of bridge connections is equaled.

The existence and treatment of rust and particularly pack rust can make bridge repair so expensive that bridge demolition may appear to be a feasible option. When such matters are expected to be at issue, agreement about the extent of removal of these materials should be reached prior to the start of work.

There is a trade-off between repair cost and extended service life. For maintenance repainting, the degree of surface preparation required depends on the new coating system and on the extent of degradation of the surface to be painted. The amount of rusting on the surface is based on the numerical scale of zero to 10, given in *SSPC-VIS 2, Standard Method of Evaluating Degree of Rusting on Painted Steel Surfaces* (2000), in which a reading of 10 indicates no rust and a rating of zero indicates more than 50% rusting. *SSPC-PA Guide 4, Guide to Maintenance Repainting with Oil Base or Alkyd Painting Systems*, (2004) suggests the minimum surface preparation needed for each degree of rusting. This guide includes a description of accepted practices for re-cleaning old, sound paint, removing rust, and feathering the edges of sound coating around the area and recoating. Additional information on

the subject may be found in *SSPC-SP COM, Surface Preparation Commentary for Steel and Concrete Surfaces* (2004).

Mill scale is a bluish, normally slightly-shiny outside residue that forms on steel surfaces during hot rolling at the steel mill. Although initially tightly adherent, it eventually cracks, pops and disbonds. As a general rule, unless it is completely removed before painting, it will, at some point, most likely crack and cause the coatings to crack, exposing the underlying steel surface. In addition, steel is anodic to mill scale and as the anode in the resultant dissimilar metals corrosion cell (with oxygen from the air and moisture) will corrode more rapidly.

At least in the short term, mill scale is somewhat unpredictable in its effect upon the performance of coatings, although, as noted, tightly adhered, intact mill scale may not have to be removed at all for steel exposed to a mild atmospheric exposure. However, if the steel surface is to be coated with primers with low wetting properties, or exposed to severe environments such as wetness or immersion in fresh or salt water, then removal of mill scale by blast cleaning or power tool cleaning is necessary.

Soluble salts are deposited from the atmosphere onto surfaces. If they are permitted to remain on the surface after cleaning and are coated over, they can attract moisture that can delaminate the coating and cause blisters. Salts, particularly chlorides, may also accelerate the corrosion reaction and underfilm corrosion. Methods for measuring the amount of salt on the surface are described in *SSPC-Guide 15, Field Methods for Retrieval and Analysis of Soluble Salts on Steel or Other Nonporous Substrates* (2005). In some circumstances, it is desirable to remove soluble salts by power washing or other methods employing wet methods prior to power tool or abrasive blast cleaning. In other circumstances, salt removal is more efficient after the member has initially been subjected to abrasive blast cleaning.

The removal of this non-visual surface contaminant is an area in which extra effort in cleaning will help immeasurably in improving coating durability.

Ultraviolet light degradation. The sun's ultraviolet light causes exposed organic coating to chalk to some extent. Chalk is the residue left after deterioration of the coating's organic binder on exposed surfaces. All loose chalk must be removed before coating in order to avoid intercoat adhesion problems.

Sharp edges, such as those which at times may occur on some rolled structural members or plates, as well as those resulting from flame cutting, welding, grinding, and shearing, could have an influence on coating performance and may need to be addressed.

Additional guidance on the subject of material anomalies and sharp edges can be found in *AASHTO/NSBA Steel Bridge Collaboration Specification S 8.1, Guide Specification for Application of Coating Systems with Zinc-Rich Primers to Steel Bridges*. (2006)

6.4.2.5.2 Coatings (Paint) Inspection

The importance of coating inspection during surface preparation and coating application cannot be ignored or under-emphasized.

As a general rule, it is not possible to visually examine a coated surface and know whether the surface preparation and coating application was done in accordance with the applicable specification and good painting practice. Determining whether the coating material was properly mixed; whether a component was substituted, adulterated, or left out completely; or whether an entire coat of paint was simply skipped, can be a tedious, costly process. Once a surface has been painted, it is usually not possible to determine whether each painter in a crew has complied consistently with the specification. It is important to recognize the value of adequate inspection.

Unless trained inspectors monitor the entire operation from start to finish, there is no way to know for sure about the level of specification compliance actually achieved; coatings systems can only perform if they are properly installed.

Certification programs for bridge paint inspectors are offered by the Society for Protective Coatings. The SSPC Bridge Coatings Inspector (BCI) training course was developed by a committee of more than a dozen DOT representatives. The course is appropriate for any level of worker in the coatings industry including apprentices, blasters, painters, foremen, superintendents, engineers, and inspectors. Details about the BCI course can be found at www.sspc.org.

Attendance at the BCI class is designed to instruct the attendees to be able to do the following:

1. Define the varying professional rules of the inspector, bridge owner, and painting contractor and the relationship to each other at the project site;

2. Identify what preparation the inspector must make prior to the start of work in order to conduct effective inspections;
3. Recognize common coating inspection and related terms;
4. Identify and properly adjust and operate commonly-used coating inspection instruments and test apparatus;
5. Identify fundamental surface preparation and coating application processes;
6. Identify key documents (e.g., specification, product data sheets, and technical bulletins; industry technical standards; and references) required to perform competent inspections;
7. Identify inspection check or hold points;
8. Create inspection documentation, including a basic inspection plan;
9. Identify processes normally inspected and documented; and
10. Identify common coating application defects.

There are a variety of other inspection training classes offered by private organizations and nonprofit societies. While all have strengths, it is noted that the SSPC BCI trained and certified coatings inspectors have received training prepared specifically for the coating of steel bridges. The SSPC-trained BCI is considered by many to be the most experienced and best-trained bridge coating inspector in the bridge coatings inspection field, and many of SSPC's coatings inspection training courses include the preparation of inspection plans in the curriculum.

In order to provide both guidance to those responsible for creating quality control plans and a training document for course participants, SSPC developed a *Guide for Planning Coating Inspection* in 2008. The planning guide begins by describing the importance of quality monitoring on a project to reduce the risk of coating failure, and describes the challenges associated with trying to assess quality after the project is complete. It also stresses the importance of planning the inspection to increase the likelihood that inspections are performed and the results properly documented. The intended purpose of the planning guide is to assist coating inspectors, quality control personnel, and owners with the development of a tool to help ensure the coating or lining installation is the best it can be.

6.4.2.5.3 Worst First Versus Engineered Maintenance Painting

Many agencies are perennially short of maintenance painting funds. As a result, the bridges that receive coating attention are those that appear to be in the most distress (worst-first). By the time a structure appears to need the most attention, it is probably well past the point at which the spot-on zone cleaning can be effectively employed. The *SSPC Technology Update TU-3, Overcoating* (2004), offers guidance when considering overcoating. In Appendix A, Subsection A2, Painting Scenarios, Subsection A.2.1 entitled Bridge Painting Using Risk Tables deals with the percent of the surface at which the cost of the repairs approached that of full removal. The percentage identified as the critical percentage beyond which the surfaces are rusted or distressed such that surface preparation is necessary is 16+% (per ASTM D610, Rust Grade 2. Note Rust Grade 10 = essentially no rust, while Rust Grade 0 = >50% rust)

SSPC reports in this document that according to the *AASHTO Guide for Painting Steel Structures*, (AASHTO 1994) "whenever the surface preparation area exceeds 15% to 20% of the surface area, the economics are such that a total removal of lead paint is the most viable option."

As noted above, from a cost perspective, the difference in cost between spot or zone cleaning and painting versus full removal is dramatic. According to industry sources, a typical lead paint removal project in the northeastern part of the United States (in 2011) averages ~ $13/sf. Of that amount, about half or $6.50/sf is attributed to surface preparation (access cost, containment, equipment, abrasive, and labor).

If spot or zone cleaning were possible, costs on a comparative basis would be about $3.90 (30%). If cleaning were performed when the surface affected was >3% (Rust Grade 4) but <10%, costs would be lower still.

It is apparent that a timely touch-up would extend the service life of the paint project and lower costs. The reasons for being forced into a "worst-first" mode vary, but the practice is common. If an engineered approach to maintenance recoating were employed, overall costs could be reduced by a large percentage and the condition of the coatings on the bridge inventory in a given city, district, or state would, in time, improve.

It is recognized that bridge maintenance activities are driven by many factors, not all of which are corrosion related. When the matter of maintenance painting is considered, the utilization of an engineering approach will help to counter the worst-first approach. If the de facto use of the worst first practice is unchanged, coatings costs will be at their highest.

If every bridge which was repainted, even if completely re-done, were placed into a master schedule of planned paint touch-up in say 20 years, eventually the worst bridges would be repainted and those structures which are able to be recoated to extend their service lives would emerge as the norm. There are further enhancements to the planning that can be employed. For example, the deterioration of one structure may be faster than another. In those cases, the examination and evaluation of a structure can be scheduled in a different cycle.

Early intervention saves money; can stretch the budget to cover additional projects; and can eventually improve the condition of the steel structures across an area, district, or even an entire state.

6.4.2.6 Bridge Maintenance Owner's Manual

Every new bridge and every rehabilitated structure should be delivered with an owner's manual containing a lifetime maintenance plan which outlines, much like an automobile owner's manual, when designated corrosion mitigation activities are to be undertaken.

For example, it might be recommended that coating touch-up of minor nicks, scratches, etc. be undertaken every five years and that every 25 or 30 years, whenever certain conditions exist, the structure be touched up and overcoated. Following this maintenance plan based on known, needed activities, would allow the owner to achieve the service life inherent in the structure.

6.5 STRATEGY SELECTION PROCESS

There are a wide variety of coating systems among which a specifier can choose. Several of these systems are recognized by the coatings industry as having a track record of successful performance in a given service environment. These systems are assembled by the coating manufacturer according to product, Note that in many cases, a given system can be used in a multitude of locations and service environments. For example, a zinc-rich primer/epoxy intermediate coat/acrylic polyurethane topcoat can be used to protect bridge steel in most areas and has a 40+ year service history.

A coating system is selected based on the prevailing service environment, the intended life of the structure, the level or degree of surface preparation possible, the intended service life of the coating, access, and any other constraints.

A chart listing common generic coating systems follows. It includes common service environments within a given structure and coating systems candidates for each. Note that this chart represents a cross-section of coating systems.

Table 6.3. Coating Systems for Highway Bridges (New Construction and Maintenance). (KTA-Tator, Inc)

Coating System	Highway Bridges (New)	Highway Bridges (Maintenance -1)	Highway Bridges (Maintenance-2)
Inorganic Zinc-Rich Primer/Polyamide Epoxy/Acrylic Polyurethane	√		
Polysiloxane	√		
Organic Zinc-Rich Primer/Polyamide Epoxy/Acrylic Polyurethane	√	√	
Organic Zinc-Rich Primer/ Polyamide Epoxy/Polysiloxane	√		
Organic Zinc-Rich Primer/ Polyamide Epoxy/Fluoropolymer	√	√	
Organic Zinc-Rich Primer/Polyurea	√	√	
Moisture Cure Urethane Zinc-Rich Primer/Moisture Cure Urethane/ Moisture Cure Urethane	√	√	
Moisture Cure Urethane Zinc-Rich Primer/Moisture Cure Urethane/Acrylic Polyurethane	√	√	
Inorganic Zinc-Rich Primer/Water-borne Acrylic	√		
Organic Zinc-Rich Primer/Water-borne Acrylic		√	
Thermal Spray Coating/Sealer	√	√	
Epoxy Sealer/Epoxy Mastic/Acrylic Polyurethane			√
Epoxy Mastic/Acrylic Polyurethane			√
Epoxy Mastic/Waterborne Acrylic			√
Moisture Cure Urethane Sealer/Moisture Cure Urethane/Moisture Cure Urethane			√
Moisture Cure Urethane/Moisture Cure Urethane/Acrylic Polyurethane			√
Alkyd/Silicone Alkyd			√
Calcium Sulfonate Alkyd (2 coats)			√

Maintenance 1: Recoating—Total removal and replacement of existing system
Maintenance 2: Overcoating—Spot or zone repair, spot or zone repriming and overcoat

Maintenance ***overcoating*** is a process in which new coating is applied over existing coating. Based on industry knowledge and DOT survey information obtained for this project, the coating systems currently in use for this purpose include acrylic, calcium sulfonate, epoxy sealer/epoxy/urethane, epoxy sealer/urethane, polyester and polyaspartic.

Maintenance ***recoating*** is a process in which a new coating system is applied over a surface from which all old coating has been removed. According to the DOT survey information described above, the most commonly used systems consist either of an organic or inorganic zinc-rich primer with an epoxy mid-coat and a urethane topcoat.

It is noted that these zinc-rich primer based systems have proven themselves in the field on thousands of structures for over 40 years. In the bridge industry, few materials have this proven track record. No doubt the demonstrated longevity of the systems has contributed to their continued use.

6.5.1 Characteristics by Coating

Table 6.4 is a chart listing common coating types and their inherent properties and characteristics. While the zinc/epoxy/urethane systems are widely used on bridges, special circumstances may dictate the use of systems which are tailored for a specific application. A description of the more commonly used coatings is included in this section.

An explanation of the chart design and an example of how the chart can be used to select a coating material based on the desired performance characteristics, follows.

The left column of the chart contains industrial coating types. Note that within a coating type category, there can be subcategories that are not shown. For example, the category of organic zinc-rich primer includes an epoxy zinc, urethane zinc, vinyl zinc, etc. This list is not exhaustive, but rather contains some of the more common coating types. The top row on the chart contains 17 common characteristics.

Once the service environment is identified and the intended use of the coating is determined, the specifier can review which generic categories of coatings are available. For example, if the specifier is considering overcoating, there are five coatings that can be considered for this application (alkyd, calcium sulfonate alkyd (CSA), epoxy mastic, moisture cure urethane (MCU), and Waterborne acrylic). However, if the overcoat material must also demonstrate abrasion resistance, then only two candidates remain, epoxy mastic and moisture cure urethane (MCU), as the other three do not possess abrasion resistant properties. If single pack paint is desirable (i.e., a product that has all ingredients in a single container) then the specifier can select a moisture cure urethane from these two, as the epoxy mastic is a two-pack product that requires mixing prior to application.

Table 6.4. Coating Characteristics Chart. (Reprinted with permission of SSPC: The Society for Protective Coatings)

Coating type	Color and Gloss Retention	Surface Tolerant	Flexible	Easy to Apply	Low Cost	Can be Modified	Acid Resistant	Caustic Resistant	Abrasion Resistant	Solvent Resistant	Fast Dry	Single Pack	Low VOC Available	Overcoat Material	Chemical Resistance	Immersion	Typical Max Service Temperature**
Alkyd*		X	X	X	X	X					X	X	X	X			250°F
Silicone Alkyd	X	X	X	X							X	X	X				250°F
CSA	X	X	X	X								X	X	X			250°F
Epoxy*						X	X	X	X	X			X		X	X	250°F
Epoxy Mastic		X				X	X	X	X	X			X	X	X	X	250°F
Urethane*	X		X			X	X	X	X	X			X		X		250°F
MCU	X	X	X	X			X	X	X	X	X	X	X	X	X		250°F
Inorganic Zinc-Rich*									X	X			X			X	750°F
Organic Zinc-Rich*									X	X		Mcuz	X			X	Binder Dependent
Waterborne Acrylic*	X		X	X	X	X						X	X	X			250°F
Polyurea	X		X				X	X	X	X	X		X		X	X	350°F
Polysiloxane	X		X					X	X	X	X		X		X	X	200-1400 °F
TSC			X				WD	WD	X	X	X	X	X		WD	X	Wire Dependent

*Most common types of coatings used on steel bridges.

** Consult coating material manufacturer's Product Data Sheet PDS

Legend:

MCU: Moisture Cure Urethane
CSA: Calcium Sulfonate Alkyd
TSC: Thermal Spray Coating
Mcuz: Moisture Cured Urethane Zinc
WD: Wire Dependent

6.5.1.1 Acrylic

Acrylic coatings can be formulated as thermoplastic, solvent deposited coatings, cross-linked thermoset coatings, and water-based emulsion coatings.

The acrylic resins, with suitable pigmentation, provide excellent film-forming coatings characterized by excellent light fastness, gloss, and ultraviolet stability. Chemical resistance to weathering environments is generally excellent, as is resistance to moisture. Most acrylic coatings are not suitable for immersion service or strong chemical environments. In some state DOTs water-based coatings are required as topcoats due to local environmental rules banning VOC's or local preferences.

6.5.1.2 Cross-Linked Thermoset Coatings

Chemically cured coatings refer to coatings that harden or cure and attain their final resistance properties by virtue of a chemical reaction either with a copolymer, or by reaction with moisture. Coatings that chemically cross-link by copolymerization include the epoxy family of coatings, including urethanes. Chemically cured coatings that react with water are moisture-cured polyurethanes and all of the inorganic zinc-rich coatings.

Coatings based on chemically cured binders can be formulated to have excellent resistance to acid, alkalis, and moisture, and to resist abrasion, ultraviolet degradation, and thermal degradation. Chemical and moisture resistance increases as the cross-linking density increases within the larger macromolecule. The rate at which the molecule cross-links is dependent not only on the reactants, but also on the cross-linking mechanism. Most importantly, external factors such as temperature and, with moisture reactions, atmospheric humidity affect the rate and extent of cross-linking. Thus, for chemically altered converted coatings, after application the coating must set through solvent, or water volatilization, and then harden and attain its final cured properties via the cross-linking reaction, which is temperature and/or moisture dependent. A reaction that is too fast may lead to an over-cured, hard, impervious coating that cannot be recoated or topcoated with a properly-adherent subsequent coat. This is always a problem in maintenance repainting when a renewal coat is applied to the original cross-linked coating system after an extended period of time. As a general rule, curing of most chemically cross-linked systems should proceed for approximately seven days at 75ºF before the coating system is exposed to severely corrosive conditions. In corrosive environments, the structure may have to be enclosed in a containment device, and the coating manufacturer's guidance on these issues should be sought.

Following is a discussion of the more common chemically cross-linked binders or resins used for coatings.

6.5.1.2.1 Epoxies

Chemically curing epoxies usually come in two packages: one consists of the epoxy resin, pigments, and some solvent, while the other is the curing agent. For bridge coatings, the two packages are mixed immediately prior to application and, upon curing, develop the large macromolecule structure which provides a tough, water resistant, durable film. However, the film is subject to chalking when exposed to sunlight, and is normally topcoated.

6.5.1.2.2 Urethane Coatings

Urethane coatings have chemical and moisture resistance properties similar to the epoxies, but can also be formulated in a variety of light-stable colors and hues that maintain their gloss and wet look upon prolonged outdoor exposure.

Acrylic urethanes are perhaps the most widely used corrosion protection urethanes for atmospheric service on bridges. These coatings, when properly formulated, have excellent weatherability, gloss, and color retention, and good chemical and moisture resistance. They can be readily tinted and pigmented to provide a variety of deep and pastel colors at a lower cost per gallon than the next most popular class, the polyester urethane. They have excellent weathering properties.

6.5.1.3 Zinc-Rich Coatings

Zinc-rich coatings are a unique class of coating materials that provide galvanic protection to a ferrous substrate. As the name implies, the binder is highly loaded with a metallic zinc dust pigment. After the coating is applied to a thoroughly cleaned substrate, the binder holds the metallic zinc particles in contact with the steel, and to each other. Thus, metal-to-metal contact of two dissimilar metals is made, resulting in a galvanic cell. In this metallic couple, zinc becomes the anode and sacrifices itself to protect the underlying (cathodic) steel.

The major advantage of corrosion protection using zinc-rich coatings is that pitting corrosion is eliminated, even at voids, pinholes, scratches, and abrasions in the coating system. This cannot be said of any non-zinc type of protective coating, and it is this protective capability that makes zinc-rich coating so unique and invaluable on bridges.

This advantage, however, comes with certain disadvantages. The underlying steel substrate must be cleaned of all mill scale, rust, old paint, and other contaminants that may interfere with metal-to-metal contact. Thus, the degree of surface preparation must be quite thorough—and blast cleaning should, at a minimum, be an SSPC-SP 6 (Commercial) blast. For more aggressive, immersion-like exposures, SSPC-SP 10 Near-White Blast Cleaning or SSPC- SP 5 White Metal Blast Cleaning is necessary.

No material is perfect. Because of the high reactivity of the zinc dust pigment, zinc-rich coatings are not suitable outside the pH range of approximately 5.5 to 11. pH ranges from 1 to 14, with pH 7 are neutral, and most bridges are located in areas that are within this range. Both strong acids and strong alkalis will attack the zinc dust pigment, and even if topcoated, penetration of the chemicals may occur through pinholes, scratches, voids, or discontinuities within the topcoat, leading to aggressive attack of the zinc-rich primer.

However, despite these disadvantages, the advantages that accrue by the elimination of pitting corrosion makes zinc-rich coatings, either topcoated or untopcoated, one of the most widely used corrosion prevention coatings for painting steel bridges.

Zinc-rich coatings can be used either as primers with topcoats, or as complete one-coat systems. Both organic and inorganic zinc-rich coatings are used extensively for steel protection on bridges and highway structures, and any area where fresh or saltwater corrosion, mild fume, and high humidity and resultant corrosion is a problem.

6.5.1.3.1 Types of Zinc-Rich Coatings

Zinc-rich coatings can be subcategorized into two types; those with or with inorganic binders. The organic types are similar in many ways to the epoxy, urethane, etc. coating systems previously discussed, except that sufficient zinc dust pigment is added to provide galvanic protection. The inorganic zinc-rich coatings, on the other hand, utilize different binder chemistry, and therefore, are quite unlike the organic zinc-rich coatings.

6.5.1.3.1a Organic Zinc-Rich Coatings

Organic zinc-rich coatings are most commonly formulated from epoxy polyamide, urethane, vinyl, and chlorinated rubber binders.

The drying, hardening, and ultimate curing of the organic zinc-rich coating is predicated on the type of binder used. The organic nature of organic zinc-rich primers makes them more tolerant of deficient surface preparation, as

they more readily wet and seal poorly prepared surfaces where residues of rust or old paint may remain. Similarly, top coating with the same generic type of topcoat is more readily accomplished, because organic zinc-rich coatings of all types generally have a less porous surface and are more akin to conventional non-zinc-rich coatings than are the inorganic zinc-rich coatings.

6.5.1.3.1b Inorganic Zinc-Rich Coatings

The SSPC has categorized inorganic zinc-rich coatings for use in the bridge industry in two major groups:

- Self-cured water base alkali metal silicates, and
- Self-cured solvent base alkali silicates.

While the binder in both cases is an inorganic silicate, essentially the same material as glass or sand, the curing of the binder is different.

6.5.1.3.1c Self-Curing Water Base Alkali Silicates

The most common of these silicate binders is based on potassium and lithium silicates, or combinations of the two. Lithium hydroxide-colloidal silica and quaternary ammonium silicate binders are also included in this category. Self-curing alkali silicate zinc-rich coatings become hard within minutes and are considered generally resistant to precipitation within half an hour after application.

When final curing is ultimately attained, most water base zinc-rich coatings experience a color change, often from a reddish-grey or light grey color to a darker bluish-grey color.

6.5.1.3.1d Solvent Reducible, Self-Cure Inorganic Zinc-Rich Coatings

The binders for this class of coatings are essentially modifications of partially hydrolyzed alkyl silicates. Of these, the ethyl silicate type is most commonly used.

During the condensation phase of the reaction, the partially polymerized silicate combines with atmospheric moisture to eliminate alcohol, which vaporizes. After complete hydrolysis, the cross-linked network forms a matrix to hold the pigment particles.

6.5.1.3.2 Durability of Zinc-Rich Primer Coated Structures

There are thousands of zinc-rich paint coated structures that have been built since the late 1960s and are in good condition. In these cases, a third of the desired service life of 100 years has already been met.

The ability of the coating to last 100 years or more appears to be achievable. Improved coating systems that have been extensively tested by NTPEP (see Section 6.5.2) can be expected to perform for 100 years. Naturally, periodic systematic planned maintenance painting must be performed. Simply put, "painting it now and coming back in 100 years" expecting to see a coating system in good condition is not believed to be achievable at this time.

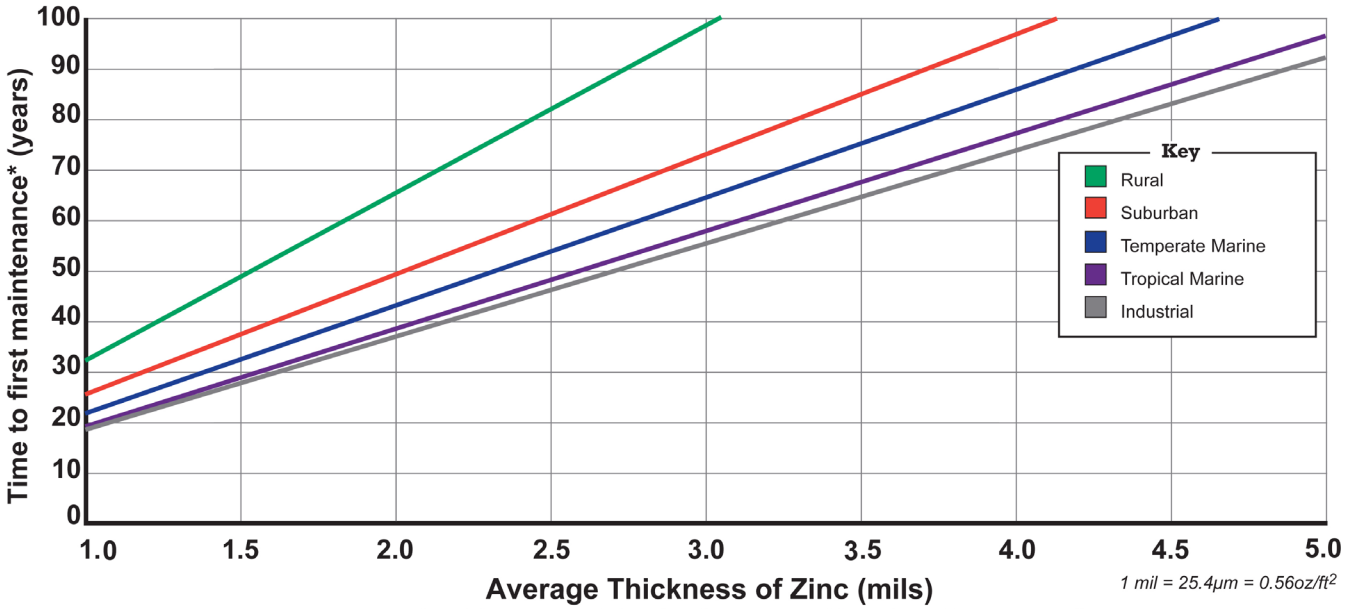
6.5.1.4 Hot-Dip Galvanizing (HDG)

Galvanizing is the process in which steel pieces or parts are immersed in a kettle, or vat, filled with molten zinc, resulting in a metallurgically-bonded alloy coating that protects the steel from corrosion.

When galvanizing is exposed to the natural wet and dry cycles of the atmosphere, it develops a zinc byproduct layer on the surface. This layer is stable and non-reactive unless exposed to aggressive chlorides or sulfides. The protective layer is a key component in the longevity of the HDG coating in the atmosphere.

The American Galvanizers Association (AGA) projects that HDG items will last 75 to 100 years. (See Figure 6.19) This figure shows the relationship between time to first maintenance and zinc thickness for various types of environments. The various environments shown in the Key are listed in the order that the lines are shown in the graph, from top to bottom.

While there are some important differences between zinc-rich coatings and galvanizing, the extensive field-performance history of zinc-rich coatings in combination with the AGA data strongly suggests that steel that has been properly coated with a zinc coating and has additional coating layers to extend the service life of the zinc coating beneath, can last 100 years or more.



*Time to first maintenance is defined as the time to 5% rusting of the steel surface.

Photo courtesy of the American Galvanizers Association

Figure 6.19. Study service-life chart for HDG coatings in various environments.

6.5.1.5 Thermal Spray Metalizing (TSM)

Zinc in wire form may be applied to clean steel by feeding it into a heated gun, where it is heated, melted and spray applied by using combustion gages or auxiliary compressed air to provide ample velocity. Metalizing may be used on any size steel object. As such, limitations due to vat size and awkward shapes are eliminated. Applying a consistent coating in recesses, hollows, and cavities does add a measure of complexity. Pure zinc can be used, but often zinc is alloyed with 15% aluminum to provide a smoother film.

Metalizing spray application generates a smaller spray pattern and application is normally slower than spray painting; accordingly, zinc thermal spray is generally more costly.

FHWA funded two studies, in 1991 and 1996, relative to zinc thermal spray and reported in Publications *FHWA-RD-91-060* (Kogler and Mott, 1992) and *FHWA-RD-96-058* (Kogler et.al., 1997). The 1992 report stated that "… metalized test systems...performed extremely well in the battery of accelerated tests."

In like manner, the 1996 study reported on the testing of 13 coating systems for 5- to 6.5-year periods at three seaside sites. Three of the systems were 100% Zn, 100% Al, and 85% Zn/15% Al metalizing. In most cases, there was virtually no rusting or undercut in the metalized coating and they were reported to have “near perfect corrosion

performance." In this study, the authors noted that the metalizing did not perform any better with or without the epoxy topcoat.

In summary, metalized coatings outperform conventional liquid applied zinc-rich coating systems, but are less easily applied and they are less aesthetically pleasing without having high gloss topcoats.

6.5.1.6 Composite Protection

It is believed that in a given exposure, galvanizing will outperform metalizing because of the alloying effect of molten zinc and steel (iron). Metalizing has been shown to produce impressive corrosion protection, and generally, zinc metalized surfaces will outperform zinc-rich paint coated surfaces. Zinc-rich paint coated members have a proven field history. When difficult conditions are foreseen, all three can be combined to provide protection from corrosion. For example, steel bearings can perhaps be hot-dip galvanized, while girder ends or cross frames could be metalized. In addition, it may be possible and desirable to coat a longer section on the girder end as opposed to the traditional 1.5 times the girder depth. The best set of practices should be set forth for the myriad of service conditions encountered, depending on the exposure conditions encountered on both a macro- and micro-environment basis.

6.5.2 Performance Evaluation of Individual Protective Coating Systems Products

Independent verification of coating system performance based on laboratory testing and/or field exposure is a critical component to selecting a coating system. For a given coating system, there may be multiple manufacturers. It is not safe to assume that all coating systems within a given generic category are created equal. Therefore, careful evaluation of coating system performance prior to full-scale field application is employed to determine which of the candidate systems will perform the best.

AASHTO oversees a materials testing branch—the National Transportation Product Evaluation Program (NTPEP)—which is comprised of highway safety and construction materials project panels. These panels are made up of state highway agency personnel, whose objective is providing quality and responsive engineering for the testing and evaluation of products, materials, and devices that are commonly used by the AASHTO member DOTs. In 1997, the Structural Steel Coatings (SSC) Panel was created to develop a standard specification, a corresponding project work plan, and a reporting system for testing industrial coating systems for use on bridge and highway structures.

Data is generated by pre-qualified independent testing laboratories, and then uploaded to a central database known as *Datamine* for DOT access. All testing is paid for by the coating manufacturer.

The advantage of this type of performance evaluation is that many agencies within a given industry can access performance data with little or no associated costs. Limitations include the ability to keep the database current as new coating systems come to market, the time associated with generating the performance data (AASHTO NTPEP SSC requires approximately 10 months), and the application of the same performance criteria to the data for a coating that will be used on a bridge structure in northern Minnesota and on a bridge in Phoenix, Arizona, two very different service environments.

Some facility owners and agencies may choose to employ a combination of performance/evaluation methods. For example, a bridge owner may subscribe to the AASHTO NTPEP SSC *Datamine* (industry-specific performance evaluation) and may also suspend or mount racks of test panels containing candidate coating systems from a bridge structure

CHAPTER 7

FATIGUE AND FRACTURE OF STEEL STRUCTURES

7.1 INTRODUCTION

This chapter introduces basic principles related to fatigue and fracture in steel bridges, and discusses factors that cause fatigue and fracture. Various available options for repairing observed cracking in steel bridges are also presented. These options are adapted from the *Manual for Repair and Retrofit of Fatigue Cracks in Steel Bridges* (Dexter and Ocel 2006), and are proposed as a guide for the detailing of repairs and retrofits for fatigue cracks. This chapter only contains summarized information from this manual and thus should not be the only means used to develop specifications needed for the repair and retrofit of fatigue damaged details. Refer to the referenced manual for additional detailed descriptions of the topics, procedures, and examples presented in this chapter. Further, this chapter should be used in combination with other existing codes, specifications, and engineering judgment.

7.2 BACKGROUND

Cracks found in steel elements of bridges can usually be attributed to fatigue. Fatigue in metals is described as the process by which cracks initiate and grow under repeated loads. These fatigue cracks can lead to failure if the remaining uncracked section can no longer carry the loads experienced by the structure. In the case of bridge structures, fatigue failure usually occurs as a result of the crack growth that initiates from existing discontinuities. In fatigue, these existing discontinuities are treated as existing cracks. All fabricated steel elements contain discontinuities and most contain high stress concentrations at weld toes. The stress levels causing the failure due to fatigue are usually considerably lower than those that can cause failure under static loading conditions. Fatigue cracks usually form under large amounts of load cycles and worsen with higher stress ranges (Fisher et al. 1998).

Cracks and discontinuities are expected in steel structures and do not necessarily mean that the member will fail as long as the proper precautions are taken. Most modern structures are redundant and allow for the excess stresses in the cracked members to be redistributed, thus keeping the fatigue crack from propagating any further without intervention. However, it is important to assess tension elements that contain cracks to determine the potential for fracture. Bridges that do not possess redundancy for the stresses to be redistributed, face failure of the entire

structure if one of the members were to fail; these members are known as fracture critical members. These structures call for more careful attention, as a fatigue crack can be detrimental to the life of the bridge (Fisher et al. 1998).

Fatigue failure often occurs very suddenly, with little warning; however, the process begins at the onset of the structure's usage, implying that fatigue is progressive. Another important aspect of fatigue is that it is a local phenomenon, occurring in areas of high stresses and strains due to load transfer, abrupt changes in geometry, residual stresses, and material imperfections. The damage caused by fatigue is permanent and is not reversible. Fatigue cracks exist in many structures, but not all of them are critical; certain criteria must be met before the cracks are detrimental to the structural element. Fracture, separation of a component into two or more parts, occurs once the remaining uncracked portion of the member can no longer handle the stresses and strains (Stephens et al. 2001).

The entire fatigue process includes the nucleation (formation) of a fatigue crack, crack propagation (growth), and final fracture (failure). The nucleation of a fatigue crack takes place at the microscopic level, dealing primarily with the microstructure of the material. Discontinuities are common sites of crack nucleation and include persistent slip bands, inclusions, pores, second-phase particles, corrosion pits, voids, and twin and grain boundaries. However, cracks primarily tend to nucleate along slip lines in the direction of planes of maximum shear (Stephens et al. 2001).

Once a fatigue crack forms and continues to undergo repeated loading, it tends to coalesce and grow along the plane of maximum tensile stress range. The crack will grow with each load cycle, even if only by a small amount. As the cracked member is loaded, the crack will open causing an increase in stress at the crack tip; this consequently drives the crack to grow even larger. Fatigue crack growth is broken up into two stages; Stage 1 and Stage 2, as seen in the Figure 7.1. Stage 1 refers to the growth in the direction of the principal shear plane, whereas Stage 2 refers to the growth along the plane of maximum principal tensile stress. Fatigue cracks tend to grow transcrystalline (through grains), but some fatigue cracks can grow intercrystalline (along grain boundaries). Crack growth mechanisms include striation formation, microvoid coalescence (MVC), and microcleavage. Striations are microscopic "ripples" that are representative of the fatigue cycles experienced by the element. Striation can be used to investigate the rate of crack growth and are very useful in forensic studies. MVC involves the formation, growth, and joining of microvoids during plastic deformation. Microcleavage is a fracture along specific crystallographic planes and tends to be a brittle fatigue mode (Stephens et al. 2001).

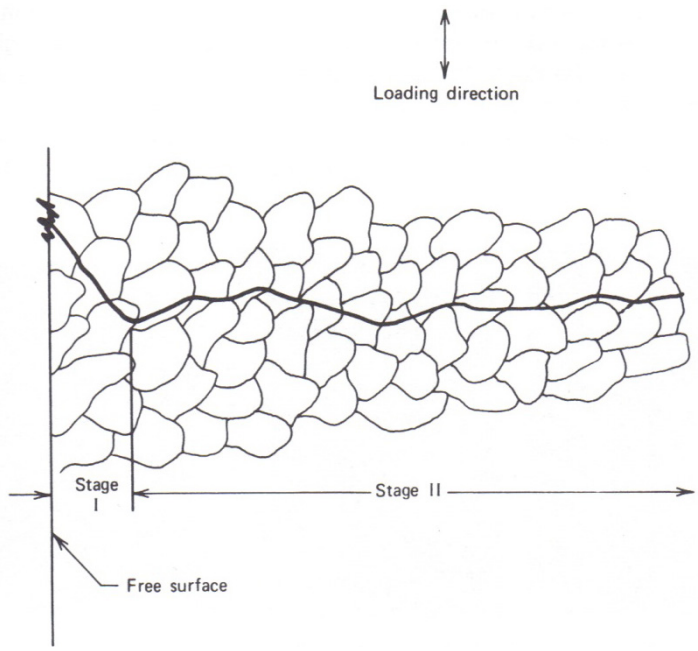


Figure 7.1. Schematic of Stages 1 and 2 of fatigue crack growth. (Stephens et al. 2001)

This section presents only a brief and very general summary of the fatigue process, but it is important for the practicing engineer to understand the principles of the fatigue damage process in order to be proficient in fatigue design.

Minimizing fatigue cracks can be achieved by first avoiding the use of known details that have proven to have low resistance to fatigue. Fatigue cracks can also be reduced by the use of better fabrication and welding processes that lower inherent defects and also allocate for the detection and repair of such cracks before the bridge is opened to the public. In-service inspection is necessary to discover new fatigue cracks and monitor existing ones. Once cracks are located through the inspection process, it is necessary to perform an assessment to determine the risk of fracture. Furthermore, for the proper repair methods to be implemented, the cause and type of crack need to be determined (Fisher et al. 1998).

Several methods exist for determining the fatigue resistance of a detail; these include: nominal stress approach, hot-spot stress approach, and fracture mechanics.

7.2.1 Nominal Stress Approach

This design approach is a simple way to determine the fatigue resistance of a detail using equations for bending and axial loads to compute the nominal stress near a weld toe. Test data from full-scale fatigue tests are needed in order to utilize this design method. Stress ranges (S) versus the number of cycles to failure (N) curves are developed

from the test data. The curves are also grouped into categories to aid in organizing the details according to their fatigue resistance. Figure 7.2 shows the S-N curves as used in AASHTO specifications. The detail categories reflect and account for the variations in the combined geometric and local notch stress concentrations. Each category has a constant-amplitude fatigue limit (CAFL), also referred to as threshold (CAFT). Stress ranges that fall below the CAFL are not expected to exhibit any fatigue failures during constant-amplitude test.

Most bridges with a service life of 75 years are designed as having an infinite-life, no occurrence of fatigue cracking. In the *AASHTO LRFD Bridge Design Specifications* (*LRFD Specifications*), the fatigue design live load is taken as 0.75 times the HS20 for finite load-induced fatigue life and 1.5 times the HS20 for infinite load-induced fatigue life. This fatigue load is used to calculate the nominal stress ranges to be used with the S-N curves. If the resulting nominal stress range is less than half of the CAFL, it is assumed the bridge is designed for infinite life. This ensures that the fatigue limit-state stress range is below the CAFL. The fatigue limit-state is the stress range in which 0.01 percent of the test data exceeds the CAFL. (AASHTO 2012)

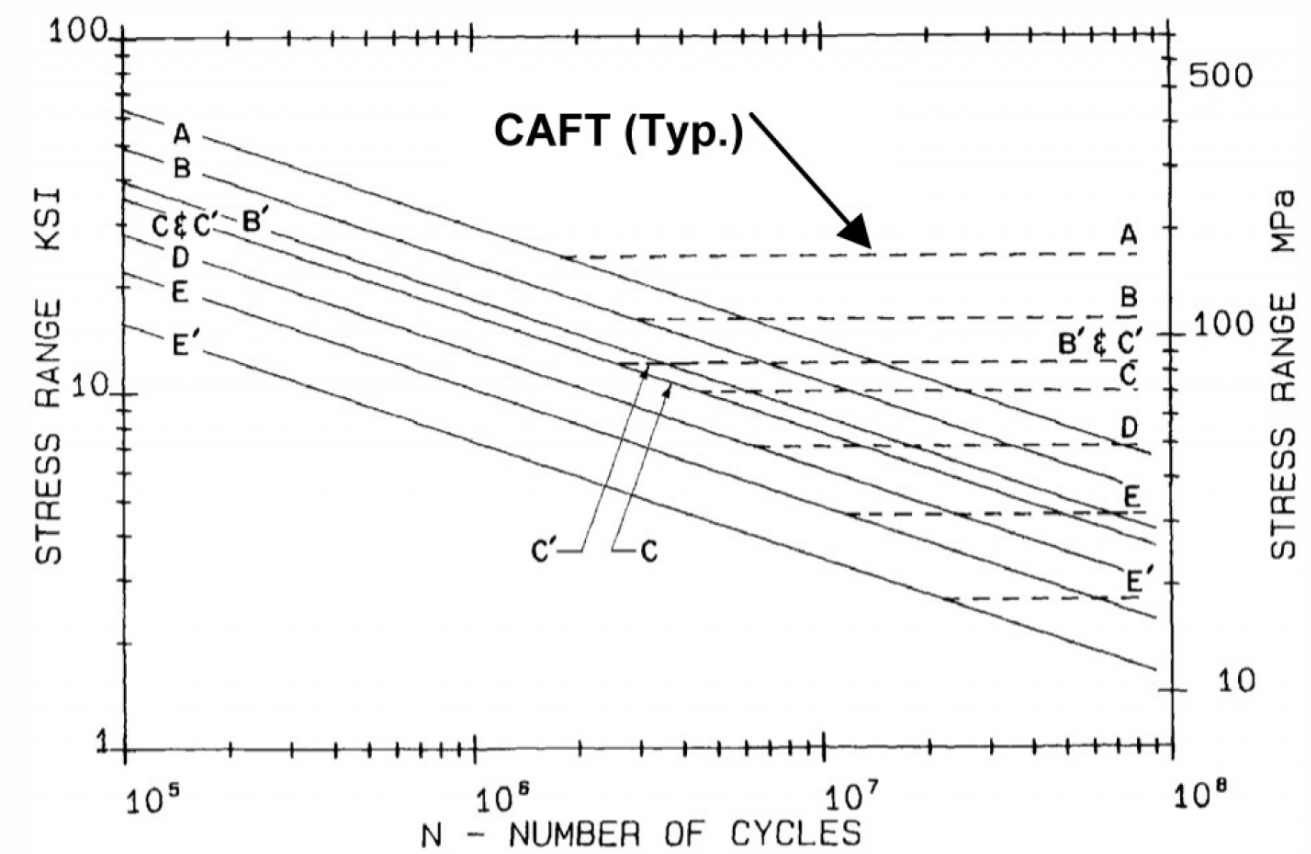


Figure 7.2. S-N curves used in AASHTO, AISC, AWS, and AREMA specifications. (AASHTO 2012)

7.2.2 Hot-Spot Stress Approach

This approach is similar to the nominal stress approach, however the S-N curves are based on the geometric stress ranges, also known as hot-spot stresses. This process is beneficial in instances where the nominal stress approach breaks down, such as offshore tubular structures where the fatigue resistance is heavily dependent on the geometry of the tubes. Using this design process involves determining the stress concentration factor using parametric equations or finite element analysis. Disadvantages arise with the variability of different hot-spot definitions, varying baseline S-N curves, and difficulties with the CAFL.

7.2.3 Fracture Mechanics Approach

In the case of bridge structures, this is the most complex approach and tends to be difficult to implement during the bridge design process. The fracture mechanics approach is divided into two main categories: Linear Elastic Fracture Mechanics (LEFM) and Plastic Fracture Mechanics (PFM). LEFM is used when remotely applied stresses are in elastic ranges. It should be noted that at the crack tip, a stress singularity is always present and stresses tend to approach infinity. In the case of LEFM, the rate of crack growth is related to stress ranges, while in the case of PFM, rate of crack growth needs to be related to a parameter related to energy dissipation or plastic strain (Azizinamini and Radziminski 1989).

For the case of LEFM, the Paris Law, shown in Equation 7.1, can be useful in determining the crack growth rate for many engineering applications.

$$\frac{da}{dN} = C * \Delta K^m \qquad \textbf{EQ 7.1}$$

Where:

$a =$ crack size (mm or inches)
$N =$ number of cycles
$C =$ material constant
$\Delta K =$ stress intensity factor range $\left(MPa - m^{\frac{1}{2}}\ or\ ksi - in^{\frac{1}{2}}\right)$
$m =$ material constant

Although fracture mechanics is rarely used in design, it can serve as a qualitative tool to give the designer a better understanding of structures containing cracks and discontinuities. Fracture mechanic is best suited when local behavior of structure in the vicinity of the crack is of interest, such as how fast crack will grow. However, addressing such problem requires detail understanding of the parameters that affect the performance of crack.

7.3 CRACK DETECTION TECHNIQUES

Cracks are not always obvious to the human eye and can be difficult to locate at times. There are several methods in practice to aid in the detection of cracks, two of which are dye penetrant and magnetic particle inspection.

Dye penetrant consists of three parts; cleaning the area of a suspected crack, application of a liquid dye, and finally the application of a white developer. Figure 7.3 shows a crack exposed using a red dye penetrant.

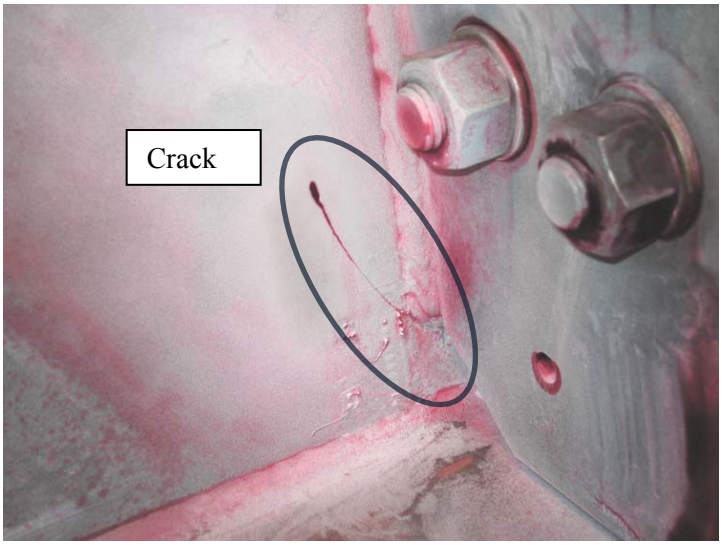


Figure 7.3. Crack exposed using a red dye penetrant

Magnetic particle inspection works by inducing a magnetic field using a handheld device around a crack. The magnetic field is disrupted at the crack and a concentration of a magnetic field results. A fine iron powder is sprinkled over the area of interest and is attracted to the magnetic field, exposing the crack.

7.4 REPAIR AND RETROFIT METHODS

This section presents several methods for the repair and retrofit of fatigue critical details. Such techniques can be categorized as surface treatments, repair of through-thickness cracks, and connection or global structure modification to reduce the causes of cracking.

7.4.1 Surface Treatments

Surface treatments are usually performed on weld toes to increase the fatigue strength of uncracked welds. Such treatments, including grinding, gas tungsten arc (GTA), and impact treatments, aim to improve the weld geometry in order to reduce stress concentrations, remove discontinuities, and/or reduce residual tensile stresses.

7.4.2 Reshape by grinding

Grinding can be used as an effective measure for increasing the fatigue life of the weld toe by removing portions of the weld that contain small cracks. Grinding has proven more effective on larger welds in structures such as offshore structures with large tubular joints. It should be noted that grinding is ineffective against microcracks because the process tends to create new microcracks as it removes the existing ones. Two types of grinding methods are commonly used; disc grinding (Figure 7.4) and burr grinding (Figure 7.5), and both have advantages and disadvantages. Disc grinding can be more effective at removing the weld material with faster speeds; however, the operator needs to use caution to avoid gouging the metal or removing too much of the weld material. Burr grinding is typically easier to operate and works in more confined spaces than disc grinding. (Gregory et al. 1989)

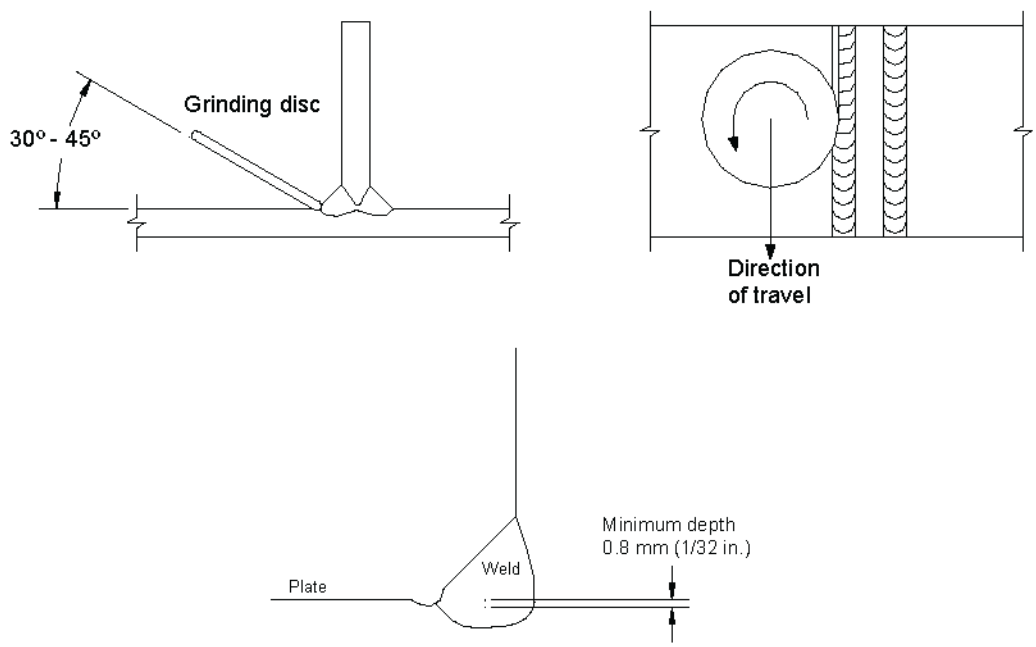


Figure 7.4. Disc grinding..

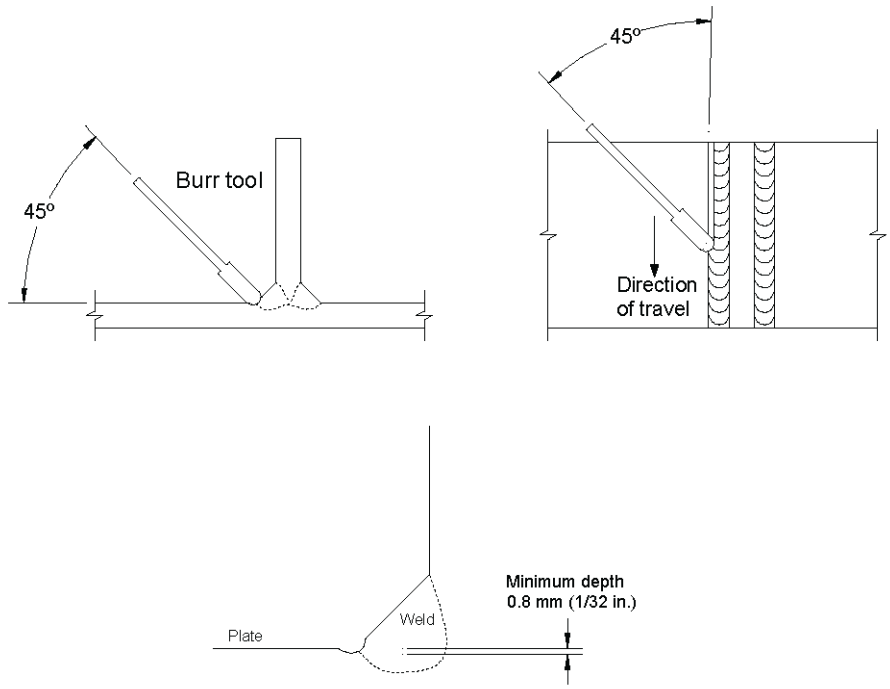


Figure 7.5. Burr Grinding.

7.4.3 Gas Tungsten Arc (GTA) or Plasma Remelting

GTA aims to reduce the stress concentrations at the weld toe and also remove slag intrusions. This process involves melting a small volume of the weld toe and base material using tungsten electrodes. In order for this process to effectively increase the weld's fatigue life, the operator needs great skill, which consequently increases the cost of this process.

7.4.4 Impact treatments

Compressive residual stresses can be induced around the weld toe using impact treatments. These compressive stresses reduce the effective tensile stress range, extending the fatigue life of welds. Since impact treatments enhance the weld profile and residual stresses, the process can only effect stresses transverse to the impacted weld. Thus, impact treatments are most effective on transversely-loaded welds and have no effect on longitudinally-loaded welds. The most common types of impact treatments include air hammer peening and ultrasonic impact treatment.

Air hammer peening utilizes an air-powered hammer with a blunt tip that plastically deforms the weld toe. This is simple method that can increase the fatigue resistance by at least one detail category. For instance a category C

detail could be improved to category B detail. Air hammer peening reduces the number of slag intrusions but at the same time creates lap-type defects. These lap-type defects can be reduced by light grinding following the peening. (Hausammann et al. 1983)

Ultrasonic impact treatment (UIT) has proven to be more effective than hammer peening by using low-amplitude and high frequency displacements. However, UIT can be more costly as it is still a proprietary method. (Tryfyakov et al. 1993; Roy et al. 2003)

7.4.5 Hole Drilling

Hole drilling is the most widely used method for the repair of fatigue cracks. The process involves drilling a hole at the tip of the crack (propagating end). The larger the hole, the more effective it is at arresting the fatigue crack from propagating, as long as the hole is not detrimental to the stiffness of the member. Holes should not be made smaller than 1 in. in diameter as holes smaller than this tend to be ineffective. Holes should be sealed against corrosion and plugged to hide the hole from the public. Equations have been developed in order to simplify the process of hole size selection for in-plane fatigue as seen below. (Fisher et al. 1980)

$$\frac{\Delta K}{\sqrt{\rho}} \leq 10.5\sqrt{\sigma_y} \text{ (for } \sigma_y \text{ in MPa)}$$
$$\frac{\Delta K}{\sqrt{\rho}} \leq 4\sqrt{\sigma_y} \text{ (for } \sigma_y \text{ in ksi)} \qquad \textbf{EQ 7.2}$$
$$\Delta K = S_r\sqrt{\pi a}$$

Where:

ΔK = stress intensity factor
ρ = radius of the hole
σ_y = yield stress of material
S_r = nominal stress range at crack tip

At the tip of cracks, singularity exists and stresses approach infinity. Drilling eliminates the high stress concentration and prevents the further crack growths.

7.4.6 Vee-and-Weld

This method is best for long, through-thickness cracks. The process includes removing the material along the crack in the shape of a V, and then filling the grove with weld material. The groove can be made using several methods, the preferred being air arc gouging. Grinding can also be done, but it tends to smear the crack path making

it harder to detect the crack and follow its path. Other methods need to be used in addition to vee-and-weld repairs in order to reduce the stress ranges at the location of the repair. This is necessary because the vee-and-weld repairs only have a fatigue life that is equal to that of the original uncracked weld. (Dexter et al. 2003)

7.4.7 Adding Doubler/Splice Plates

Doubler plates can be added at crack locations to increase the cross-sectional area and therefore reduce the stress ranges experienced by that section. (See Figure 7.6.) Doubler plates are designed to restore the section properties of the cracked section to the uncracked state using design processes identical to those of field splice connections.

Figure 7.6. Photo of bolted doubler plate repair

7.4.8 Posttensioning

Posttensioning methods that are applied to cracked sections can prolong the fatigue life of the structure. Post-tensioning induces forces on the cracked section that put the effective stress ranges into compression, keeping the crack closed and unable to propagate. It is recommended to drill a hole at the crack tip in addition to one of the various types of posttensioning that is to be used.

7.4.9 Detail Modification

Detail modification is used when it is necessary to lower the effective stress range in order to repair the cracked section. This can be achieved in a number of ways, among them increasing the cross-sectional area, changing connection geometry, or eliminating sharp corners from details.

7.5 FATIGUE DUE TO SECONDARY STRESSES

Secondary stresses can arise when a structure is designed as a series of individual components and the designer does not account for the global system behavior. These stresses can cause unexpected fatigue cracking. This section discusses these stresses and the methods used to repair the fatigue cracks that form under secondary stresses.

7.5.1 Out-of-Plane Distortion

Differential displacements between girders and lateral bracing elements introduce fatigue to the web-gap regions of the girders. This phenomenon causes highly localized bending of the web-gap, as shown in Figure 7.7, causing fatigue cracks. In order to properly repair the fatigue cracks the out-of-plane bending needs to be reduced or eliminated. It is important to note that web-gap fatigue retrofits need to maintain symmetry.

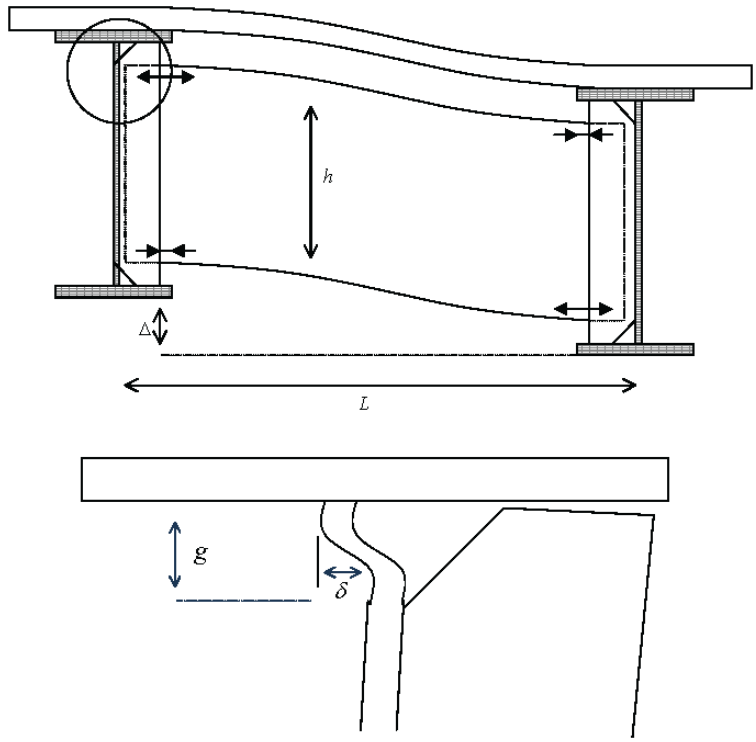


Figure 7.7. Web-gap fatigue mechanism from displacement continuity. Top: Differential girder displacements cause a force couple to develop within the diaphragm. Bottom: Zoomed view of web-gap deformation girder displacement.

7.5.1.1 Repair Methods Specific to Out-of-Plane Distortion

7.5.1.1.1 Hole Drilling

This method can be effective at reducing the crack growth but not eliminating the cause of the fatigue. See the above section on hole drilling for additional details; however, note that the hole sizing equations were developed for in-plane fatigue and may not have the same effect on out-of-plane distortion induced fatigue.

7.5.1.1.2 Diaphragm or Crossframe Removal

Diaphragms and crossframes transfer the secondary forces between girders when differential displacement of the girders occurs. Removing these members eliminates the causes of the fatigue-induced cracks in the web-gaps. However, several issues of concern have arisen from the removal of such bridge elements. It has been shown that the removal of the lateral bracing elements can be detrimental to the structure when not properly removed. In negative moment regions, the lateral bracing keeps the compression flange from buckling. Also, if the diaphragms and crossframes were removed, as the bridge deck needs to be replaced, no lateral bracing would exist to keep the girders stable when the deck is removed. Some studies show that crossframes are effective to some extent in distributing the

applied traffic loads (Brakke 2001; Flemming 2001). However, extensive investigation of crossframes indicated that it is the stiffness of the deck that is mainly responsible for distribution of truck loads between girders, and that crossframes are not contributing to load distribution. (Azizinamini et al. 1995a; Azizinamini et al. 1995b)

7.5.1.1.3 Diaphragm Repositioning

It has been shown that lowering diaphragms closer to the bottom flange of the girders (in negative moment regions) and reducing the number of connecting bolts can reduce the effective stress range. This was seen in a case study of I-35W Bridge over the Mississippi River in Minneapolis, Minnesota. (Bergson 1998) See Figure 7.8.

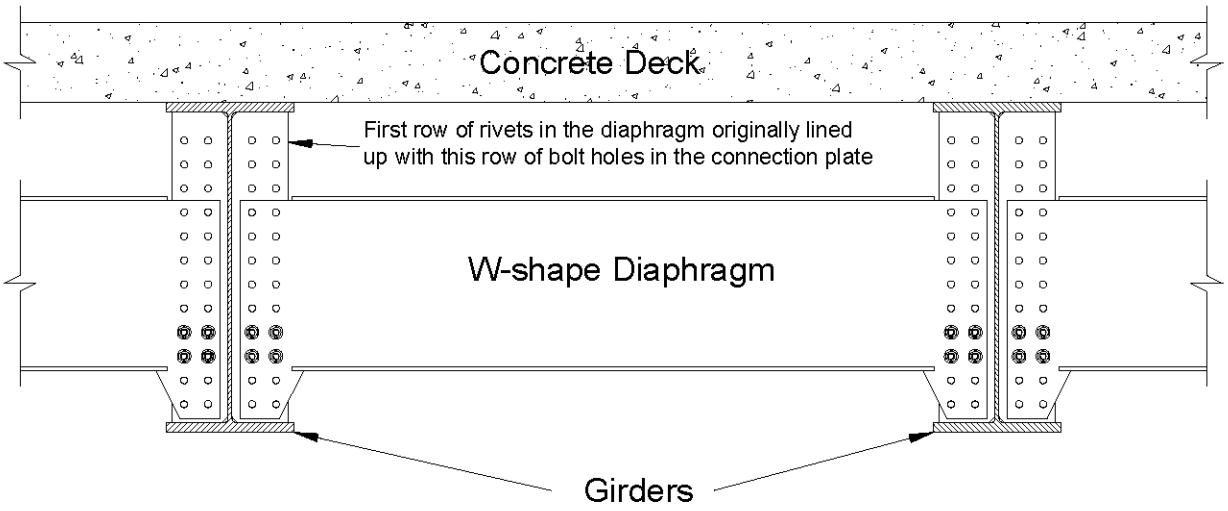


Figure 7.8. Schematic of diaphragm repositioning retrofit specified on Minnesota Bridge.

7.5.1.1.4 Bolt Loosening

Loosening the connection bolts can reduce the effect of the out-of-plane displacements. The holes are specified to be larger than the size of the bolts, and this extra space negates the effects of small differential displacements. However, the effectiveness of loosening the bolts is limited to the extra space provided by the oversized holes and whether or not the holes are in bearing due to misaligned connection plates. Additional measures are needed to ensure that the loosened nut does not fall off of the bolt due to vibrations of the structure. (Wipf et al. 1998)

7.5.1.1.5 Web-Gap Stiffening

Permanently attaching the connection plate to the girder flange can reduce or eliminate the effects of out-of-plane displacements. Several types of methods exist for the attachment of the connection plate to the girder flange: all welded, all bolted, welded and bolted, adhesives, and nails.

7.5.1.1.6 Welded Attachment

The all-welded retrofit for connection plates can be difficult to implement. For instance, the welded connection itself can cause fatigue cracks. (Keating et al. 1996) In addition, it can be very difficult to properly weld high strength steels as well as flanges that are embedded in concrete. Although AASHTO now requires transverse welds or bolted connections on both the girder flanges for the positive attachment of the connection plate, the all-welded attachment retrofit has rarely been specified for reasons previously mentioned. (AASHTO 2012) Figure 7.9 shows a fillet welded connection plate to girder detail.

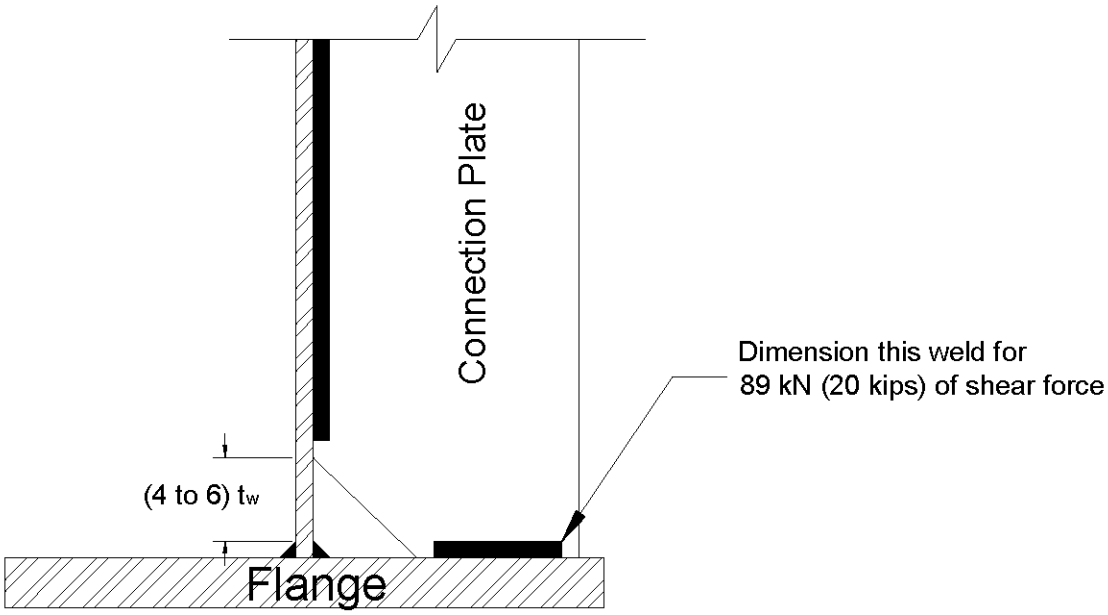


Figure 7.9. Connection plate-to-girder fillet weld detailing. (Keating 2001)

7.5.1.1.7 Bolted Connections

The connection plate can be bolted to the girder flange using angles or sections. It is important to properly size the angles and tee sections and number the amount of bolts needed in order to provide for the proper stiffness of the section. Bolted tee sections are preferred over double angles because they provide greater stiffness. (Fisher et al. 1990) See Figure 7.10.

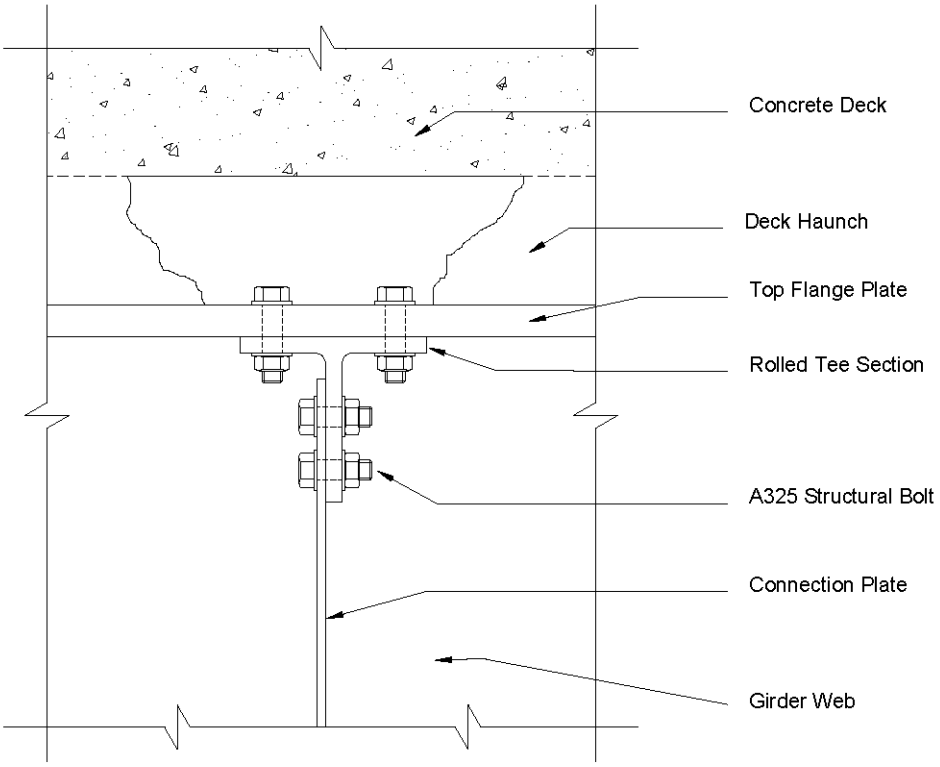


Figure 7.10. Schematic of concrete deck haunch removal to allow for bolt installation. (Keating 2001)

7.5.1.1.8 Hybrid Connections

These types of connections utilize both welded and bolted connections and may be more beneficial in areas with certain clearance issues.

7.5.1.1.8a Adhesives

Adhesives become attractive when short-term positive attachments are needed. They can be less expensive than the bolted or welded options because they do not require the removal of any concrete. (Hu 2005) See Figure 7.11.

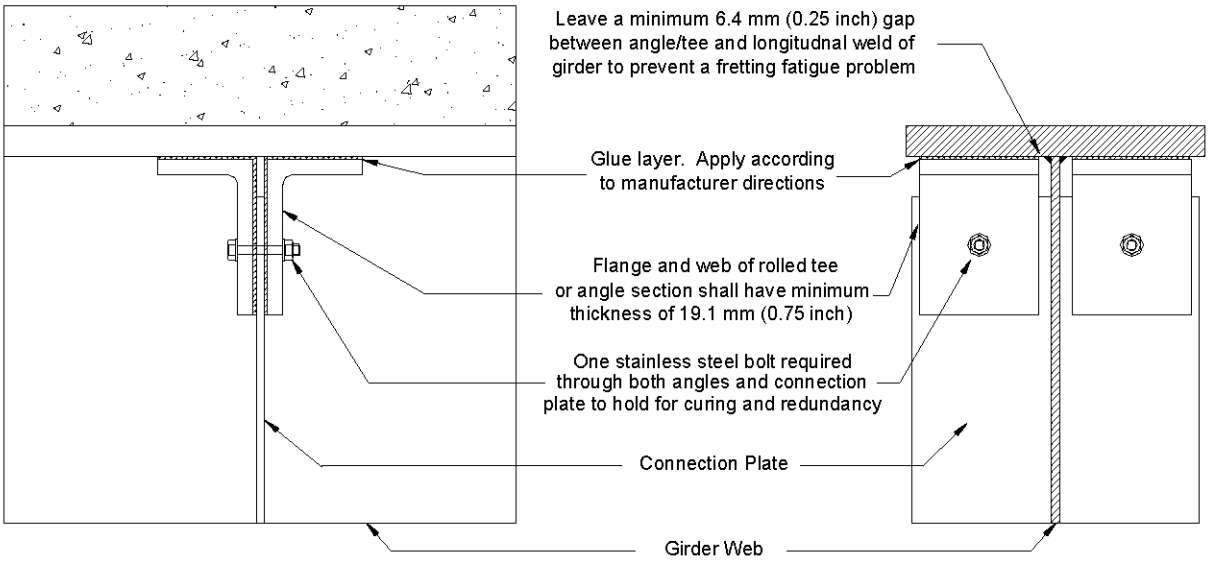


Figure 7.11. Work plan for stiffening retrofit of web-gaps with adhesives. (Hu 2005)

7.5.1.1.8b Nails

Powder-actuated fasteners are the newest and perhaps the best alternative for stiffening web-gaps. These fasteners are made of high strength materials and are propelled into the girder flanges using explosive discharges. Concern has been raised on the possible fatigue issues of these powder-actuated nails, but research has shown that the fasteners perform adequately with little detriment to the members. Due to dimensional issues, nails are only used for the flange connection while bolts are used for attachments to the connection plate as shown in Figure 7.12. When determining the number of nails to use, it is imperative that the manufacturer's recommended nail shear resistance be used. (Niessner and Seeger 1999)

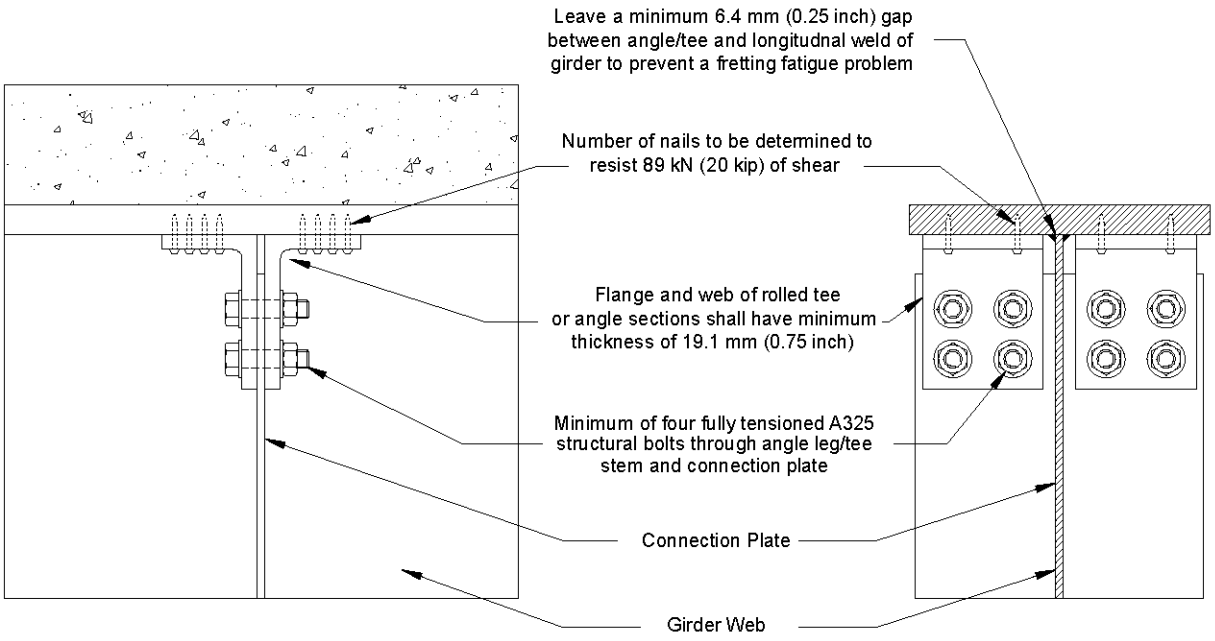


Figure 7.12. Work plan for web-gap retrofit using nails.

7.5.1.1.9 Web-Gap Softening

Web-gap softening entails the removal of portions of material to make the web-gap more flexible. See Figure 7.13.

A portion of the connection plate can be removed in order to increase the size of the web-gap and therefore reduce the stresses due to out-of-plane distortion. After flame cutting the connection plate it is important to grind the portion of the web smooth and flush where the connection plate was previously attached.

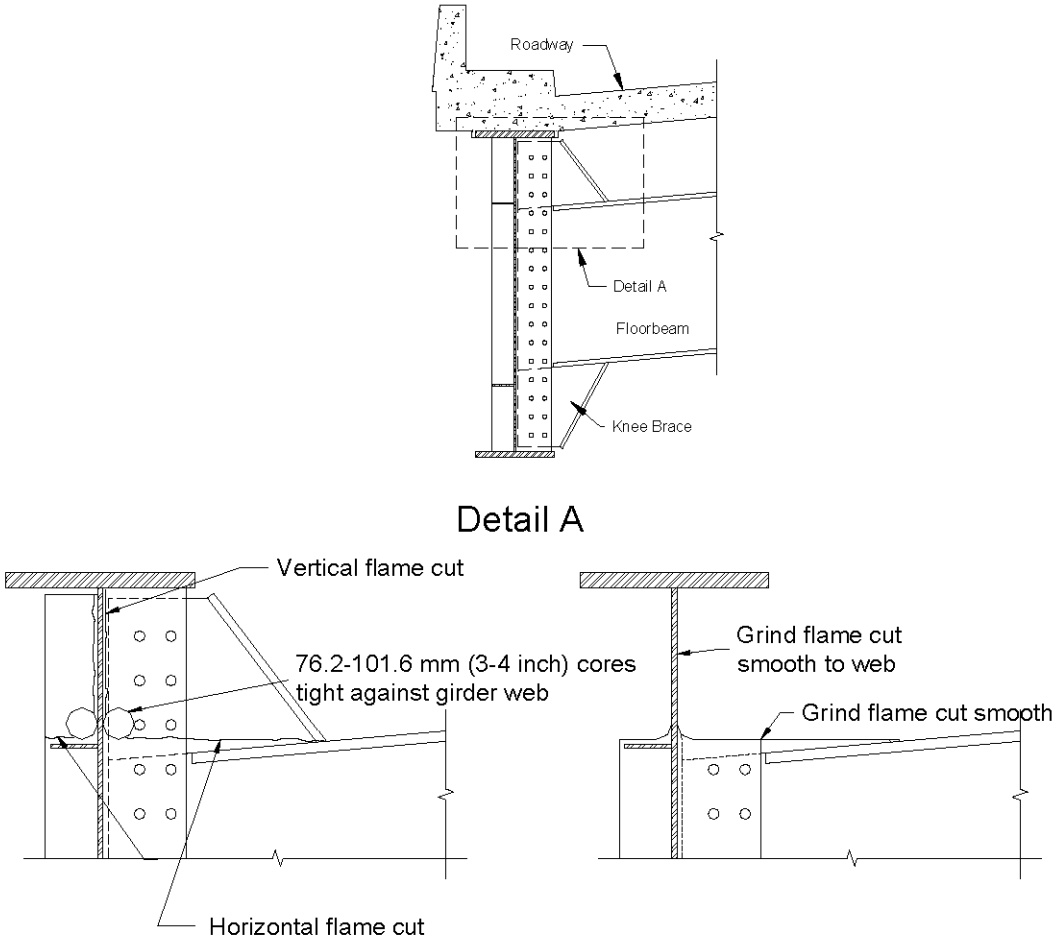


Figure 7.13. Work plan for web-gap softening used on Poplar Street Bridges in East St. (Koob et al. 1998)

A simpler and faster approach to softening the web-gap would be drilling large holes in the web of the girder close to the web-gap. See Figure 7.14. This process is much like the smaller hole retrofits discussed earlier; however, the larger diameter holes are able to capture several cracks as opposed to the "Swiss cheese" method of numerous smaller holes.

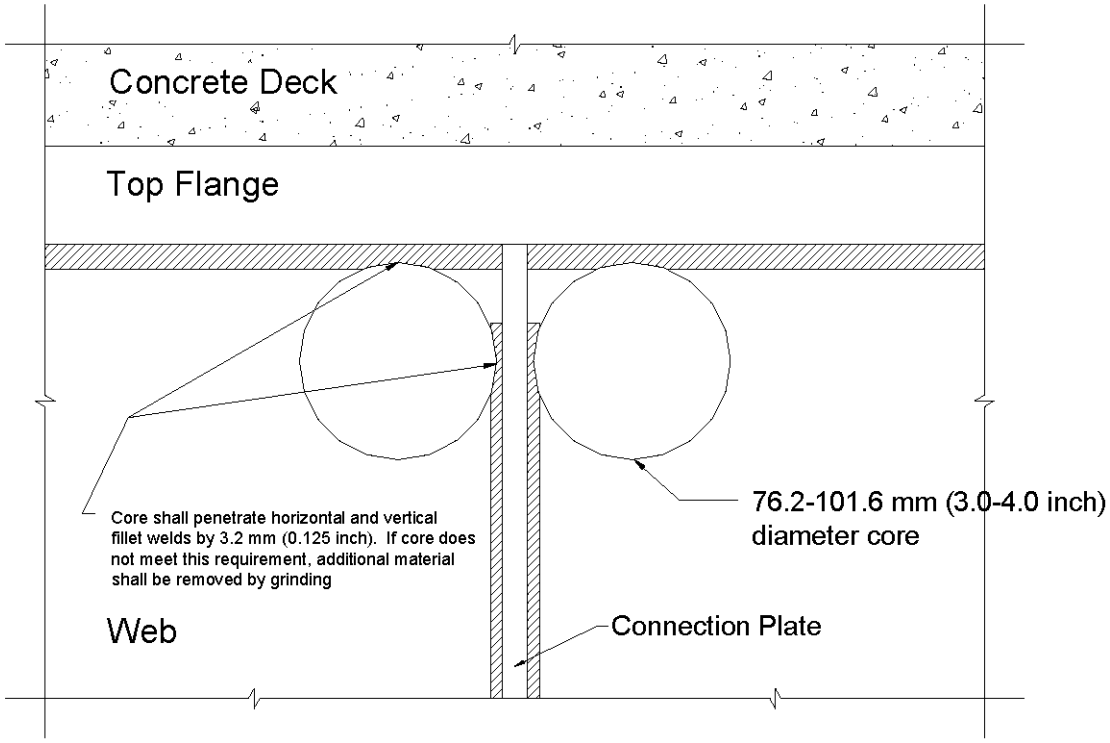
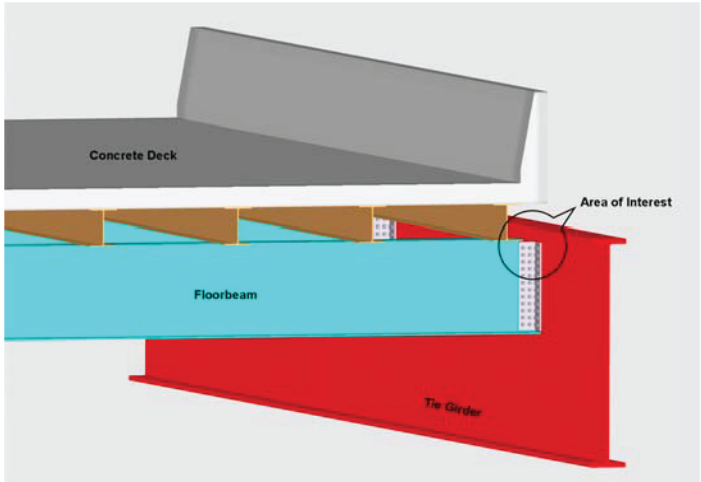


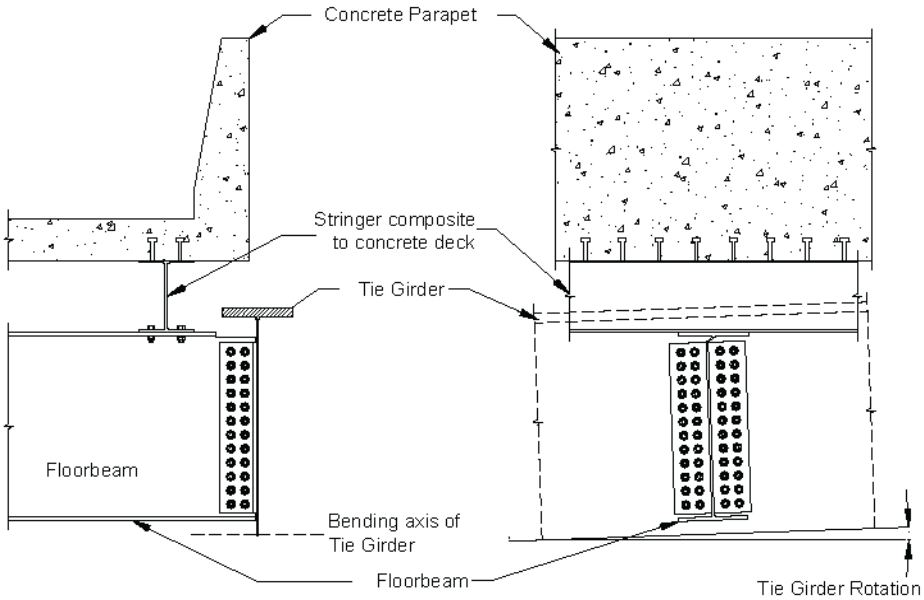
Figure 7.14. Schematic of typical large diameter hole retrofit. (Koob and McGormely 1998)

7.5.2 Tie Girder/Floor Beam Connection

Tied arch bridges exhibit a specific type of web-gap fatigue in the connections between the floor beams and the tie girders. This fatigue arises from the displacement incompatibility between the floor beams that are composite with the bridge deck and the non-composite tie girder. The mode of deformation is illustrated in Figure 7.15. Several retrofits have been implemented in the field and subsequently studied. These studies are presented in the report from which this section has been adapted. (Dexter and Ocel 2008)



(a)



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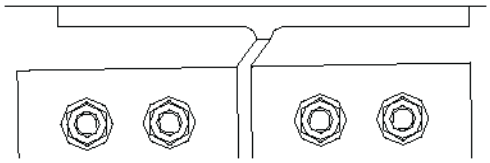


(c)

Figure 7.15. Schematic of tie girder to floor beam cracking driving force. (a) A generic deck system of an arch bridge using a tie girder. (b) Close-up view of members showing deformation caused by tie girder rotation. (c) Close-up view of web-gap deformation of floor beam web.

7.5.3 Cantilever Bracket Cracking

Floor beam cantilevered brackets are used on bridges with large deck overhangs and can be susceptible to secondary stress fatigue. Several retrofit options exist for reducing the displacement incompatibility of the girder and floor beams. These retrofits deal mainly with the modification of the tie plates that span over the girders. The main idea is to either remove any positive attachments between the girders and tie plates or add spacer plates to create a gap between the elements. Figures 7.16 through 7.18 show the deformation modes and possible retrofit options.

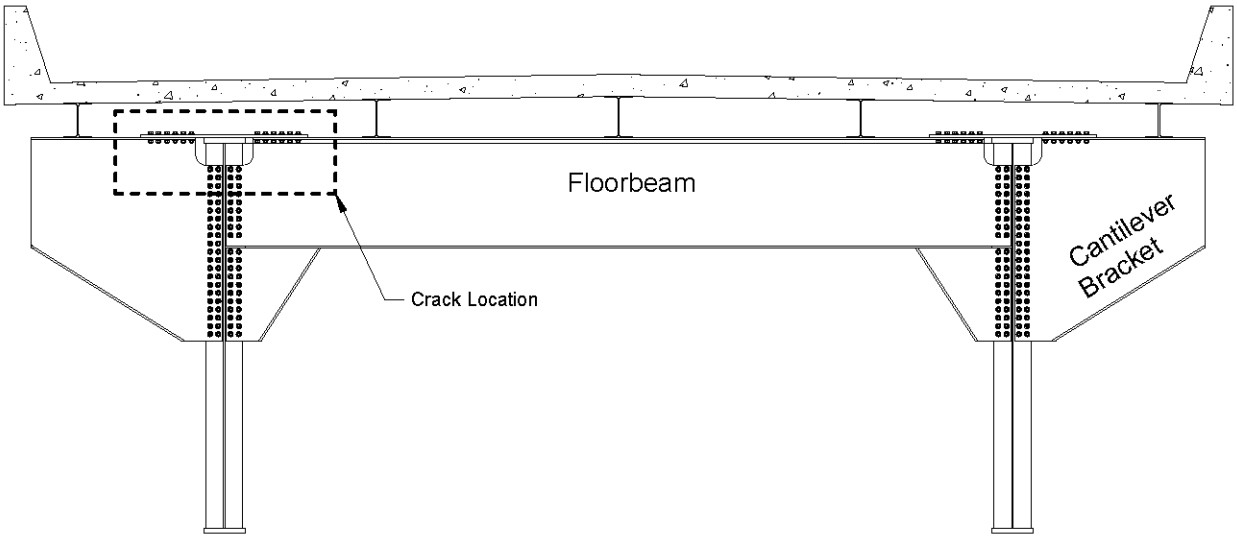


Figure 7.16. Typical cross-section of a two-girder bridge with cantilever bracket outriggers.

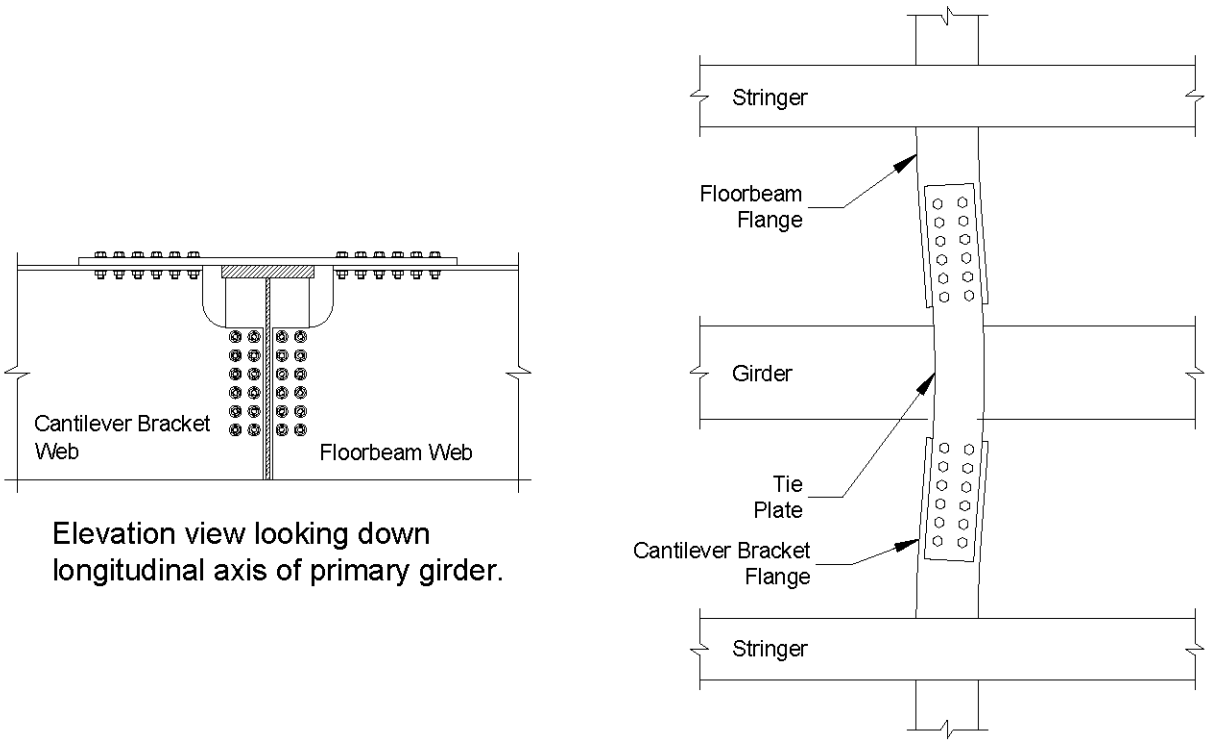


Figure 7.17. Zoomed-in view of tie plate detail (left) and deformation mode that causes cracking (right).

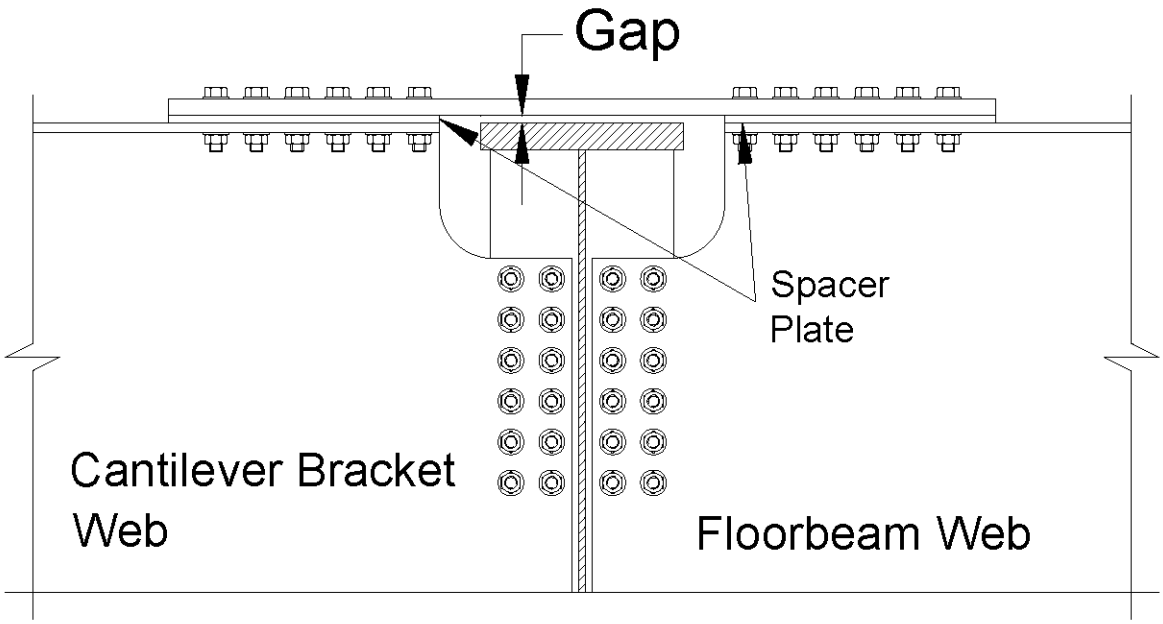


Figure 7.18. Retrofit of tie plate cracking through addition of spacer plates.

7.6 RETROFIT VALIDATION OF SECONDARY STRESS FATIGUE

Due to the unknown nature of retrofitting web-gap fatigue, it is necessary to validate particular retrofits before retrofitting an entire bridge. The simplest plan to validate a particular retrofit is to first instrument an uncracked detail, then perform the necessary retrofit and validate whether or not the retrofit adequately lowered the stress ranges and out-of-plane displacements. Typical instrumentation includes the use of strain gauges and/or displacement measurement devices.

Strain gauges are very common and effective instruments used to determine the effective stress ranges of bridge elements. Strain gauges can be either spot welded or glued to the element of interest. Figure 7.19 shows preferable strain gauge layouts for retrofit validation.

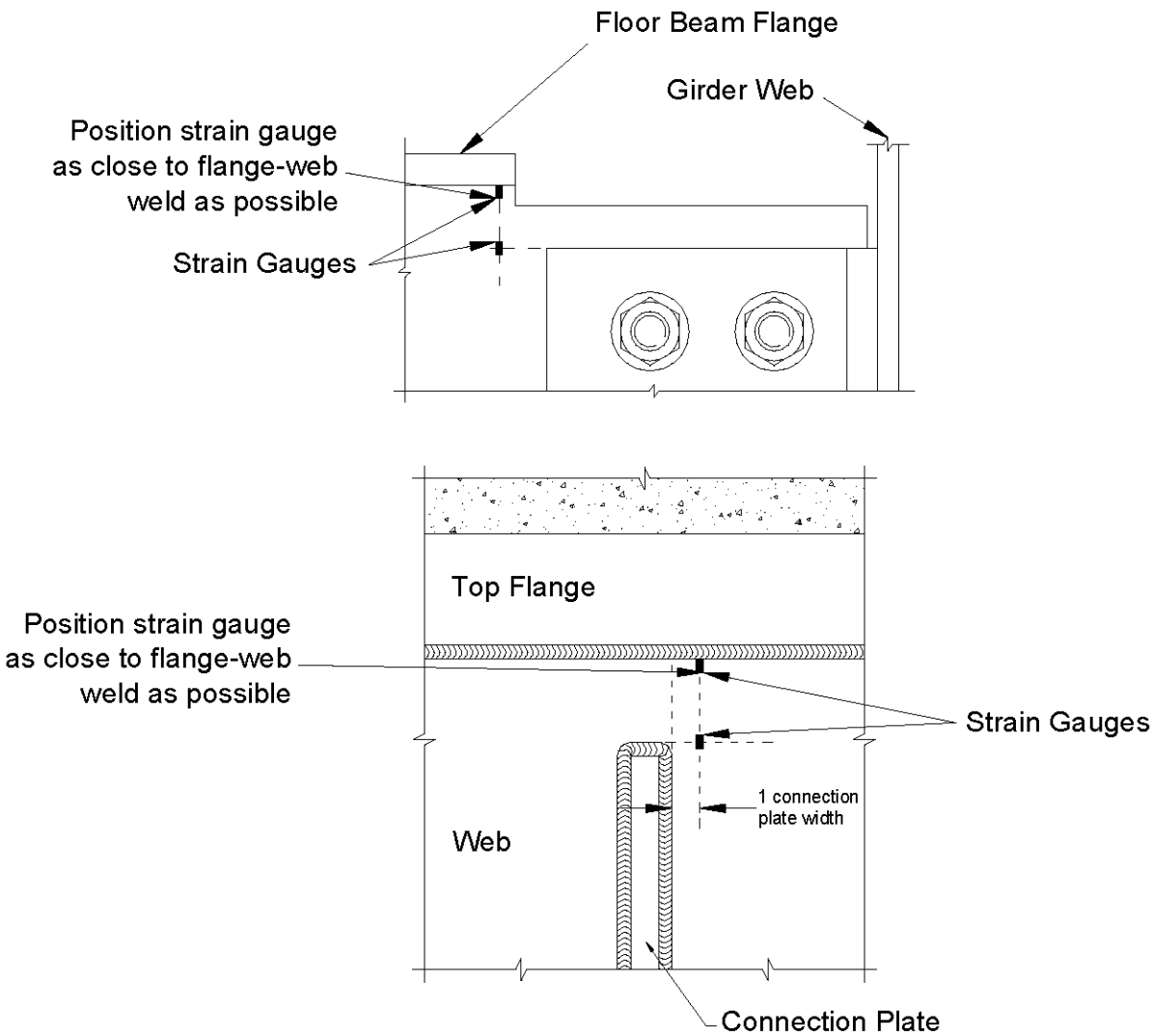


Figure 7.19. Recommended strain gauge placement for retrofit validation. Bottom: Strain gauge placement for out-

of-plane distortion. Top: Strain gauge placement for floor beam/tie girder connection or beam cope cracking.

Measuring the displacements has become the preferable method for validating retrofits because displacement measurements are quicker and more cost-effective than strain measurements. That displacement gauges are only usable for stiffening retrofits should be taken into account. Two common types of displacement measuring devices are linear variable differential transformers (LVDTs) and dial gauges. Figure 7.20 shows a schematic for the placement of the devices to measure the web-gap displacement. It should be noted that maximum distortion-induced fatigue strains/stresses do not always correlate with the largest values of differential deflection, and an instrumentation plan based primarily on displacements may not capture the whole picture.

In the final analysis, however, the choice of instrumentation to validate the retrofit type is context sensitive. The designer must select an instrumentation type that best matches the retrofit type.

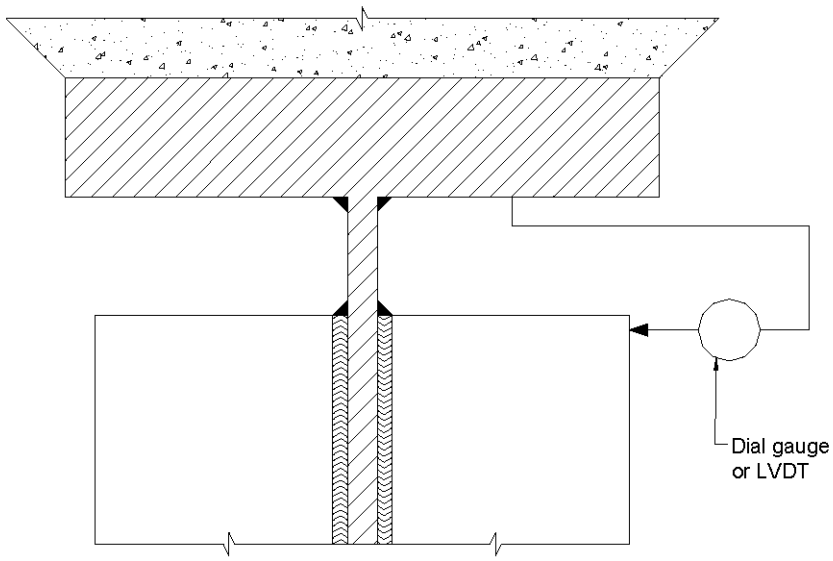


Figure 7.20. Recommended web-gap displacement instrumentation.

7.7 LOAD CONTROLLED FATIGUE CRACK REPAIR

7.7.1 Coverplates

Several methods have been explored in the retrofitting of coverplates including grinding, air hammer peening, GTA, and bolted splice plates. Grinding has proven ineffective and therefore is not recommended. Bolted splice plates are an effective option for girders with severed flanges and in instances in which the aforementioned methods are undesirable. See Figure 7.21.

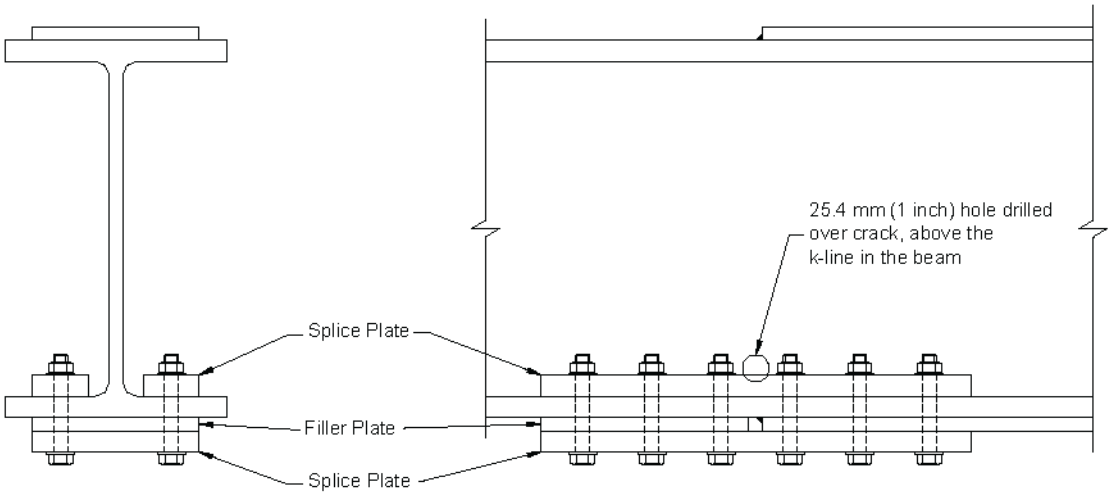


Figure 7.21. Detailing of splice plate retrofit for cracked coverplate details.

7.7.2 Eyebars and Hangers

Eyebars are long slender bars or rods with forged eyes at the ends and are commonly used as tension members in truss bridges. Hangers are vertically-oriented tension members supporting load. Both eyebars and hangers are often the sole component supporting the particular tensile load and are therefore usually classified as fracture critical members and inherent flaws can lead to fatigue cracks.

Assessing the integrity of eyebars, especially in old truss bridges, is very difficult. An example of eyebar with extensive corrosion is shown in Figure 7.22.

Figure 7.22. Corrosion of eyebar connection.

In the case of truss bridges, it is recommended to add, rather than replace truss members. Removing even single truss member or connection could easily lead to bridge failure. In retrofitting many existing old truss bridges, extensive corrosion of eyebars in tension could be addressed by adding additional tension members, while keeping the existing ones in place. Figure 7.23 shows a possible retrofit alternative for eyebars in tension in existing truss bridges. (Azizinamini 2002)

Figure 7.23. Retrofit option for eyebar connection.

7.7.3 Temporary Tack Welds

Tack welds used to temporarily hold members in place during construction can be sources of concern for fatigue cracks. Fatigue cracks tend to form at the ends of the tack welds and then propagate into the base metal. The most susceptible types of tack welds are longitudinal, and those located at the end of tack-welded members. Tack welds can be removed using grinding methods.

7.7.4 Connection Angles

Connection angles used to connect diaphragms to girder webs are susceptible to fatigue cracking due to differential girder displacements. See Figure 7.24. The thickness of such angles needs to be reduced in order to reduce the flexural rigidity of the elements and negate the causes of the differential girder displacements. The following equation helps determine the proper angle thickness to prevent cracking. (Dexter and Fisher 1999)

$$t \leq 12\left(\frac{g^2}{L}\right) \qquad \textbf{EQ 7.3}$$

Where:

t = angle thickness
g = bolt gauge
L = distance between girders

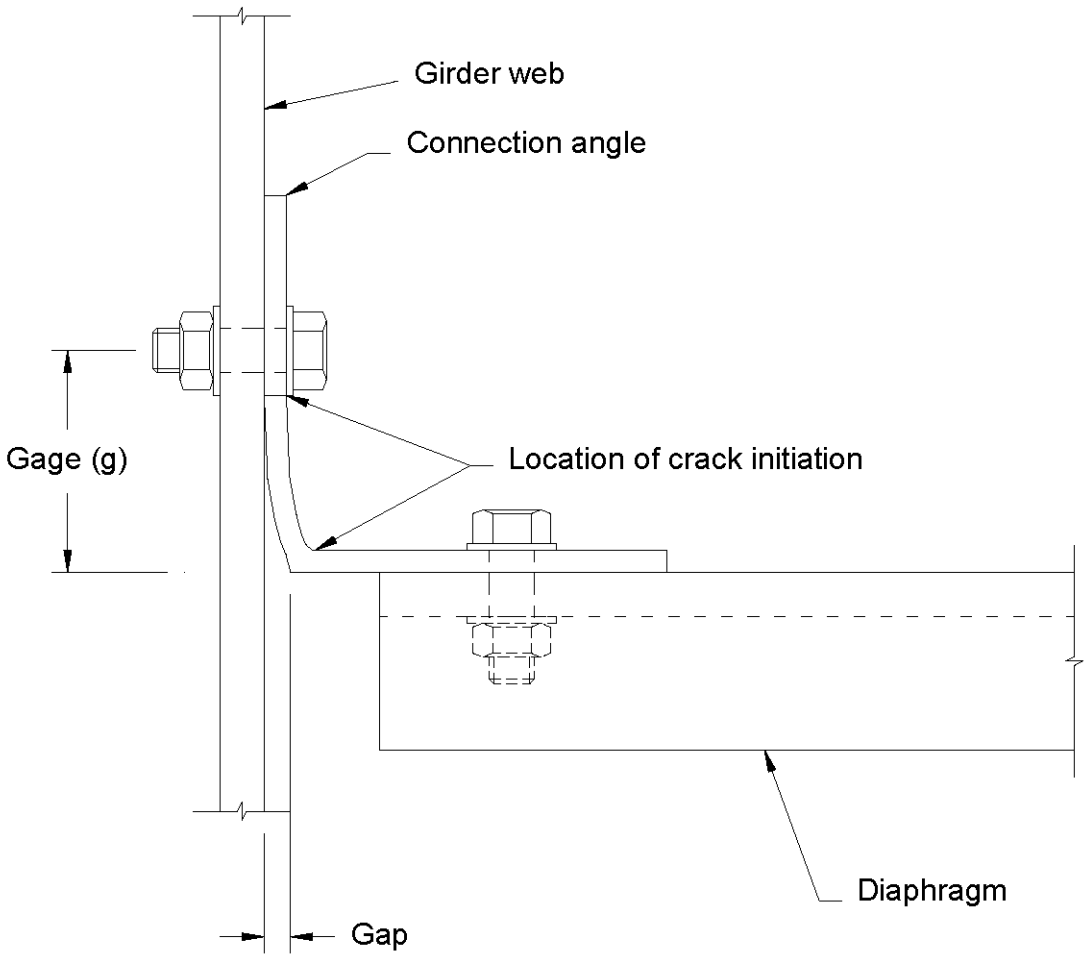


Figure 7.24. Deformation mode of connection angles due to displacement compatibility.

7.7.5 Web Gusset Plates

Another source of fatigue cracks are web gusset plates, which can experience fatigue damage due to weld root defects and locations of intersecting welds. Intersecting welds can be retrofitted by coring holes at the intersections; this will not only remove the intersecting welds, but also reduce the web constraint. See Figures 7.25 and 7.26.

Fatigue cracks have also been found to initiate from the ends of the gusset plates and propagate into the girder webs. This issue can be solved using impact treatments or grinding of the weld termination. See Figure 7.27.

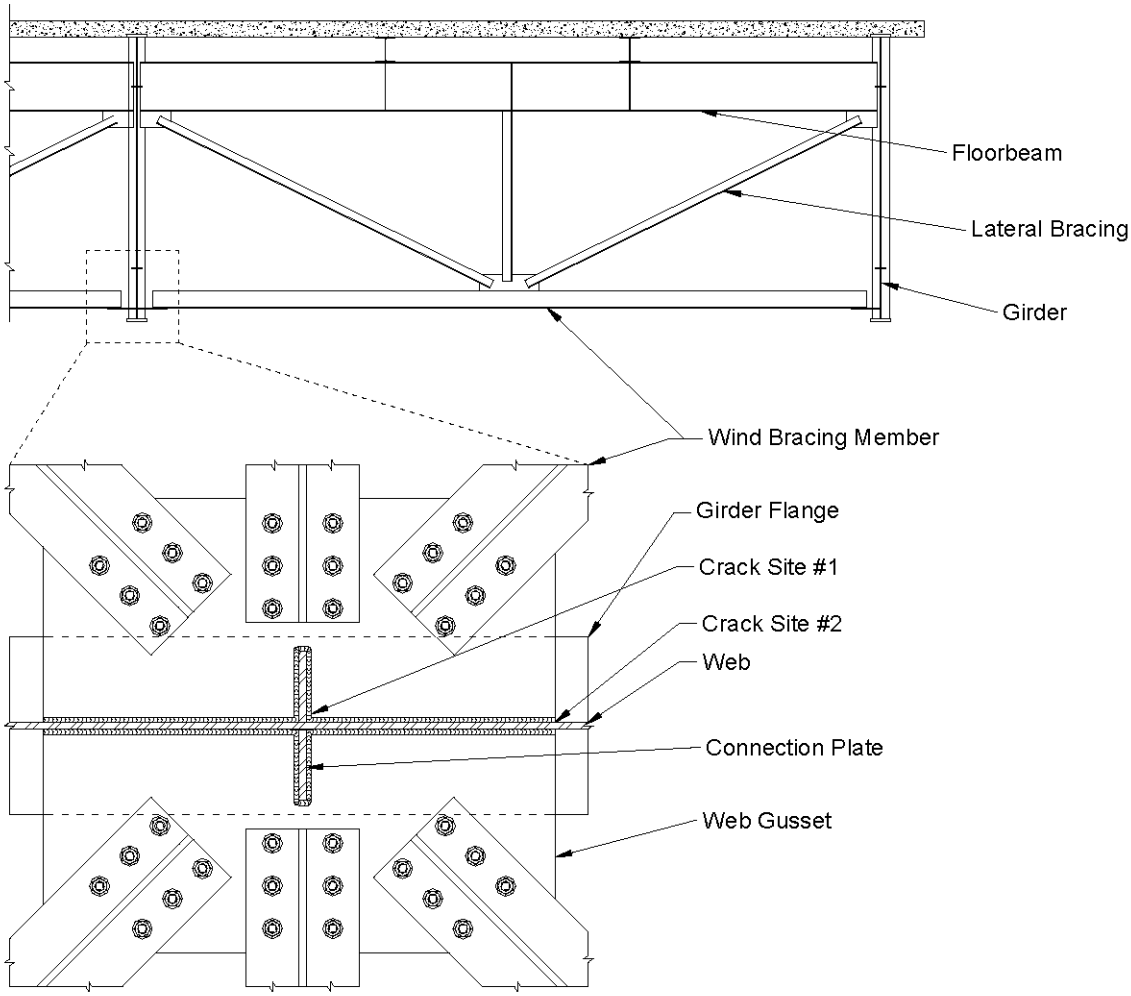


Figure 7.25. Typical cross-section of a deep girder bridge (top) and plan view of the web gusset detail (bottom).

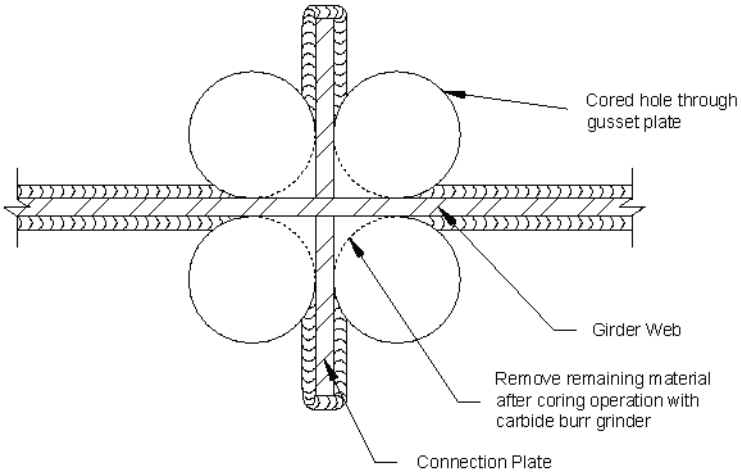


Figure 7.26. Retrofit detail for gusset plates with intersecting welds. (Crack Site #1)

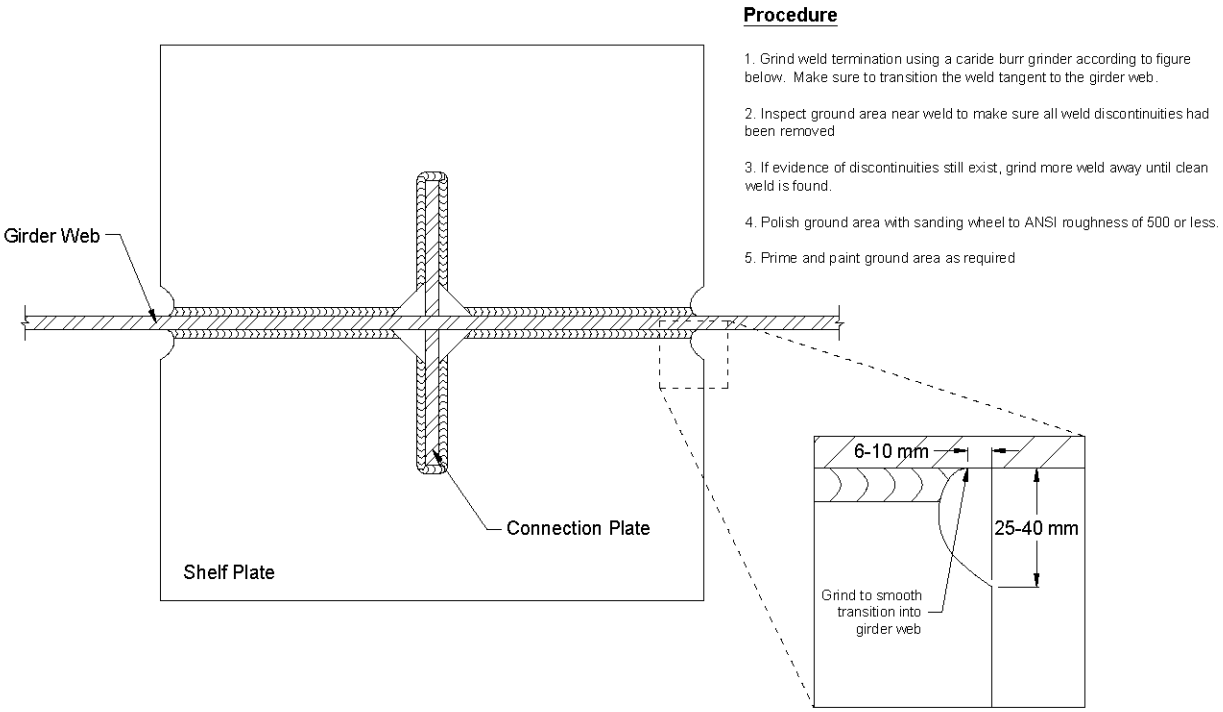


Figure 7.27. Detailing of crack site #2 retrofit.

7.7.6 Longitudinal Stiffeners

Fatigue cracks can form at the butt welds of longitudinal stiffeners, primarily due to poor workmanship and inherent defects of the welds. Drilling a large diameter hole into the longitudinal stiffener located next to the girder has proven to keep the crack from developing into the girder web. See Figure 7.28.

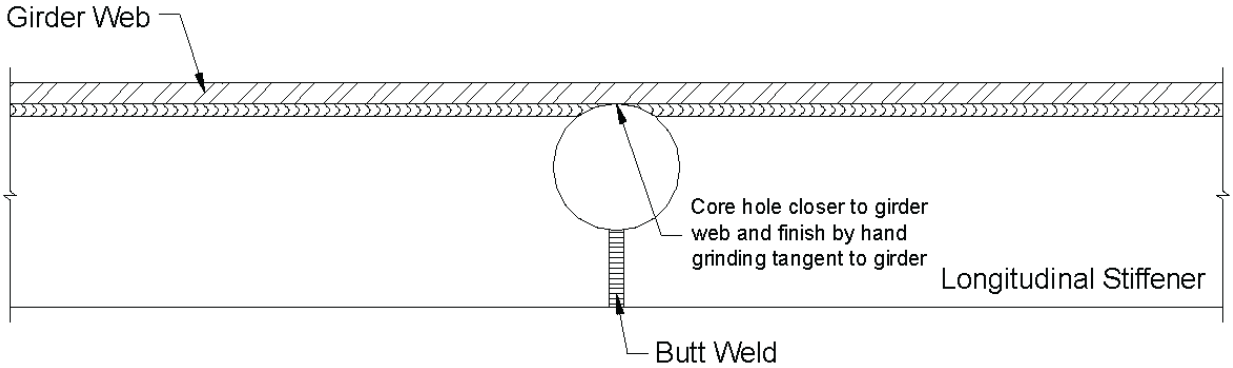


Figure 7.28. Work plan of longitudinal butt weld retrofit.

7.7.7 Coped Beam Ends

Fatigue cracks have also been observed in coped beam ends. These cracks can be attributed to stresses from flame cut copes not ground smooth, or the presence of bending moments at these copes which were designed as simply supported. Square cut copes have the least fatigue resistance. Holes can be drilled as a retrofit option or the cope can be cut back with a radius.

CHAPTER 8

JOINTLESS BRIDGES

8.1 INTRODUCTION

A jointless bridge has a continuous deck with no expansion joints over the superstructure, abutments, and piers, and is also commonly referred to as an integral bridge. In this type of bridge structure, all movement due to thermal, creep, and shrinkage strain is accommodated either within the system itself or at the ends of the approach slabs where the slabs abut the roadway pavement. Because there are no joints, ride quality is improved and maintenance can be greatly reduced.

Leaking deck joints have been a major cause of bridge deterioration and reduced service life, especially where roadway drainage carrying deicing chemicals can spill onto bridge elements below. Elimination of bridge deck expansion joints is therefore an important consideration in bridge system selection to provide long-term service life, as discussed in Chapter 2.

This chapter provides a summary of design, construction and maintenance provisions related to the use of jointless bridges.

8.2 HISTORY OF JOINTLESS BRIDGES

A detailed history of jointless bridges is provided by Burke, Jr. (2009). On the basis of a nationwide mail survey of state and province transportation departments, it appears that the Ohio highway department was one of the first agencies to initiate the routine use of continuous construction for multi-span bridges (Burke, Jr. 2009). However, these bridges had expansion joints at abutments. Ohio began this practice in 1930.

In conjunction with the development and adoption of continuous construction for all moderate length highway bridges, Ohio Department of Transportation (DOT) was also the first to routinely eliminate deck joints at abutments. The first integral bridge in the United States was the Teens Run Bridge. It was built in 1938 near Eureka in Gallia County, Ohio, and consisted of five continuous reinforced concrete slab spans supported by capped pile piers and abutments. Since that time, construction of integral bridges has spread throughout the United States and abroad. The

United Kingdom recently adopted them for routine applications, Japan completed its first two in 1996, and South Korea completed its first such bridge in 2002.

The Tennessee DOT now is leading the way in construction of continuous bridges. For example the Long Island bridge of Kingsport was constructed in 1980 using 29 continuous spans without a single intermediate deck expansion joint.

Continuous integral bridges with steel main members have performed successfully for years in the 300 ft range in such states as North Dakota, South Dakota, and Tennessee. Continuous integral bridges with concrete main members 500 to 800 ft long have been constructed in Kansas, California, Colorado, and Tennessee.

As of 1987, 11 states reported building continuous integral bridges in the 300 ft range. Missouri and Tennessee reported even longer lengths. Missouri reported steel and concrete bridges in length of 500 and 600 ft respectively. Tennessee reported lengths of 400 and 800 ft for similar bridges. Sixty percent of those departments responding to the 1987 survey were using integral construction for continuous bridges.

More recently, the Tennessee DOT completed the Happy Hollow Creek Bridge, a seven-span prestressed concrete curved integral bridge with a total length of over 1175 ft. In that bridge, tall flexible twin circular column piers support the superstructure. A single row of steel H-piles is used to support each abutment. Although to some engineers the length of this structure may seem extreme, it is well within the Tennessee DOT's bridge design policy statement regarding the length of integral bridges.

Seamless bridges are another type of jointless bridges introduced by *SHRP 2 R19A* for practice in the United States, which allows elimination of expansion joints even at the end of the approach slab. The seamless bridge system was first introduced by Russell Bridge et al. (2000) in Australia for use with continuously reinforced concrete pavement for the approach roadways. Most commonly used pavements in the United States, however, are jointed plain concrete (JPCP) and flexible pavements, which require a modified application. Seamless bridges do not have any joints, even at the ends of an approach slab (hence seamless). Instead, a pavement transition zone is used to dissipate the thermal displacements of the bridge. The transition zones can be rather lengthy relative to the bridge. The benefit however, is that movements at the end of the transition zone are very small.

8.3 TYPES OF JOINTLESS BRIDGES

Three main types of jointless bridges are described in this chapter: integral and semi-integral jointless bridges, which are commonly used in practice, and a new class of jointless bridges, referred to as seamless jointless bridges. The main characteristic of the seamless system is that expansion joints are eliminated altogether and the bridge deck is connected to the approach road pavement with no joint.

Figure 8.1 shows a rendering of a typical layout of a jointless bridge, shown with the superstructure cast in an integral abutment.

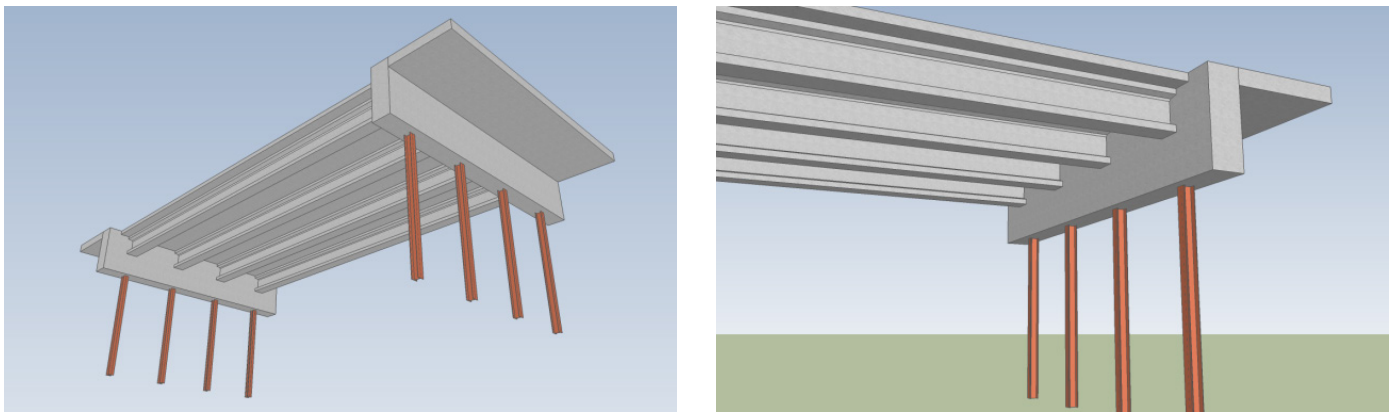


Figure 8.1. Elements of jointless integral bridges.

8.3.1 Integral Bridges

Integral bridges have superstructure constructed monolithically with the abutments, encasing the ends of the superstructure within the backwall. The main characteristics of integral bridges are their jointless construction and flexible abutment foundations. The system is structurally continuous and the abutment foundation is flexible longitudinally. The movement of the superstructure is accommodated by the foundation.

Figure 8.1 shows, schematically, the main elements of an integral bridge system, which consist of bridge deck, girders, integral cast abutments, and approach slabs. The bridge movement is accommodated at the ends of the approach slabs. Also, sleeper slabs are commonly used to provide vertical support for the ends of the approach slab where the slabs abut the roadway pavement (not shown in Figure 8.1). In addition, jointless integral bridges can be continuous multi-span structures with intermediate piers (also not shown in Figure 8.1). Various details are described in greater detail in Section 8.7, which also includes further discussion of sleeper slabs in Section 8.7.3.

8.3.2 Semi-Integral Bridges

Semi-integral bridges are defined as having an end diaphragm serving as the abutment backwall and that is cast encasing the superstructure ends. In this system, the superstructure rests on expansion bearings and the end diaphragm is not restrained longitudinally with respect to the pile cap or abutment stem. The deck may be sliding, or cast monolithically with the backwall, but does not have a joint above the abutment. The foundation is rigid longitudinally, where superstructure movement is accommodated through bearings.

The main elements of a semi-integral bridge system consist of bridge deck, girders, abutment stem and bearing seat, integral cast diaphragm backwall, approach slab, and sleeper slab. The bridge movement is accommodated at the ends of the approach slabs. Greater detail is provided in in Section 8.7.

8.3.3 Seamless Bridges

The seamless bridge system is characterized by eliminating the need for expansion joints, even at the ends of the approach slabs, while limiting the longitudinal expansion and contraction of the bridge superstructure. Imposing the limitation on longitudinal expansion and contraction of the bridge superstructure results in development of longitudinal forces that need to be resisted with appropriate design features. It should be noted that longitudinal expansion and contraction is limited, but not eliminated. This philosophy is used to reduce the level of longitudinal forces that can be developed, while making the gap between end of transition zone and start of pavement manageable (to less than about 0.25 in.). The foundation requirements are very similar to those of integral abutments. A seamless bridge system for jointed pavement types used in the United States was developed by *SHRP 2 R19A* project and described in Appendix E. Additional information on this system can be found in Ala (2011), as well as in Ala and Azizinmanini (2013a, 2013b).

Figure 8.2 shows, schematically, the main elements of the seamless bridge system developed by *SHRP 2 R19A* (Ala 2011; Ala and Azizinamini 2013a; Ala and Azizinamini 2013b). The main elements of the system consist of bridge deck, transition zone and roadway pavement. The bridge movement is accommodated within the transition zone, and the movement at the end of the transition zone is relatively small. This eliminates having expansion joints at the end of transition zone, where the roadway pavement starts. The thickness of the transition zone and approach slab near the abutment is increased to account for lack of support from soil below. The assumption is that the transition

zone near the abutment has no support and resists the imposed loads by flexure. Appendix E provides detailed information about this new system.

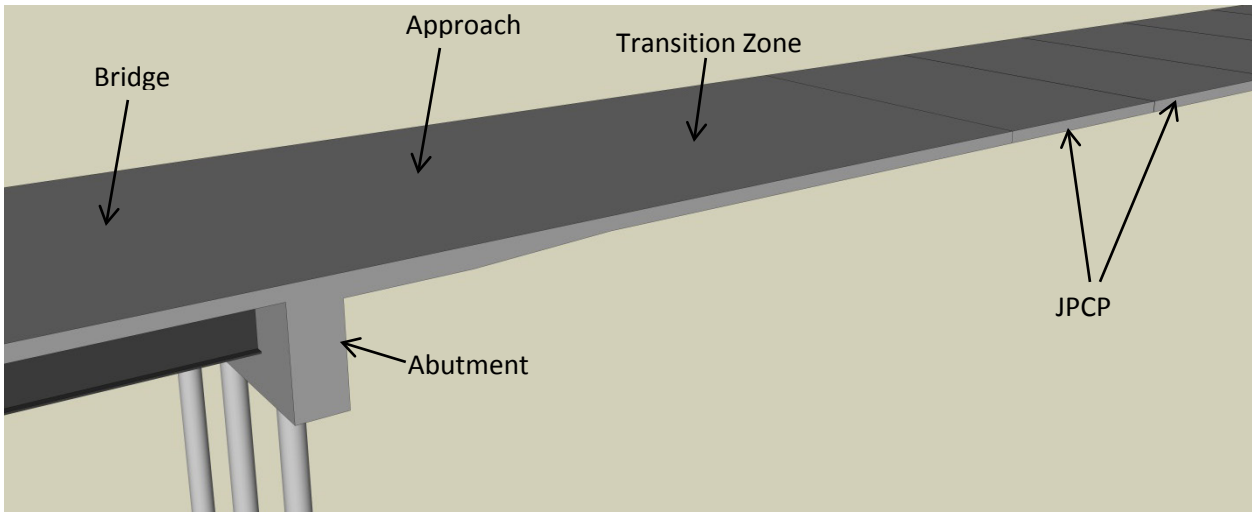


Figure 8.2. Seamless bridge system.

8.3.4 Advantages of Jointless Bridges

Henry Derthick, former engineer of structures at the Tennessee DOT, once stated, "The only good joint is no joint." In keeping with this statement, known advantages of the jointless bridge systems include:

- Lower initial cost,
- Lower maintenance cost,
- Prevention of leakage of moisture to bridge elements below deck resulting in longer service life,
- Improved ride quality,
- Easier and faster construction,
- Easier inspection,
- Simplified bridge details,
- Elimination of bearings (except for semi-integral),
- Ideally suitable for bridges with skew and curvature or located in high seismic areas, and

- Enhanced buoyancy resistance of the bridge.

Because of these advantages, many DOTs have started using jointless bridges; however, the design provisions vary significantly from one state to another.

8.3.5 Cost Effectiveness of Jointless Bridges

Jointless bridges have a significant cost savings advantage as compared to traditional bridges with expansion joints. As mentioned, cost savings are realized both during initial construction and throughout the life of the bridge with reduced maintenance. This is particularly true for bridges with integral abutments.

Most components of typical bridges with joints and jointless bridges are similar in construction and cost (deck, beams, cross frames, etc.). Thus, a comparison is made relative to the different components that distinguish each type of construction (e.g., the costs of the abutments and expansion joints). In addition, since unit pricing of each item is neither consistent from region to region nor over time, a qualitative comparison is made using relative costs.

It is recognized that different states and municipalities have different specifications and construction techniques; however, initial construction of a typical abutment with an expansion joint will most often include the following:

- Excavation;
- Two rows of piling (in some cases);
- Concrete cap, stem, diaphragm, and backwall with reinforcing (three pours);
- Elastomeric bearings, per beam;
- Precision-cast bridge seats for bearings;
- Expansion joint; and
- Porous backfill.

Similarly, typical construction of an integral bridge includes:

- Excavation;

- One row of piling;
- Concrete cap, integral backwall, and diaphragm (two pours);
- Sleeper slab with reinforcement; and
- Porous backfill.

The important differences between an integral abutment and a traditional abutment with a jointed deck include a lack of expansion joint, no bearings, reduced number of required piles, reduced number of concrete pours, and inclusion of a sleeper slab. Taking these differences into consideration, the cost savings are readily apparent. The sleeper slab adds a few cubic yards of concrete and an extra detail to the cost, but removal of the expansion joint, removal of the second row of piling (overall reduced number of piles), reduction in the required concrete and reinforcing in the stem/backwall unit, and elimination of beam bearings at the abutment greatly reduce the cost relative to adding the sleeper slab. The overall reduction in initial construction cost can be over 40% for each abutment. (The percent difference was estimated using Ohio 2010 bid planning costs for a typical 32ft wide bridge.)

The life-cycle cost of the two types of abutments differs as well. A service life of 100 years is used for the comparison, although it is acknowledged that differences in estimated service life can affect the parameters. For standard jointed bridges, common armored expansion joints typically require gland replacement on the order of eight to 12 years depending on condition severity. Additionally, the entire joint including armor will need to be replaced along with the deck at least once (based on an estimated life of a deck between 30 to 50 years). Again, this depends on the severity of the conditions. It is also expected that the bearings will need replacing at least once over the life of the bridge, the cost of which includes jacking of the bridge.

Similar to expansion joints, the sleeper slab joint seal, if used, will require replacement on the same, or at least similar, schedule. Likewise, the deck will need replacing on a similar schedule. Deck replacement of an integral bridge does require additional consideration of certain construction items, but does not require a significant increase in construction cost as compared to traditional deck replacement.

Therefore, for comparative purposes, consider that a typical bridge abutment with expansion joints will require:

- Expansion gland replacement every ~10 years,

- Deck replacement every ~50 years,
- Expansion joint replacement every ~50 years, and
- Bearing replacement every ~50 years.

A typical jointless bridge abutment will require:

- Sleeper slab joint seal, if used, replacement every ~10years; and
- Deck replacement every ~50 years.

Cost is similar for deck replacement, and gland and seal replacement; however, there is a significant increase in cost when replacing the expansion joint.

A qualitative cost comparison is presented in Figure 8.3 and Figure 8.4. Both Figures consider the difference in the initial cost at year zero, and the accumulated cost difference over the life of the bridge. The figures show the difference in costs, that is, similar costs have been removed from the equation (i.e., the cost of replacing the deck itself is removed from the equation since it is similar). Figure 8.3 shows the estimated cost comparison through the 100-year life of the structure. Figure 8.4 shows the cost comparison differentiating the initial difference in the construction costs, the lifetime maintenance costs, and the overall total difference over the life of the bridge.

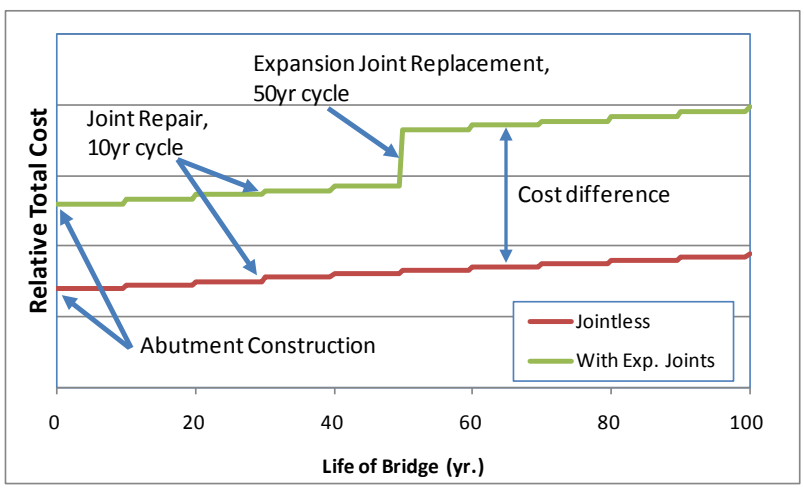


Figure 8.3. Lifetime cost analysis of jointed vs. jointless bridge over time.

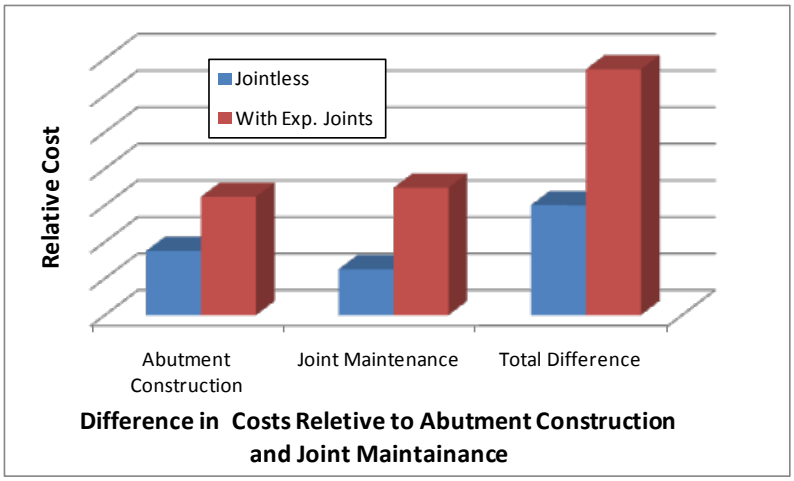


Figure 8.4. Lifetime cost differential analysis of jointed vs. jointless bridge.

8.4 FACTORS AFFECTING PERFORMANCE OF JOINTLESS BRIDGES

Factors affecting the performance of jointless bridges include curvature, skew, bearing, and connection of superstructure and substructure. Other factors that should be considered include site conditions, deterioration of piles and abutment walls.

8.4.1 Curvature

Horizontal curvature changes the internal forces of integral abutment bridges. These changes are more important for bridges with either of the following conditions in which the length and radius are measured at the centerline of the bridge:

- Length-over-radius ratio greater than 0.5, or
- Radius of curvature less than 1000 ft.

If a curved bridge that does not have either of these conditions, the response of the curved bridge can be estimated by the response of a straight bridge of the same length (Doust 2011). This estimation is not valid for the internal forces during construction.

8.4.2 Skew

In skewed integral abutment bridges, the soil passive pressure developed in response to thermal elongation can prevent the transverse movement of the bridge. Appendix B provides additional detail on this subject. However, if the friction created by contact between soil and abutment wall is insufficient, and depending on the transverse stiffness of

the abutment, either significant transverse forces or significant transverse movements could be generated (Oesterle et al. 2005).

8.4.3 Bearings

In the case of multi-span integral bridges with rigid piers, the superstructure is commonly seated on piers through use of bearing devices. In curved bridges or wide, straight bridges, fixed bearings are not recommended except at the points of zero movement. In curved integral bridges, there may be no point of zero movement throughout the bridge. Also, guided bearings are not recommended for curved and wide, straight integral bridges because the displacements do not happen in just one particular direction. In such cases, guided bearings behave like fixed bearings, creating large internal forces at the piers.

Multi-directional elastomeric or sliding bearings are the proper types of pier bearings for integral bridges. If such bearings are used, the superstructure movement is mainly controlled by the integral abutments.

8.4.4 Connection between Superstructure and Substructure

The choice of how the superstructure is connected to the substructure has a significant impact on how the bridge will behave. Choosing the abutment type sets the major design considerations for the bridge with respect to jointless behavior. Methodology for designing the abutments for the various types of jointless bridges is presented in Section 8.6.

The consideration for the connection to the piers is equally important. The superstructure can be made integral with a pier or designed to transfer loads to the pier with more traditional assumptions. What is important to note in the planning stages is how the pier will react as the bridge expands and contracts. Piers must be sufficiently designed, whether intended to flex with the structure (slender piers), or designed to resist the movement (stout piers). The latter case is generally not preferable as it often leads to overdesigned substructures since the movement from the continuous deck superstructure can generally be accommodated by simply using an expansion bearing to accommodate the movement. Design of different pier types is discussed in more detail in Section 8.6.2.9.

8.4.5 Other Considerations

Other factors that can affect the performance of jointless bridges are primarily associated with foundation conditions.

8.4.5.1 Site Condition

Integral abutments for jointless bridges are usually supported on a single row of piles to provide flexibility. Also, piles are typically used to minimize settlement of the abutment and differential settlement within the superstructure. However, when rock is close to the substructure bearing surface, a different type of type of foundation may be required. One solution is to use semi-integral abutments, described in Section 8.3.2, in which the abutment foundations are supported directly by and keyed into the rock. The end diaphragm serving as the abutment backwall and encasing the superstructure ends rests on bearings supported by the abutment foundations. The deck and approach slabs are cast monolithically with the backwall. The abutment foundations are rigid and the longitudinal movement of the superstructure is accommodated through the bearings.

As an alternate to the semi-integral abutments, spread footings may potentially be considered for integral abutments when rock is close to the surface, particularly for single-span bridges, and when the foundation is assumed to slide. However, significant friction forces would have to be overcome and this concept has typically not been considered. Differential settlement would be another concern for use of spread footings on soils to support abutments for multi-span continuous bridges but, Moulton et al. (1985) and Hearn (1995) indicate that the magnitude of settlement for abutments supported by spread footings is similar to that for abutments supported by piles. However, there is very little experience with the actual use of spread footings for integral abutments either on rock or on competent soil near the surface. Hence, it is recommended that experience be gained by starting with relatively short simple-span bridges. Use can then progress to longer structures and multi-span structures as successful experience is gained.

The following recommendations pertain to abutments supported by shallow spread footings, in which end movement may be accommodated by sliding:

- For footings founded on rock, a layer of granular fill should be used (on top of a leveling layer of fill concrete, as needed) between the footing and rock to facilitate sliding. The footing should not be keyed into rock.

- The abutment wall should be designed for shear and moments resulting from both expansion and contraction movements. The resistance to contraction should include friction on the bottom of the footing and passive soil pressure from the berm soil on the front face of the abutment.

- Sufficient drainage, distance from the face of the slope, and slope protection are essential in order to keep soil from washing out below the footing. For footings supported on a layer of granular soil for sliding on rock, use of geotextile material may be considered to contain the granular soil. For footings supported on soil, mechanical stabilization of the soil below the footing may be appropriate.

Another possible solution for use in conditions in which rock is close to surface is to drill large diameter holes in the rock and use piles, which would consequently allow the use of typical integral abutment construction. It must be noted that the concepts of integral abutments supported on spread footings or supported on piles placed in holes drilled into rock are not common practice. These two concepts are suggested for consideration when site conditions would otherwise inhibit use of typical jointless construction.

8.4.5.2 Deterioration of Piling

Accelerated pile deterioration is generally not considered except in specialized corrosive locations. Designers should consult with either a geotechnical engineer or geologist to mitigate possible impacts for this condition. More commonly, corrosion is generally thought of as a minimal concern for piles but it has been recorded (Beavers and Durr 1998) and more recently evaluated (Decker et al. 2008). Additionally, the state of Iowa has been investigating deterioration of piles just below the pile cap of integral abutments (Clark 2011). Initial results note that the State has discovered corrosion immediately below abutment footings of what would be considered normal conditions.

Piling deterioration is of increased importance for integral abutments due to the additional strains placed on the substructure from the longitudinal expansion of the superstructure. The potential for section loss based on soil conditions should be accounted for as presented by Article 10.7.5 of the *AASHTO LRFD Bridge Design Specifications* (*LRFD Specifications*) which states minimum considerations for the effects of corrosion and deterioration of piling (AASHTO 2012). Adhering to these guidelines should provide sufficient protection against advanced corrosion and thus failure of the integral abutment system.

8.4.5.3 Jointless Bridge Abutments with MSE Walls

For locations where it is impractical to set the abutment on top of an embankment slope or to reduce the total bridge length, full height abutments with MSE retaining wall may be considered in the design of jointless bridges.

When MSE walls are used, steps must be taken to prevent excess pressure on the retaining wall introduced by the movement of the backwall and pile.

For integral abutments, per *FHWA Demonstration Project 82* (Elias et al. 1997) the horizontal force and its distribution with depth may be developed using pile load/deflection methods (*p-y* curves) and added as a supplementary horizontal force to be resisted by the MSE wall reinforcements. This force will vary depending on the level of horizontal load, pile diameter, pile spacing, and distance from the pile to the back of the panels.

Per the *FHWA Demonstration Project,* the following additional design details have been used successfully:

- Providing a clear horizontal distance of about 1.5 ft (0.5m) between the back of the panels and the front edge of the pile; and
- Providing a casing around piles, thru the reinforced fill, where significant negative skin friction is anticipated.

Where pile locations interfere with the reinforcement, specific methods for field installation must be developed. Simple cutting of the reinforcement is not permissible.

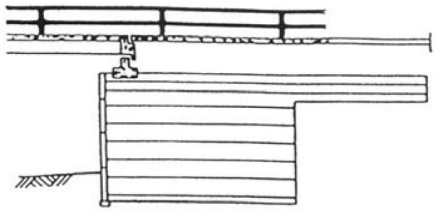
For integral abutments and for seamless bridges, MSE walls can still be used, however they must be sufficiently isolated from the soil movement caused by the movement of the piles. Alternatives suggested by Nicholson (1997) for the use of MSE walls with jointless bridge abutments are shown in Figure 8.5.

Figure 8.5a illustrates the use of a semi-integral abutment or stub integral abutment on spread footings. In this approach, the MSE reinforcement should be designed for the sliding forces in the bearings of the semi-integral abutment or the frictional sliding forces of the spread footings.

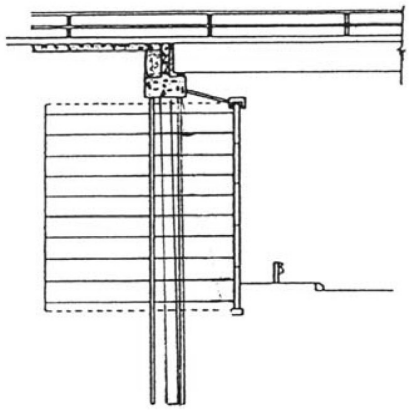
Figure 8.5b illustrates use of pile encased in a pressure-relieving sleeve that isolates the pile movements from the surrounding soil. Hassiotis (2007) has reported tests with an integral abutment supported on pile encased in corrugated steel sleeves backfilled with sand. Lui et al (2005) indicate that the Iowa DOT criteria for use of MSE walls with integral abutments requires each pile to be encased in a corrugated metal sleeve. The reinforced soil should include sand up to the bottom of the sleeve and the remainder should be filled with bentonite to the top of the sleeve.

Figure 8.5c illustrates the use of a semi-integral abutment supported on a pier in front of the MSE wall. No additional considerations are necessary for semi-integral abutments since the lateral movement is dissipated through the bearings.

(a) Semi-integral or stub integral abutment on spread footing. The reinforced soil must be allowed to settle before constructing the bridge and approach pavement.



(b) Piled integral abutment. The piles are surrounded by an earth-pressure relieving sleeve.



(c) Semi-integral abutment in front of reinforced soil.

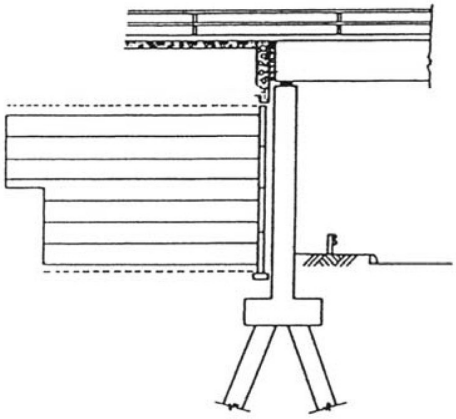


Figure 8.5. Alternatives to integral full-height wall abutments using reinforced soil retaining structure. (Nicholson et al. 1997)

8.5 STRATEGY SELECTION PROCESS

Each type of jointless construction has a range of parameters that is appropriate for particular bridges or provides various advantages over another type of system. The following tables provide guidance in selecting bridge type based on limiting parameters. Table 8.1 assists in the selection of the primary system and provides three options with

regards to foundation types: integral, semi-integral and seamless. The maximum length of each system is not set. Rather it is based on design calculations. Typical details that could be employed with each system are provided and corresponding sections are noted. Major advantages and disadvantages of each system are briefly described in Table 8.1 Relatively, the semi-integral abutment type provides larger longitudinal movement capabilities as compared to integral and seamless systems. The trade-off is the need to add sliding bearing which will result in reduction in service life. As indicated in Table 8.1, the relative maintenance of integral and seamless abutment types is lower than maintenance costs for the semi-integral abutment type. The main reason is the need for incorporating bearing at abutment. All three abutment types are applicable to existing bridges where it is desired to eliminate the expansion devices at the abutment. In some situations, cost may prevent use of integral or semi-integral abutment types.

Table 8.1. Strategy Table for Abutment Type Selection in Jointless Bridges—Straight Bridges.

Strategy	Maximum Bridge Length Used	Typical Details	Advantages	Disadvantages	Qualitative Longitudinal Movement Demand	Qualitative Maintenance Ranking	Applicability to Existing Bridges
Integral	Established based on design provisions stated in the *Guide*	Figure 8.31, Figure 8.33, Figure 8.34, Figure 8.35	Eliminates need for bearings	Difficult to inspect damage to piles due to pile movement	Low	Low	Yes
Semi-Integral	Established based on design provisions stated in the *Guide*	Figure 8.36, Figure 8.37, Figure 8.38	Provides reduced longitudinal force transfer to piles	Needs bearings	Medium	Medium	Yes
Seamless	Established based on design provisions stated in the *Guide*	Figure 8.25, Figure 8.26	Eliminates need for expansion joints. Eliminates concerns when there is skew or curvature. Eliminates need for bearings and sleeper slab.	Possible, initial higher cost. Difficult to inspect damage to piles due to pile movement; long transition zone off bridge	Low	Low	Yes

Table 8.2 provides further guidance on the substructure type that is appropriate for use with each type of jointless bridge. As indicated in Table 8.2, for integral abutment type, H or Prestressed piles or concrete filled tube (CFT) piles could be used. In the case of prestressed piles, the relative lateral displacement movement is lower compared to H or CFT piles. In the case of prestressed piles, cracking and corrosion is of concern. In the case of H pile and CFT piles, corrosion is of concern. These concerns are reflected qualitatively in assessing the potential for each abutment type to achieve 100 years of service life.

Table 8.2. Strategy Table for Foundation at Abutments in Jointless Bridges-—Straight Bridges.

Strategy		Typical Details	Advantage	Disadvantage	Qualitative Longitudinal Movement Demand	Potential for Achieving 100+ Years of Service Life
Integral Abutment	**H-pile**	Figure 8.33	Economical for small movements. Easy to construct	Relatively Low strength, ductility and buckling capacity	Medium	Medium
	Prestressed Pile	Figure 8.32	Very stiff and high axial load capacity	Prone to concrete deterioration and corrosion of strands	Low	Medium
	CFT Pile	Figure 8.32	CFT has high strength and ductility and higher buckling capacity, which will accommodate much larger bridge length.	Higher initial cost	High	Medium
Semi-Integral Abutment	Any foundation type could be used with the semi-integral abutment.					
Seamless	The strategy for selection of seamless foundations is the same as those for integral abutments. The detailing for seamless bridges is primarily in the transition zones at the ends of the bridge as detailed in Section 8.6.					

Table 8.3 provides guidance on the types of connections and bearings used at the piers when used in a jointless bridge. Considering the discussions provided in previous sections, the last four columns of Table 8.3 provide qualitative rankings of each option with respect to maximum longitudinal movement capabilities, relative maintenance cost, applicability to existing bridges, and potential to achieve 100-year-plus service life.

Table 8.3. Strategy Table for Connection between Piers and Superstructure in Jointless Bridges—Straight Bridges.

Strategy		Detail Figure	Advantage	Disadvantage	Qualitative Longitudinal Movement Demand	Qualitative Maintenance Ranking	Degree Of Difficulty To Apply To Existing Bridges	Potential For Achieving 100 Plus Years Of Service Life
Girders continuous over pier	**Integral-Frame Action**	Figure 8.27a, Figure 8.29	Eliminates need for bearings over pier	May cause transverse cracking in the pier	Low	Low	Medium	High
	Fixed Bearing (Rotational Movement Allowed)	Figure 8.27b	No longitudinal movement requirement for bearing over pier	May cause transverse cracking in the pier	Low	Mediu m	Medium	Medium
	Expansion Bearing	Figure 8.27c	Reduced bending of pier column	Bearing designed for both rotation and longitudinal movement	High	Mediu m	Low	Low
Girders not continuous over pier	**Simple for Dead and Continuous for Live Load**	Figure 8.27b, Figure 8.27c, Figure 8.28, Figure 8.30	Eliminates joints and protects girder ends; viable option for seismic retrofit	Restraint moments and cracking in diaphragms .	Varies with bearing type	Low	Low	High
	Link Slab	Figure 8.27d, Figure 8.31	Low cost	May crack and cause leakage over joint	Varies with bearing type	High	Low	Medium

8.6 DESIGN PROVISIONS FOR JOINTLESS BRIDGES

Design procedures for integral abutment bridges can range from a simplified method of analysis to a more detailed approach. This section includes provisions for both. Section 8.6.1 provides requirements for using the

simplified approach and Section 8.6.2 describes detailed methods of analysis that should be used if Section 8.6.1 requirements are not met.

8.6.1 Simplified Method of Analysis

The simplified analysis method is provided to eliminate many design steps for simple bridges that do not require detailed analysis. A bridge should meet the following requirements for use of the simplified analysis method (VTrans 2009):

- The skew angle should be less than or equal to 20 degrees;
- Bridge can be straight or curved, but with parallel girders;
- Abutments and piers should be parallel;
- Abutment height should be limited to 13 ft.;
- Heights of the abutment at the bridge ends should be approximately the same (max difference 20%);
- The slope of the bridge in longitudinal direction should be less than 5%;
- The length of the wing wall, attached to abutment, should be less than 10 ft; and
- The length of the pile should be greater than or equal to 16 ft.

The main characteristics of the simplified method of analysis:

- The internal forces of the superstructure and substructure are obtained using a 2D analysis.
- Conservatively, the superstructure may be assumed to be simply supported at the two abutment ends.
- The design of the pile can be accomplished by separating the pile from other bridge elements and treating it as an axial member. The moment capacity of the pile section is affected by the applied axial load. As the pile axial load increases, the moment which causes the formation of the plastic hinge in the pile will decrease. Once the plastic hinge is formed the pile head can be assumed to act as a pin. In the simplified approach the pile is modeled as an axial element with one end (end close to abutment) subjected to axial load and constant

moment equal to moment capacity of the pile cross section. The interaction equation in *LRFD Specifications* Article 6.9.2.2 can be employed to determine the capacity of the pile section.

8.6.2 Detailed Method of Analysis

If requirements for the simplified method of analysis are not met, the bridge must then be analyzed using a detailed analysis approach. In this approach, there is no limitation on total skew angle, etc. For years, jointless integral abutment bridges were designed with an imposed maximum limitation on total bridge length, with maximum length of steel structures less than that of concrete. Design provisions for the detailed method of analysis, as outlined in this chapter, do not include bridge length limitations; rather, one must meet the specified design provisions.

In the detailed method of analysis, the superstructure and substructure are modeled as an integral system using 3-D finite element analysis, with girder webs modeled using shell elements. Use of grid type analysis should be carried out with caution and is not recommended primarily because the torsional stiffness of line elements in some of the available commercial programs does not include the contribution of warping torsional stiffness.

8.6.2.1 Loads

8.6.2.1.1 Dead Loads

Dead loads include the weight of all components including superstructure and substructure elements and include all permanent loads in accordance with *LRFD Specifications* Article 3.5. The dead loads are distributed to the foundation through traditional assumptions or in accordance with the owner's bridge design provisions.

8.6.2.1.2 Live Loads

Live loads and the associated impact are applied in accordance with *LRFD Specifications* Article 3.6, or in accordance with the owner's bridge design provisions. Note that for integral abutments and piers, application of live loads will cause rotation and induce moments that will need to be considered in the design.

Horizontal live load (braking force and centrifugal force) are subject to distribution with respect to the stiffness of the integral and semi-integral abutments. In traditional design, longitudinal forces are distributed to the substructure based on bearing fixity (expansion vs. fixed against horizontal movement) and relative substructure flexibility. For jointless bridges, the backfill is in full contact with the end diaphragm (backwall) and provides a significant amount of stiffness relative to the other substructure components. For integral abutments, in which the bearing condition is

fixed, it is acceptable to assume for bridges with one to three spans that the longitudinal forces are absorbed by the passive pressure and stiffness provided by the backfill soil. This should be verified by a geotechnical engineer. As bridges get longer and additional substructure units are introduced, a relative stiffness analysis should be performed. However, even with multiple piers with some having fixed-expansion bearings, integral abutments can be expected to absorb as much as 80% of the longitudinal force.

8.6.2.1.3 Soil Loads

8.6.2.1.3a Soil Load on Abutment

The magnitude of soil pressure behind the abutment wall and the nonlinear distribution of this pressure depend on wall displacement, soil type, depth, pile stiffness, and also direction of the displacement (Faraji et al. 2001). As a wall moves toward the backfill, passive pressure is engaged, and when it moves away, active pressure and surcharge pressure may be generated.

Full passive pressure builds up for relatively long bridge lengths. For shorter bridge lengths, only part of the passive pressure is developed for expansion as thermal expansion is limited. For all bridges, the maximum passive pressure force, P_p is calculated as

$$P_p = \frac{1}{2} K_p \gamma H^2 \qquad \textbf{EQ 8.1}$$

Where:

P_p = passive pressure force
K_p = the passive pressure coefficient
γ = unit weight of soil
H = height of soil face

K_p is not necessarily the maximum K_p associated with full passive pressure. The value of K_p should be calculated using Figure 8.6 and Figure 8.7 (Clough and Duncan 1991). The extreme values for expansion and contraction are proportional to the height of the wall. The movement required to reach the maximum passive pressure is on the order of 10 times the movement required to reach the active soil pressure. The movement required to reach the extreme pressures are larger for loose soils than that for dense soils (Figure 8.6 and Figure 8.7). Table 8.4 highlights the movements required to achieve maximum pressures.

The force-deflection relationship should be based on the design curves (Barker et al. 1991) shown in Figure 8.6 and Figure 8.7 (Clough and Duncan 1991). The stiffness of the springs behind the abutment wall is nonlinear and depends on the type of the soil.

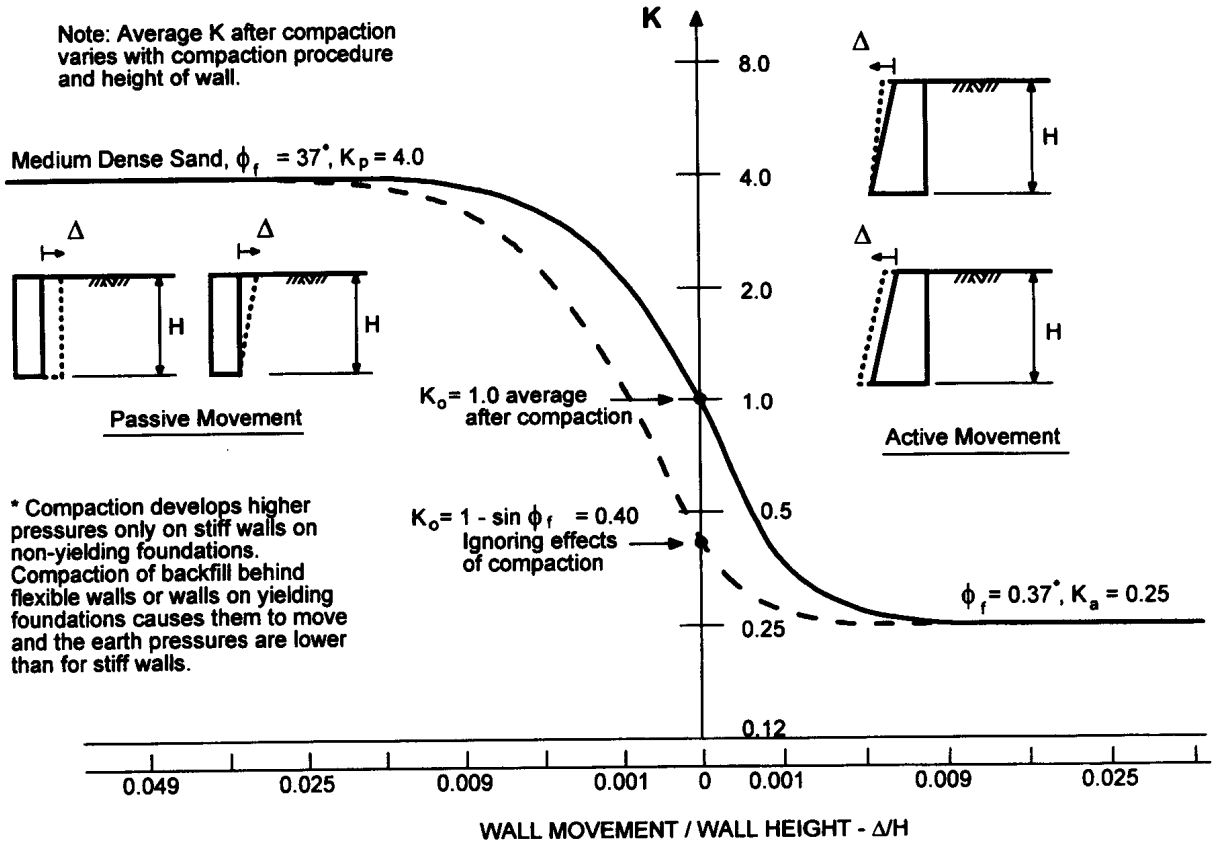


Figure 8.6. Relationship between wall movement and earth pressure. (Clough and Duncan 1991)

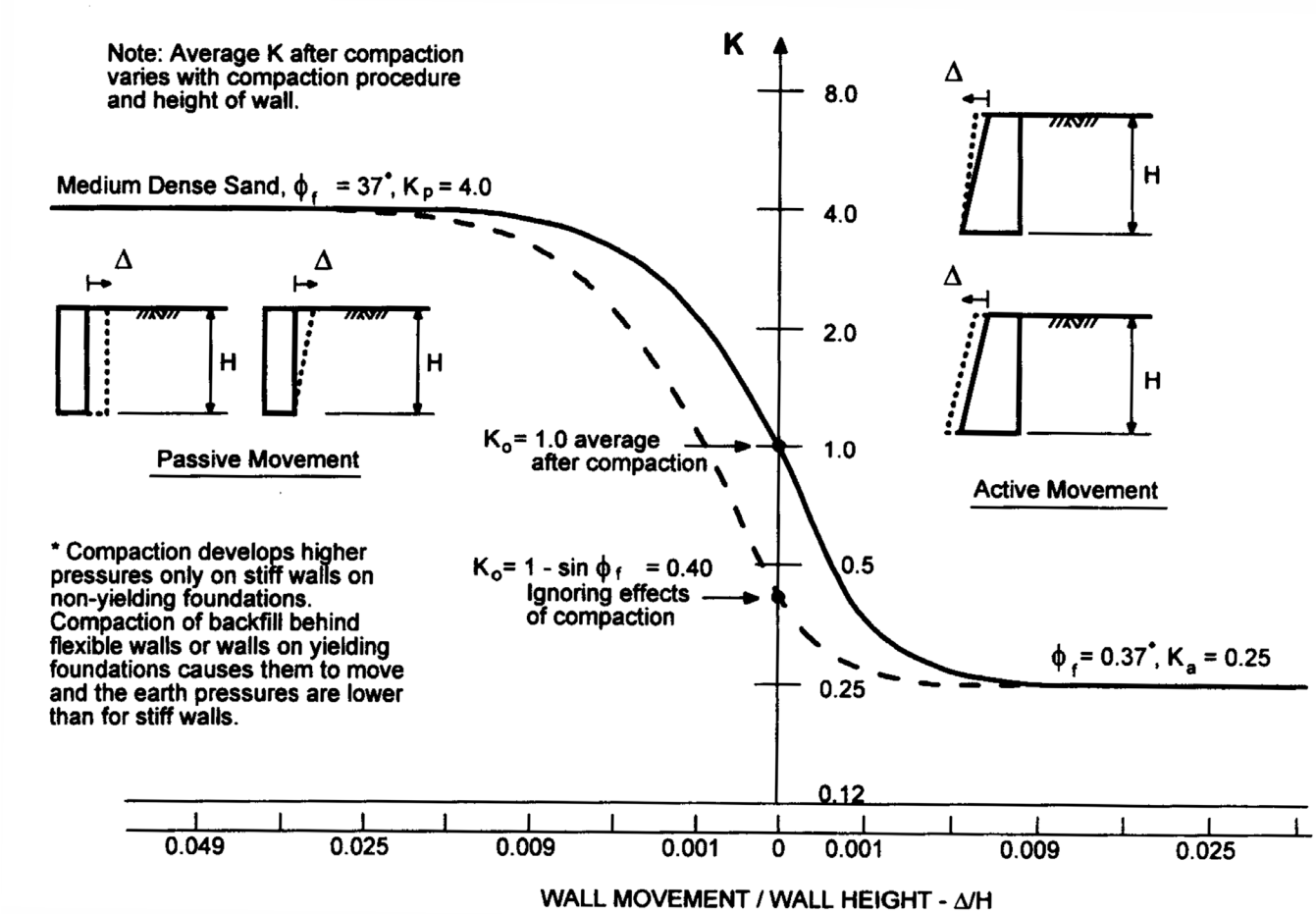


Figure 8.7. Relationship between wall movement and earth pressure for a wall with compacted backfill. (Clough and Duncan 1991)

Table 8.4. Approximate Magnitudes of Movements Required to Reach Extreme Soil Pressure Condition. (Clough and Duncan 1991)

	Values of Δ/H[(a)]	
Type of Backfill	**Active**	**Passive**
Dense Sand	0.001	0.01
Medium-Dense Sand	0.002	0.02
Loose Sand	0.004	0.04
Compacted Silt	0.002	0.02
Compacted lean clay	0.01[(b)]	0.05[(b)]
Compacted fat clay	0.01[(b)]	0.05[(b)]

(a) Δ = movement of top of the wall required to reach extreme soil pressure, by tilting or lateral translation, H = height of the wall.
(b) Under stress conditions close to the minimum active or maximum passive pressures, cohesive soils creep continually. The movement shown would produce temporary passive pressures. If pressures remain constant with time, the movements shown will increase. If movement remains constant, active pressures will increase while passive pressures will decrease.

8.6.2.1.3b Soil Load on Piles

The design of piles should consider soil-structure interaction using *p-y* curves such as the procedure recommended by the American Petroleum Institute for offshore platform design (API 1993).

Soil-structure interaction analysis of piles can be performed using available software—LPILE, COM624P, FB-MultiPier are several that utilize this approach. Further information on this topic is provided in Article 10.7 of the *LRFD Specifications*.

8.6.2.1.4 Thermal Loads

In order to account for the effect of temperature changes in design of jointless bridges, two different effects should be considered: the effect of uniform temperature change and the effect of temperature gradient within the structure. These two effects are explained in the following subsections.

8.6.2.1.4a Uniform Temperature Change

The calculation of uniform temperature changes should be in accordance with *LRFD Specifications* Article 3.12.2 in which two procedures are recommended, Procedure A and Procedure B, as described in the following. Either procedure may be used for concrete deck bridges that have concrete or steel girders. For all other types of bridges, Procedure A should be employed.

Procedure A

Table 8.5 presents the temperature ranges to calculate the design thermal movements. The difference between these values and the base construction temperature should be used to calculate thermal movements.

Table 8.5. Procedure A—Temperature Changes. (*LRFD Specifications* Table 3.12.2.1-1)

Climate	Steel or Aluminum	Concrete	Wood
Moderate	0 to 120 ^{o}F	10 to 80 ^{o}F	10 to 75 ^{o}F
Cold	-30 to 120 ^{o}F	0 to 80 ^{o}F	0 to 75 ^{o}F

Procedure B

The range of temperature change is the difference between maximum design temperature and minimum design temperature. The maximum design temperature for concrete girder bridges with concrete deck is provided by Figure 8.8 and the minimum design temperature is given in Figure 8.9. The maximum and minimum design temperatures for steel girder bridges are given in Figure 8.10 and Figure 8.11 respectively.

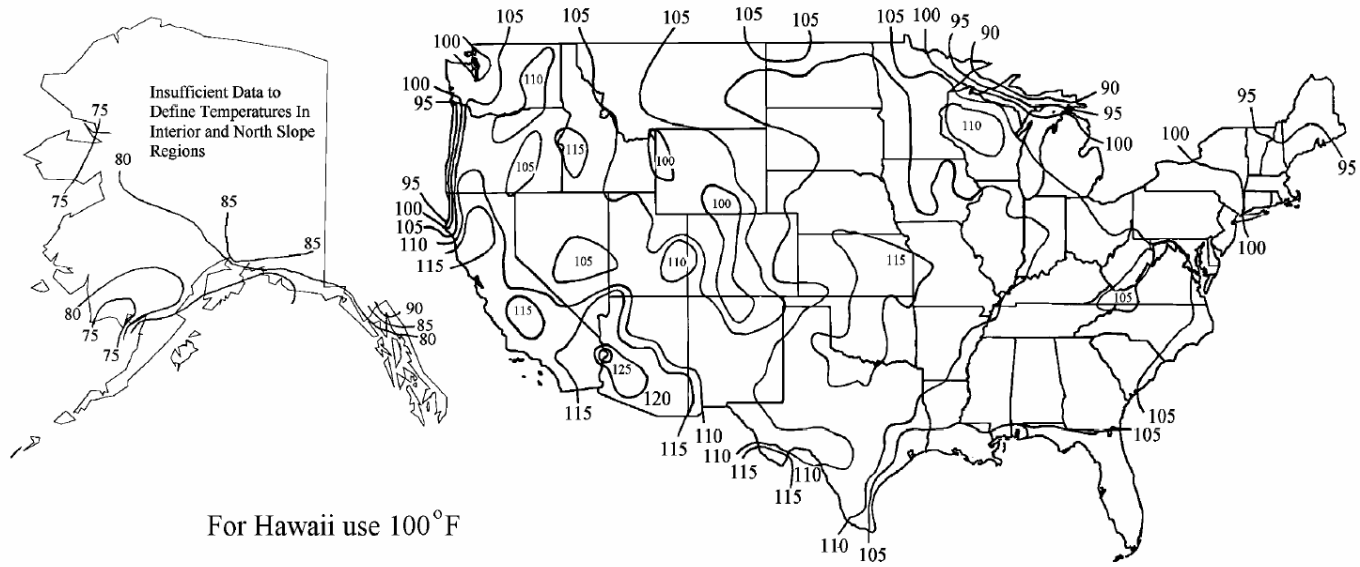


Figure 8.8. Maximum design temperature for concrete girder bridges. (*LRFD Specifications* Figure 3.12.2.2-1)

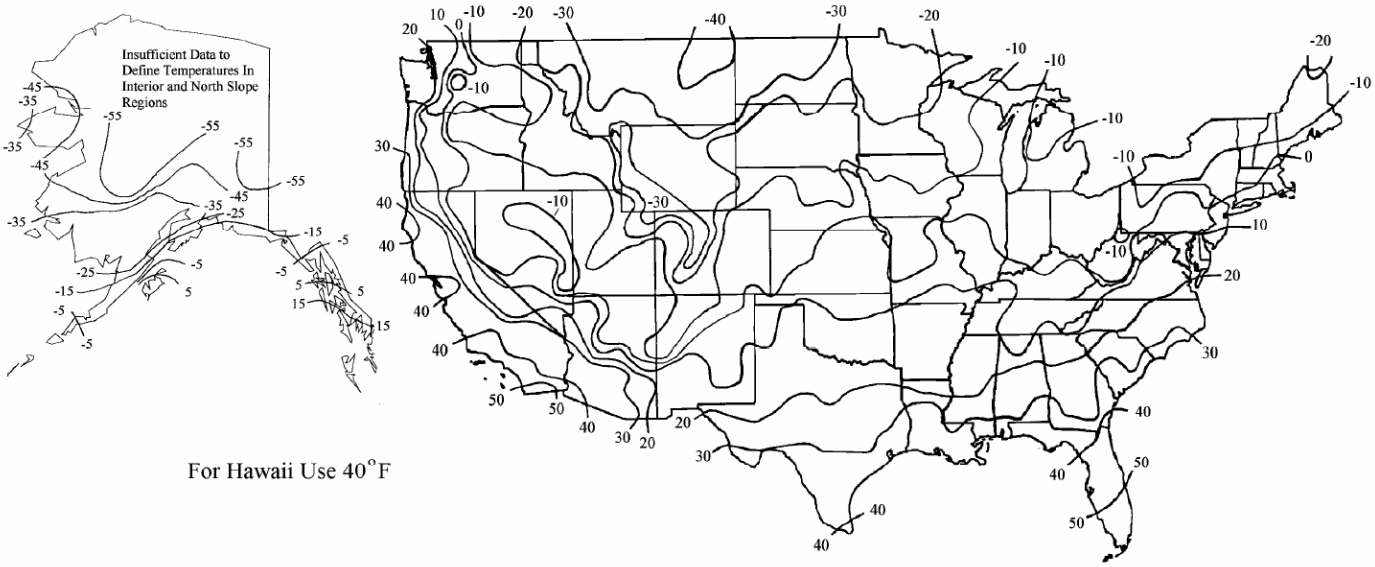


Figure 8.9. Minimum design temperature for concrete girder bridges. (*LRFD Specifications* Figure 3.12.2.2-2)

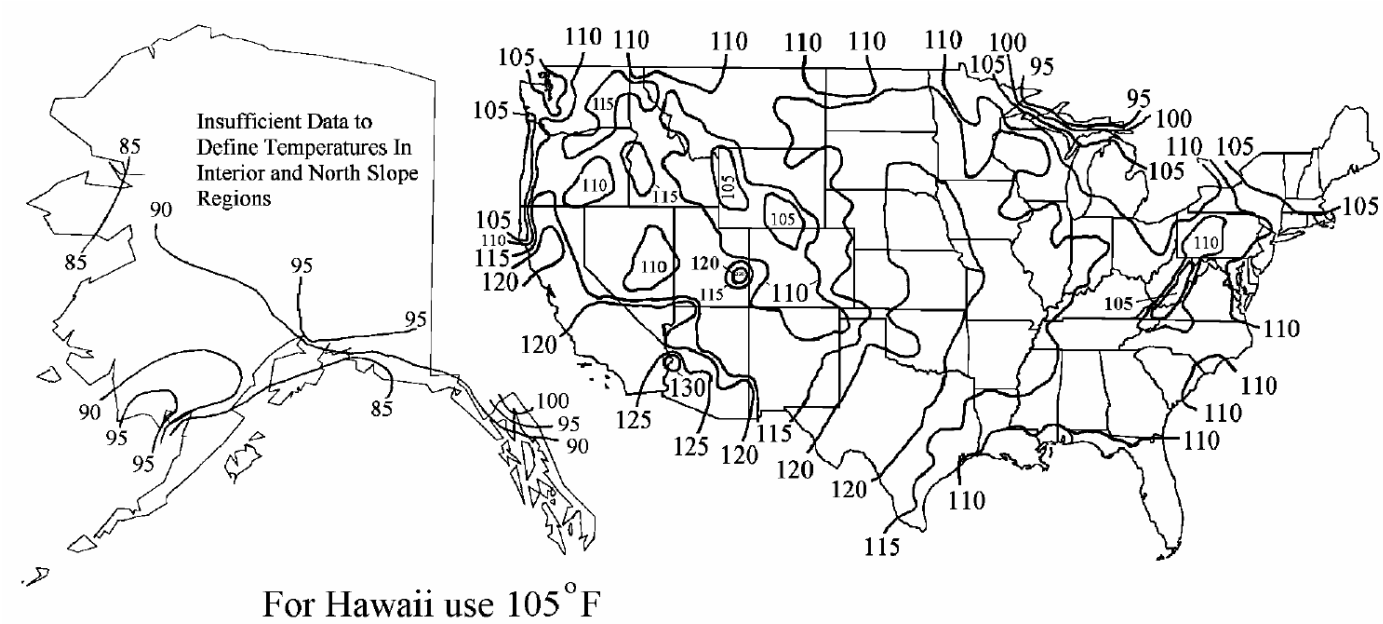


Figure 8.10. Maximum design temperature for steel girder bridges. (*LRFD Specifications* Figure 3.12.2.2-3)

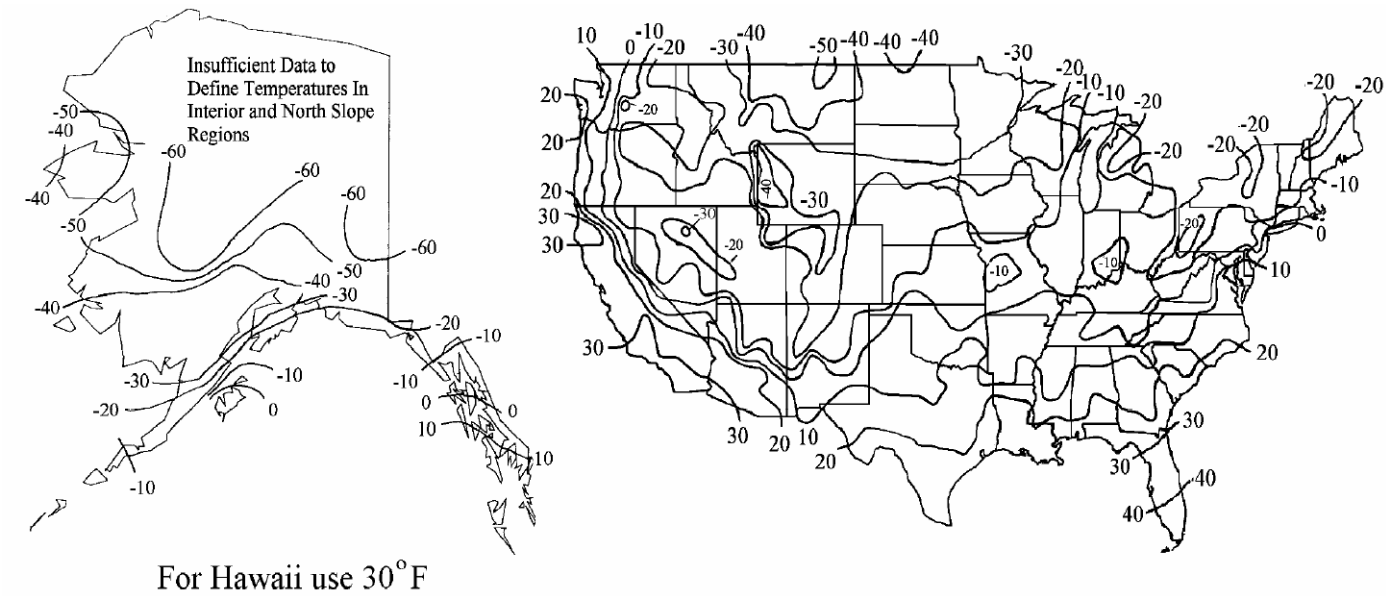


Figure 8.11. Minimum design temperature for steel girder bridges. (*LRFD Specifications* Figure 3.12.2.2-4)

8.6.2.1.4b Temperature Gradient

The effect of temperature gradient may typically be ignored; however, if the designer decides to consider the effect of temperature gradient the following provisions, taken from *LRFD Specifications* Article 3.12.3, are recommended. The profile of the temperature in steel and concrete girder bridges may be taken as shown in Figure 8.12; in which t is the thickness of concrete deck. Dimension A in this figure should be taken as:

- 12.0 in. for concrete superstructures deeper than 16 in.,

- (Depth of superstructure minus 4.0 in.) for concrete superstructures shallower than 16 in., and
- 12.0 in. for steel superstructures.

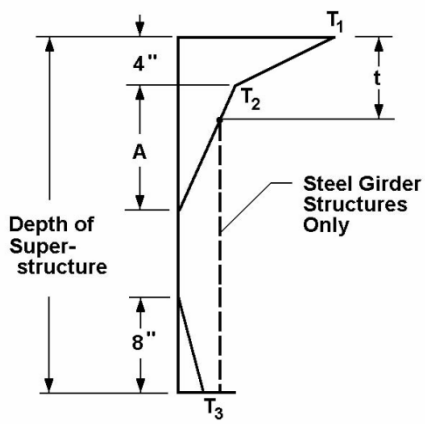


Figure 8.12. Positive vertical temperature gradient in concrete and steel superstructures. (*LRFD Specifications* Figure 3.12.3-2)

The values for T1 and T2 are given in Table 8.6 and vary by solar radiation zone as determined from the map shown in Figure 8.13. The values in Table 8.6 are positive temperature values. The negative temperature values are obtained by multiplying the values from the same table by -0.3 for plain concrete decks and by -0.2 for decks with asphalt overlay. The value of T_3 should be taken as 0°F, unless a specific field study is carried out to determine this value, in which case T_3 should not exceed 5°F.

Table 8.6. Basis for Temperature Gradients. (*LRFD Specifications* Table 3.12.3-1)

Zone	T_1 (°F)	T_2 (°F)
1	54	14
2	46	12
3	41	11
4	38	9

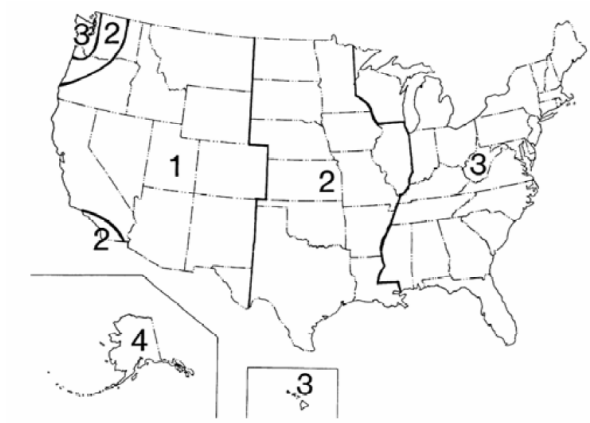


Figure 8.13. Solar radiation zones for the United States. (*LRFD Specifications* Figure 3.12.3-1)

When considering temperature gradient in the section profile, the analysis should consider axial extension, flexural deformation, and internal stresses (*LRFD Specifications* Article 4.6.6). The response of the structure to temperature gradient can be divided into three parts as follows:

Axial Expansion. This component is due to the uniform portion of the temperature gradient and can be calculated as (*LRFD Specifications* Equation C4.6.6-1):

$$T_{UG} = \frac{1}{A_c} \int\int T_G \, dw.dz \qquad \textbf{EQ 8.2}$$

Where:

T_G = temperature gradient (Δ°F),
T_{UG} = temperature averaged across the cross-section (°F),
A_c = cross-section area—transformed for steel beams (in.^{2}),
w = width of element in cross-section (in.),
z = vertical distance from center of gravity of cross-section (in.).

The corresponding uniform axial strain is then taken as (*LRFD Specifications* Equation C4.6.6-2):

$$\varepsilon_u = \alpha \left(T_{UG} + T_U \right) \qquad \textbf{EQ 8.3}$$

Where:

α = coefficient of thermal expansion (in./in./°F),
T_U = uniform specified temperature (°F).

Flexural Deformation. The consequence of temperature gradient is the development of curvature, ϕ, over the cross section and can be calculated using Equation 8.4.

$$\phi = \frac{\alpha}{I_c} \int\int T_G \, z \, dw . dz = \frac{1}{R} \qquad \textbf{EQ 8.4}$$

Where:

I_c = inertia of cross-section—transformed for steel beams (in.^{4}),

R = radius of curvature (ft).

Additional stresses. Any additional stresses because of curvature, created by thermal gradient can be calculated as:

$$\sigma_E = E\left[\alpha T_G - \alpha T_{UG} - \phi z\right] \qquad \textbf{EQ 8.5}$$

Where:

E = modulus of elasticity (ksi).

8.6.2.1.5 Creep

Concrete creep strains should be calculated using *LRFD Specifications* Article 5.4.2.3.2. Time dependence and changes in concrete strength should be taken into account in determining the effect of concrete creep. The creep coefficient, Ψ, can be determined using *LRFD Specifications* Equation 5.4.2.3.2-1.

$$\Psi(t,t_i) = 1.9 k_s k_{hc} k_f k_{td} t_i^{-0.118}$$ **EQ 8.6**

In which:

$$k_s = 1.45 - 0.13\frac{V}{S} \geq 1.0$$ **EQ 8.7**

$$k_{hc} = 1.56 - 0.008H$$ **EQ 8.8**

$$k_f = \frac{5}{1 + f_{ci}^{'}}$$ **EQ 8.9**

$$k_{td} = \left(\frac{t}{61 - 4f_{ci}^{'} + t}\right)$$ **EQ 8.10**

Where:

H = relative humidity (%). In the absence of better information, H may be taken from Figure 8.14,

k_s = factor for the effect of the volume-to-surface ratio of the component,

k_f = factor for the effect of concrete strength,

k_{hc} = humidity factor for creep,

k_{td} = time development factor,

t = maturity of concrete (day), defined as age of concrete between time of loading for creep calculations, or end of curing for shrinkage calculations, and time being considered for analysis of creep or shrinkage effects,

t_i = age of concrete at time of load application (day),

V/S = volume-to-surface ratio (in.),

$f_{ci}^{'}$ = specified compressive strength of concrete at time of prestressing for pretensioned members and at time of initial loading for nonprestressed members. If concrete age at time of initial loading is unknown at design time, $f_{ci}^{'}$ may be taken as 0.80 $f_c^{'}$ (ksi).

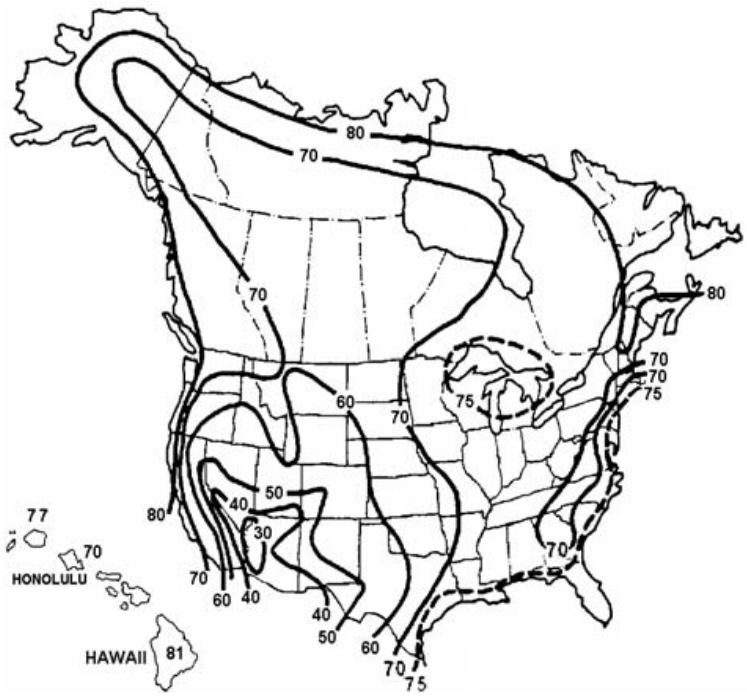


Figure 8.14. Annual average ambient relative humidity in percent. (*LRFD Specifications* Figure 5.4.2.3.3-1)

8.6.2.1.6 Shrinkage

Concrete shrinkage should be calculated in accordance with the provisions of *LRFD Specifications* Article 5.4.2.3.3, where appropriate. For concrete elements, shrinkage strain ε_{sh} can be calculated using Equation 8.11(*LRFD Specifications* Equation 5.4.2.3.3-1)

$$\varepsilon_{sh} = k_s k_{hs} k_f k_{td} 0.48 \times 10^{-3} \qquad \textbf{EQ 8.11}$$

In which:

$$k_{hs} = (2.00 - 0.014H) \qquad \textbf{EQ 8.12}$$

Where:

k_{hs} = humidity factor for shrinkage.

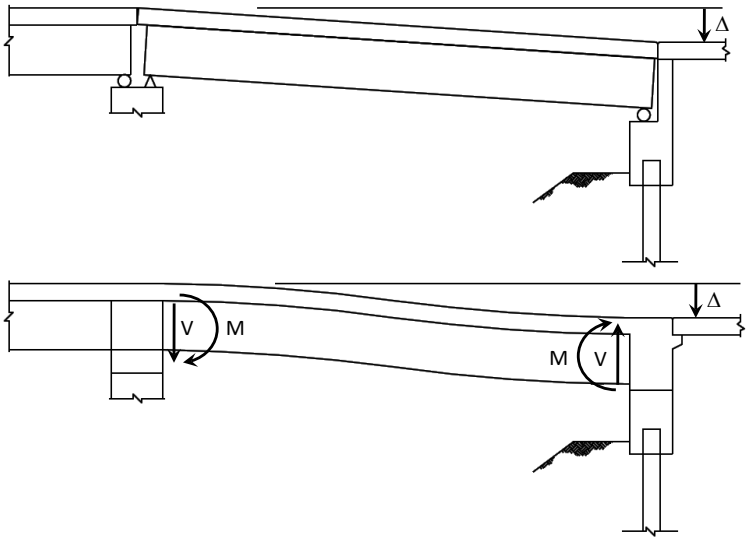
This article states that if the concrete is exposed to drying before five days of curing have elapsed, the shrinkage as determined in Equation 8.11 should be increased by 20%.

8.6.2.1.7 Settlement

Settlement is not a deterrent to the use of jointless bridges if sufficiently accounted for in the design of the effected components. AASHTO provides guidance on estimating settlement for structures in *LRFD Specifications* Article 10.7.2.3.

It must be recognized that bridges with simple spans and simple abutment bearings are able to accommodate the shifting and the associated rotation of the end spans with flexibility of the bearings. With continuous jointless superstructures and integral abutments, vertical or longitudinal movement of the foundation will introduce additional stresses in the superstructure, deck, or both. Also with semi-integral abutments, vertical movement of the foundation will introduce additional stresses in the superstructure, deck, or both. Figure 8.15 demonstrates this concept with an exaggerated illustration showing a settlement, Δ. In instances in which traditional bearings are used, the superstructure is free to rotate to accommodate the movement. On the other hand, when the superstructure is integral with the substructure, the superstructure is not permitted to rotate or shift and thus forces are introduced from the fixed end displacement.

The best approach to address settlement, in general, is to increase pile length such that settlement is not a design consideration. If increasing the pile length is not an option and design for settlement must be considered, then one of two strategies can be used to reduce or eliminate the effect of settlement: 1) evaluate the anticipated settlement and account for the resulting forces in the design, or 2) determine the maximum permissible displacement allowable by design and take measures to ensure that that settlement limit is not exceeded.



(Not to Scale)

Figure 8.15. Illustration comparing of settlement effects on the superstructure.

8.6.2.1.8 Wind

Wind load needs to be considered in accordance with *LRFD Specifications* Article 3.8. As with braking and centrifugal forces, longitudinal and transverse forces resulting from wind loads should also take into consideration the considerable stiffness of the integral abutments (see Section 8.6.2.1.2).

8.6.2.1.9 Other Loads

All other loads prescribed by the *LRFD Specifications,* such as collision forces and water and ice loads, need to be applied to jointless structures in the same manner as other structures. As with all designs, it is the engineer's responsibility to determine and apply the necessary load conditions appropriate for the unique situation of each jointless bridge.

8.6.2.2 Load Combinations and Limit States

This section provides a summary of available information in the *LRFD Specifications* and those developed by *SHRP 2 R19A* project related to load combinations and limit states to be considered for jointless bridges.

8.6.2.2.1 Load Combinations

The following loads should be considered for jointless bridges:

DC = dead load of structural components and nonstructural attachments,

DW = dead load of wearing surfaces and utilities,

EH = horizontal earth pressure load,

LL = vehicular live load,

WS = wind load on structure,

WL = wind on live load,

TU = uniform temperature,

CR = creep,

SH = shrinkage,

TG = temperature gradient, and

SE = settlement.

8.6.2.2.2 Load Factors and Combinations

Table 8.7 lists load combinations required in the design of jointless bridges based on *LRFD Specifications*.

Table 8.7. Load Combinations and Load Factors. (from *LRFD Specifications* Table 3.4.1-1)

Load Combination Limit State	DC DW EH	LL	WS	WL	TU CR SH	TG	SE
Strength I	γ_p	1.75	-	-	0.50/1.20	γ_{TG}	γ_{SE}
Strength II	γ_p	1.35	-	-	0.50/1.20	γ_{TG}	γ_{SE}
Strength III	γ_p	-	1.40	-	0.50/1.20	γ_{TG}	γ_{SE}
Strength IV	γ_p	-	-		0.50/1.20	-	-
Strength V	γ_p	1.35	0.40	1.00	0.50/1.20	γ_{TG}	γ_{SE}
Service I	1.00	1.00	0.30	1.00	1.00/1.20	γ_{TG}	γ_{SE}
Service II	1.00	1.30	-	-	1.00/1.20	-	-
Service III	1.00	0.80	-	-	1.00/1.20	γ_{TG}	γ_{SE}
Service IV	1.00	-	0.70	-	1.00/1.20	-	1.00

Table 8.8. Load Factors for Permanent Loads, γ_p. (from *LRFD Specifications* Table 3.4.1-2)

Type of Load and Foundation Type	Load Factor	
	Maximum	Minimum
DC: Component and Attachments	1.25	0.90
DC: Strength IV only	1.50	0.90
DW: Wearing Surface and Utilities	1.50	0.65
EH: Horizontal Earth Pressure		
Active	1.50	0.90
At-Rest	1.35	0.90

The *LRFD Specifications* state that the load factor for temperature gradient, γ_{TG}, should be considered on a project-specific basis or may be taken as:

- 0 at the strength limit states,
- 1.0 at the service limit states where live load is not considered, and
- 0.50 at the service limit state when live load is considered.

Since effects of γ_{TG} are typically self-limiting and do not significantly affect strength or ductility at strength limit states for the types of bridge girders typically used in jointless bridges, γ_{TG} can commonly be taken as 0 for the design of foundations in integral and semi-integral abutments.

Similarly, the load factor for settlement, γ_{SE}, should be considered on project-specific information or may be taken as 1.0. Load combinations which include settlement should also be applied without settlement.

8.6.2.3 Bridge Movement

Three methods are provided for calculating bridge maximum end displacements. The first approach is applicable to straight bridges, the second approach addresses transverse movement of skewed bridges, and the third approach is a general method to calculate the movement of curved girder bridges.

8.6.2.3.1 Displacement of Straight Bridges (Non-Skew)

Bridges expand and contract because of temperature changes and time-dependent volume changes associated with concrete creep and shrinkage. In jointless bridges, it is important to estimate the maximum expansion and contraction at each end of a bridge to determine the longitudinal displacement expected for the abutment piles. It is also important to predict the movement at each pier and the joint width needed between the approach slab and the pavement. Another important movement is the maximum total thermal movement at each end resulting from the total effective temperature range. The starting point to determine the maximum passive pressure should conservatively be at the maximum contraction (Oesterle, et al. 2005). The maximum passive pressure is related to the end movement, with re-expansion for the full effective temperature range.

Calculation of the length change for a prestressed concrete bridge can be accomplished through use of typical design values for the coefficient of thermal expansion combined with creep and shrinkage strains. However, the overall variability of these factors adds uncertainty to the calculated end movements. Although a coefficient of thermal expansion for concrete is typically assumed to be 5.5×10^{-6} to 6.0×10^{-6} /°F, it is known that this value can range from approximately 3.0×10^{-6} to 7.0×10^{-6} /°F (Kosmatka and Panarese 1988). Also, the variability of creep, shrinkage, and modulus of elasticity of concrete is known to be significant (Bazant and Panula 1980). In addition, resistance to length change from abutments and piers, combined with the variability of the restraint (primarily caused by the variability of the soil), leads to unequal movement at each end of a bridge (even in theoretically symmetrical bridges) and uncertainty as to the magnitude of the movement at each end. Finally, the effective setting temperature of the bridge and the age of concrete girders at completion of the superstructure are typically unknown, making the

relative magnitude of expansion and contraction and the starting point for temperature, creep, and shrinkage calculations uncertain.

To investigate the effects of the variability of these parameters, and to provide guidance in formulating recommendations for design calculations, Monte Carlo studies were carried out to calculate bridge movements in order to generate a large number of computer analyses using the statistical variation of material parameters affecting the movement (Oesterle 2005; Oesterle and Volz 2005). Within each analysis, values for the coefficient of thermal expansion, temperature at construction, creep and shrinkage parameters of concrete, modulus of elasticity of concrete, and soil stiffness were selected based on statistical distributions of the values of these parameters. The variations in calculated bridge end abutment movements were then used to determine a 98 percent confidence interval for the maximum calculated movements. These maximum values were used to determine magnification factors, referred to as Γ factors, for modification of calculated values to account for uncertainty in the various parameters affecting results.

Procedures presented in the following sections outline how to determine the maximum end movements of jointless bridges including use of these Γ factors. In these calculations, it is assumed that the bridge has unknown construction timing and that no specific data on material properties are available.

For prestressed concrete bridges the following steps should be used to estimate the longitudinal movement.

- Determine the average construction temperature using Section 8.6.2.1.4a;
- Determine the maximum and minimum effective bridge temperatures based on the recommendations of Procedure B in Section 8.6.2.1.4a; and
- Assume the parameters for concrete presented in the following table.

Table 8.9. Concrete Parameters. (Oesterle et al. 2005)

	Coefficient of Expansion	Modulus of Elasticity
Value (English)	6.0x10-6/oF	$57000\sqrt{f_c'}$ (psi)
Value (Metric)	10.8x10-6/oC	$4700\sqrt{f_c'}$ (MPa)

- Determine point of zero movement of fixity point of the bridge based on the stiffness of the piers and the abutments. Use Section 8.6.2.1.3a provisions (Clough and Duncan 1991) to estimate the backfill passive

pressure and *p-y* method to evaluate the nonlinear behavior of the soil surrounding the piles. It should be noted that for symmetric bridges the middle of the bridge will be the point of fixity.

- Use the following equations to calculate the strain values in the bridge:

$$\varepsilon_{th} = \alpha \, \Delta T \qquad \textbf{EQ 8.13}$$

$$\varepsilon_{sh} = \varepsilon_{sh,girder} + \frac{\varepsilon_{sh,deck} - \varepsilon_{sh,girder}}{1 + \dfrac{(EA)_{girder}}{(EA)_{deck}}} \qquad \textbf{EQ 8.14}$$

$$\varepsilon_{cr} = \varepsilon_{cr,girder} \left[\frac{1}{1 + \dfrac{(EA)_{girder}}{(EA)_{deck}}} \right] \qquad \textbf{EQ 8.15}$$

$$\Delta\lambda = \Gamma \varepsilon_{total} \lambda \qquad \textbf{EQ 8.16}$$

Where:

$\Delta\lambda$	=	maximum end movement.
ε_{th}	=	thermal strain,
ε_{sh}	=	shrinkage strain,
ε_{cr}	=	creep strain,
α	=	coefficient of thermal expansion,
E	=	modulus of elasticity,
A	=	cross section area,
λ	=	length from the point of fixity to the end of the bridge. Note that for un-symmetrical bridge two different λ are involved,
Γ	=	magnification factor to account for uncertainty listed in Table 8.10,

$$\varepsilon_{total} = \varepsilon_{th} - \varepsilon_{sh} - \varepsilon_{cr} \text{ for expansion}$$

$$\varepsilon_{total} = -\varepsilon_{th} - \varepsilon_{sh} - \varepsilon_{cr} \text{ for contraction}$$

- For maximum expansion, which occurs shortly after construction, use the temperature difference between the maximum effective bridge temperature and the mean construction temperature for the bridge location based on the Federal Construction Council *Technical Report No. 65* (Science 1979). For creep and shrinkage calculations, assume the girders are 90 days old. Based on Monte Carlo simulation, Γ should be 1.6 to account for uncertainties with 98% confidence that the movement will be less than the calculated value.

- For maximum contraction, which occurs after several years of service, use the temperature difference between the minimum effective bridge temperature and the mean construction temperature. For creep and shrinkage, assume ultimate values with the girder to be 10 days old at the time of casting the deck. Based on Monte Carlo simulation, Γ should be 1.35 to account for uncertainties with 98% confidence that the movement will be less than the calculated value.

- For maximum thermal re-expansion from a starting point of full contraction, use the full effective bridge temperature range without any creep and shrinkage movements. Based on Monte Carlo simulation, Γ should be 1.2 to account for uncertainties.

- It should be noted that Γ values in the first two columns of Table 8.10 for maximum expansion and maximum contraction are relatively large and possibly over conservative because they are affected by the relatively large uncertainty of the construction or setting temperature. Further studies to include a more deterministic method to incorporate the construction temperature for a given bridge may reduce these magnification factors for a more efficient design approach.

Table 8.10. Summary of Recommended Magnification Factors. (Oesterle 2005)

	Maximum Expansion	Maximum Contraction	Maximum Thermal Re-Expansion
Prestressed Concrete Bridges	$\Gamma = 1.6$ creep+shrinkage+thermal	$\Gamma = 1.35$ creep+shrinkage+thermal	$\Gamma = 1.2$ thermal
Reinforced Concrete Bridges	$\Gamma = 1.6$ shrinkage+thermal	$\Gamma = 1.4$ shrinkage+thermal	$\Gamma = 1.2$ thermal
Composite Steel Bridges	$\Gamma = 1.7$ shrinkage+thermal	$\Gamma = 1.5$ shrinkage+thermal	$\Gamma = 1.2$ thermal

For reinforced concrete bridges, the same procedure as that used for prestressed concrete bridges should be used to calculate bridge end movements. In the case of shortening, movement caused by creep is not a factor. Magnification factors for different cases are listed in Table 8.10.

For steel girder bridges, the same procedure as that used for prestressed concrete bridges should be used to calculate bridge end movements, except that the extreme effective bridge temperatures should be calculated using the recommendations of Section 8.6.2.1.4.

Other steel material parameters are provided in Table 8.11.

Table 8.11. Recommended Steel Parameters. (Oesterle, Tabatabai, Lawson, Refai, Volz and Scanlon, 2005)

	Coefficient of Expansion	Modulus of Elasticity
Value (English)	$6.5 \times 10^{-6}/^{o}F$	2.9×10^{7} (psi)
Value (Metric)	$11.7 \times 10^{-6}/^{o}C$	2.0×10^{5} (MPa)

The effective coefficient of thermal expansion for steel composite bridges can be estimated as (Emanuel and Hulsey 1977):

$$\alpha_e = \frac{(\alpha EA)_{girder} + (\alpha EA)_{deck}}{(EA)_{girder} + (EA)_{deck}} \qquad \textbf{EQ 8.17}$$

Magnification factors for different cases are listed in Table 8.10.

8.6.2.3.2 Displacement of Skewed Bridges

For information on displacement of skewed bridges, refer to Appendix B.

8.6.2.3.3 Displacement of Curved Bridges

A procedure has been developed (Doust 2011) to determine the magnitude and direction of bridge end displacement in the case of curved integral abutment bridges. The related material is provided in Appendix F which explains the assumptions and limitations of the approach.

8.6.2.4 Design of Pile Foundation

Following are the main steps in design of piles.

- Based on subsurface explorations develop a soil profile for the site. Details of strength profiles, compressibility characteristics, stress history, and geology of the subsurface materials should be included. Further, identify favorable and unfavorable strata in the effected subsurface zones.

- Estimate the loads for the strength and the serviceability limit states.

- Determine the water profiles for the site and the expected depth of scour during 100-year and 500-year flood events.

- Select technically feasible pile types and pile lengths based on constructability and consider the strength, serviceability and extreme event limit states. Then eliminate the unsatisfactory alternatives.
- Make a general comparison between the technically feasible piles. Then design with the most cost effective alternative based on the following steps.
- Estimate the axial and lateral pile nominal resistance considering soil and structural capacity.
- Determine the required number of piles and their spacing.
- Estimate the resistance of the pile group based on pile group interaction. If the group resistance is not sufficient, modify the number of piles and/or the pile spacing.
- Check the possibility of punching of the pile into any weak stratum that may be present beneath the bearing stratum.
- Determine the tolerable deformations of the structure and estimate its vertical and lateral deformations. If the deformations are greater than the tolerable magnitudes, increase the length of the piles or number of the pile spacing.
- If the pile group is subject to uplift, check its uplift lateral.
- Determine the loads on top of pile under design lateral displacements to determine design forces for interaction with the pile cap.
- Determine whether pile load tests are needed to verify the design and apply the appropriate resistance factors.

These requirements are summarized in Table 8.12 and categorized as to whether the requirement applies to the strength or serviceability limit state. In certain cases the extreme event limit state does govern design of piles.

Table 8.12. Summary of Strength, Serviceability, and Extreme Event Limit States That Must Be Considered in the Design of Pile Foundations. (Adapted from Barker et al. 1991)

Design Consideration	Strength Limit State	Serviceability Limit State	Extreme Event Limit State
Structural capacity of single pile			
Bearing capacity of single pile			
Bearing capacity of pile groups			
Punching into lower weak stratum			
Settlement of pile groups			
Tensile capacity of piles during uplift			
Uplift capacity of single piles			
Structural capacity of piles under lateral loading			
Lateral movement of pile groups when subjected to lateral loads			

8.6.2.4.1 Pile Orientation

8.6.2.4.1a Straight Bridges

Abutment piles of straight bridges should be oriented so that the strong axis of the piles is perpendicular to the longitudinal direction of the bridge (Doust 2011). This orientation results in weak-axis bending of the piles due to longitudinal movement of straight non-skew bridges.

8.6.2.4.1b Curved Bridges

A procedure has been developed to determine the optimum abutment pile orientation in the case of curved girder integral bridges (Doust 2011). Appendix F provides the suggested approach and current limitations.

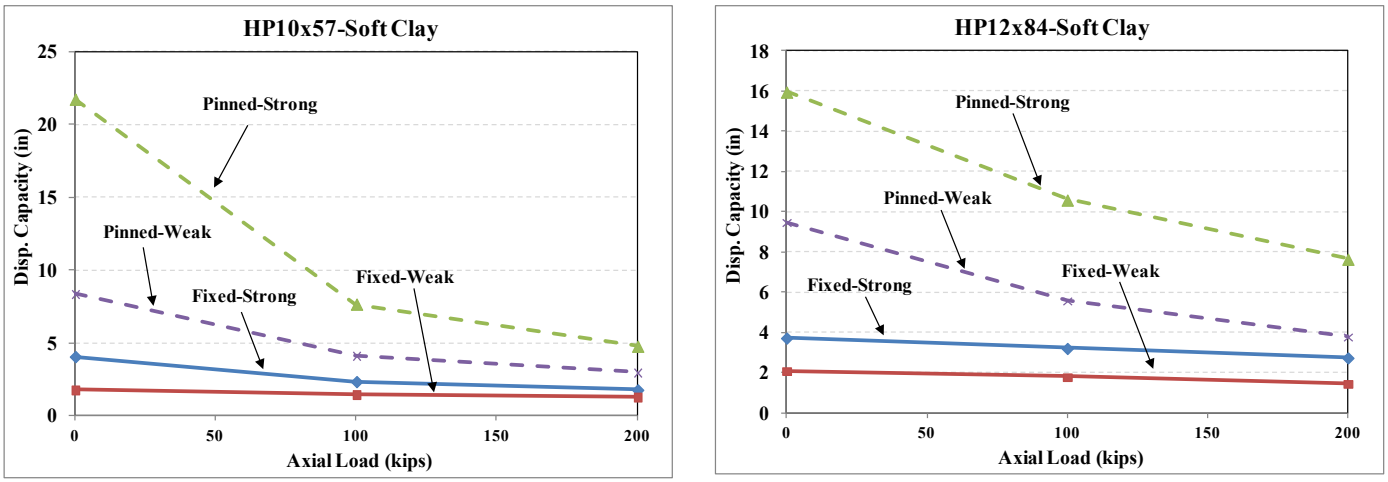
8.6.2.4.2 Pile Design

Design of piles should consider: strength; ductility; fatigue; stability; pile group interaction; and minimum penetration length required to satisfy the requirements for uplift, scour, down-drag, liquefaction, lateral loads, seismic forces and other extreme event loadings.

Figures 8.16 and 8.17 provide the design aids for design of piles for integral abutment systems. These design aids are based on research study conducted within *SHRP 2 R19A* (Sherafati and Azizinamini 2013; Sherafati 2011). Summary of the steps in developing these design aids, are provided in Appendix C. Figure 8.16 and 8.17 provides four charts that allows determination of maximum lateral movement capacity of a single pile versus the applied axial load to the pile. The design aids are provides for for two HP piles (HP 10x57 and HP 12x 84) and four soil conditions. The yield strength of the HP piles is assumed to be 50 ksi. Design aids provided in Figure 8.16 and 8.17 makes design

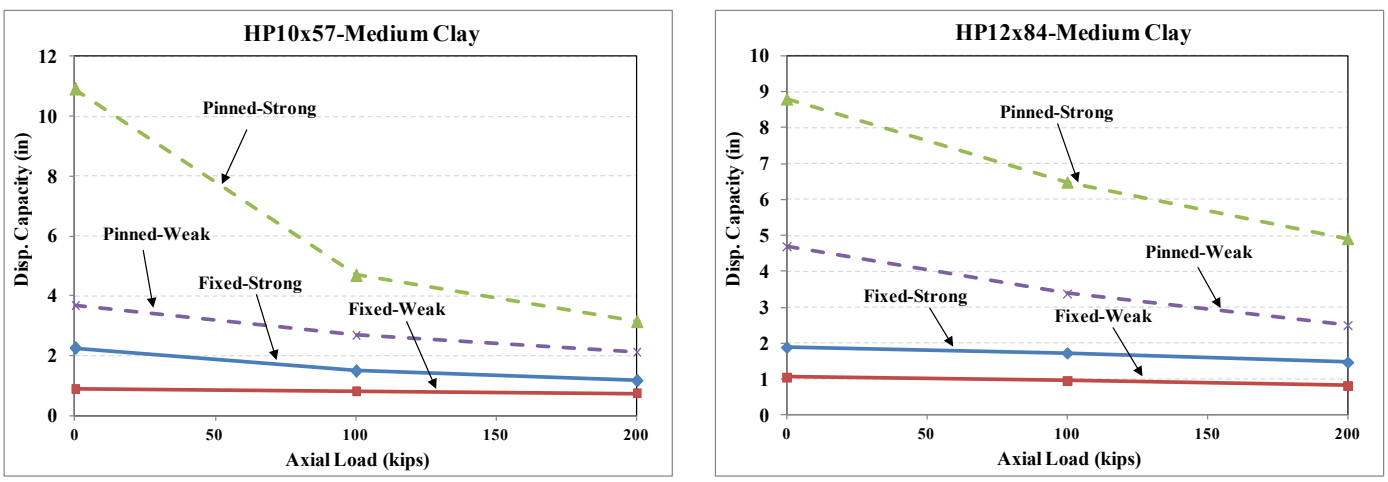
of piles for integral abutment system a very easy process. Knowing the applied axial load to pile, allows determination of maximum lateral movement that pile can accommodate.

The development of design aids shown in Figure 8.16 and 8.17 consider strength, fatigue, local and global stability, as described in Appendix C.



(a) (b)

Figure 8.16. Maximum displacement of compact HP sections in soft clay (c_u = 2.9 psi). (a) HP10x57. (b) HP12x84.



(a) (b)

Figure 8.17. Maximum displacement of compact HP sections in medium clay (c_u = 5.8 psi). (a) HP10x57. (b) HP12x84.

Geotechnical axial Resistance

The axial nominal resistance of a pile is the sum of its tip and friction resistance minus the weight of the pile.

$$Q_{nom} = Q_s + Q_t - W \qquad \textbf{EQ 8.18}$$

Where:

Q_{nom} = nominal bearing capacity of a pile
Q_s = pile shaft resistance ($A_s q_s$),
Q_t = pile tip resistance ($A_t q_t$)
W = weight of the pile
A_s = surface area of the pile shaft
q_s = unit skin resistance of the pile
A_t = area of the pile tip
q_t = unit tip resistance of the pile.

In most situations (except for large concrete piles in pile bent piers), the weight of the pile is small compared to the other terms and is usually disregarded.

8.6.2.4.2a Global Stability

Global stability is referred to as buckling of pile between end supports as opposed to local flange or web buckling. In general, the global stability is not a governing design provision unless a significant length of pile is above ground level and is unsupported against lateral buckling. (Sherafati et al. 2012).

8.6.2.4.2b Lateral Deformation of Pile Groups

Provisions of *LRFD Specifications* Article 10.7.2.4 should be used when *p-y* method of analysis is utilized to evaluate pile group horizontal movement.

8.6.2.4.2c Minimum Penetration Length

LRFD Specifications Article 10.7.1.5 specifies the provisions for the minimum penetration length necessary to satisfy the requirements for uplift, scour, settlement, down-drag, liquefaction, lateral loads, seismic response, and other extreme event loading conditions. This guidance is also appropriate for the design of jointless bridges and should be followed by the designer.

Tensile loading of a foundation (Uplift) may be caused by swelling soils, frost heave, buoyancy, lateral loads, and tensile loading during construction activities. Piles subjected to uplift should be designed to withstand tensile stresses and pullout from the subsurface materials.

Tensile loading for a foundation design and is well covered in the *LRFD Specifications*, which provides guidance to design against uplift for both single piles and pile groups. The design of single piles, drilled shafts and micropiles and in groups are addressed in Articles 10.7.3, 10.8.3, and 10.9, respectively.

Scour around the foundation is an important issue that should be considered in the design. In geotechnical analysis, it should be assumed that the subsurface materials above the scour line does not exist to provide bearing or lateral support. Three scour types should be considered in design (Barker et al. 1991):

- Aggradation and degradation are long-term effects. Aggradation is defined as the deposit of stream bed material eroded from other portions of a stream. Degradation is the removal of stream bed material and thus lowering the bed elevation.

- General scour and contraction scour are distinguished by removal of bed material across the entire width of the stream as a result of increasing flow velocities.

- Local scour occurs when bed material is removed from a small portion of the width of the stream. Bridge piers and abutments induce acceleration of the flow because of obstruction of the flow and cause vortices that wash away the bed material.

Scour is usually evaluated for a design flood with a return period of 100 years with a check flood not to exceed the 500-year event, or from an overtopping flood of lesser recurrence (AASHTO 2012).

Also, to increase the safety against pile failure due to scour, a few longer piles should be used rather than many short piles.

Settlement is not a deterrent to the use of jointless bridges if accounted for in the design of the effected components (see Section 8.6.2.1.7). Minimum penetration lengths with respect to settlement calculations for the foundation are not an additional concern for jointless bridges.

8.6.2.4.3 Analysis Tools

This section provides a general discussion of different analysis approaches and available tools designers can use to analyze jointless systems.

8.6.2.4.3a Simplified Analysis (p-y Method)

The ability to estimate the response of laterally loaded piles is of great importance in the design of jointless bridges. This design consideration is similar to a beam-on-elastic foundation model. If the piles are deep enough, modeling soil with Winkler springs is a useful method. In this method the soil is considered as a series of independent layers providing resistance (p) to the pile deflection (y). This resistance (p) may be a highly nonlinear function of the deflection (y). The proper form of a p-y relation is influenced by many factors, including:

- Variation of soil properties with depth,
- Shape of pile deflection,
- The state of stress and strain throughout the affected soil zone, and
- The rate sequence and history of load cycles.

8.6.2.4.3b Finite Element Analysis

Finite element modeling can be used to analyze a jointless bridge. There can be several different levels of finite element analysis for such a structure ranging from a simplified analysis to a refined analysis.

In a simplified finite element analysis, different elements including composite girders, abutment walls, piers, and piles are modeled using frame elements. The modeling can be two-dimensional (2-D) or three-dimensional (3-D); however, a 3-D analysis is preferred. The soil-structure interaction should be modeled by means of springs. Each spring's load-deflection curve can be assumed to be linear for a simplified model. In the case of 2-D models, the girder distribution factors should be calculated using appropriate equations from the *LRFD Specifications*.

On the other hand, a refined finite element analysis is a 3D modeling of a jointless bridge. In this approach, shell elements can be used to model the bridge elements. The soil-structure interaction can be modeled using nonlinear springs, which can model the abutment-soil interaction of the gap created between the abutment wall and soil due to contraction.

Based on the importance and complexity of the bridge, the level of detail included in the finite element model can vary. Engineering judgment should be excersized in developing the 3-D finite element model.

8.6.2.5 Design of Other Foundation Types

It is recognized that other foundation types may be appropriate for jointless applications depending on the requirements for the individual bridge. Following are additional considerations for other foundations.

8.6.2.5.1 Drilled Shafts

Drilled shafts should be designed considering the same design requirements as piles. Note, however, that traditional drilled shaft diameters of 30 in. and larger may prove to be too stiff for longer bridge lengths. Semi-integral abutments may be designed using drilled shafts with no additional consideration. Refer to *LRFD Specifications* Article 10.8 for more information on the design of drilled shafts.

8.6.2.5.2 Spread Footings

Use of spread footings directly over rock with integral abutments is not common practice and is not recommended.

8.6.2.5.3 Micro-Piles

Micro-piles may be a viable option for jointless bridges. Note that micro-piles, similar to regular piling, should only be used in a single row for integral abutments. Additionally, the micro-pile design must include consideration of the cyclic nature of the bending load resulting from the integral abutment configuration. Multiple rows of micropiles should only be used for semi-integral abutments.

8.6.2.6 Design of Pile Cap

Pile caps of jointless bridges may require special consideration based on the selected jointless system. The pile cap of an integral abutment no longer serves solely as a transfer for gravity loads. The pile cap must transfer longitudinal movements and other forces introduced by making the abutment integral.

8.6.2.6.1 Integral Pile Cap Design

Pile head elevation design for integral abutments can take one of two forms. The first option is to fix the pile head against rotation. Alternately, as recently demonstrated by the *SHRP 2 R19A* project (Sherafati and Azizinamini 2013;

Sherafati et al. 2013), the pile head can be fitted with an elastomer-based collar. This collar allows for limited end rotation and displacements to occur that alleviate some of the stresses induced by bending of the piles.

8.6.2.6.1a Encased Piles (Fixed Head Condition)

The pile cap for integral abutments must take into account and be able to develop the moment resulting from the restraint of the embedded pile head. Figure 8.18 illustrates how the shear restraint develops as a moment over the pile length due to fixity (assuming no soil support) as the force couple develops. In turn, this force is resisted by the pile cap, as shown in Figure 8.19. Note that the shear (V) and bending resultant (C_m) will be additive. Wasserman and Walker (1996) indicate that the depth of the resulting stress block is:

$$a_p = 0.85\left(\frac{l_{pe}}{2}\right) \qquad \textbf{EQ 8.19}$$

Where:

a_p = depth of the stress block

l_{pe} = pile embedment length within the cap

It can intuitively be seen that increasing the embedment length will directly decrease the bending resultant stresses on the cap, both by increasing the moment arm and the length of a_p.

Taking M_p as the plastic moment of the pile, the force couple balance is represented by:

$$M_p = C_m D' \qquad \textbf{EQ 8.20}$$

Or,

$$M_p = a_p b\, f_{cb2}(l_{pe} - a_p) \qquad \textbf{EQ 8.21}$$

Where:

C_m and D' = given in Figure 8.19

b = the either the pile section depth (weak axis bending) or flange width (strong axis bending), respective to pile orientation; or pipe diameter.

f_{cb2} = the bending resultant stress

It follows then that the maximum stress on the concrete cap is the combined resultant of the bending and shear stresses (f_{cb1})

$$f_{cb1} = f_{cb2} + V/a_p b \qquad \textbf{EQ 8.22}$$

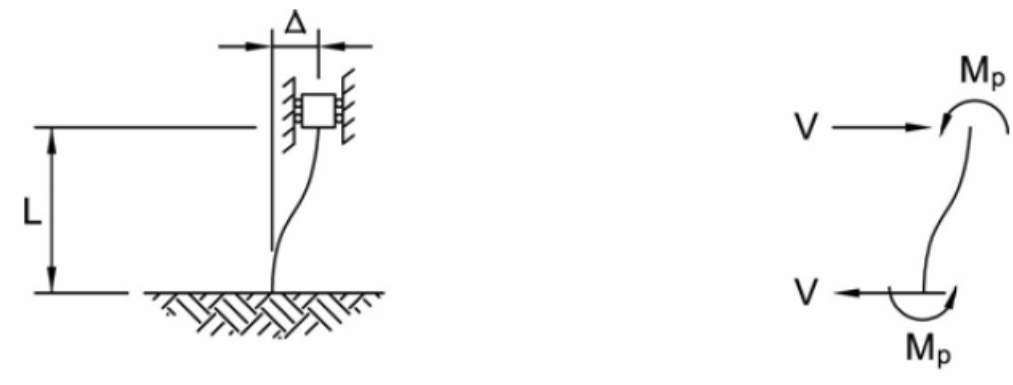


Figure 8.18. Moment transfer from pile to cap.

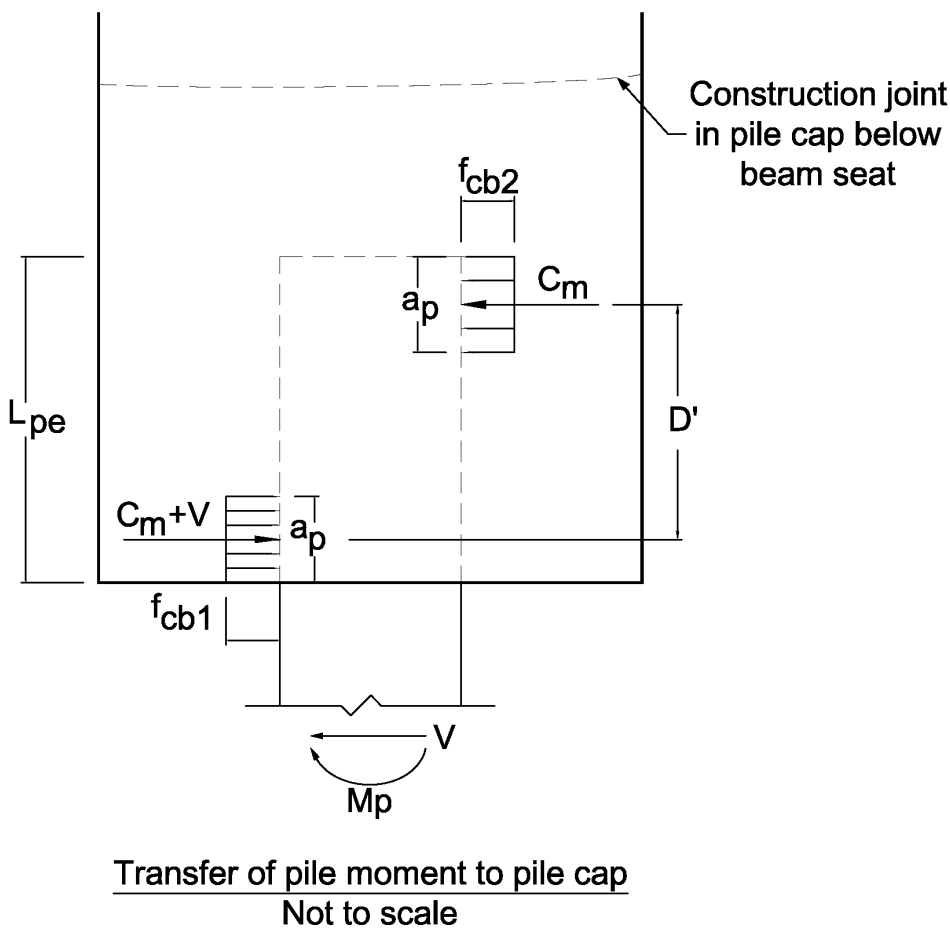


Figure 8.19. Moment transfer from pile to cap; internal force balance.

8.6.2.6.1b Pin Head Piles (Flexible Condition)

By providing rotational capacity at the pile head, the stiffness of the piling system is reduced, and also the moments developed in the pile as a result of lateral movement are decreased, since the pile will deform in a single curvature shape rather than double curvature. Since the major criterion limiting the application of jointless bridges is the capacity of the piles to accommodate lateral movement, the proposed detail can allow the application of jointless construction to longer bridge lengths (Sherafati and Azizinamini 2013).

Pile Cap Detail. The proposed detail consists of an elastomeric casing at the pile head. To alleviate the stress concentration at the top of the pile due to rotation, steel plates that slide by each other are key to the design detail. One of these plates is welded to the end of the pile while the other one is embedded in the concrete with shear studs. Figure 8.20 shows the pin head detail for the case of concrete filled tube (CFT) pile. However, the suggested detail can be adopted for H-piles as well.

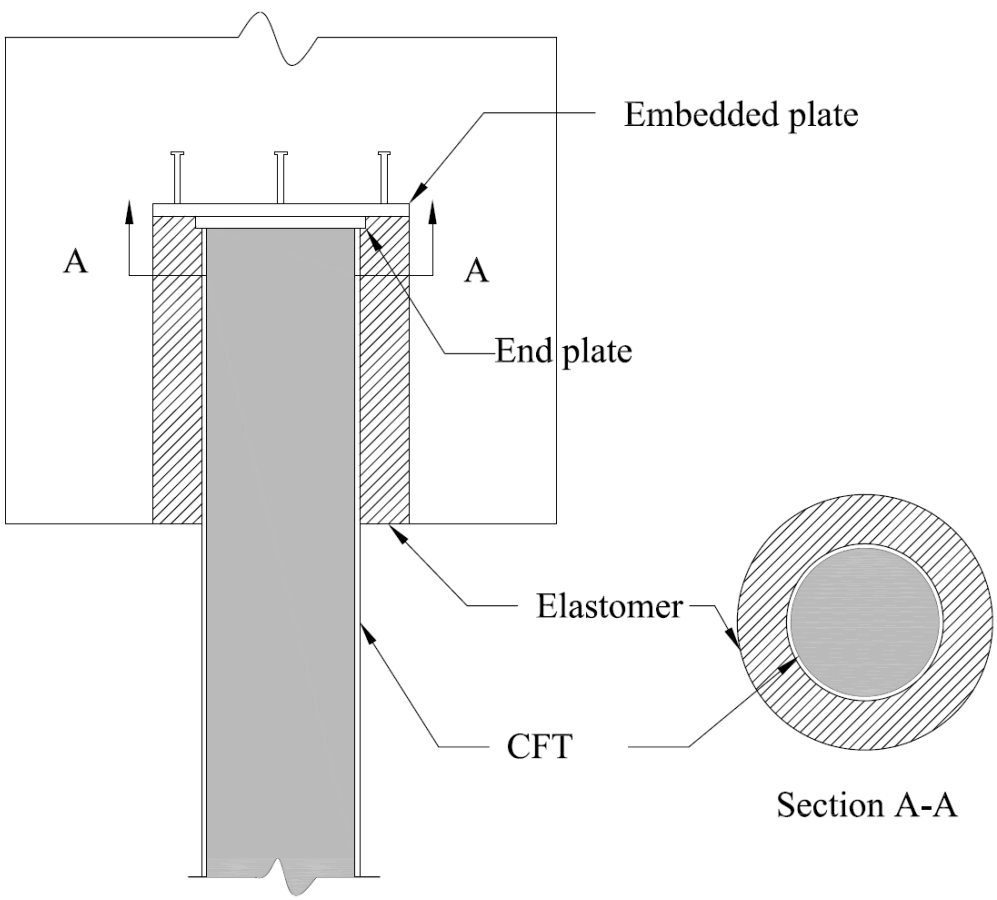


Figure 8.20. Proposed detail.

There are several advantages of this system:

- Can provide longer service life (by allowing integral construction for longer bridges);
- Can effectively be used for jointless skewed or curved bridges;
- Allows construction of longer jointless bridges;
- Develops smaller forces in the abutment and superstructure, since the lateral stiffness of the pile is reduced; and
- Reduces construction cost and time.

(a) **(b)**

Figure 8.21. Prefabricated pile cap.

Design Considerations. The material around the pile head is intended to have a very low elastic modulus to provide rotational capacity. Since the material for the detail experiences large strains, it must be able to accommodate these strains when subjected to the applied cyclic rotations.

Elastomeric material, regularly used as bearings for girder bridges, is recommended for the detail (Sherafati et al. 2013). Sufficient thickness needs to be provided to ensure the efficiency of the detail. Preliminary results indicate that the minimum thickness of elastomer should be 4 in.

8.6.2.6.2 Semi-Integral Pile Cap Design

The pile cap in semi-integral bridges is mainly subjected to axial load and possibly the moment created by axial load eccentricity applied to the pile cap.

8.6.2.6.3 Seamless Details

The design of the pile cap for a seamless bridge should follow the same procedure as for integral abutment bridges.

8.6.2.7 End-Diaphragm (Backwall) Design

In addition to being integral with the superstructure, the end diaphragm acts as a backwall for integral and semi-integral jointless bridge systems. As such, the end diaphragm is designed to resist forces resulting from soil loads and will henceforth be referred to as the backwall. The soil loads include the passive pressure force that is created by superstructure thermal expansion. The calculation of this passive pressure, P_p, is shown in Section 8.6.2.1.3.

Modeling, as shown in Figure 8.22 can be used to design the backwall.

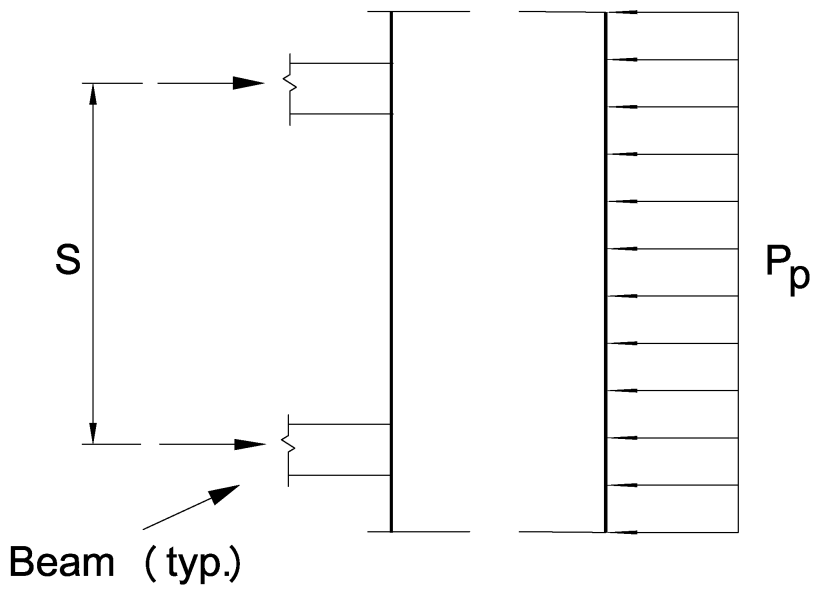


Figure 8.22. Lateral pressure restraint by superstructure. (Oesterle et al. 2005)

The following subsections discuss additional backwall design considerations that vary for each jointless bridge type.

8.6.2.7.1 Integral

The backwall for an integral bridge abutment must be designed to adequately transfer forces across the construction joint and into the foundation cap for each direction in which the pile bends. This transfer of forces is illustrated through an example of strut-and-tie model in Figure 8.23. In this figure, Section AA shows the local section recommended for a local region ($d_p + b$) over which the forces can be transferred and a suggested reinforcing pattern. The length d_p is the distance from the forward face of the pile cap to the face of the pile.

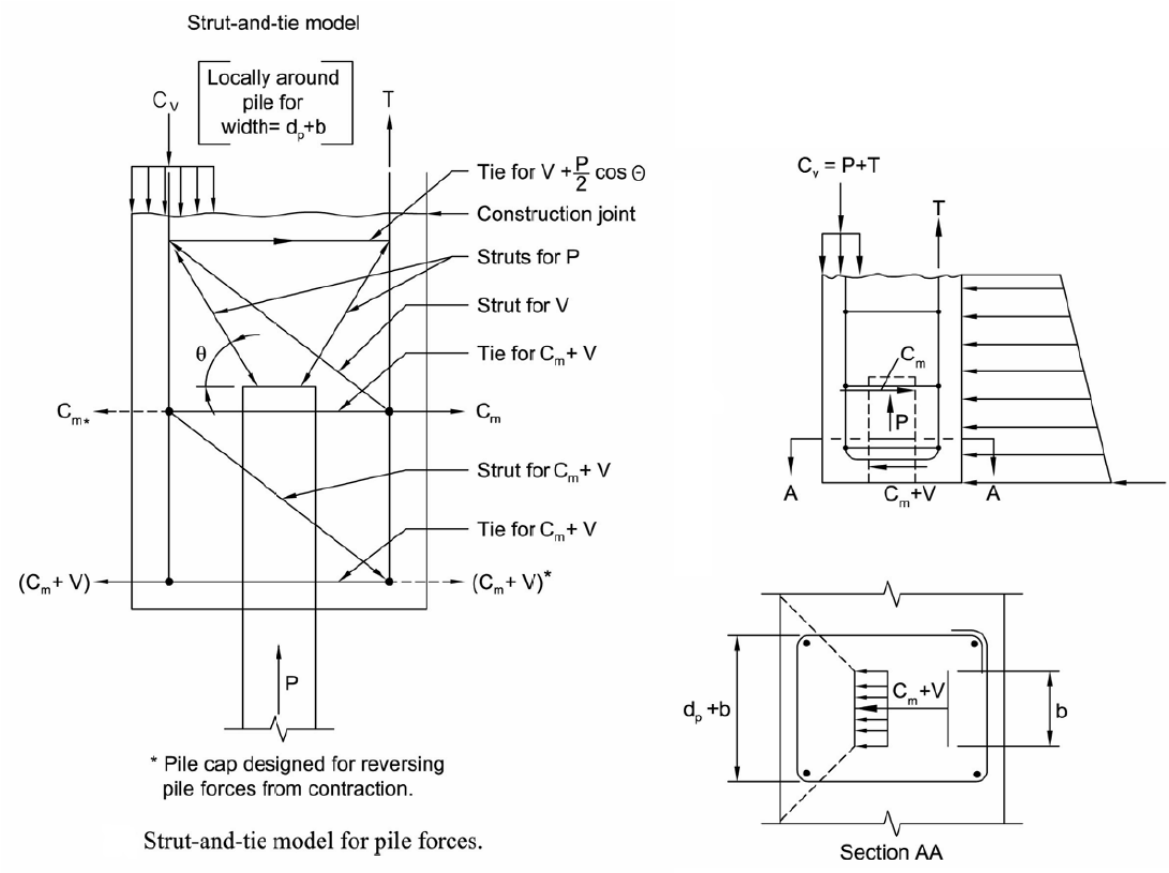


Figure 8.23. Lateral pressure restraint by superstructure. (Oesterle et al. 2005)

8.6.2.7.2 Semi-Integral

Semi integral backwalls do not require additional considerations above those outlined in Section 8.6.2.7.1. The one item of note, however, is that if removable forms are not used to form the bottom of the backwall over the foundation cap, the joint fill material used should be sufficiently stiff to support the concrete weight, yet flexible enough to not interfere with the movement permitted by the bearings. This has been successfully accomplished with expanded polystyrene filler.

8.6.2.7.3 Seamless

The design of backwalls for seamless bridges should be the same as for integral bridges.

8.6.2.8 Approach Slab Design

Jointless bridges require approach slabs for two main reasons: 1) the slab needs to be positively attached to the deck and/or substructure to eliminate the joint over the abutment, and 2) the slab must span the area behind the

abutment where the potential for backfill settlement exists. Backfill settlement will occur and introduce voids regardless of the degree of compaction and must be considered in design (Schaefer and Koch 1992)

8.6.2.8.1 Integral and Semi-Integral

For both integral and semi-integral abutments, the length of the approach slab is determined by the extent of the backfill. Gangarao and Thippeswamy (1996) determined that the rate of backfill settlement decreased significantly beyond 20 ft. from the back face of the backwall. This is a typical standard approach slab dimension shown in several state standards. The study by Schaefer and Koch (1992) demonstrated that backfill movements occur within a 1.5 horizontal to 1.0 vertical line from the bottom of the abutment for integral abutments. A general recommendation for the design length of the approach slab is to conservatively set at a 2.0 horizontal to 1.0 vertical slope from the bottom of the abutment. A 20 ft minimum should be considered for both integral and semi-integral abutments as shown in Figure 8.24.

Additionally, experience from several states has found that the approach slab should be positively attached to the backwall by at least No. 8 reinforcing bars anchored with a hook as shown in Figure 8.24. The condition shown in the figure allows for a separate pour of the approach slab designed as a simple span. Creating a moment connection between the approach slab and the deck slab is not recommended. The connection should be detailed to act as a pin with tension steel transferred across the approach span into the backwall for integral and semi-integral abutments. If a moment connection is desired, it is recommended to use a seamless deck transition for the design (see Section 8.6.2.8.2).

A final consideration for the approach slab is the development of compression forces. Sufficient allowance for expansion of the superstructure must be accommodated in the sleeper slab. (See Section 8.7.3 for sleeper slab details.) Otherwise, compression can be introduced into the slab resulting from closing the expansion gap and then activating the passive pressure behind the sleeper slab or contact with the adjacent roadway pavement, which is a major issue for spalling and buckling of adjacent pavement.

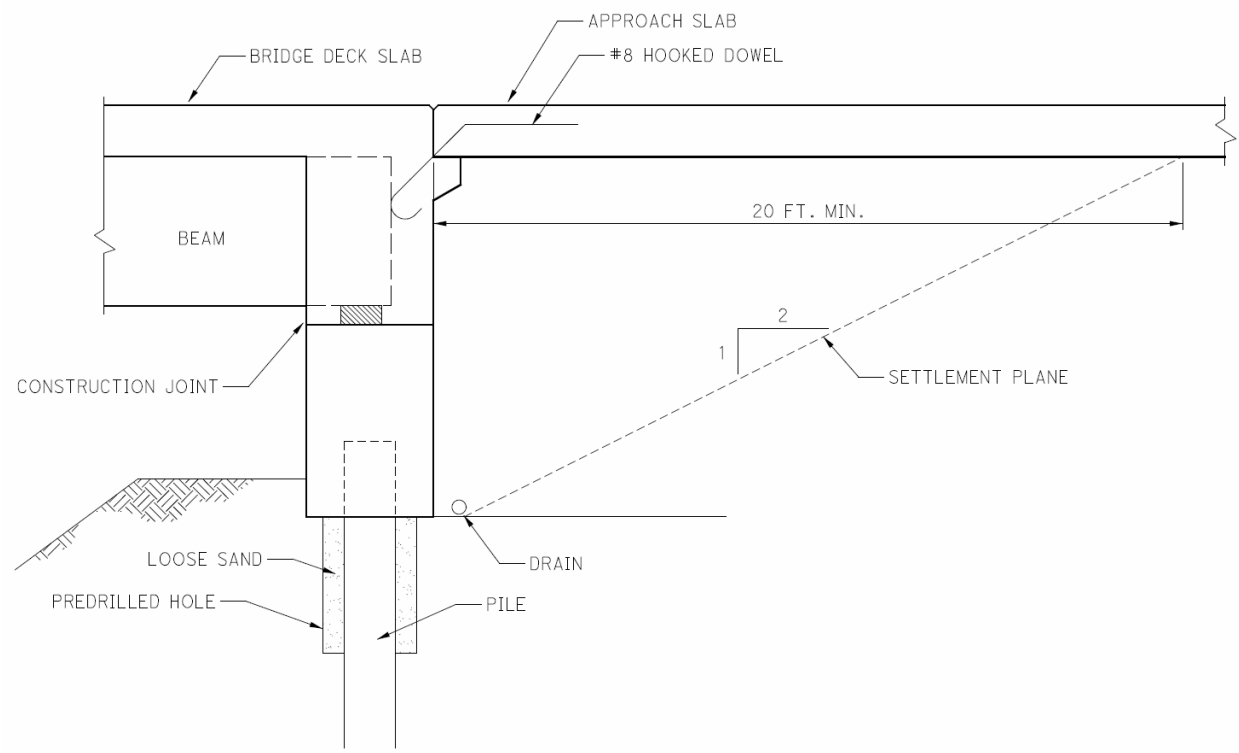


Figure 8.24. Determination of approach slab length.

8.6.2.8.2 Seamless Deck Transition Zone

Details of seamless systems developed by *SHRP 2 R19A* are provided in Appendix E. The system is shown in Figure 8.25 and Figure 8.26.

Beyond the abutment a "Transition Zone" is required which replaces the approach slab. The proposed transition introduces simplicity and ease of construction (Jung et al. 2007). The concept slowly transitions from a heavily reinforced region to a plain jointed condition over an extended transition length. Within the heavily reinforced region, crack spacing is quite small. As the level of reinforcement is reduced, the crack spacing increases. These cracks may be allowed to occur naturally or may be forced by shallow saw cuts in the pavement.

Immediately adjacent to the bridge is a thickened and reinforced approach zone. The approach zone behaves similar to a reinforced concrete slab bridge and is intended to carry flexural forces that may arise as a result of settlement.

The design of the approach zone is similar to the design of an approach slab for an integral or semi-integral bridge. The transition zone is not designed, per se, but the reinforcing spacing is reduced in specified stages. This transition zone reinforcement is shown in Figure 8.26.

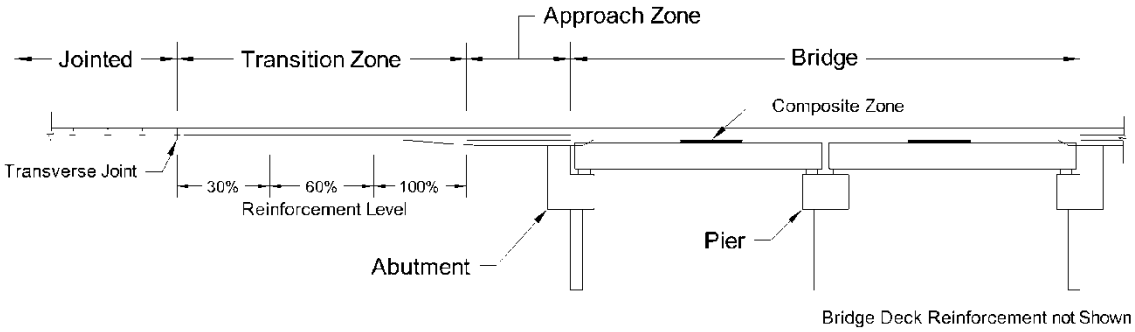


Figure 8.25. Seamless paving over bridge transitioning to jointed pavement.

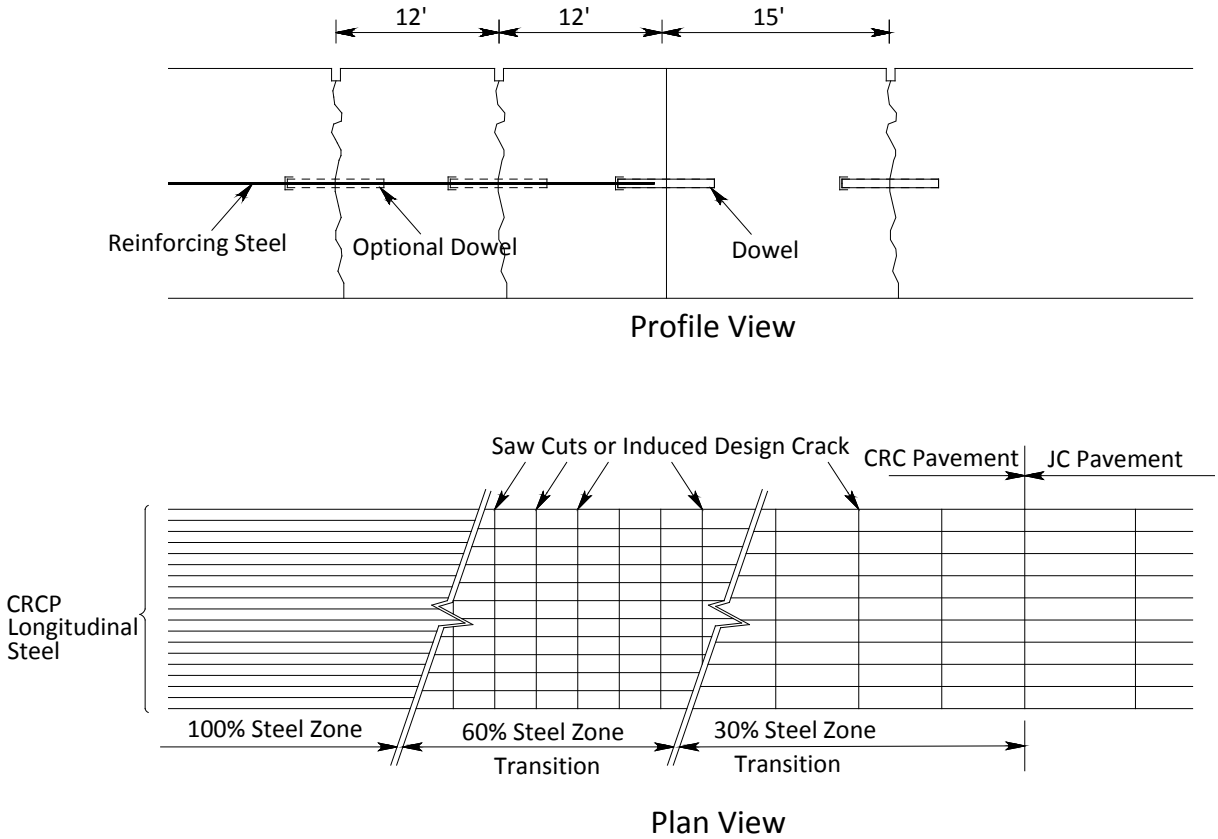


Figure 8.26. Continuously reinforced to jointed pavement. (Jung et al. 2007)

8.6.2.9 Design of Superstructure-Pier Connection

By definition, bridge decks in jointless bridges are continuous, including the region over the piers. The connection between the piers and the bridge deck could be integral, pinned, or expansion types, or be connected with a link slab. Figure 8.27 shows these different configurations conceptually.

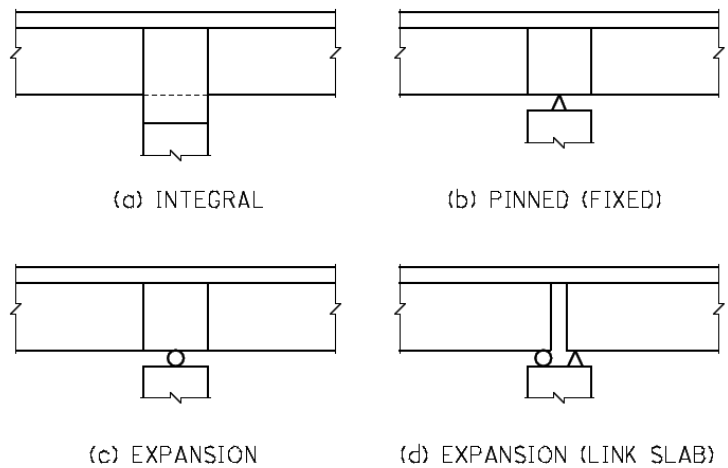


Figure 8.27. Integral, pinned, and expansion type bearings for jointless bridges.

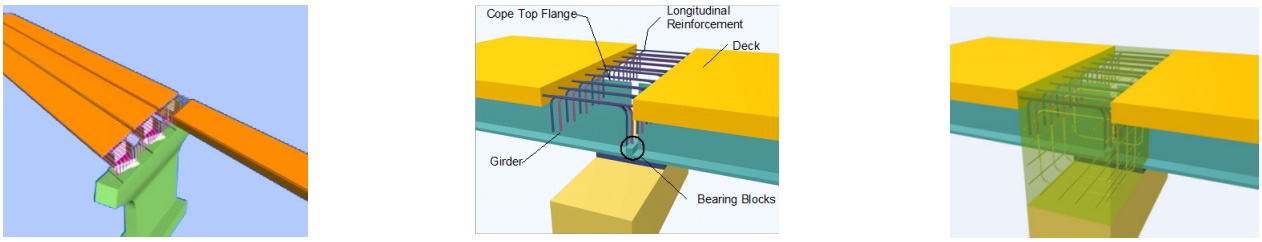
In integral type connections (Figure 8.27a), the pier and superstructure are monolithic with frame action developed between the superstructure and substructure. The advantage of this type of connection is the elimination of bearings. Further, the system provides higher levels of redundancy, especially in highly seismic areas. The longitudinal movement of the bridge superstructure is not affected by making the piers integral with superstructure. However, the longitudinal expansion of the deck must be considered in the design of the pier columns, pier foundations and their connection to the superstructure.

In pinned connections (Figure 8.27b), bearings are used to restrict longitudinal movement. Rotation at the bearing is allowed. Although designated as a pin type connection, typical bridge terminology in which a bearing is not permitted to move longitudinally is designated a fixed bearing, commonly denoted as "F" (Fixed) in traditional design plans. For this connection, longitudinal movement between superstructure and pier is not permitted. Similar to integral type connections, the longitudinal expansion of the deck must be considered in the design of the pier.

In expansion connections (Figure 8.27c), bearings are necessary and are required to accommodate both rotation and longitudinal movements. This detail uses traditional expansion bearings as determined by design requirements.

For the three connection types shown in Figure 8.27, integral (a), pinned (b), and expansion (c), the superstructure is made continuous over the pier. This can be accomplished in one of two ways: 1) the superstructure splices can be positioned such that they are made at or near the dead load inflection points for the continuous bridge, or 2) a continuity splice can be used over the pier. This second option is commonly referred to as simple-for-dead-load, continuous-for-live-load. This construction method is shown conceptually in Figure 8.28 for an integral pier. The

beams are placed as simply supported over the pier; the beams are either spliced mechanically or additional reinforcing is provided for the diaphragm; and finally, a closure pour is made.



(a) **(b)** **(c)**

Figure 8.28. Simple for dead and continuous for live pier detail after the placing of girders (a and b), and after the closure pour (c). (Azizinamini et al. 2008)

The last construction option is the expansion condition using a link slab (Figure 8.27d). A linkage slab is used where the beams are not positively connected, as is the case with the other details shown in Figure 8.27. For this condition, the superstructure is designed and constructed with traditional bearing considerations.

The design considerations for each of these pier cap connections are provided in the following sections.

8.6.2.9.1 Integral Pier Cap

When making the superstructure truly integral with the pier cap, it must be recognized that both positive and negative moments will be introduced to the cap from live loads and other transient loads. As such, sufficient strength needs to be provided through the deck, integral diaphragm, and pier cap. While this type of connection eliminates the need for bearings at the piers and can increase clearance, it introduces more complex forces in the superstructure and the piers (see Figure 8.29).

Figure 8.29. Integral cap as completed.

The longitudinal deflection movement of the foundation and the pier accommodates the total longitudinal movement expected at the top of the integral piers. (See Section 8.6.2.10 for additional information.) When designing the integral cap, the connection between the beams, continuity diaphragm, pier cap, and pier column must be sufficient to transfer the moments resulting from this deflection. Resolution of these forces should be computed by an analytical method or structural model with the ability to properly capture the behavior of the whole bridge system.

8.6.2.9.2 Fixed (Pinned) and Expansion Pier Caps

Similar to the integral pier cap, the superstructure is made continuous over the pier for both fixed (pinned) and expansion pier caps. The difference is that it is not made integral with the pier caps. The connection between the superstructure and the pier is treated with traditional bearings.

Various details are used over the interior supports of multi-span bridges to eliminate joints. One of the concepts implemented by a number of owner agencies for concrete bridges has been the use of simple spans for dead load made continuous for live load (Freyermuth 1969; Oesterle et al. 1989) The girders are simply-supported for dead load, but continuity is achieved with deck steel as negative moment reinforcement over the piers. Also, the girders are made integral with the interior pier diaphragms and commonly positive moment reinforcement is included as shown in Figure 8.30.

Badie et al. (2001) discussed the alternate use of an interior steel pier diaphragm with prestressed girders to speed construction and achieve better overall design economy. The concept of a simple span made continuous has also been

applied to eliminate interior joints and improve the construction speed and design economy for short- and medium-span steel girder bridges (Azizinamini et al. 2008).

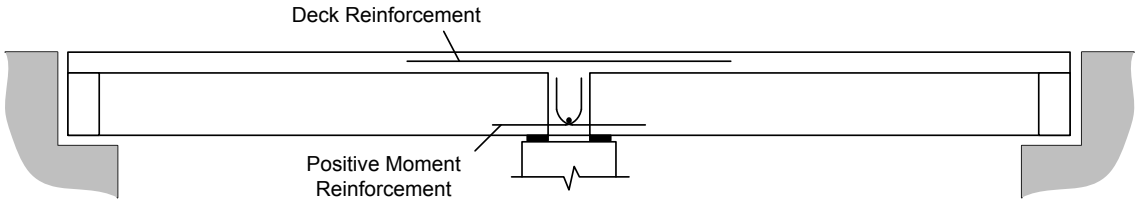


Figure 8.30. Precast, prestressed girders connected with live load continuity.

As discussed in Section 8.6.2.4, attention must be paid to the effects of providing positive moment restraint at the diaphragms. Some simple-made continuous prestressed concrete girder bridges have experienced severe cracking in the girders near the interior diaphragms. One example that has been studied extensively was on the Francis Case Memorial Bridge spanning the Washington Channel of the Potomac River in the District of Columbia (Telang and Mehrabi 2003). The prime cause of this distress was the restraint of upward camber of the prestressed girders under the influence of prestressing and temperature gradient. According to Telang and Mehrabi (2003), "By providing a large amount of positive moment reinforcement at the diaphragms, designers inadvertently make the diaphragm area stronger than the adjacent girder sections, thereby forcing the cracking to occur in far more critical but weaker areas of the girder span." The article states, "In closing, it is important to note that this seemingly simple transformation of simple-span prestressed girders to continuous spans should be attempted with caution, and significant attention must be paid during analysis and design to include loading conditions that can cause counterintuitive behavior such as secondary positive moments at the piers. Most importantly, positive moment reinforcement should be designed and detailed such that any cracking, if it occurs, should be limited to the relatively less critical diaphragm region of this type of structural system." Further discussion of this problem and solutions to avoid it have been published by Oesterle et al. (2004a) and Arockiasamy and Sivakumar (2005) and are discussed in Section 8.6.2.4.

8.6.2.9.3 Link Slab Expansion Pier Cap

A link slab is a type of detail used in conjunction with existing or new bridges having girders that act as simple beams for both dead and live loads. In this type of deck detail, the slab spans continuously over the longitudinal gap between the adjacent span girders while the girders are kept as simple-spans. The length of the deck connecting the two adjacent simple-span girders is called a link slab (Caner and Zia 1998). Link slabs generally require less deck

reinforcement, but have more girder positive moment demands than simple-made-continuous designs. Limited analysis and laboratory experiments were carried out and design recommendations are provided in Caner and Zia 1998. The use of this detail has been very limited due to field observed cracking. In fact, link slabs are not common in snow belt states. A crack is invariably formed due to deck slab rotation as the bridge is loaded with live loads.

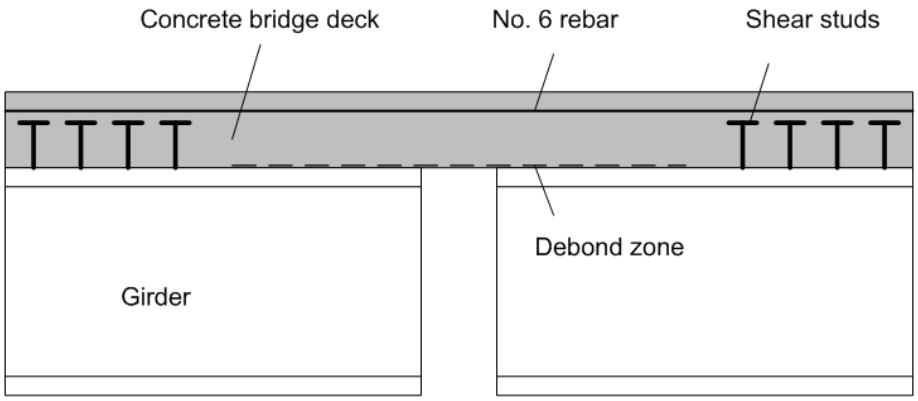


Figure 8.31. Conceptual detail for link slab.

SHRP 2 R19A research studies on the link slab indicated that it offered negligible rotational end restraint to the bridge girders and that the link slab can be analyzed as a beam subjected to the same end rotations as the adjacent girders. The researchers found that under service-load conditions, the link slab would crack primarily due to bending. In addition, prior research by Gastal (1989) and El-Safty (1994) were capable of predicting the forces, stresses and crack widths in the link slab due to thermal and creep and shrinkage effects. Caner (1996) modified the procedures developed by Gastal and El-Safty to properly capture the link-slab actions. All of these solutions were based on beam theory. The reinforcing bar stresses compared reasonably well with the data measured from the experimental tests. The predicted crack widths were somewhat larger than the measured crack widths. The researchers concluded that bending and cracking under live load plus impact are the governing factors that must be considered in the design of the link slab.

Caner and Zia (1998) suggested design of the link slab using only one layer of rebar placed near the top of the deck, but suggested that two layers could be used to improve performance in bridges having horizontal restraints.

8.6.2.10 Design of Integral Piers

As previously discussed in the section on design of integral pier caps the total longitudinal movement expected at the top of integral piers is accommodated by two modes of deformation: longitudinal movement via rotation of the foundation system and flexural deflection of the pier. Pier deflection can be both elastic and inelastic in response.

8.6.2.10.1 Foundation Rotation

For spread footings, Zederbaum (1969) provides an equation to estimate the rotational stiffness of the soil or rock responding to an applied moment:

$$K_\theta = \frac{3E_s I_f}{b} \qquad \textbf{EQ 8.23}$$

Where:

K_θ = rotational stiffness of the foundation
b = one-third of the spread footing width
E_s = compression modulus of the soil or rock
I_f = the moment of inertia of the footing base

For pile-supported and drilled shaft-supported foundations, the rotational stiffness is estimated from the elastic stiffness of the pile or shaft group. Rotation of the foundation can be attributed to the elastic shortening and elongation of the piles or shafts for multiple rows. Note that the elongation and shortening add additional uplift and downward forces, respectively, that must be accounted for in the foundation design. In a single row of piles or drilled shafts, the rotational stiffness is based on the cantilever response of the single row. The length of the cantilever is based on the soil-structure interaction at the foundation and can be based on the assumed or calculated point of fixity for the pile or shaft.

8.6.2.10.2 Pier Displacement

The differential between the pier displacement at the integral cap and the rotation of the foundation is the deflection of the pier column. The resulting design moment can be estimated by the following. First, the expected movement of the superstructure at the pier cap should be calculated, as outlined in Section 8.6.2.3. Alternately, this can be sufficiently approximated by determining the point of zero movement on the bridge and multiplying the end displacement by the ratio of the distance from the fixed point to the pier to the distance from fixity to the end support. Second, assume that 30% of the expected lateral deflection is accommodated by the foundation rotation. Thirty

percent is based on a parametric study that demonstrated foundation rotation can vary from 30% to 80% with an average close to 45%. Third, the anticipated bending moment should be calculated using the equation:

$$M = \frac{6EI_e \Delta b}{H^2} \qquad \textbf{EQ 8.24}$$

Where:

E = concrete modulus
I_e = effective section modulus
Δb = lateral deflection at pier cap
H = height of the pier

Note that the value of the effective section modulus may depend on the applied loading and thus a simultaneous or iterative solution may be required. For fixed (pinned) continuous piers, divide the result of Equation 8.24 by 2 (fixed end moment for a fixed-guided beam is one half that of a fixed-fixed beam). The value of the effective section modulus for reinforced concrete piers can be obtained from Equation 8.25.

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \qquad \textbf{EQ 8.25}$$

Where:

M_{cr} = cracking moment
M_a = applied moment
I_g = moment of inertia of gross section
I_{cr} = moment of inertia of cracked section

8.6.2.11 Design of Wingwalls

The design of wingwalls depends on their orientation relative to the abutment stem, their method of support, and the abutment skew. There are various possible configurations for wingwalls, but the traditional configurations include U-shaped, straight, or flared, the latter being some degree of angle between the other two.

Oesterle et al. (2005) indicates that the U-shape configuration is preferable for wingwalls in that this configuration inherently reduces the passive pressure introduced by the longitudinal movement of the abutment end diaphragms. Additionally, they note that the U-shape configuration conveniently contains the soil behind the abutment and decreases bulging of the embankment soil.

Use of both straight and flared walls leads to the development of passive pressure on the wingwalls as the jointless abutment moves. Oesterle et al. (2005) note that this pressure can be expected to decrease as the distance

from the abutment increases, but that the degradation cannot be effectively predicted. Thus, the wingwalls need to be designed for the same passive pressure as that of the abutment end diaphragm across the length.

For integral and seamless bridges, additional considerations for wingwalls include the loading effect they have on the bridge structure. When cantilevered from the abutment stem, the weight of the wingwalls will create additional torsion and/or bending along the length of the abutment. These forces are resisted by a counteracting negative moment at the end of the external beam or girder.

If wingwalls for integral abutments are placed on supports, such as piles or spread foundation, the support must be able to accommodate the movements of the jointless bridge as well. For this condition, Oesterle et al. (2005) note that the shear and moment developed in the wingwall foundation must be transferred through the wingwall structure to the abutment and superstructure. They also note that U-shaped wingwalls on piles create significant resistance to abutment rotation, which creates partial fixity for beam end moments on the exterior beams or girders. These additional moments need to be included in the design of the connections of the exterior beams to the integral abutment.

8.7 DETAILS

The introduction of different mechanisms for transferring force to the foundations requires that additional details be considered when designing jointless bridges. The following section presents specific details for each jointless bridge type. In this section, the term backwall is used to describe the end diaphragm that resists soil loads.

8.7.1 General Abutment Details for Jointless Bridges

In this section, details that have been used successfully in the past by some states are presented along with general concepts. The figures presented represent recent research efforts and the accumulated experience of several states that have used jointless bridge technology.

8.7.1.1 Integral Abutments

Figure 8.32 shows the overall concept for an integral bridge abutment including the typical layout with the beam, end diaphragm (backwall) and pile cap, all integral. Although it is not necessary in all cases, the beam shown in this figure is sitting on a temporary pedestal to achieve proper alignment before being cast integral with the rest of the abutment. Alternatively, the cap can be stepped to accommodate elevations prior to pouring the backwall. For proper

alignment and to allow for rotations that occur when placing the beam, a small elastomeric pad should be placed at the girder bearing even though each beam will eventually be cast composite with the abutment. Note that the need to design the pads and for what capacity has not been thoroughly studied. The pads need not be designed to meet the criteria for rotational capacity, which is now addressed as a shear strain component. The maximum rotation of the pad is realized during placement of the beam. A reasonable assumption is to design the pad to accommodate only non-composite bearing pressure.

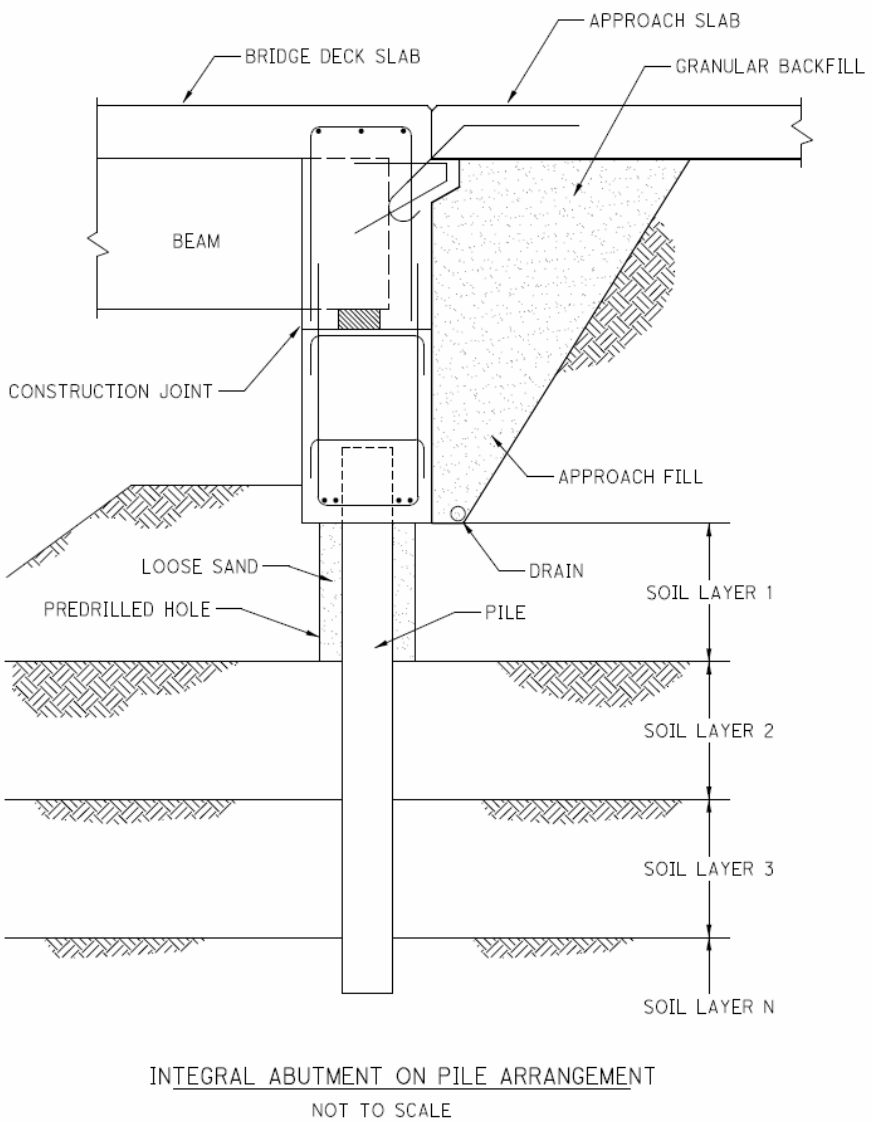


Figure 8.32. General integral abutment concept.

Drainage is also important to avoid ice expansion and removal of the backfill by washout. A drain pipe should be placed at the appropriate location to properly remove any water that might otherwise accumulate behind the backwall.

Additional end diaphragm details are presented in Figure 8.33. Note that an H-pile foundation is shown in the figure; however, each of the foundation types noted in the strategy table in Section 8.5 can be interchanged. The minimum embedment length of 2 ft., within pile cap, should be maintained for H-piles, prestressed piles, and CFT piles, as shown in Figure 8.34. Also shown in the figure is an approximate cap height of 5 ft, typical of the cold weather regions, which allows for embedment below the frost depth and to provide 2 ft between the finished grade and the bottom of the beam. A depth of 3 ft to 3.5 ft is more common where frost depth need not be considered. Another alternative to the holes through the beam shown in the figure is to use threaded inserts, which are preferred by some precast concrete companies to ease securing them in the forms.

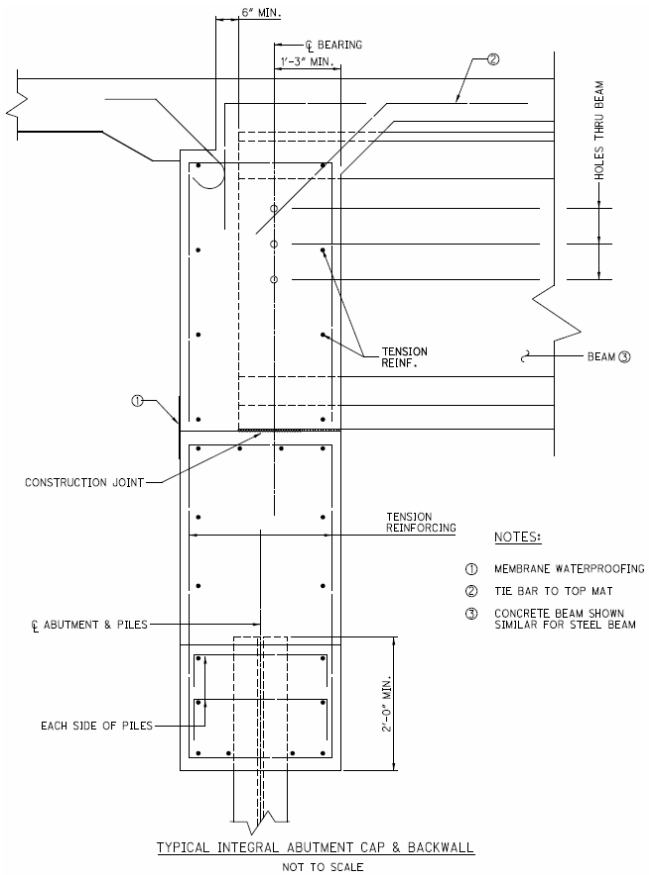


Figure 8.33. Integral abutment details.

Figure 8.34 is an adaptation from an Ohio DOT standard drawing showing a prestressed concrete beam. Now a standard detail for most prestressed girders includes providing holes through the beam for reinforcing. This reinforcing provides continuity though the backwall for bending and limits the differential deflection between the superstructure and backwall where tension forces develop in the top portion of the web.

In contrast to the design recommendations in Section 8.6.2.7, the state of Ohio allows for rotation in the backwall across the construction joint instead of designing rebar to transfer the forces through the stem. The configuration is used to accommodate the rotation of the superstructure as shown in Figure 8.34. At the centerline of bearing, reinforcing is crossed at the bearing pivot location, and expansion joint material is placed so as to permit a limited amount of rotation. Note that Ohio limits the length of their bridges with integral abutments to 250 ft, so consideration of this limit should be made before adopting this detail for other bridges.

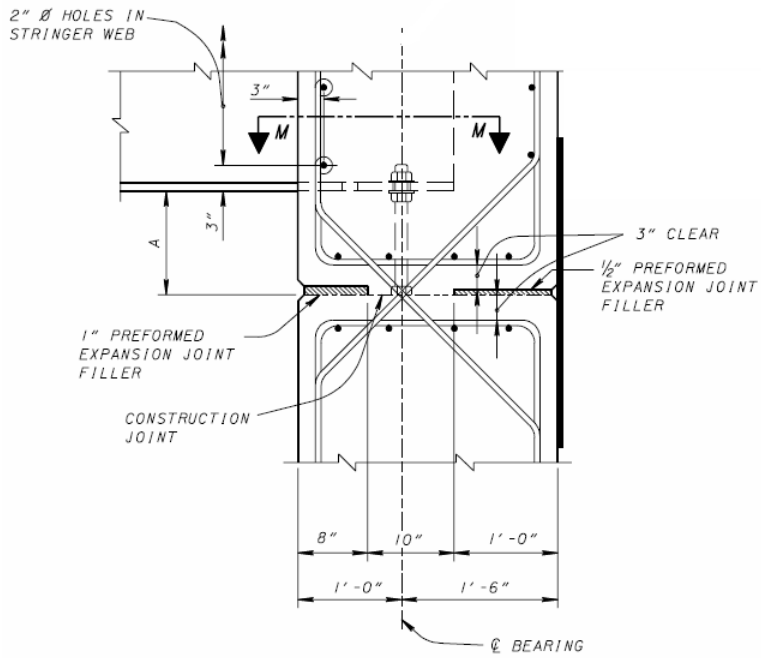


Figure 8.34. Integral abutment rotation detail.

Figure 8.35 presents another standard integral detail drawing from the New York State Throughway Authority, which shows a steel beam connection. This detail is more typical of DOT design standards in that the reinforcing is continuous across the construction joint. Additionally, when comparing this detail with Figure 8.33, although both details have had repeated success, there are two obvious differences: 1) Figure 8.33 shows a bent hook bar connecting the approach slab, whereas Figure 8.35 shows that continuity is maintained by a straight bar connecting the approach slab to the deck; and 2) the Figure 8.33 detail utilizes a shear key, while the Figure 8.35 detail relies solely on the continuity of the reinforcing across the construction joint. Each detail has demonstrated success in application and the designer should consider which option may be more appropriate for each bridge's unique situation.

For more information on backwall detailing see Section 8.7.2.

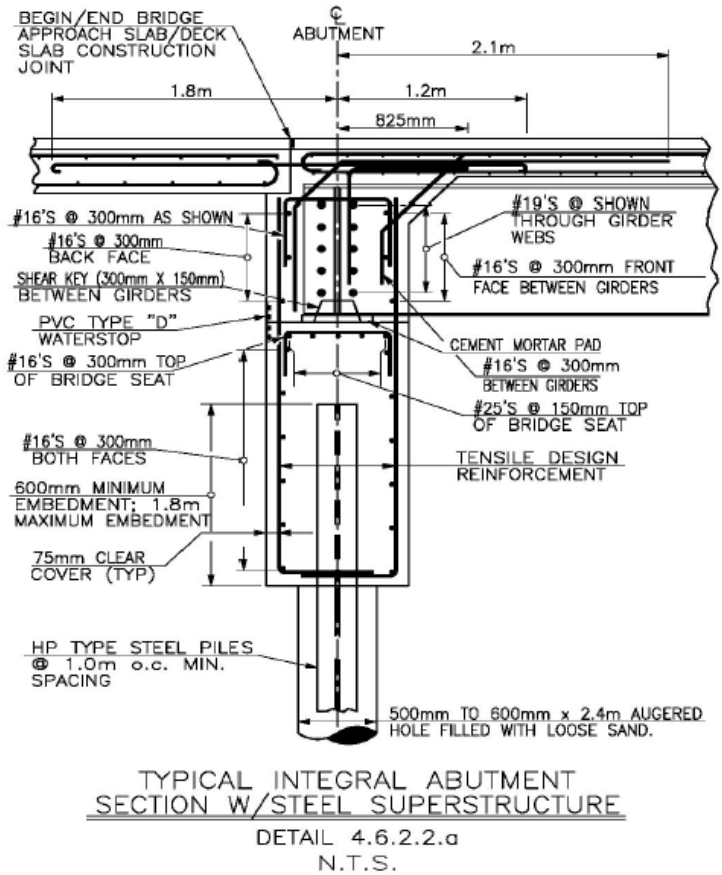
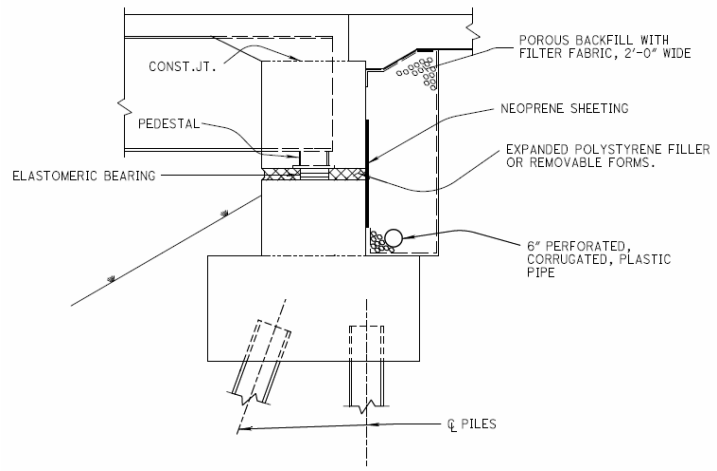


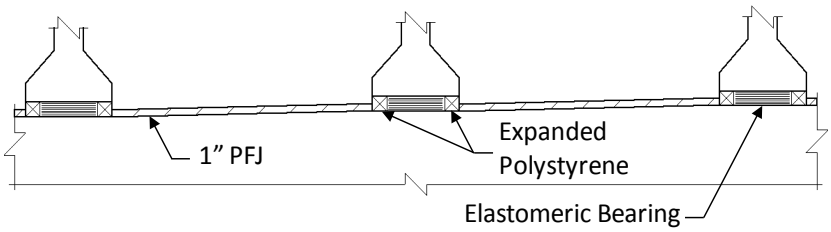
Figure 8.35. Integral abutment details. (New York DOT)

8.7.1.2 Semi-Integral Abutments

Figure 8.36 shows the overall concept for a semi-integral bridge abutment. It includes the typical layout with the beam and end diaphragm (backwall) cast integral.



(a) Section (showing pedestal).



(b) Elevation (with no pedestal).

Figure 8.36. General semi-integral abutment concept.

Drainage and porous backfill are necessary for the same reasons as for integral abutments—formation of ice and integrity of the backfill. In semi-integral abutments, two bearing strategies have been used successfully: 1) the pile cap may be cast level and the superstructure superelevation can be accommodated through the use of bearing pedestals, and 2) the second method is to step the pile cap. In this case, the polystyrene filler must be used on both the top of the cap and on the sides of the step to allow for movement. Due to the nature of the superstructure movement it is recommended that the first case with pedestals be used for locations of high skew (larger than 20°) and bridges on a curve. If it is desired to inspect the bearings during the life of the bridge, removable filler material should be placed in front of the bearings.

Figure 8.37 shows the successful detailing strategies that have been used in various states. The foundation shown is for a drilled shaft, but other foundation types are equally applicable. Similar to integral abutments, dowel holes are placed through the beam or girder. Unlike integral abutments, bearings are used to accommodate movement between the superstructure and the foundation. Efforts must be made to seal the gap between the cap and backwall yet still accommodate movement. This seal has traditionally been a preformed filler surrounding the bearing area and a layer

of waterproofing applied to the rear face of the seam prior to placing the backfill. For more information on backwall detailing see Section 8.7.2.

Other than the backwall and treatment of the bearing area, detailing for the rest of a semi-integral abutment is the same as traditional design.

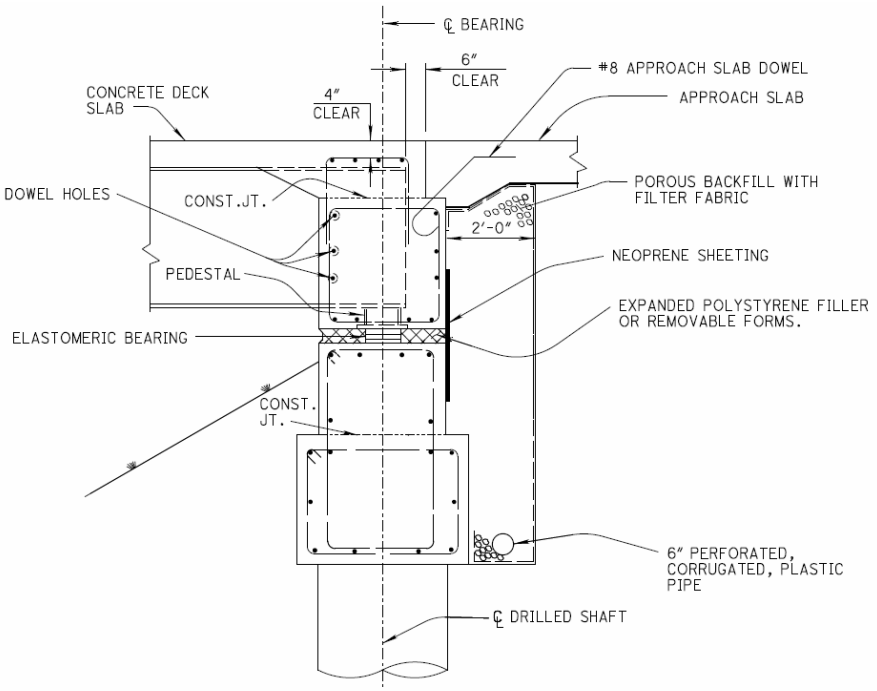


Figure 8.37. Semi-integral details.

Figure 8.38 shows an alternate detail used in instances in which the diaphragm is extended and a lip is dropped down over the pile cap. This detail replaces the neoprene sheeting that provided the barrier between the porous backfill and the expanded polystyrene filler surrounding the bearings. Preformed elastomeric material is placed between the extended diaphragm and the abutment stem.

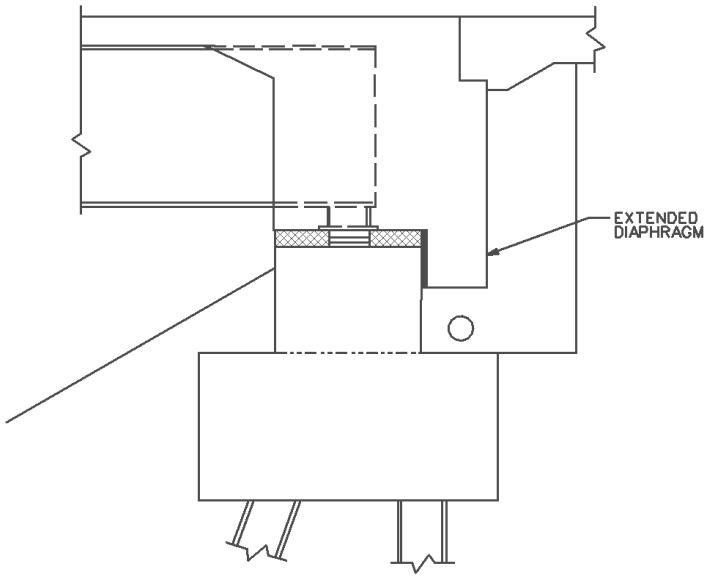


Figure 8.38. Semi-integral details with extended diaphragm.

8.7.1.3 Seamless Abutments

Detail recommendations for the transition zone are not well established and thus no standard details are available for reference. However, the recommendations for the abutment cap and backwall are the same as those presented in Section 8.7.1.1.

8.7.2 Pile Cap and Backwall

Oesterle et al. (2005) recommend that vertical reinforcement for the moment from the soil load be distributed with 75% of the bars within 25% of the beam spacing on either side of the beam. Furthermore, for crack control they recommend the center-to-center spacing of the flexural reinforcement not exceed (in inches):

$$s \le \frac{540}{f_s} - 2.5c_c \quad or$$
$$s \le 12\left(\frac{36}{f_s}\right) \qquad \textbf{EQ 8.26}$$

Where:

c_c = clear cover from the nearest surface in tension

f_s = calculated stress (ksi) at service load, or alternately as $0.60F_y$

This limitation is taken from *ACI 318-05* Section 10.6.4 rules for the distribution of flexural reinforcing to control cracking in one-way slabs. Further commentary on this requirement can be found in that section.

8.7.3 Sleeper Slab

A sleeper slab is appropriate for all integral or semi-integral bridges and is placed at the roadway end of the approach slab. The intent of this slab is to provide a relatively solid foundation for the far end of the approach slab and to provide a location for limited expansion and contraction (see Figure 8.39). Although no formal design is suggested, a typical suggested detail has been provided by Wasserman and Walker (1996).

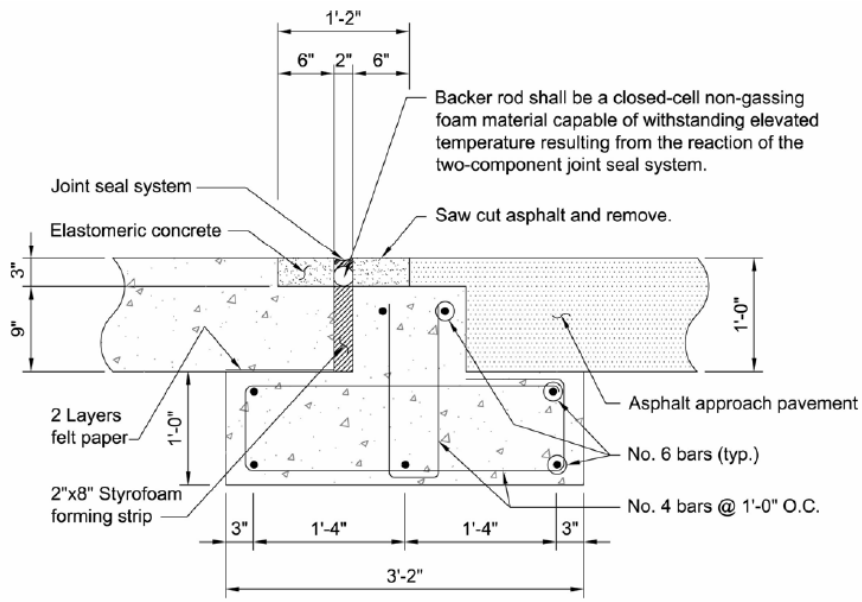


Figure 8.39. Suggested helper (sleeper) slab details. (Wasserman and Walker 1996)

A potential problem with Figure 8.39 is that it presents a potential for cracking in the approach pavement where it suddenly transitions to the thin piece above the sleeper slab. Whereas this might ease final grading, it is preferable to have the stem of the inverted "T" of the sleeper slab extend to final grade and thus avoid any sharp transitions.

The state of New York has adopted this sleeper slab detail and modified it to marry the adjoining pavement design based on the type of surfacing used. Figure 8.40 and Figure 8.41 show the sleeper slab for concrete and asphaltic pavements respectively. In these figures, note how the state formed the joint such that both the pavement and approach slab are both graded at full depth up to the sleeper slab that provides the transition.

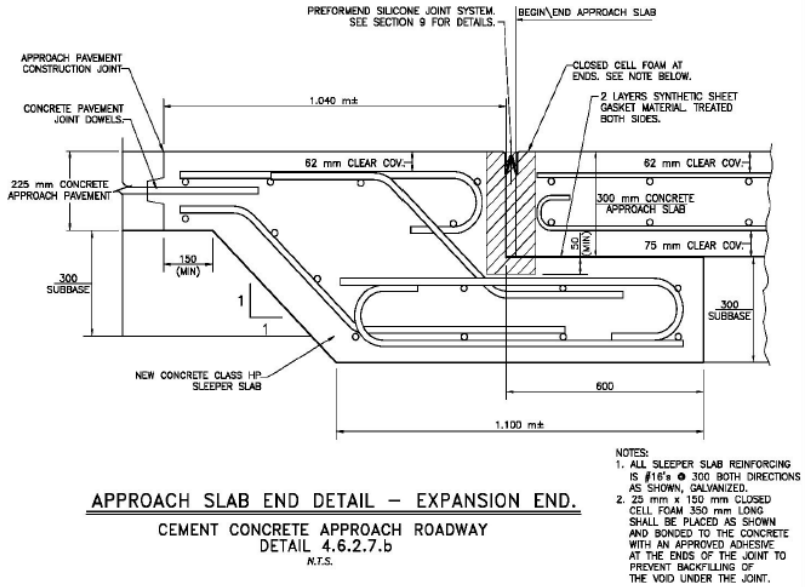


Figure 8.40. Sleeper slab with concrete pavement approach. (New York DOT)

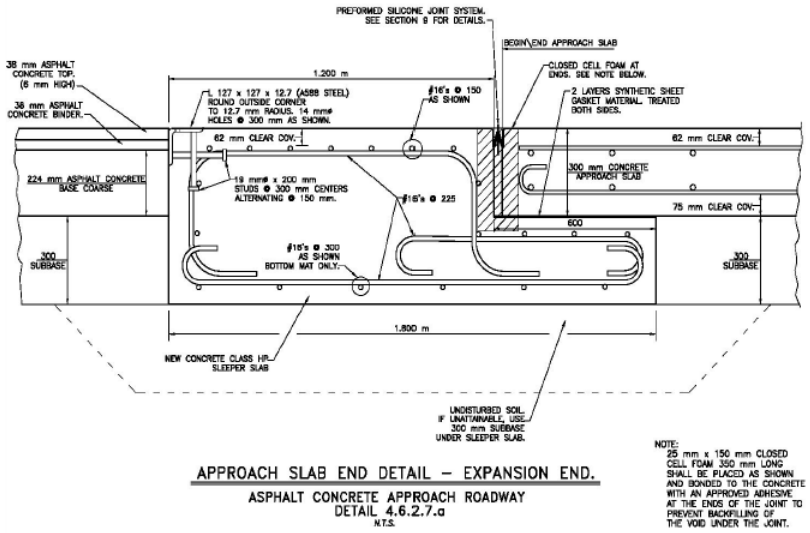


Figure 8.41. Sleeper slab with asphalt pavement approach. (New York DOT)

The location of the sleeper slab should be placed so that the entirety of the slab is outside the failure plane as discussed in Section 8.6.2.8.1.

8.7.4 Details for Skewed and Curved Bridges

Transverse movements of integral abutments associated with large skews or horizontal curves should be accommodated by the details for barrier walls, drainage structures, and the ends of the approach slabs. In addition, the

foundation and pier structure stiffness will likely be significant for movement parallel to the pier cap. Therefore, it is recommended that the connection between the bottoms of the girders and diaphragms and the pier caps be flexible in this direction. This approach, however, may not be appropriate for seismic design, in which case the design of the diaphragms should consider the interior pier restraint of the rigid body rotations that result from passive abutment restraint of longitudinal thermal expansion.

8.8 CONSTRUCTION

8.8.1 Construction Stability

Due to concerns about the repetitive bending stresses on the pile, it is recommended that no seam (weld) be placed at the top 30 ft of the pile. This will ensure proper ductility and eliminate the possibility of having a poor fatigue detail near the region of higher bending response. Additionally, this will better ensure proper alignment of the pile at the cap.

The order of construction is also important as described in section 8.8.4.

8.8.2 Utilities

Non-flexible utilities should not be permitted to pass through integral and semi-integral abutments. Multiple DOTs report experiencing problems in which the flexibility of the integral cap creates issues with rigid utilities. Only utilities that are able to sufficiently flex with the movement of the integral abutment should be permitted, but it is preferable to locate all utilities adjacent to the bridge structure.

8.8.3 Cracking Control

Vertical cracks have often been found at the bottom of diaphragms between precast beams over the piers, in the positive moment connection region near the external (fascia) girders. On the interior girders encased in the diaphragm, spalling of the diaphragm has been observed near the bottom flange. This spalling resulted from the bottom flange slipping outward (away from the diaphragm) due to the end rotation of the girder associated with creep and thermal changes. These vertical cracks in the diaphragm and end rotation of the girders serve to relieve tensile stresses due to creep, shrinkage, and thermal movement and are not detrimental to the integrity of the structure. Attempts to control this cracking through over-reinforcing may result in cracks in less desirable locations.

Horizontal cracks and efflorescence have been found on the forward face of integral abutments at the construction joint on top of the pile cap. This can be alleviated by placing adequate sealing from water behind the stem across the construction joint.

Settlement of the approach slab is common and this can cause cracking and further damage to the barrier rail. Rails that are attached to both the deck and approach slab should be jointed to accommodate the differential settlement.

8.8.4 Construction Sequencing

Guidelines for concrete bridge deck materials and construction to control transverse cracking in concrete bridge decks are presented in *NCHRP Report 380* (Krauss 1996). Among the issues that affect deck cracking are weather, time of placement, curing, vibration, finishing, loads, and placement sequencing. Certain current practices are presented here for jointless bridges.

For jointless bridges the construction sequence should generally be as follows:

1. Embankment should be completed prior to pile driving and allow for consolidation (if required).
2. Piling should be placed and pre-drill holes filled and forms constructed (if used).
3. Abutments and wingwalls should be constructed to the elevation of bearing seat.
4. Semi-integral elastomeric bearings should be set; or for integral beam pads should be set allowing for rotation from beam and deck dead load.
5. Beams should be set.
6. The deck slab and the integral backwall should be cast. The ends of the slab should be poured last in order to minimize locked-in stresses at the supports.
7. Drainage and backfill should be placed behind the abutments after the deck has achieved the appropriate strength. It is important that the backfill be placed simultaneously behind each abutment so the bridge is not inadvertently shifted in the unsupported direction.
8. The approach slab should be cast, ideally with the bridge in the thermally contracted position (i.e., early morning). This avoids putting the slab into tension until the concrete has gained sufficient strength.

It should be emphasized that simultaneous placement of the abutment backfill (7) is particularly important for semi-integral abutments. The reason for this emphasis for semi-integal bridges is that the superstructure sits on flexible bearings rather than being positively attached to the abutment, and is more likely to move due to the pressure from the compacting procedures.

8.8.5 Fill Compaction

Construction can follow normal compaction procedures as specified by the owner agency except as noted in the construction sequencing. Fill compaction has been modified and adjusted using several variables, including the use of specialized material. However, general experience has indicated that properly compacted normal fill material is sufficient for jointless bridge construction, and proper drainage behind the backwall is more important.

8.9 MAINTENANCE AND REPAIR

8.9.1 Problems with Jointless Construction

Although adoption of integral-type bridges will eliminate some of the more troublesome problems associated with jointed bridges and yield significantly more durable structures, they will not eliminate endemic highway construction problems that are somewhat related to accelerated construction, all-weather construction, marginal construction supervision, and other construction issues identified in Chapter 3 on materials and Chapter 4 on bridge deck.

Transverse and diagonal deck slab cracks, stage construction issues, lateral rotation of superstructure, erosion of embankments, marginal quality of structure movement systems, and other problems have appeared to trouble design, construction, and maintenance engineers. Except for early-age deck slab cracking, these problems are generally the result of failure to anticipate and apply typical design and construction provisions to achieve trouble-free construction and more durable structures.

8.9.1.1 Deck Cracking

Diagonal deck slab cracks located at acute corners of integral-type bridges are occasionally reported. When constructing integral-type bridges, stationary abutments and moving superstructure must be joined together by cast-in-place continuity connections. Consequently, these connections could be stressed and cracked if a substantial temperature drop were to occur during initial concrete setting, or if concrete placement sequences were not suitably controlled. To address this problem, one or more of the following procedures should be used: place continuity

connections at sunrise, place deck slab and continuity connections at sunrise, place continuity connections after deck slab placement, or use crack sealers.

8.9.1.2 Lateral Rotation of Semi-Integral Bridges

One of the primary aspects of semi-integral bridges that must be considered and addressed is the design and proper orientation of guided bearings for the superstructure of skewed bridges. Unfortunately, many of the retention devices currently being used are not fully functional because of friction and binding and consequently, the long-term stability of some abutments, especially those not supported by rigid foundations, may not have been provided for effectively. However, it appears inevitable that this aspect of the semi-integral bridge concept will be improved when bearing manufacturers and bridge design engineers combine their expertise to design and manufacture more functional structure movement systems for these applications.

8.9.1.3 Approach Slabs

Shortly after the state of Ohio adapted the integral concept to continuous steel beam bridges in the early 1960s, slab distress was experienced. Where the bridges in question were constructed adjacent to compressible asphalt concrete approach pavements, approach slab seats at the ends of bridge superstructure were found to be fractured, approach slabs had settled, and the vertical discontinuity in the roadway surface at the approach slab/superstructure interface was hindering movement of vehicular traffic.

8.9.1.4 Drainage

Washout has been noted on several existing structures in which drainage was not properly designed or maintained, including some where the piles became exposed. It is imperative that proper drainage material including geotextiles and perforated piping be placed behind the abutments. The preferred alternative is to direct water away from the bridge approach, but it is acknowledged that this can be difficult to accomplish in many cases.

Additionally, improper drainage can lead to washout at one end of the bridge and not the other. For semi-integral abutments, this leads to an unbalanced soil pressure, which can lead to additional maintenance issues at the bearing locations.

Drainage can also affect settlement of the sleeper slab and create settlement of the approach slab. It is recommended that runoff be intercepted or diverted so that it does not reach the end of the approach slab.

In regions that experience freezing temperatures, proper drainage is also important to minimize the potential for frozen soil behind the abutment. The magnitude of the potential restraining force is unknown for frozen soil, but it will be minimized with proper drainage (Briaud et al. 1997).

8.9.1.5 Cycle-control Joints

Probably the most significant unresolved problem regarding integral and semi-integral bridges is the availability of cost-effective functional and durable cycle-control joints, which are the moveable transverse joints used between approach slabs of integral-type bridges and approach pavements. The usual pavement movement joints, composed of preformed fillers, are currently being used for the shortest bridges. For the longest bridges, finger-plate joints with easily maintainable curb inlets and drainage troughs have been successfully employed. However, for intermediate-length bridges, development of a suitable cycle-control joint is still in the evolutionary stages.

8.9.2 Deck Replacement

Figure 8.42 shows what can happen when the proper procedures and sequences for deck replacement and integral abutment backfilling are not followed. It should be anticipated that large compressive forces are acting on the whole structure as a result of soil pressure on the abutments and restrained expansion of the girders. In order to ensure the global stability of the structure, one of two procedures must be followed. The first procedure, which should always be used for whole deck replacement, is to use proper construction sequencing as follows:

1. Remove the approach slab.
2. Remove backfill to the bottom of the stem for integral or to the bottom of the end diaphragm for semi-integral abutments. Excavation should be done simultaneously behind both backwalls.
3. Remove the deck.
4. Replace the deck according to the guidance provided in Section 8.8.4.

The second option is to calculate the stress applied by the passive pressure of the abutment backwalls. This force can then be applied to the superstructure with portions of the deck removed to check the stability of the system and each structural item that might be affected by the removal of the deck. This includes checking both local and global buckling stability. It is recommended that this be used only for partial width deck repair.

Figure 8.42. Buckled girder flanges due to improper deck sequencing.

8.9.3 Bearing Replacement for Semi-Integral

An additional factor when detailing semi-integral jointless bridges, should bearing repair or replacement be required, is to incorporate appropriate features at the initial design stage to facilitate superstructure jacking. In general, since superstructure and abutments in semi-integral bridges are separated by elastomeric bearings, it would be easy to place flat jacks between the abutment and the end diaphragm to raise the superstructure and approach slab and allow replacement of the bearing.

8.10 RETROFITS

A large percentage of existing bridges are simple span bridges that rely on expansion joints at piers and abutments to accommodate longitudinal movements. Most of the deficient bridges in the United States include these jointed structures, which require upgrade and repair. Retrofitting existing jointed bridges to jointless ones is highly recommended.

The following considerations are required in integral conversion (Leathers 1990):

1. The existing structural elements should be able to properly function without the expansion joint.
2. Movement calculation should be based on the *LRFD Specifications*.
3. Continuity can be achieved by making either the deck or the girders continuous.
4. All obsolete and/or deteriorated bearings should be replaced with elastomeric bearing devices.
5. If the abutment is unrestrained, a fixed integral condition can be developed for many of the shorter bridges. Abutments that are free to rotate are considered unrestrained, such as a stub abutment on one

row of piles or an abutment hinged at the footing. A semi-integral condition can be developed for restrained abutments.

8.10.1 Details over the Pier

Two practical options that can be used with or without integral abutments are available for retrofitting existing jointed bridges into jointless bridges.

1. Provide beam continuity for live load only. In this case, the negative moment continuity is provided over the piers, with or without positive moment continuity at these locations.
2. Only provide deck slab continuity. In this option, although the deck is continuous, beams are technically, simply supported. This method involves removing some length of slab at the ends of the adjacent beams, splicing the existing reinforcement and adding new bars, then recasting that part of the deck.

8.10.1.1 Link Slab

When retrofit of an existing open joint is considered, the following approach may be used as shown in Figure 8.43. (Note for this detail only the deck is made continuous.)

1. Remove concrete as necessary to eliminate existing armoring.
2. Add negative moment steel at the level of existing top-deck steel sufficient to resist transverse cracking.
3. Reconstruct with regular concrete to original grade.

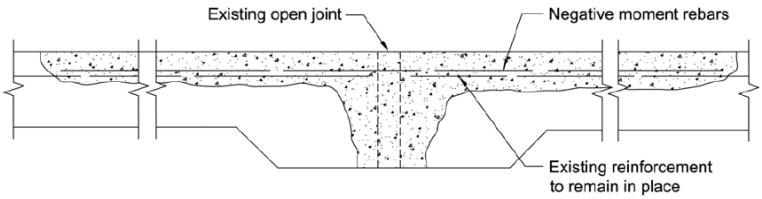


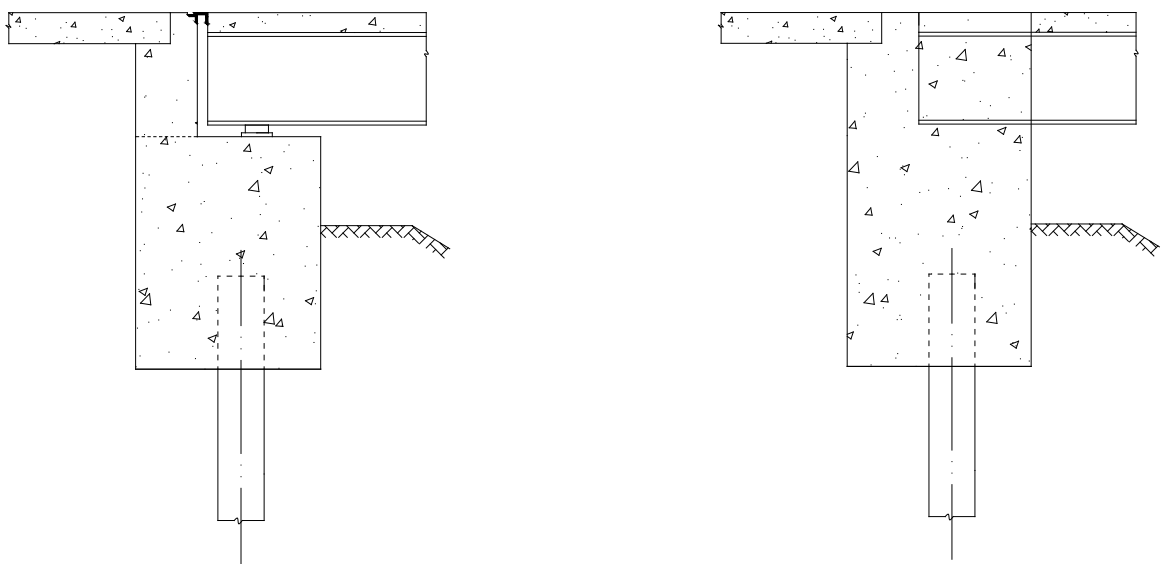
Figure 8.43. Integral conversion at piers. (Leathers 1990)

Since the deck slab would be exposed to longitudinal flexure due to rotation of beam ends responding to the movement of vehicular traffic, cracks will occur over the link slab. However, for short and medium span bridges, the deck cracking associated with such behavior is preferred over long-term consequences associated with open moveable deck joints or poorly executed joint seals.

8.10.2 Details over the Abutment

For existing stub abutments with a single row of piles, the following procedure shown in Figure 8.44 should be used (integral abutment retrofit).

1. Check the capacity of piles and pile-cap connection for the expected movement.
2. Remove the approach slab, and excavate backfill to the elevation of existing ground on front face.
3. Demolish the existing backwall to top of bridge seat. Cast reinforced concrete around beam ends.
3. Then provide drainage, backfill, and new approach slab behind the new abutment.

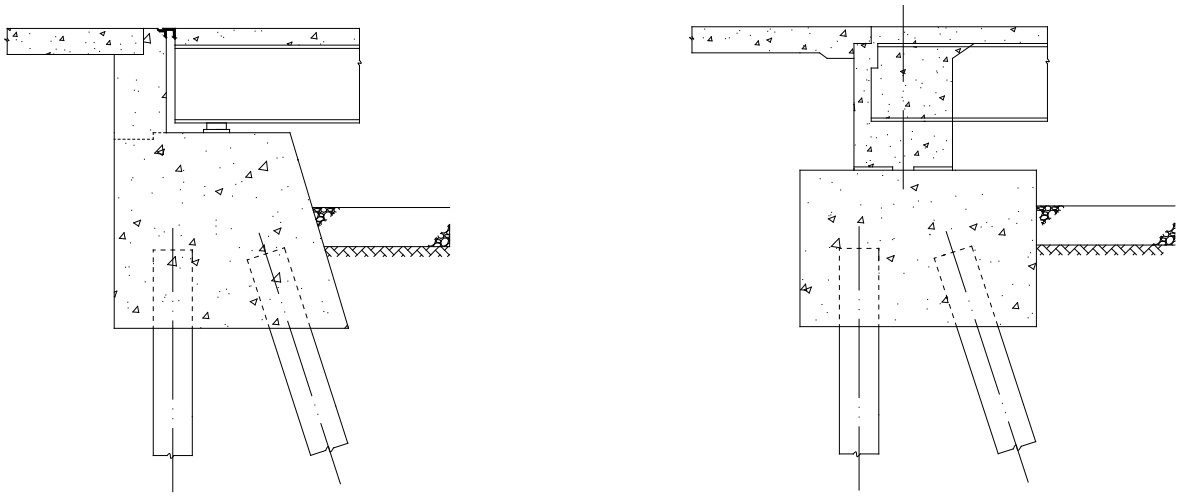


(a) **(b)**

Figure 8.44. Conversion of a bridge with moveable deck joints at the superstructure-abutment interface with integral abutment (a) before conversion, and (b) after conversion.

For existing stub abutments with rigid foundation or existing full height wall abutments, the following procedure should be used (semi-integral abutment retrofit).

1. Remove the approach slab and excavate the backfill to the elevation of existing ground on front face.
2. Remove the existing abutment backwall to the top of bridge seat.
3. Provide a sliding surface between the pile cap and the abutment stem which is cast integrally with the beam ends and approach slab.
4. Provide details for both horizontal and vertical sliding joints using lateral guide bearings, sheet seals, and drainage and backfill.



(a) (b)

Figure 8.45. Conversion of a very short span bridge with moveable deck joints at the superstructure-abutment interface with integral abutment (a) before conversion, and (b) after conversion.

8.10.3 Converting Jointed Bridges to Jointless Bridges

General experience has shown that most common bridge types can be converted to jointless bridges to enhance their performance with the same goal as new construction (i.e., joint elimination). Examples of candidates that have already been converted are pin-and-hanger bridges and multi-span, simple span bridges for both steel and concrete superstructures.

Several states have had success converting old pin-and-hanger expansion joints to a bolted full moment connection, thus eliminating the expansion joints.

The state of New Mexico has presented several case studies (Maberry et al. 2005). In one project, they converted simple span concrete girders by incorporating a link slab. The project demonstrated that attention must be paid to the bearings. The greatly increased expansion that would transfer to the outer bearing locations was overlooked by the retrofit assessment. Subsequently, the resulting expansion loads were absorbed by the pile caps, which quickly deteriorated.

The key to any conversion is the ability of the bridge to withstand the new continuity loading and expansion demands introduced by the changed load path. Due to the complex nature of the converted structure, it is recommended that conversions be treated with the same level of analysis as required for a new design.

CHAPTER 9

EXPANSION DEVICES

Bridge elements are subjected to various loads, including traffic and environmental loads that result in movement of bridge elements. One of the key factors affecting the service life of bridges is how to address thermal expansion and contraction of the bridge elements. This design issue is handled in two distinct ways. One option is to provide expansion joints at designated locations within the superstructure. By doing so, the designer forces the entire thermal deformation to take place at these discrete locations. The other option is to make the superstructure and deck continuous and assume that the thermal movement will be accommodated by the flexibility of the substructure, such as with the use of integral abutments. In such cases, the joint is usually moved away from the bridge abutment and placed near the end of the approach slab.

This chapter provides essential information for enhancing the service life of bridge expansion joints. The process begins with developing an understanding of the types of bridge joints that may be considered for a particular project. The viable expansion joint types are further evaluated for factors that adversely affect their service life, and individual strategies are then developed to mitigate these adverse effects. The overall strategy selection is then developed, blending these individual strategies that are sometimes in conflict. The components of an overall strategy should include:

- Identification of appropriate design methodologies,
- Selection of durable expansion device types considering life-cycle costs,
- Specification of best practices for construction, and
- Development of an effective maintenance plan.

Section 9.1 provides a general description of the different types of expansion devices used in practice, as well as their various advantages and disadvantages. Section 9.2 discusses reported potential factors that could affect the service life of the different expansion devices, and Section 9.3 provides strategies for enhancing expansion device service life.

9.1 DESCRIPTION OF VARIOUS EXPANSION JOINT DEVICES

There are many types of expansion joints to better meet the needs of a wide array of bridge types. However, it is important for the designing engineer to select the proper expansion joint, as this is the most crucial step in enhancing the service life of an expansion device. When selecting the proper expansion device it is important to consider the total anticipated displacements, bridge skew, and other special considerations, such as proprietary system requirements and accommodations for deicing agents, etc.

Following are characteristics of an ideal expansion joint (Lee 1994):

- Accommodate thermal expansion and contraction,
- Accommodate movement due to traffic-induced loads,
- Provide a smooth ride,
- Prevent the creation of hazards and safety issues,
- Accommodate needs during snow removal,
- Prevent leaking of moisture and other chemicals to elements below the superstructure,
- Have a long service life,
- Be maintenance-free or require minimal maintenance, and
- Be cost effective.

There are several ways to categorize expansion joint systems. One approach is to divide the various joints based on the maximum longitudinal movements that they can accommodate. There are many different joint types and often the terminology used to describe them differs from agency to agency or manufacturer to manufacturer. Expansion joints can be classified into three categories based on the maximum longitudinal movements as follows:

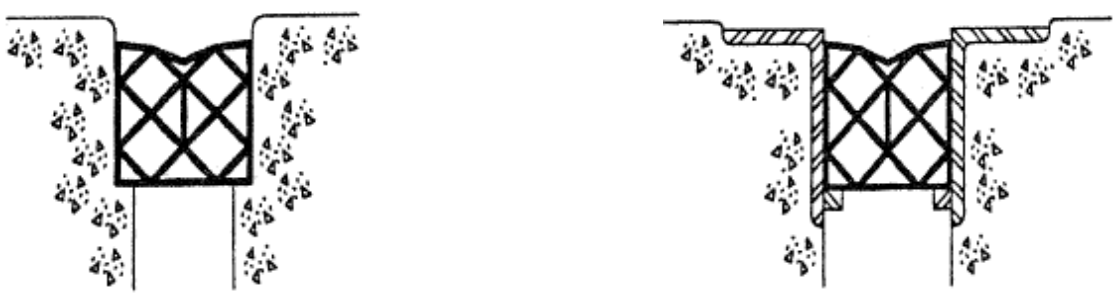
1. Expansion joints capable of accommodating small longitudinal movements (less than about 3 in.),
2. Expansion joints capable of accommodating medium longitudinal movements (between 3 and 5in.), and
3. Expansion joints capable of accommodating large longitudinal movements (in excess of 5 in.).

Descriptions of the various joint types follow.

9.1.1 Expansion Joints with Small Movement Capabilities

9.1.1.1 Compression Seal

In this type of expansion joint, the opening throughout the entire bridge deck width is filled with a neoprene elastomeric section and forms a waterproof joint, as shown in Figure 9.1 (Purvis 2003). The neoprene elastomeric is installed by squeezing and inserting the seal into a preformed joint opening (Chang and Lee 2002). Steel or special concrete materials may or may not be used to strengthen the joint face. To facilitate the movement of the joint, they are usually made semi-hollow with internal diagonals and vertical neoprene webs (like a truss structure). However, some types are made of closed section foam. The squeezing of the neoprene elastomeric section is meant to ensure that it will be in compression, thereby sealing the joint. Figure 9.1(b) shows an armored version of a compression seal joint. A recent trend within transportation agencies has been to eliminate the horizontal leg of the angle used and to attach the vertical leg to the deck using shear studs. This expansion joint type allows total movement of up to 3 in.



(a) Plain **(b) Armored**

Figure 9.1. Compression seal. (Purvis 2003)

9.1.1.1.1 Features

- Movement from 0.25 in. to 3.0 in. is allowed.
- The seal needs to be made the right size for the needed range of movement.
- The semi-hollow cross-section with the internal diagonal and vertical neoprene webbings forms a truss action, which allows the section to compress toward the sides of the joints and become watertight.

- Sometimes foams are used instead of semi-hollow cross-sections, which form a closed section; however, the movement range is the same.
- The compression seal fits into the sides of the joint using a lubricant material that also functions as an adhesive that bonds the seal to its place.
- Splices should be avoided in this type of seal.

9.1.1.1.2 Advantages

- Some agencies report minimal required maintenance and a good life span for these seals.
- This type of joint is recommended mostly for areas with moderate temperature extremes.

9.1.1.1.3 Disadvantages

- It is reported that large sustained compressive movement forces air from the seal material, which may not recover when the joint opening expands.
- Some agencies do not report good performance for this type of seal, citing damage due to debris, traffic, and snowplows.
- Leakage has also been reported shortly after installation.
- It has been reported to dislodge and lose compression over time.
- This type of joint is not recommended for extreme temperature ranges.
- The seal needs to be installed at relatively low temperatures or it is more difficult to install and prone to damage.
- According to some reports, the compression seal loses its capability to retain initial compression recovery due to loss of resilience, particularly if the movement range is large.

9.1.1.2 Poured Sealants

The poured sealant systems used today mainly consist of two parts: a polyethylene foam backer rod and a pourable silicone sealer (Purvis 2003), as shown in Figure 9.2. The backer rod keeps the sealant from spilling through the opening. During installation, it is very important to keep the joint edges clean so that the silicone sealant bonds tightly. The performance of this type of joint will improve if it is poured when the ambient temperature is at the middle of the historical range.

Traditionally, this type of joint could handle movements up to 3/16 in., but newer systems can accommodate up to 3 in. of movement (Purvis 2003; Chang and Lee 2001).

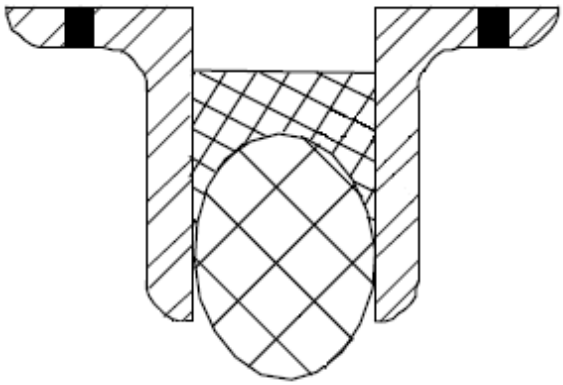


Figure 9.2. Poured sealant joint type.

9.1.1.2.1 Features

- Traditional systems were good for shorter spans where the movement need is 3/16 in (5 mm) or less. However, newer systems accommodate larger movements depending on the sealant material.
- The most common sealant used today is silicone.

9.1.1.2.2 Advantages

- This type of joint is maintaining popularity because, according to reports, newer systems are serving well if installed properly.
- The silicone polymer used is self-leveling and rapid-curing.
- The performance of this type of seal is not affected by joint walls that are not perfectly made.

- They are easy to repair.
- It is easy to remove a portion of the sealant, to clean the sides of the joint, and to refill the joint.
- The quick maintenance process makes traffic disruption very short.

9.1.1.2.3 Disadvantages

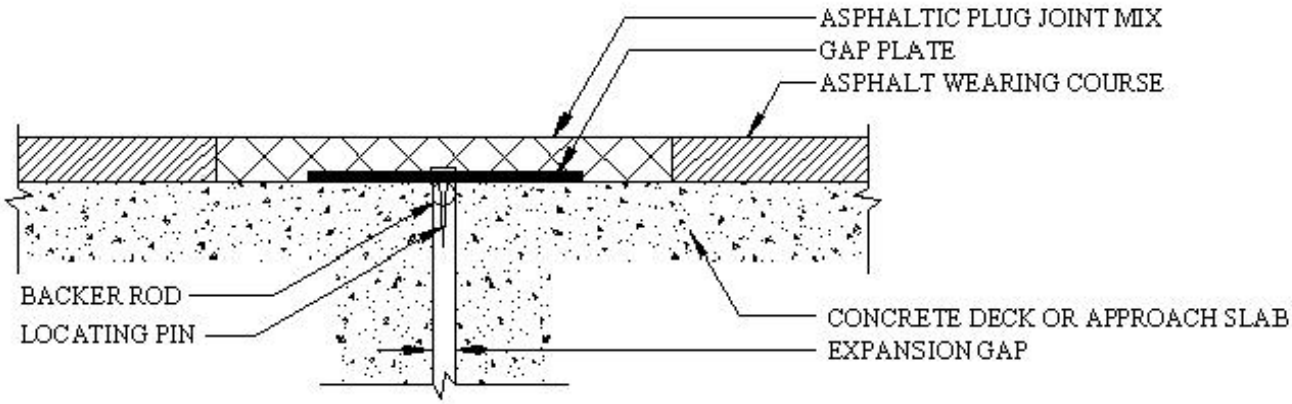
- The first poured seal materials were heated asphalt or coal tar products, which did not perform satisfactorily for many transportation agencies.
- Polymer materials used to have problems such as debonding, splitting, and damage from noncompressible debris.
- Earlier, damage to the deck edge also caused the joint sealant to fail.

9.1.1.2.4 Requirements

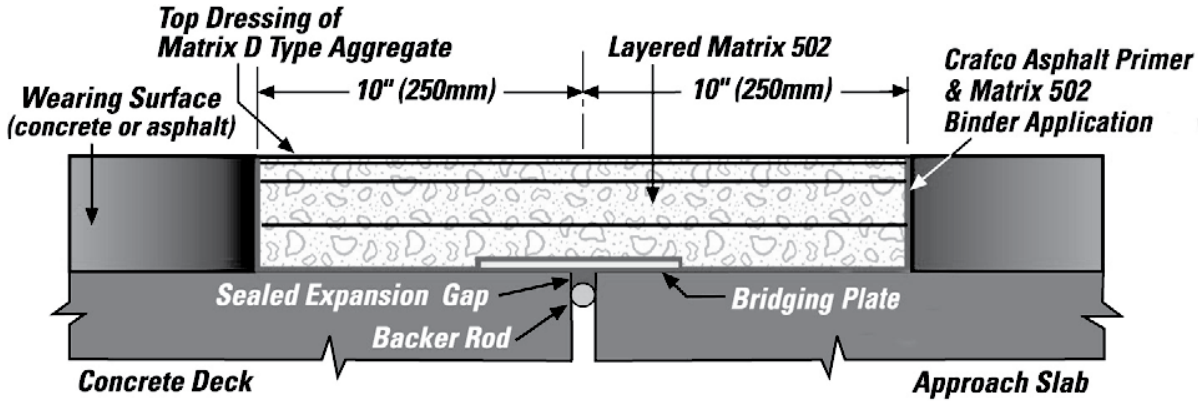
- The thickness of the silicone at the center should be no more than half the width of the joint.
- It is important that the bottom of the silicone does not bond to the material below.
- Poured sealants perform best if the seal is poured when the ambient temperature (which must be above 40ºF) is at the middle of the historical range or the joint opening is at the midpoint.

9.1.1.3 Asphaltic Plug Joint

9.3 shows a typical detail for the asphaltic plug joint. It is a simple detail that can be used with concrete deck overlays. In this system, a center opening around the joint is prepared (about 20 in. wide and 2 in. deep), and a steel plate is placed in the opening as shown in 9.3. The asphaltic material is then placed over the steel plate to seal the joint. The maximum movement allowed by this system is about 2 in (Pemmaraju et al. 2006; Mogawer and Austerman 2004).



a) Typical detail. (Mogawer and Austerman)



b) Commercially available details. (Courtesy D.S. Brown)

9.3. Asphaltic plug joint.

9.1.1.3.1 Features

- Movements less than 2 in (50 mm) are allowed.
- The system is suitable for concrete decks, especially when used with an overlay layer.
- "A popular application is on decks where a waterproof membrane, topped by bituminous (asphalt) concrete overlay, has been added." (Purvis 2003)
- This system may work better in restricted climate conditions.

9.1.1.3.2 Advantages

- Installation and repair are easy.
- Installation and repair are low cost.

- There is a low susceptibility to snowplow damage.

- The system can be cold milled.

9.1.1.3.3 Disadvantages

- "This seal was developed almost exclusively for bridge deck joints without curbs, barriers, parapets, etc., and does not provide an effective method of sealing joint upturns, especially for longer decks and skewed deck joints where joint movement will degrade the system, resulting in early system failure." (Purvis 2003)

- Some problems have also been reported, among them are softening in hot weather, debonding of the joint-pavement interface, and cracking in very cold weather.

- In areas of heavy traffic volumes, this seal may rut or delaminate over time.

- Rapid temperature changes can damage this type of seal.

9.1.1.4 Sheet Seals

As shown in Figure 9.4, the sheet seal joint consists of a sheet of fiber-reinforced elastomeric membrane, tied to both sides of the joints. This joint type can accommodate up to 4 in. of longitudinal movement. The kink in the membrane allows the expansion and contraction of the joint, while keeping it watertight. This system could be used for rehabilitation of medium-span bridges. The membrane used in this system is provided in a variety of shapes, configurations, and sizes (Chang and Lee 2001).

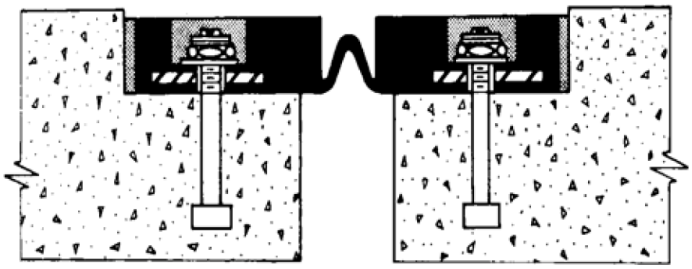


Figure 9.4. Sheet seal expansion joint. (Yuen 2005)

9.1.1.5 Sliding Plate Joint

Figure 9.5 shows a typical sliding plate joint. According to some literature (Purvis 2003), this type of joint could also be classified as an open joint, as it does allow some level of leakage while preventing most debris from passing

though the opening. This system consists of a steel plate spanning an open joint and embedded in adjoining deck slabs. It can accommodate movements up to 3 in. Currently, these joint types can even handle larger movements, depending on the thickness of the plate used. At present, these joint types are not used without a trough below.

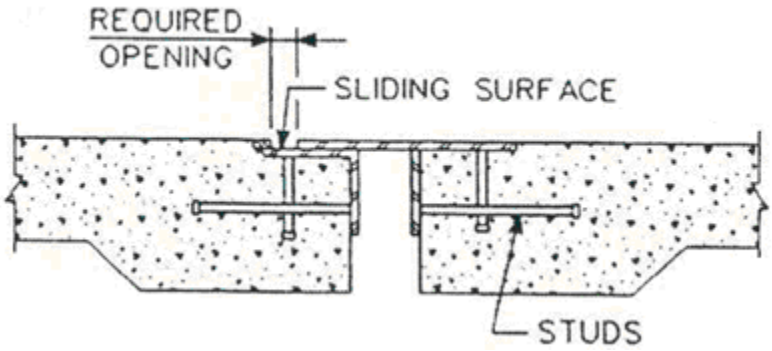


Figure 9.5. Sliding plate joint system. (Yuen 2005)

9.1.1.5.1 Advantages

- Sliding plate joints effectively prevent debris from passing through the opening.

9.1.1.5.2 Disadvantages

- The system is noisy under traffic due to loosening over time.
- It is prone to safety hazards due to detaching.
- Anchorage problems can occur between the plate and the concrete.
- Anchors can be corroded over time and are prone to fatigue damage due to traffic loads.
- Damage accumulated at the unsupported edge of the sliding plate causes a weak spot for traffic impact loads.
- Snowplow blades can damage both the sliding plate and the anchors.
- The impact on the plate due to traffic loads can be increased due to deterioration of the roadway surface close to the joint.
- Sliding joints are not recommended for highways with considerable truck traffic loads.
- It is not considered a watertight joint.

9.1.1.6 Open Joints

Figure 9.6 shows an example of an open joint with trough and armoring angles. The early versions of open joints were constructed without trough and armoring angles; however, most departments of transportation (DOT)s are no longer using open joints without a trough. These joint types are best suited for small movements (less than 1 in.). Armoring provides protection against traffic impact and the trough protects bridge elements below the joints from moisture and debris that cause deterioration.

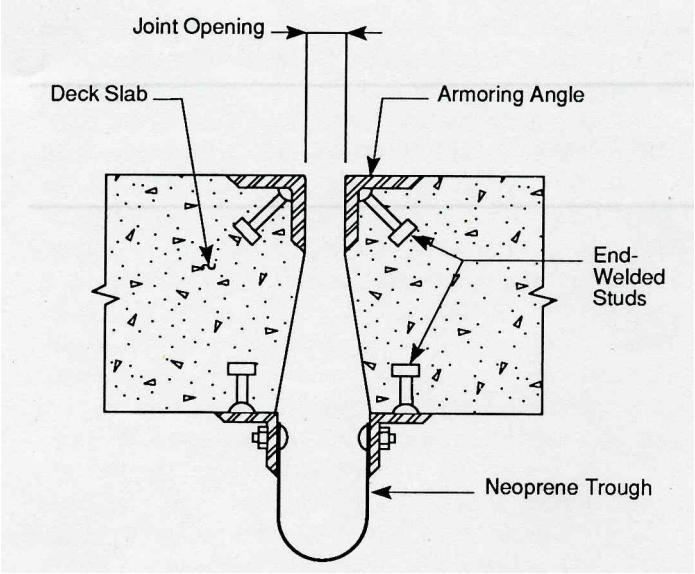


Figure 9.6. An example of an open joint with trough and armoring angles.

9.1.2 Expansion Joints with Medium Movement Capabilities

9.1.2.1 Strip Seals

Figure 9.7 shows a typical configuration for a strip seal joint. The main elements are a V-shaped membrane made using elastomeric material mechanically attached to an extruded metal piece anchored to joint edges using studs (Purvis 2003). The maximum movement that can be accommodated by this joint system is about five in (Chang and Lee 2002). Strip seal joints have gained popularity in recent years and most DOTs are relatively more satisfied with strip seal joints as compared to other joint types.

Figure 9.7. Strip seal expansion joint.

9.1.2.1.1 Advantages

- Strip seals are the most positively evaluated seals by agencies surveyed during the study done by *NCHRP 319*.
- The seal is watertight if properly installed.
- Under desirable conditions, strip seals have a longer service life than any other kind of seal.

9.1.2.1.2 Disadvantages

- Replacement is difficult.
- Strip seal splices should be avoided.
- Snowplow blades can damage the seal.
- Problem areas for this type of seal usually occur at locations of rapid cross section change, both in the vertical or horizontal direction of the deck, like the gutter line.
- During expansion, noncompressible material at tiny cracks could create membrane tears. This material can also cause ruptures, resulting in loss of water-tightness.
- Occasionally, the seal comes loose from the extruded metal holding piece.

9.1.3 Expansion Joints with Large Movement Capabilities

9.1.3.1 Modular Expansion Joints

Modular joints were developed to accommodate large longitudinal expansion and contraction movements by combining multiple neoprene strip elements. Depending on the number of combined strips, modular joints can accommodate movement from 6 to 28 in. Modular expansion joints are mainly used for bridges with more complex movements. They consist of several metallic pieces that make them prone to fatigue. The modular expansion joint system consists of sealer, separator beams for sealers, and support bars for separator beams. Sealers can be of strip, compression, or sheet seal type. Separator beams are usually extruded or rolled metal shapes to join the seals in a series. The separator beams are supported on support beams at frequent intervals (Chang and Lee 2001). Figure 9.8 shows a schematic of one commercially available modular joint.

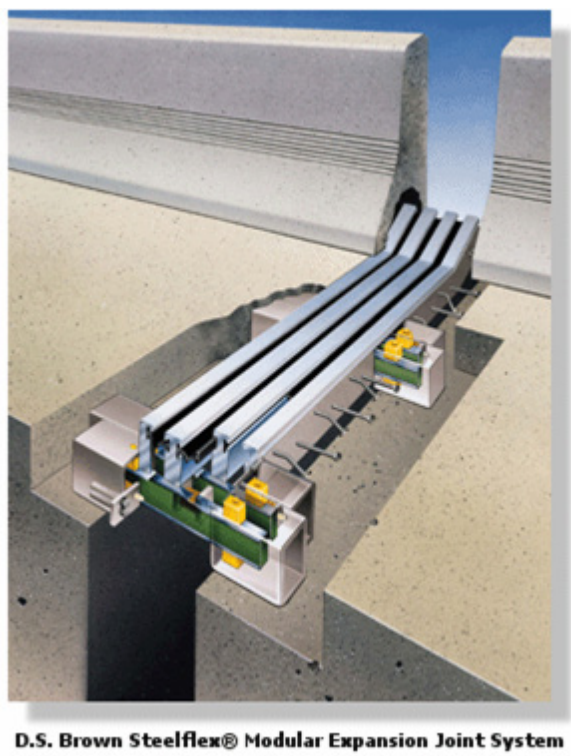


Figure 9.8. Modular expansion joint system. (Courtesy D.S. Brown)

9.1.3.1.1 Features

- The system consists of three elements: sealers, separator beams, and support bars.
- Modular joints are considered a closed joint system, so they protect the components below from damage due to water, salt, and other problems associated with deck runoff.

- They are used for all types of long spans. It is the best alternative for large movements.

9.1.3.1.2 Advantages

- They can accommodate large movements over 4 in (100 mm), but the typical movement is between 6 and 24 in.
- They can be sized according to the magnitude of movement and have been designed to accommodate movements of more than 7 ft (very long span).
- Joint seals have been continuously improved.
- The system is improving with experience and research. New design provisions in the *AASHTO LRFD Design Specifications* incorporate recent fatigue related research.

9.1.3.1.3 Disadvantages

- Problems have included fatigue cracking of welds (older designs); damage to the equalizing spring, the neoprene sealer material and the support; and damage from snowplows.
- The system is complex and must be capable of expanding and contracting to accommodate very large movements and must remain watertight while be supported by a movable framing system.
- The high initial and maintenance cost. Many DOTs have returned to using finger joints for large movements and placed additional emphasis on proper drainage.
- There are only two primary suppliers, which limits competition.
- Mixed performance results have been reported.

9.1.3.1.4 Performance Requirements for Modular Expansion Joints

Modular expansion joints, by some agencies, are exclusively used to accommodate large movements. Water tightness and fatigue resistance are important considerations and some states have or are developing specifications to ensure these considerations.

9.1.3.2 Finger Plate Joints

Figure 9.9 shows a typical finger plate joint system. Two pieces of steel are anchored either to the deck or the steel superstructure and cover the joint opening. Grooves in each of the plates fit loosely together like fingers. The majority of the finger plate joint systems in use have a trough as shown in Figure 9.9 to catch water and debris and carry it away from the superstructure to prevent deterioration (Purvis 2003; Chang and Lee 2001). A variation of this concept is to offset the location of the finger assembly to one side of the deck opening, whereby the fingers from one side slide over an embedded plate in the deck on the other side. The advantage of this design is that the deck opening is covered and limits the amount of debris that can fall through.

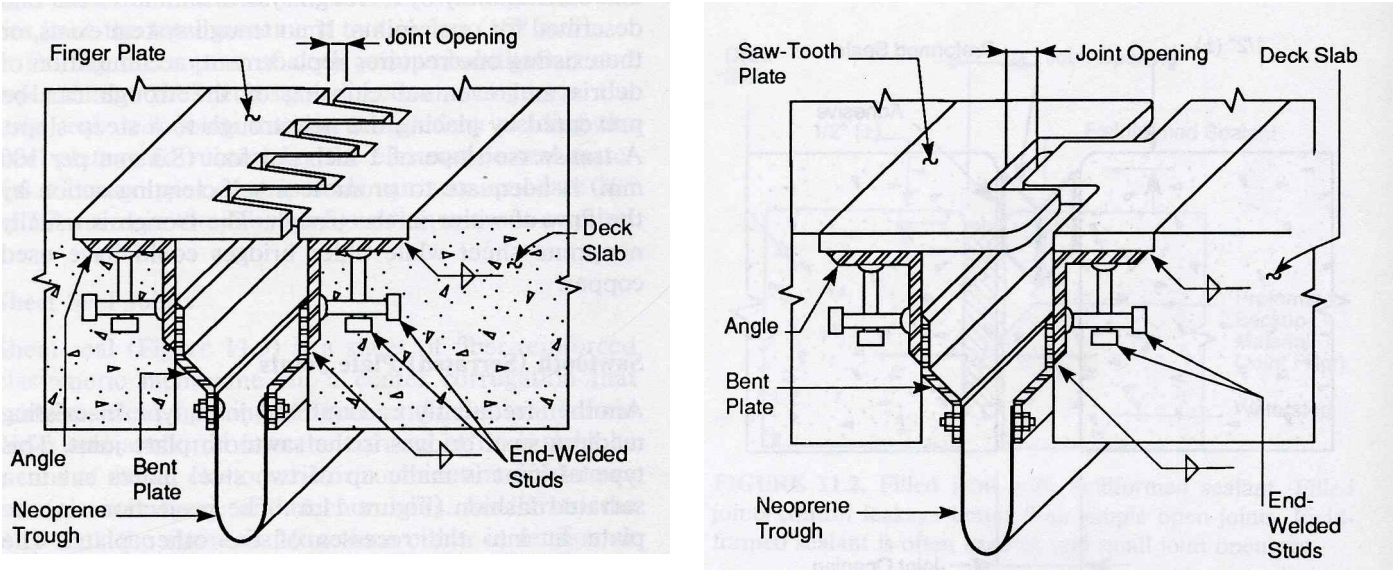


Figure 9.9. Examples of a finger plate joint system.

9.1.3.2.1 Features

- The system consists of finger like plates attached to both sides of the superstructure and usually includes a drainage trough.
- It is considered an open joint system.

9.1.3.2.2 Advantages

- These joints can accommodate large movements over three in.
- The system experiences fewer durability problems in comparison with modular joints, and tends to have fewer problems than many other joints.

- DOTs have opted to use finger joints on longer spans.

9.1.3.2.3 Disadvantages

- The upward bending of the ends of the fingers results in increased noise, a rough riding surface, and occasionally broken fingers.
- The most common problem is concrete deterioration around the joint.
- The only corrosion protection for the components below is the drainage trough.
- Problems are usually caused by poor drainage trough design.
- Cleaning and flushing are often difficult and are rarely performed.

9.1.3.2.4 Performance Requirement s for Finger Plate Expansion Joint

Water tightness and good drainage is an important consideration for finger plate expansion joints. Use of appropriate slope (min 1%) can easily address this requirement. When using finger plate expansion joints, following the installation requirement specified by manufacture is important. Maintenance of the trough and routinely cleaning the debris is essential when using finger plate expansion joint.

9.2 FACTORS AND CONSIDERATIONS INFLUENCING EXPANSION JOINT SERVICE LIFE

Joint performance is perhaps the most important factor affecting the deterioration of bridge elements. A leaky joint allows salt and other chemicals to penetrate below the deck and causes many maintenance and deterioration problems. A study conducted by the FHWA (Fincher 1983) reports that over a five-year period, more than 60% of joints evaluated were leaking water, and the other 40% had problems that were actually decreasing their service lives. Another study conducted by Wallbank in 1989 evaluated 200 concrete bridges and found leaking expansion joints to be the major cause of bridge element deterioration.

The reduced service life of bridge expansion devices is primarily related to deficiencies, which may be load-induced, caused by man-made or natural hazards, or result from production defects in construction processes and/or design details, or operational procedures. These deficiencies are illustrated in the fault tree shown in Figure 9.10.

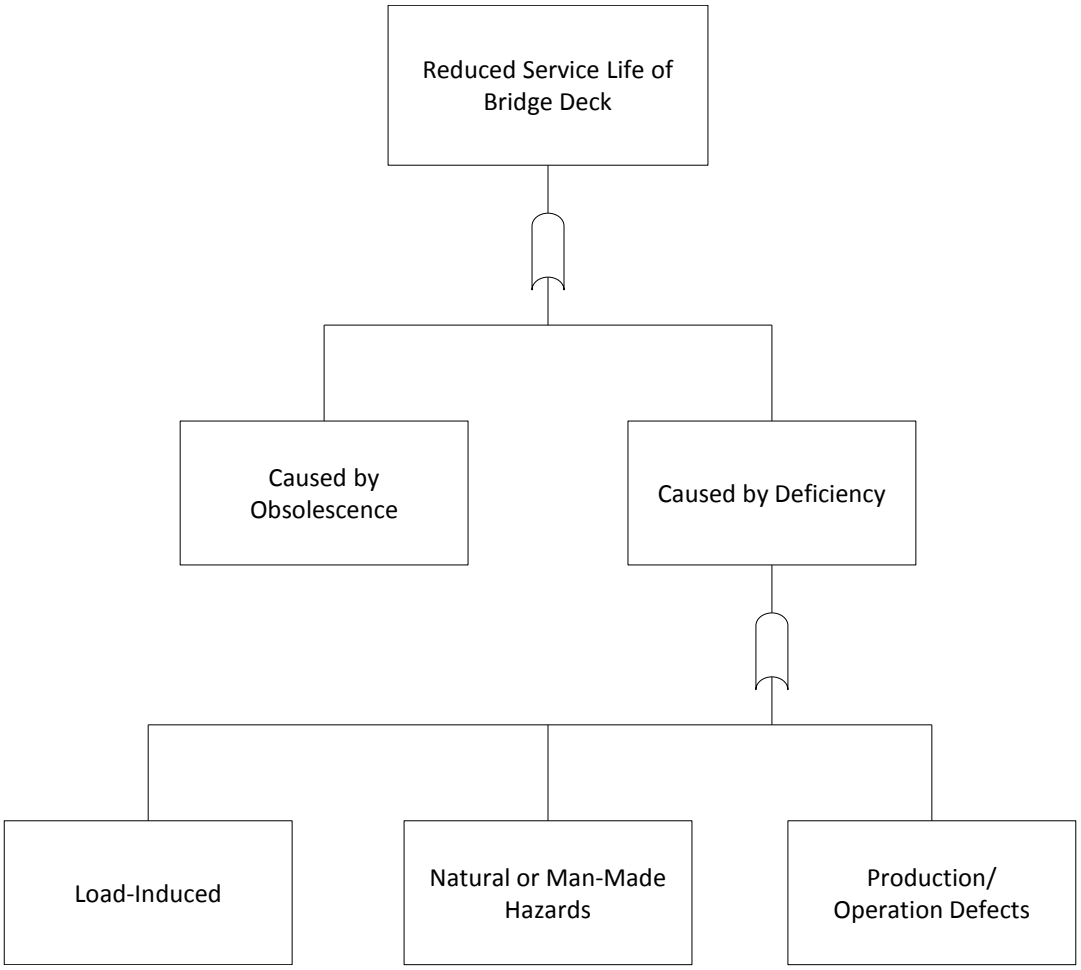


Figure 9.10. Expansion joint reduced service life fault tree.

9.2.1 Load-Induced Expansion Joint Considerations

Load-induced bridge deck deterioration can be attributed to either loads induced by the traffic that the bridge deck expansion device carries, or by characteristics dependent on the overall bridge system. These load-induced factors are introduced in the fault tree provided in Figure 9.11.

9.2.1.1 Traffic-Induced Load Considerations

Traffic-induced loads include the effects of truck and other vehicle traffic on the riding surface of the bridge. Bridge deck loading, and thus the loading on the expansion devices, has a degree of uncertainty that must be addressed during design of the bridge including vehicle weights and frequency of application, vehicle axle and tire spacing, vehicle location on the deck, and variability in vehicle suspension systems that can affect impact assumptions. The assumptions in loading must be carefully reviewed when establishing the criteria to be used, and

checked for bridge expansion device design. Typically the service life of bridge deck expansion devices will be affected by fatigue response to repetitive loading, overload, and wear and abrasion.

9.2.1.1.1 Fatigue

Fatigue is caused by the repetition of applied loads that result in a degradation of the strength resistance of the components used to resist tensile stresses that occur in expansion devices (especially in modular expansion joints and other large movement joints).

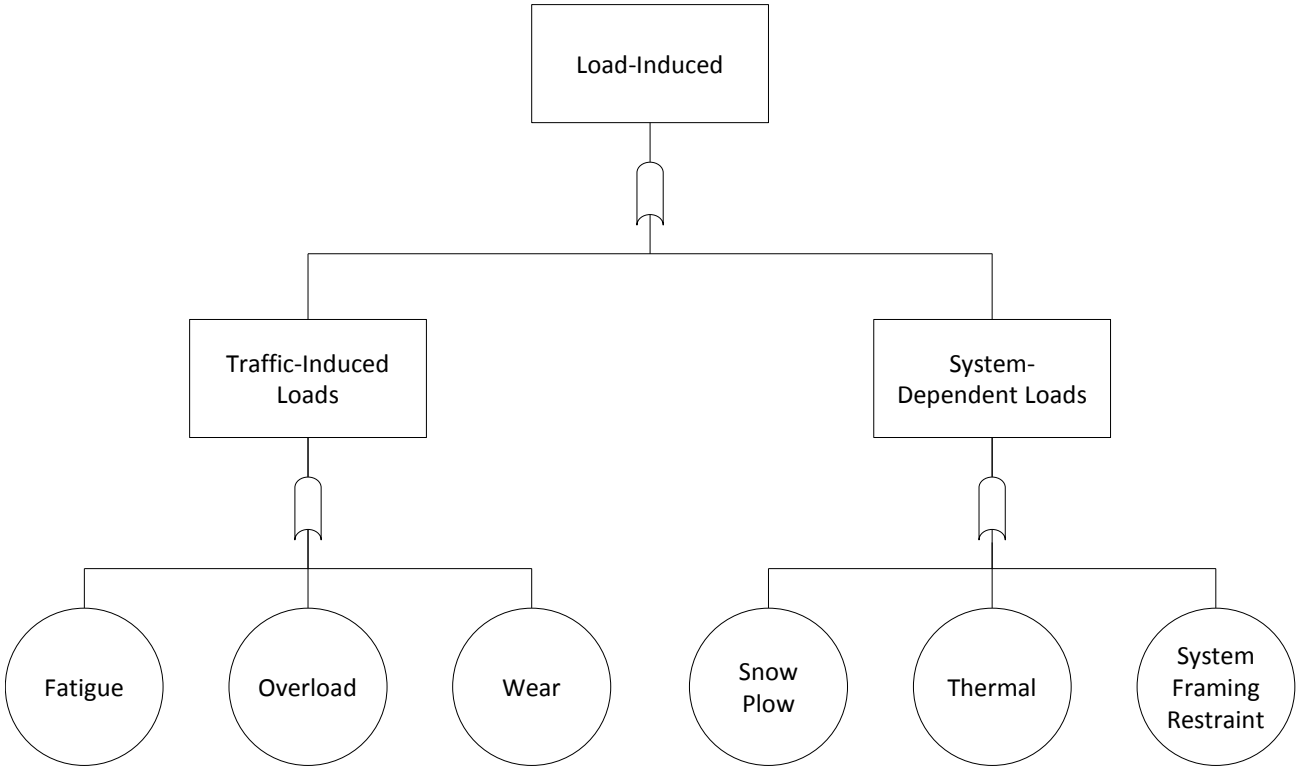


Figure 9.11. Load-induced deficiency fault tree.

9.2.1.1.2 Overload

Despite weight limit regulations in most states that define load limits for permit and legal truck configurations, overloads exceeding these limits are not specifically controlled. There are few weigh stations for checking these loads and they are usually only found on major roadway facilities, such as Interstate highways. Enforcement of the laws against these overloads on other facilities is limited to spot checks by code enforcement officials.

Overloads result in additional flexural stresses in bridge decks and expansion devices that can cause excessive cracking and movements not accommodated by the original design. Heavier tire loads may also affect the wear and

abrasion on the structure. Multiple applications of these loads can also affect the fatigue behavior of the deck expansion devices.

9.2.1.1.3 Wear and Abrasion

Wear and abrasion is typically affected by high traffic volumes, high tire loads, and the types of tires used by the vehicles. In cold climates, tires may have enhanced features to aid in traction, such as deep grooves, studs, and chains. These added tire features, while aiding traction, can abrade the surface of the bridge expansion joints.

9.2.1.1.4 Snow Plow

Impacts from snow plows are special considerations that need to be addressed in the selection of expansion joints. Repeated impacts from snow plows can cause expansion devices to function incorrectly and even make driving over them unsafe.

9.2.1.2 System-Dependent Loads

System-induced loads include the effects of the bridge system configuration on the behavior of the bridge deck expansion devices. These effects are accentuated by restraints provided through the bridge system and bridge deck boundary conditions, and can result in significant locked-in stresses.

9.2.1.2.1 System Framing Restraint

Boundary conditions for the longitudinal and transverse restraint of bridges can result in tension that may lead to cracking in bridge decks under cold weather conditions. These boundary condition restraints are introduced through design by the choices made relating to bearing types and their fixity and sliding assumptions. Fixed bearings provide an anchor that is intended to restrict “walking” of the superstructure resulting from shrinkage and cycling of expansion and contraction, and are usually located longitudinally near the point of zero movement of a supported multi-span superstructure unit. These fixed bearings, used in combination with bearings designed to allow the superstructure to either move or slide over the top of the substructure, reduce restraining forces that would otherwise be required to resist the movement.

9.2.1.2.2 Thermal

Temperature changes can result in changes in bridge movement, which can affect the service life of expansion joints.

9.2.2 Natural or Man-Made Hazard Considerations

The environment to which the bridge deck is subjected can have a significant influence on the service life of bridge decks and consequently the expansion devices.

These environmental influences include hazards from both natural and manmade sources and include effects from areas with adverse thermal climate, coastal climates, and chemical climates, as well as from chemical properties of the materials and outside agents, such as fire. These natural and manmade hazards are introduced in the fault tree provided in Figure 9.12.

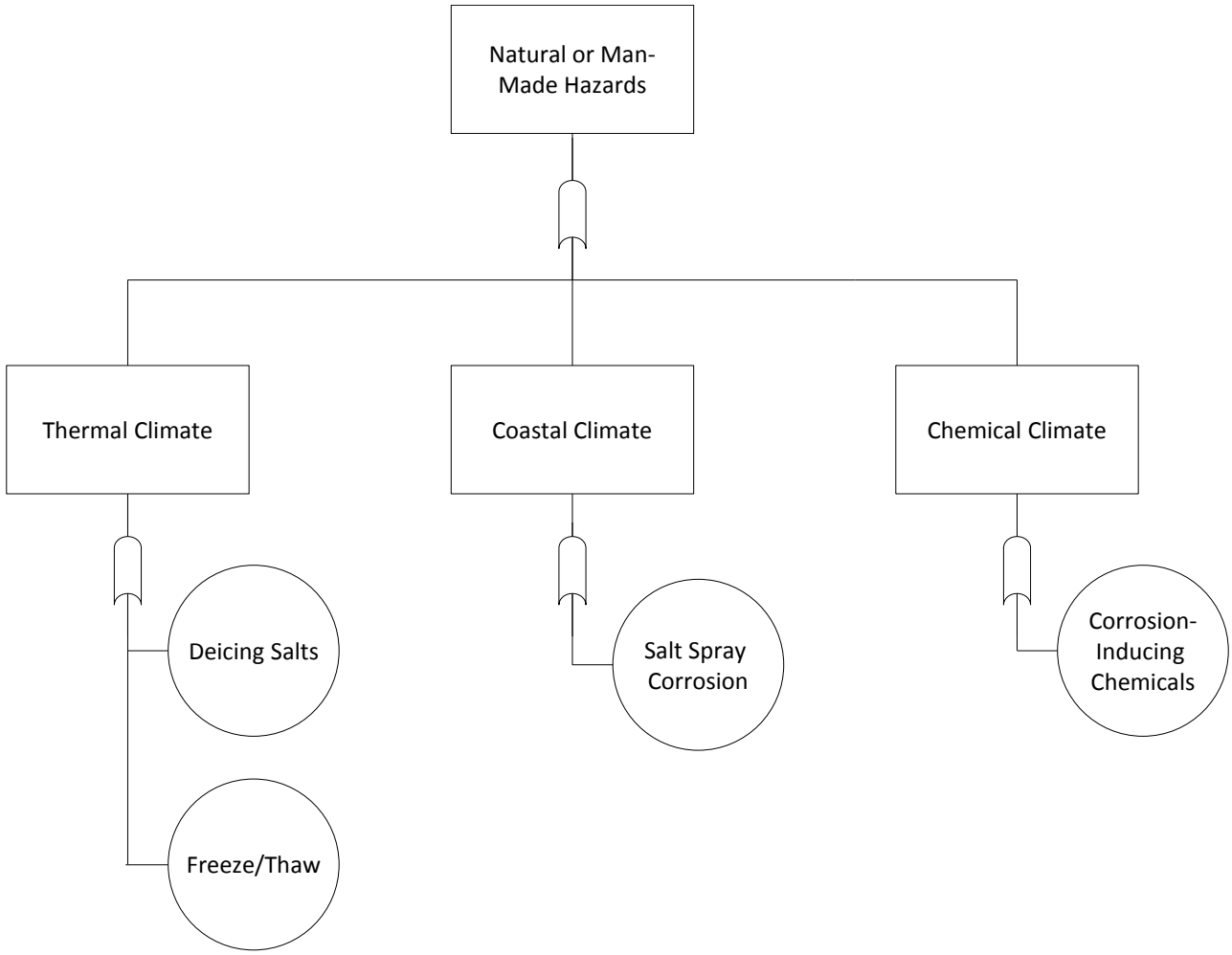


Figure 9.12. Natural or man-made hazard fault tree.

9.2.2.1 Thermal Climate

Thermal climate influences on bridge deck expansion device service life performance are primarily due to cold weather climates. These influences are both man-made from the application of deicing salts, and natural, in the case of freeze/thaw.

9.2.2.1.1 Application of Deicing Salts

Agencies in cold weather climates dealing with ice and snow on roadways and bridges have traditionally applied deicing salts to melt the ice and snow to facilitate tire traction on their facilities. These chloride-laden compounds can cause corrosion of expansion joint steel components.

The cross slope built into bridge decks for drainage purposes causes salt to wash down towards the bridge gutter adjacent to the traffic railing barriers bounding the bridge. Removal equipment that scrapes snow from the bridge deck will also deposit residual snow laden with deicing salts at this location, resulting in a very high concentration of chlorides.

9.2.2.1.2 Freeze/Thaw

Spalling and cracking of the concrete deck due to freeze/thaw at the sites of expansion joints could have adverse effects on the joint-to-deck connections and possibly on the performance of the device as well.

9.2.2.2 Coastal Climate

Coastal climate influences on bridge expansion joint service life performance are primarily due to chlorides introduced through salt spray and effects from high humidity. These influences both occur naturally.

9.2.2.2.1 Salt Spray

Coastal regions are subjected to a chloride-laden saltwater environment and a combination of wind and wave action that causes these chlorides to become airborne as salt spray. The susceptibility of the bridge expansion joints to these environmental influences depends on the height of the bridge deck above the water elevation. The salt spray wets the surfaces leaving a chloride residual that can cause steel components of expansion devices to corrode.

9.2.2.3 Chemical Climate

Chemical climate influences on service life performance can be attributed to corrosion-inducing chemicals or other chemicals that can have detrimental effects on exposed neoprene elements. These influences can occur naturally or can be manmade.

9.2.2.3.1 Corrosion-Inducing Chemicals

Corrosion-inducing chemicals can be introduced to the bridge deck and joints from adjacent industries in which residuals from pollution can attribute to reduction in bridge expansion joint service life. As an example, oil and coal

burning facilities release sulfur dioxide and nitrogen oxide into the air, which causes acid rain consisting of sulfuric and nitric acids. These acids can dissolve cement compounds in the cement paste and calcareous aggregates, and leave crystallized salts on concrete surfaces that can lead to spalling and the corrosion of reinforcing bars at the site of expansion devices, as well as components of the joints themselves.

9.2.3 Production and Operation Bridge Deck Considerations

Decisions made for the production of bridge expansion devices and activities during its operation can have a significant influence on the service life of bridge expansion joints. These production and operation influences are introduced in the fault tree provided in Figure 9.13, and include decisions made during the design and detailing of the bridge expansion joints, and decisions regarding the quality of construction, the level of inspection and testing performed during operations, and maintenance implementation.

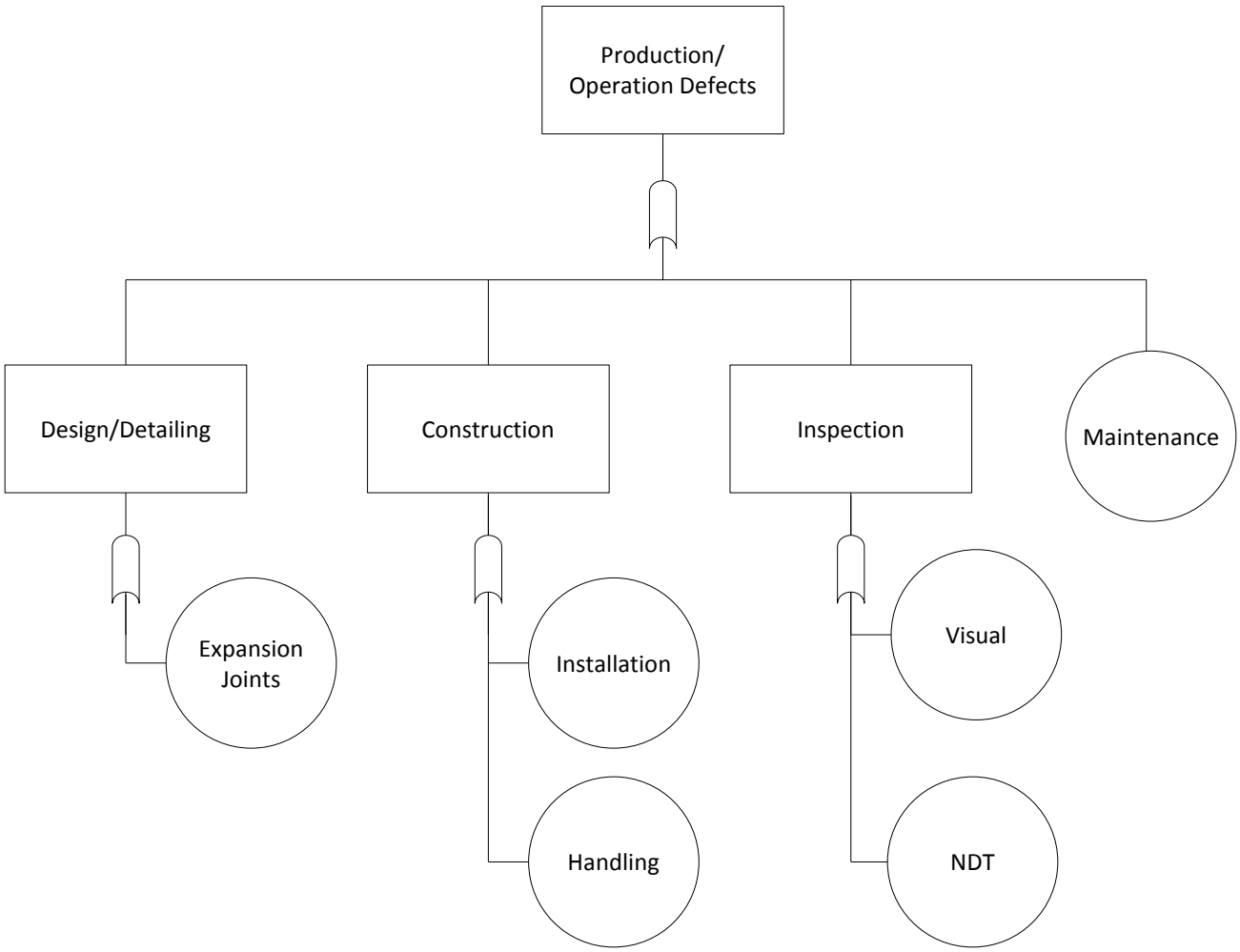


Figure 9.13. Production/operation and design/detailing defects fault tree.

9.2.3.1 Design and Detailing Bridge Deck Considerations

Decisions made during the design and detailing phase of a bridge project can significantly impact the service life of the bridge. It is incumbent upon designers to understand the implications of these decisions in order to make rational choices that will improve the service life of bridge expansion devices.

9.2.3.2 Construction

Attention to good practices during construction and installation is crucial to the long-term durability of expansion devices. Well-qualified, trained and executed workmanship increases productivity, reduces material waste, and provides expected service life. Proper use of test methods is needed to ensure that quality expansion joints are achieved.

9.2.3.3 Maintenance Plan—Monitoring of Condition

An effective maintenance plan must include provisions to adequately monitor the structure in order to identify deficiencies at an early age so that appropriate repairs can be performed before the deficiency propagates into an irreparable condition. The plan should address proper inspection, collection of data, and prompt action upon reported deficiencies.

9.2.3.4 Inspection Requirements and Intervals

Inspection is a valuable tool for increasing the service life of bridges. Identification of deterioration at an early stage is essential to providing corrective actions in a timely manner in order to maximize the owner's investment in the structure. The scope and interval of these inspections should be calibrated based on bridge expansion device performance history for the applicable deck hazard exposures. Inspection personnel should be properly trained to identify this deterioration.

9.3 OVERALL STRATEGIES FOR ENHANCED BRIDGE EXPANSION JOINTS SERVICE LIFE

Providing bridge expansion joints with enhanced service life requires a full understanding of the potential deterioration mechanisms. These mechanisms are associated with load-induced conditions, local environmental hazards, production-created deficiencies, and lack of effective operational procedures. This process is presented in this section.

The intention of this section is to provide guidelines for selecting the most appropriate individual strategy to achieve the desired service life.

9.3.1 Design Methodology

With limited funds available for bridge construction, first-cost economy is often an overriding factor in critical bridge joint selection decisions. However, to take advantage of the long-term advantages of durable expansion joints, service life enhancement strategies must apply a cascading series of economy, design, construction, and maintenance measures. The success of the strategy selection process is dependent on the ability to predict service life and the incorporation of best practices to enhance service life.

9.3.2 Expansion Joint System Selection

In general, selection of bridge expansion joints is historically based on locally accepted construction practices and procedures, which result in an economical deployment. Proposed enhancements to these systems would need to be assessed on the specific long-term performance of these systems. As noted in Section 9.2, there are numerous deficiencies that result in a reduction of service life, and these conditions may not necessarily occur at the specific project site under consideration. It is the design engineer's responsibility to understand local practice and its performance history, and to perform sufficient investigation of the project site, its environment, and other design conditions prior to selecting an expansion joint and its potential enhancements.

9.3.3 Life-Cycle Cost Analysis (LCCA)

Once a series of strategies has been developed, the evaluation of these strategies is performed through a life-cycle cost analysis. LCCA provides an assessment of the overall long-term cost of a strategy throughout the target service life of the structure. LCCA is discussed in detail in Chapter 11 of this guide.

The analysis should consider all costs associated with the construction required, implementation trials and testing, and all future maintenance actions required through the life of the structure.

9.3.4 Construction Practice Specifications

Once the bridge expansion joint system is selected, a proper set of specifications must be developed to ensure the appropriate standard of care is used during construction. It should be noted that many agencies and manufacturers

have specific requirements for temperature setting for expansion joints to allow maximum range of movements, which need to be considered during design and construction.

9.3.5 Maintenance Plan

An effective maintenance plan must be developed to ensure the assumptions regarding maintenance upkeep made in the bridge expansion joint selection process are properly identified for staff and budget requirements. If the bridge owner cannot commit to such a program, then strategies for low maintenance life-cycle costs should be recommended.

9.3.6 Retrofit Practices for Expansion Devices

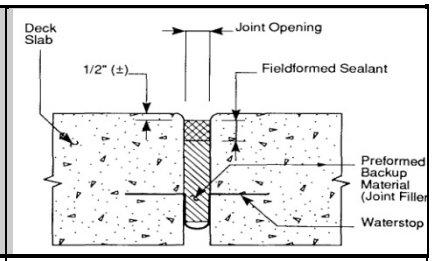
Expansion joints in bridges are not typically retrofitted as it is common practice to replace the joints when they reach the end of their service life. Due to the perishable nature of these expansion joints, it is important that they be properly maintained in order to maximize the life of such devices.

9.3.7 Technology Tables

The following technology table is provided to summarize service life and durability issues related to expansion joints, and to aid the designing engineer in selecting the proper expansion device. Table 9.1 provides service life issues related to most expansion joints used in practice. The information could be used in several different ways at the design or maintenance level. The purpose of the technology table is to identify the most common types of service life problems encountered in commonly used expansion joints, and this information can subsequently be utilized to provide possible solutions to service life problems encountered with expansion joints.

Table 9.1. Bridge Joints Technology Table.

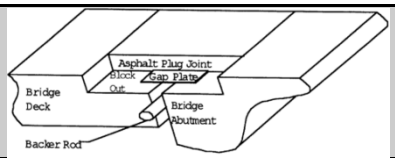
Field Molded (or Field Formed) Joints
Range of Movement: Less than 1 in.
Expected Range of Service Life: 8 to 9 years



Service Life Problems	Solutions	Advantages	Disadvantages	System Preservation Requirements
Spalling and cracking of adjacent concrete	Deeper notch	Field molded joints are inexpensive and easy to install. They are best suited for single-span bridges with a maximum length of about 100 ft This detail is an economical solution for simple span bridges with spans of less than about 100 ft, especially in low traffic areas where a few hours of interruption to traffic is not a major problem.	It requires maintenance and has a short service life. In most cases, it needs complete replacement every 2 to 3 years.	It requires regular 3-year inspection and replacement in most cases
	Filling with silicone			
	Beveling edges of concrete			
	Placement of correct silicone thickness			
	Correctly mixing silicone material			
Installation problems	Training installation technicians			
	Providing detailed installation plans by the manufacturer or contractor. Having representative of the joint supplier onsite during installation.			
Snowplow damage	Not allowing inferior quality of bonding agents			
	Installation slightly below the top of the deck elevation			
Debris accumulation	Inspection and cleaning regularly			
Water leakage	Draining water away from the joint			
	Insulation			
Hot weather damage	Using high quality silicone sealer			

Table 9.1 (Continued). Bridge Joints Technology Table

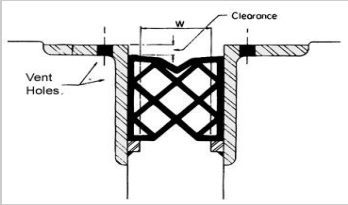
Plug Seal Joints
Range of Movement: Less than 2 in.
Expected Range of Service Life: 1.5 to 20 years



Service Life Problems	Solutions	Advantages	Disadvantages
Seal material does not always fill the joint completely	Reapplication of the joint	Low instances of snowplow damage. Low cost of installation and maintenance	Damage due to sudden changes in temperature
Early joint failures in skewed and/or long decks	Avoid the system in skewed and/or long decks		
Softening in hot weather	Proper selection of materials considering climate conditions		
Debonding of the joint-pavement interface	Reapplication of the sealant		
Cracking in very cold weather	Proper selection of materials considering the climate conditions		
Rutting or delamination over time due to heavy traffic volumes	Reapplication of the joint		

Table 9.1 (Continued). Bridge Joints Technology Table

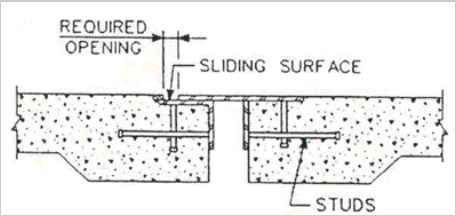
Compression Seal Joints
Range of Movement: 0.25 in. to 2.5 in.
Expected Range of Service Life: 2 to 20 years



Service Life Problems	Solutions	Advantages	Disadvantages
High deformation due to extreme temperature variation	Use only at moderate temperature extremes	Ease of installation. Not very prone to snow plow damage. Have received very good performance ratings from various agencies	Poor workmanship, leading to service life problems, is reported. In some instances, poor workmanship has resulted in joints leaking immediately after installation.
Damage due to snowplow	Use better consolidation of concrete around the steel armors		
Leakage after installation	Use high-solids urethane adhesive prior to insertion of the seal		
	Prevent twisting during installation		
Dislodging over time	Use better adhesive materials to anchor the seal to its place		
	Installing at low temperature		
Lose compression over time	Use material with lower creep property		
	Size using a working range of 40% to 85% of its uncompressed width		
Ozone sensitive	Use material with high resistance property for ozone side effects		

Table 9.1 (Continued). Bridge Joints Technology Table

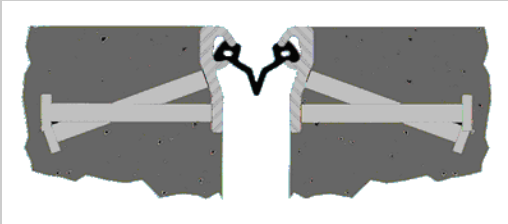
Sliding Plate Joints
Range of Movement: Less than 4 in.
Expected Range of Service Life: up to 30 years



Service Life Problems	Solutions	Advantages	Disadvantages
Corrosion of steel material used	Determine selection of corrosion-resistant material (anchor)	Comparable to armored butt joints, except that the sliding plate prevents, for the most part, the movement of debris below the joint	Movement of the sliding plate can easily be hindered by debris that accumulates in the slot
Bending and fatigue of the sliding plate	Use thicker plates		
	Better design of surround material		
	Use high tensile strength steel		
Drainage (clogging)	Trough with steep slope		
	Conduct regular maintenance		

Table 9.1 (Continued). Bridge Joints Technology Table

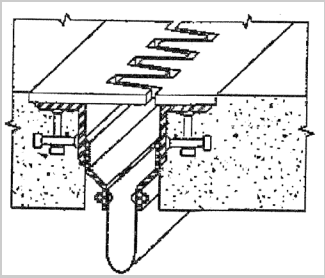
Strip Seal Joints
Range of Movement: Less than 5 in.
Expected Range of Service Life: up to 20 years



Service Life Problems	Solutions	Advantages	Disadvantages
Tearing of neoprene membranes	Replace neoprene membrane	Good performance record. High degree of water tightness	If the membrane is set low, the debris can accumulate faster than it can be maintained. If it is set high, it will be prone to snow plow damage. High initial cost and replacement
Corrosion	Determine selection of corrosion-resistant material (armoring)		
Debris accumulation	Perform regular maintenance		
Damage due to snowplow	Set the neoprene membranes lower		

Table 9.1 (Continued). Bridge Joints Technology Table

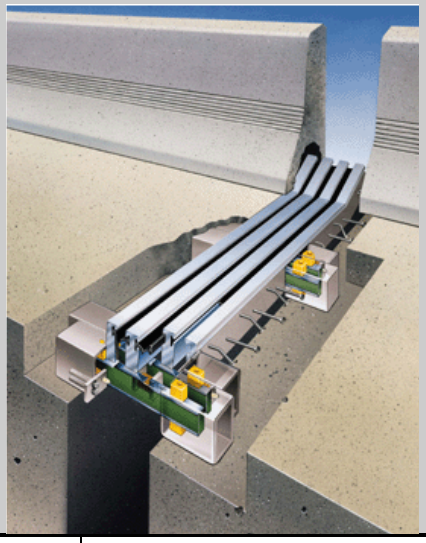
Finger Plate Joints
Range of Movement: More than 4 in.
Expected Range of Service Life: 15 to 50 years



Service Life Problems	Solutions	Advantages	Disadvantages
Clogging of the joint	Trough with steep slope	Suitable for large movements. Allows flushing of debris and prevents leakage and overflow. Have a good performance record	High initial cost and prone to damage due to snowplow if not designed correctly. Horizontal misalignment during construction can cause the fingers to jam. Vertical misalignment can result in poor ride quality and noise.
	Perform regular maintenance		
Anchorage failure	Construction QC		
	Design		
	Use stronger materials		
Bent fingers (load related)	Use thick plates		
	Use replaceable fingers		
Misaligned fingers	Ensure proper installation and avoid horizontal or vertical misalignments		
Corrosion	Design (corrosion resistant material)		

Table 9.1 (Continued). Bridge Joints Technology Table

Modular Expansion Joints
Range of Movement: Greater than 5 in.
Expected Range of Service Life: 10 million cycles



Service Life Problems	Solutions	Advantages	Disadvantages
Fatigue cracking of welds	High purity materials, new fatigue design and details	Can accommodate very large movement	Incorporates many components and is high maintenance; therefore, many agencies are resorting back to finger plate joints.
Water leakage at seal splice	Curbs to guide drainage away		
Debris accumulation in seals	Perform maintenance crew clean out once per year		
	Run joint through opening in parapet with a catch basin		
	Direct drainage away from joints in order to wash debris away		
Reflective cracking in concrete above support box	Use a thicker top plate to support box		
	Use an increased concrete cover over support box		
	Use a #3 reinforcing bar over anchorage detail to minimize cracks		
Drainage build-up	Curbs or upturns		

Review of the technology tables indicates that the following modes of failure are observed when various expansion joints are used:

- Damage to various expansion joint components, such as steel armors, due to snowplow usage;
- Inadequate design, installation, and maintenance;
- Accumulation of debris, leading to limited movement;

- Lack of timely maintenance;
- Limited service life; and
- Leakage.

The ultimate consequence of these service life problems is that joints start leaking and causing damage to bridge elements below the deck.

9.3.8 Strategy Table for Expansion Joints

Selecting the appropriate expansion device for the required movement range is the most critical step in enhancing the service the life of these joints. Table 9.2 summarizes the information presented in this chapter and proposes recommended strategies when expansion devices are to be used. The table is intended to serve as a guide to the designer in selecting bridge expansion devices.

Table 9.2. Strategy Table for Expansion Joints.

Maximum Longitudinal Movements (in.)	Strategy	Potential Deterioration Mode	Options to Improve Service Life	Life-Cycle Cost		Difficulty Associated with Replacement	Performance Record	Expected Service Life (years)
				Initial	Maintenance			
Less than 1 in.	Field molded or other equivalent joint types	See technology tables	Provide second layer protection	Low	High	Low	Good	1 to 3
Between 1 and 3 inches	Strip seal joints	See technology tables	Provide second layer protection	Medium	High	High	Very Good	3 to 30
	Compression seal joint	See technology tables	Provide second layer protection	Medium	High	High	Very Good	3 to 30
Larger than 4 in.	Finger plate joints	See technology tables	See technology tables	High	High	High	Very Good	10 to 50
	Modular expansion joints	See technology tables	See technology tables	Very High	Very High	Very High	Good	10 to 50

CHAPTER 10

BRIDGE BEARINGS

10.1 INTRODUCTION

Bearings are a critical element within overall bridge systems. Although they represent only a small part of the overall structure cost, they can potentially cause significant problems if they don't function properly or if possible maintenance, retrofit, or replacement strategies are not envisioned and well planned at the design stage. Bridge superstructures experience translational movements and end rotations caused by traffic loading, thermal effects, creep and shrinkage, wind and seismic forces, initial construction tolerances, and other factors. Bridge bearings are designed and built to accommodate these movements and rotations while supporting required gravity loads, transmitting those loads to the substructure, and providing necessary restraint for the superstructure. Proper functioning of bridge bearings is assumed in the analysis and design of overall bridge systems. Bearing failure or improper behavior can lead to significant changes in load distribution and overall structural behavior that are not accounted for in the design, and can significantly affect the superstructure/substructure interaction.

This chapter describes various bearing types and provides information concerning factors affecting and increasing their service life. Methods for design for service life are discussed along with needs for future inspection, maintenance, and possible replacement.

10.2 BEARING TYPES

Many different bearing types have been developed, primarily to provide efficient, economical ways to accommodate various levels of load and movement. Each has certain advantages and potential disadvantages. The following table identifies commonly used bridge bearing types that will be discussed further in this chapter:

Table 10.1. Bearing Types.

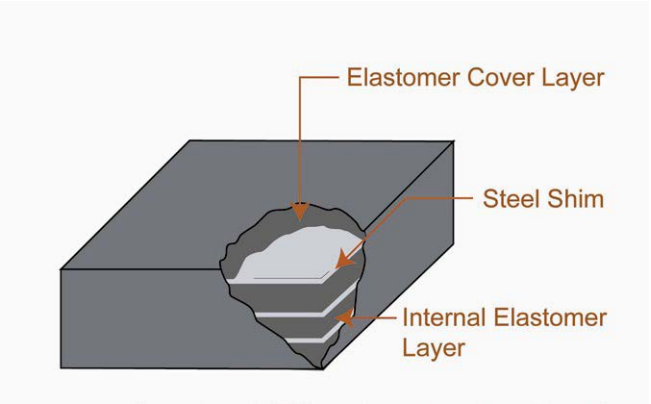
General Category	Bearing Type
Elastomeric Bearings	Plain Elastomeric Pads
	Steel Reinforced Elastomeric Pads
	Cotton Duck Pads
Sliding Bearings	Polytetrafluorethylene (PTFE)
	Alternative Sliding Materials
High Load Multi-Rotational Bearings	Pot Bearings
	Disc Bearings
	Spherical Bearings (Cylindrical for Uni-Directional)
Fabricated Steel Mechanical Bearings	Fixed Pin
	Rocker or Roller Expansion

10.2.1 Elastomeric Bearings—Plain and Reinforced

Elastomeric bearings have become the most common type of bearing in recent years because of their desirable performance characteristics, durability, low maintenance requirements, and relative economy. Elastomeric bearings have no moveable parts and accommodate movement and rotation by deformation of an elastomeric pad, which can be neoprene or natural rubber. Lateral and longitudinal movement is accommodated by the pad's ability to deform in shear. These bearings can accommodate combined movements in both longitudinal and transverse directions, and circular elastomeric bearings have been used to accommodate multi-rotation requirements. Existing bridges utilizing elastomeric bearings with more than 50 years of very good service performance are reported.

Plain, unreinforced elastomeric pads are used for short spans where loads and movements can be accommodated by a single layer of elastomer.

As vertical load and movement requirements increase, thin reinforcing plates are combined with multiple layers of elastomer to form a laminated reinforced elastomeric assembly (see Figure 10.1). Steel and fiberglass reinforcement layers have been used; however, fiberglass is weaker, more flexible, and does not bond as well to the elastomer as does steel reinforcement. As a result, the use of thin steel plate reinforcement has become more common.



(Courtesy D.S. Brown) **(Courtesy NSBA)**

Figure 10.1. Laminated elastomeric pad.

Neoprene is the most widely used elastomer, but some states also use natural rubber (Stanton 2004), particularly in colder climates, to meet AASHTO low temperature requirements. Natural rubber generally stiffens less than neoprene at low temperatures. Neoprene has greater resistance to ozone and a wide range of chemicals than natural rubber, making it more suitable for some harsh chemical environments.

AASHTO LRFD Bridge Design Specifications (LRFD Specifications) currently provide two design methods for the design of elastomeric bearings: Method A, which is the simpler method and has fewer testing requirements; and Method B, which requires greater design effort and more extensive testing (AASHTO 2012). Method A leads to viable designs for bridges up to 150 ft (Stanton et al. 2004) and Method B is generally used when a reasonable bearing cannot be designed using Method A. The majority of states use Method A (Stanton et al. 2004).

10.2.2 Cotton Duck Pads (CDP)

Cotton duck bearing pads are another type of elastomeric bearings that are occasionally used in some states, typically for pre-cast concrete I-girder bridges and with span lengths up to the 150 ft to 180 ft range. CDPs are preformed elastomeric pads consisting of very thin layers of elastomer [less than 0.4 mm (1/60 in.)] interlaid with fabric. The fabric can be cotton or polyester. They are stiff and strong in compression, giving them much larger compressive load capacities than plain elastomeric pads, however CDP's shear deflection capability is very limited. The CDP bearings provide a high stiffness in the direction of applied compressive force and are helpful in limiting problems encountered during construction of heavy girders, because of rotational instability, generally observed with other elastomeric bearing types. For large shear strain, CDPs may split and crack, or result in girder slip on the CDP.

The limited shear deflection capacity is frequently overcome by the addition of a Polytetrafluorethylene (PTFE) sliding surface to accommodate large movement. When PTFE surfaces are used, they are often combined with stainless steel sliding surfaces, similar to that shown in Figure 10.2. The overall capacities depend on the stiffness and deformation capacity of the CDP and vary from manufacturer to manufacturer. To assure adequate performance from CDP, quality control testing measures and design recommendations have been developed and incorporated into *LRFD Specifications* (Lehman et al. 2003).

10.2.3 Sliding Bearings

10.2.3.1 Polytetrafluorethylene (PTFE)

When horizontal movements become too large for elastomeric bearings to reasonably accommodate in shear, PTFE sliding surfaces can be used to provide additional capacity (see Figure 10.2). As previously stated, they are commonly used to provide movement capability with cotton duck pads, and they are also used to provide for horizontal movement in combination with other bearing systems that internally provide for compression and rotation such as high load multi-rotation (HLMR) pot and disc bearings (Figures 10.3 and 10.4). They are also used to accommodate large translations and rotations when combined with spherical or cylindrical bearings.

PTFE has low frictional characteristics, chemical inertness, and resistance to weathering and water absorption, making it an attractive material for bridge bearing applications.

The sliding movement is typically provided by a very smooth stainless steel plate sliding on a PTFE surface. The stainless steel surface is larger than the PTFE surface so that the full movement can be achieved without exposing the PTFE. The stainless steel is typically placed on top of the PTFE to prevent contamination with dirt or debris. PTFE sliding bearings may be guided, allowing movement in only one direction, or non-guided, allowing multi-directional movement. When PTFE sliding surfaces are combined with elastomeric pads, the elastomeric pad must be designed to accommodate the shear force that is needed to overcome the PTFE friction resistance.

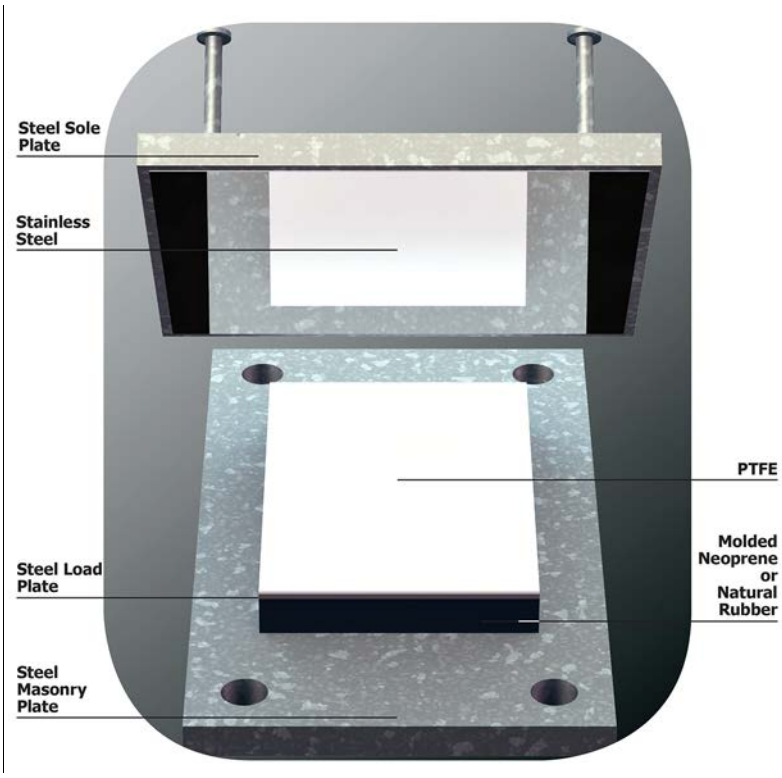


Figure 10.2. Elastomeric bearing with PTFE sliding surface. (Courtesy D.S. Brown)

Sliding surfaces develop a frictional force that acts on the superstructure, substructure, and bearing. As a result, friction is an important design consideration, and the low frictional resistance of PTFE is what makes it so useful for this application. The coefficient of friction of PTFE increases with decreasing temperature and with decreasing contact pressure. It also increases if the mating surface is rough or contaminated with dust or dirt. Proper design, fabrication, and field installation are all essential for proper performance.

Plain, unfilled PTFE is the most common material used for sliding bearings. Filled PTFE, with the addition of glass fibers, carbon fibers, or other chemically inert filler reinforcement, is used sometimes. Filled PTFE has significantly greater resistance to wear and creep, but also has a higher friction coefficient by as much as 25% to 30%. Unfilled PTFE in the form of a woven fabric is occasionally used to provide higher bearing strength, longer wear, and increased creep resistance.

Lubrication significantly reduces the coefficient of friction, and dimpling of the PTFE surface has been used as a means to facilitate lubrication. Dimples, which are spherical indentations, maximum of 0.32 in. diameter by 0.08 in. min. deep and covering 20% to 30% of the surface area, are machined into the PTFE surface to act as reservoirs for storing lubrication. Silicone greases are specified because they are effective at low temperatures and do not attack

the sliding material. Dimpled and lubricated PTFE has been used in Europe, but in the United States, it has been used only in special cases on large spherical bearings in which a very low coefficient of friction requirements is needed to reduce friction loads on substructures. Dimpled and lubricated PTFE demands a routine maintenance plan, as coefficient of friction will significantly increase as the lubrication material is depleted. This increase in coefficient of friction can have an adverse effect on service performance of other parts of the bridge system.

10.2.3.2 Alternative Materials to Plain PTFE

Maurer sliding material (MSM®) is an alternative sliding material developed in Germany as a better-performing substitute for current PTFE-based sliding material, mainly for high-speed rail applications (Maurer Söhne 2003). The new material is an ultra high molecular weight polyethylene that has performed well in recent field applications and experimental testing in Europe where it is one of the most popular sliding surfaces in use.

MSM® was primarily developed to accommodate bridge movements and related wear caused by high-speed trains, which induce high rates of movement due to girder end rotations resulting in large accumulated movement over time. Initial specifications required the bearing material to accommodate a rate of movement up to 15 mm/sec and provide 80 years of service life.

Experimental testing in Europe with dimpled and lubricated specimens subjected to high loading rates has shown MSM® to out-perform PTFE in regard to compressive strength, coefficient of friction, and rate of wear. But because this material is relatively new, there is no long-term data available. More recently, research conducted under SHRP 2 Project R19A compared coefficient of friction and wear between lubricated and unlubricated MSM® and plain PTFE specimens at high movement rates. Unlubricated specimen tests showed MSM® to have significantly greater wear resistance than plain PTFE, but with greater coefficient of friction. The *SHRP 2 Project R19A* testing also compared coefficient of friction and wear of a glass reinforced PTFE, Fluorogold®, with plain PTFE and MSM®. Like MSM®, the Fluorogold® material had significantly greater wear resistance, but with a smaller increase in coefficient of friction.

10.2.3.3 Service Life Design Method for Sliding Surfaces

Appendix G provides further information regarding a potential service life design method for sliding surfaces that considers a pressure-velocity (PV) factor in determining an effective wear rate for the surface material. The

method requires test data to establish material wear characteristics; therefore, its application as a design method will be subject to the availability of sufficient existing test data to establish reliable wear rate curves for different sliding materials. The proposed design provisions are based on research conducted by the *SHRP 2 R19A* project (Ala et al. 2012)

10.2.4 High-Load Multi-Rotation (HLMR) Bearings

When design loads and rotations exceed the reasonable limits for elastomeric bearings, HLMR bearings have typically been considered. High-load, multi-rotation situations often occur with longer spans, with curved or highly skewed bridges, or with complex framing, such as with straddle bents. In these cases the axis of rotation and/or the direction of movement are either not fixed or may be difficult to determine.

HLMR bearings include pot, disc, and spherical bearing and each is unique in how they accommodate large loads and rotations. All are fabricated in fixed and expansion versions. The expansion versions accommodate translational movement by means of PTFE sliding elements. Expansion versions may be guided, allowing movement in only one direction, or non-guided, allowing multi-directional movement. The following describes and compares each HLMR bearing type.

10.2.4.1 Pot Bearings

The pot bearing was first developed in Germany in the early 1960s and use began in the United States in the early 1970s (Fyfe et al. 2006). The main elements of these bearings include a shallow steel cylinder, or pot, which contains a tight-fitting elastomeric disc that is thinner than the depth of the cylinder. A machined steel piston fits inside the cylinder and bears directly on the elastomeric disc. Brass rings are used to seal the elastomer between the piston and pot components (see Figure 10.3).

Vertical load is carried through the piston of the bearing and is resisted by compressive stress in the elastomeric pad. The pad is deformable but almost incompressible in its confined condition and is often idealized as behaving hydrostatically. In practice, the elastomer has some shear stiffness and so this idealization is not completely satisfied. Rotation can occur about any axis and is accommodated by deformation of the elastomeric pad. Horizontal loads on a pot bearing are resisted by direct contact between the pot wall and the piston.

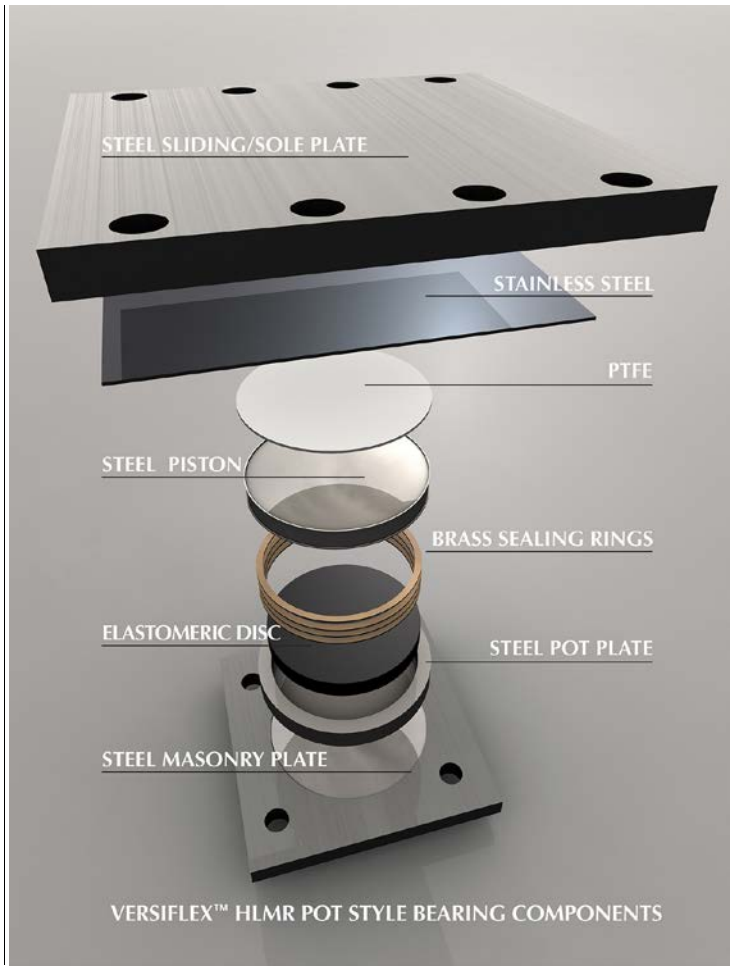


Figure 10.3. Pot bearing components. (Courtesy D.S. Brown)

To achieve satisfactory performance, pot bearings require a high degree of quality control in the fabrication and field installation process and an accurate determination of design loads and displacements. Through the years, they have been the most economical and most common HLMR bearing, and have been implemented on bridges throughout the country.

10.2.4.2 Disc Bearings

The disc bearing was developed and put into service in Canada in 1970 (Fyfe et al. 2006) and was a proprietary, patented device until recent times. It consists of a hard polyether urethane disc between upper and lower steel plates with a center shear pin device to resist horizontal load (Figures 10.4 and 10.5). The discs are stiff enough to support compressive load, yet can deform to permit rotation. However, rotational stiffness for a disc bearing is several times that of a pot bearing.

Figure 10.4. Disc bearing components. (Courtesy R.J. Watson)

Disc bearings are reasonably economical, but widespread use has been limited because of their originally patented and proprietary status, which made them available only from a single source. Now that there are additional bearing manufacturers that can supply disc bearings, their use has increased.

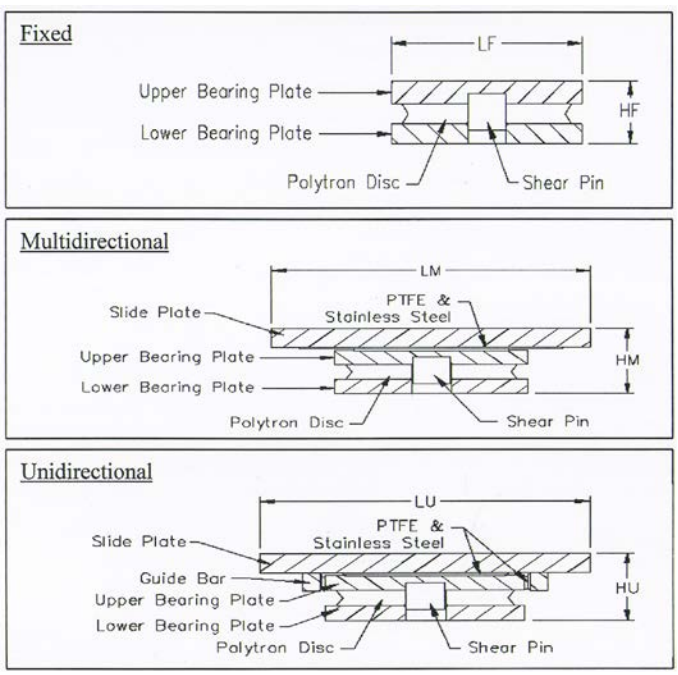


Figure 10.5. Typical disc bearings. (Courtesy R.J. Watson)

10.2.4.3 Spherical/Cylindrical Bearings

Spherical bearings are used primarily for accommodating large rotations about multiple or unknown axes. Sometimes referred to as curved sliding bearings, spherical bearings permit rotation about any axis, and cylindrical bearings permit rotation about one axis. In these bearing types, rotation is developed by sliding a convex metal surface (lower element) against a concave PTFE surface (upper element) (see Figure 10.6). The rotation occurs about the center of the radius of the curved surface, and the maximum rotation is limited by the geometry and clearances of the bearing. Translational movement is accomplished by incorporating a flat PTFE sliding surface. Horizontal loads may be partially resisted by the curved geometry of the spherical head, however large horizontal loads may require additional external restraint.

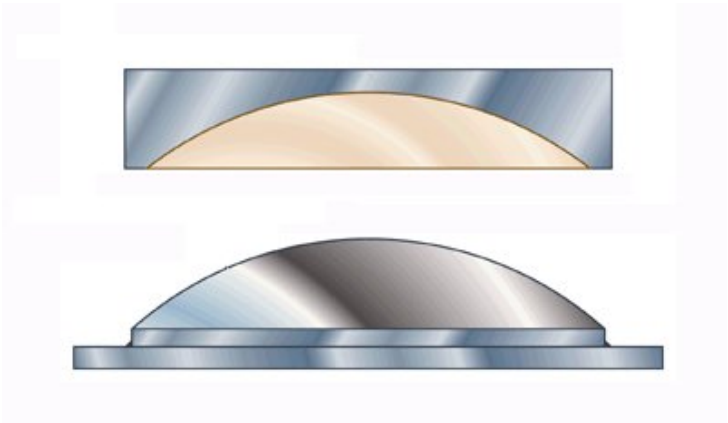


Figure 10.6. Typical spherical bearing. (Courtesy D.S. Brown)

Spherical bearings require highly machined fabrication and are more sensitive to the quality of the initial manufacture and installation than other HLMR bearings. Consequently, although they are generally the most expensive HLMR type, their advantage is their ability to accommodate higher gravity loads and rotations.

10.2.5 Fabricated Steel Bearings

Fabricated steel mechanical bearings have been used for both fixed and expansion conditions (see Figure 10.7), and have been the longest-used of any other bearing type. Many existing bridges have these types of bearings and some states still use them for new construction. When functioning properly, mechanical steel bearings generally provide the closest representation of assumed structural end conditions of all bearing types, and transmit loads through direct metal-to-metal contact. Most fixed bearings rely on a pin or knuckle to allow rotation while restricting translational movement. Rockers, rollers and sliding types are common used expansion bearings. Typically, steel

bearings are expensive to fabricate, install and maintain, which in part accounts for their popularity. Further, steel bearings typically provide only uni-directional movement. These types of bearings are fully designed by the engineer to accommodate loads, movements, and rotations, and can be developed to accommodate large requirements.

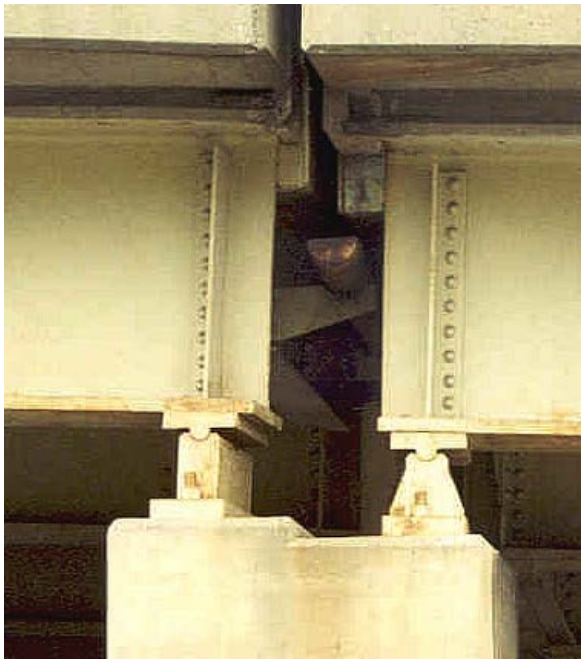


Figure 10.7. Fabricated steel bearings—rocker expansion and fixed conditions. (Courtesy HDR)

Bronze lubricated plate bearings have been used in conjunction with steel bearings to accommodate smaller amounts of movement at expansion ends, but are not used much today. PTFE sliding surfaces have replaced bronze sliding plates because of a much lower coefficient of friction and lower cost.

10.3 FACTORS INFLUENCING SERVICE LIFE OF BEARINGS

This section discusses various factors influencing the service life of bearings utilizing a fault tree analysis approach that first identifies service life issues that generally pertain to all bearing types. This is followed by specific discussions of service life issues pertaining to individual bearing types.

10.3.1 Factors Affecting Service Life of All Bearing Types—Fault Tree Analysis

A general description of the fault tree analysis approach for identifying factors affecting service life is given in Chapter 1 on general framework. Chapter 2 on system selection, further applies the fault tree analysis to identify factors affecting service life of overall bridge systems, which includes deck, superstructure, and substructure

components. This section applies specific parts of the fault tree analysis to identify service life factors that apply to bearings.

Figure 10.8 shows an overall fault tree diagram that identifies factors affecting service life for bridge bearings. The diagram identifies factors at descending levels that cause or contribute to service life reduction.

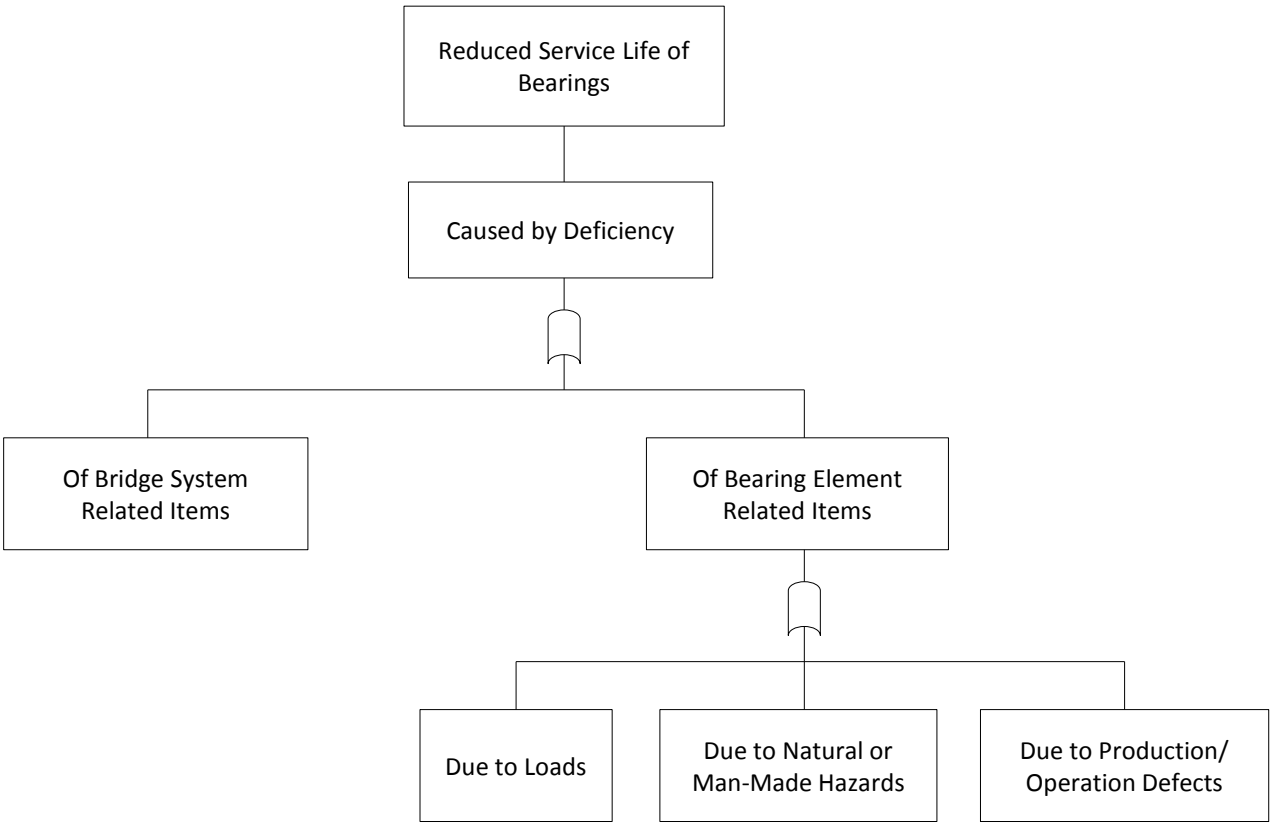


Figure 10.8. Fault tree analysis for factors affecting service life of bearings.

As discussed in Chapter 2, the first level affecting service life of a bridge system is either obsolescence (relating to function or operation) or deficiency (relating to deterioration or damage). In relation to bearings, however, service life issues are typically caused by deterioration or damage, not by obsolescence, so the fault tree moves directly to deficiency. Following this, deficiencies can be either:

- System-related (i.e., related to other items within the bridge system or to the layout of the system); or
- Bearing element related (i.e., related directly to bearing element performance).

Deficiencies can then be subcategorized to that caused by loads, natural or man-made hazards, or production/operation defects.

10.3.1.1 Deficiency of System-Related Items

System-related items whose deficiencies can directly affect bearings are primarily attributed to system production/operation defects, and are illustrated in Figure 10.9.

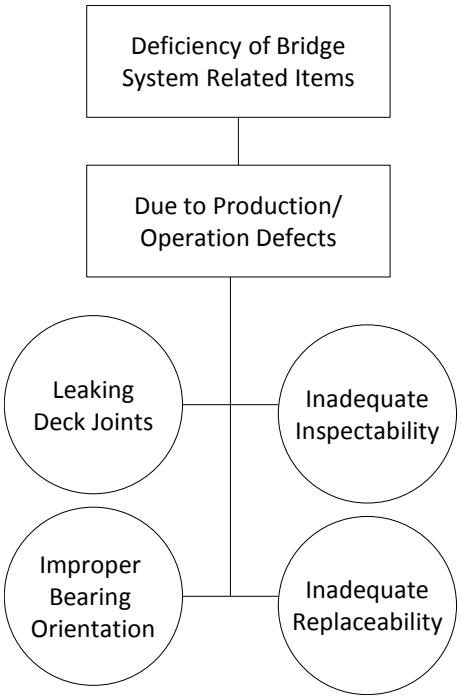


Figure 10.9. Bridge system-related deficiencies.

These factors typically relate to deficiencies in elements, details, and general layout of the overall bridge system that can adversely affect bearing performance. Other types of production/operation defects are discussed in Section 10.3.1.2.3 as they relate to individual bearing elements.

10.3.1.1.1 Leaking Deck Joints

Of all system-related deficiencies, leaking deck expansion joints can have the greatest negative impact on bridge bearings. This applies to both open and sealed joints.

Open joints, such as finger dams or sliding plate dams, typically allow drainage to pass through and be collected in troughs and below-joint drainage systems. Failure or clogging of these drainage systems allows deck drainage and debris to spill onto all bridge elements below, including bearings.

Sealed joints, such as compression joints, strip seals, or large modular joints, are intended to prevent deck drainage from spilling through. However, failure or damage to these types of joints can also allow deck drainage to leak through onto bridge elements below.

Bearings located below leaking deck joints—particularly those in northern wet climates—are subject to deck drainage, deicing chemicals, and other deck debris, which is a leading cause of deterioration and reduced service life. Drainage and deicing chemicals cause corrosion of exposed steel elements, and debris buildup affects proper rotation and expansion movement.

10.3.1.1.2 Improper Bearing Orientation

In skewed, curved, and wide bridges, bearings are subjected to multi-directional movements and/or rotations. Improper bearing orientation and/or inadequate multi-direction movement capacity can lead to higher stresses, wear, and reduced service life.

Bridges wider than three lanes can experience significant transverse thermal movement. Guides and keeper assemblies should be limited to the interior portions of the bridge that do not experience large transverse movements. Bearing details for outer portions on wide bridges should be designed to accommodate transverse movement.

In the case of skewed steel bridges, a phenomenon, commonly referred to as layovers exists during construction. The layovers subject the bearings to rotation of the steel girder about longitudinal axis of the girder (twisting of the section). This subjects the bearing to rotation that is not generally considered in design and subjects the bearing to multi-rotation.

10.3.1.1.3 Inadequate Inspectability

Proper inspection of bearings during their service life is critical in order to evaluate proper performance, wear, or deterioration. Early detection of problems can allow maintenance or repair before more serious conditions can develop. Shallower bearing types can be difficult to properly inspect, particularly when limited headroom prevents close access. Consideration should be given in overall bridge system design to allow access for proper inspection of bearings.

10.3.1.1.4 Inadequate Replaceability

Regardless of expected service life, bearings are subjected to severe service conditions, and have a high potential for unintended consequences related to improper design, manufacturing, installation, and maintenance that can lead to shorter service lives than other bridge elements. Consideration should be given in the overall bridge system design

to allow for easy replacement of bearings with minimal traffic disruption. AASHTO and NSBA provide recommended bearing details that facilitate replacement (AASHTO/NSBA 2004).

10.3.1.2 Deficiency of Bearing Related Items

Reduced service life of bearings is often caused by deficiencies of individual bearing elements themselves. As illustrated previously in Figure 10.8, bearing deficiency can be caused by loads, natural or man-made hazards, or production/operation defects.

10.3.1.2.1 Bearing Deficiency due to Loads

Figure 10.10 illustrates factors affecting bearing service life due to loads.

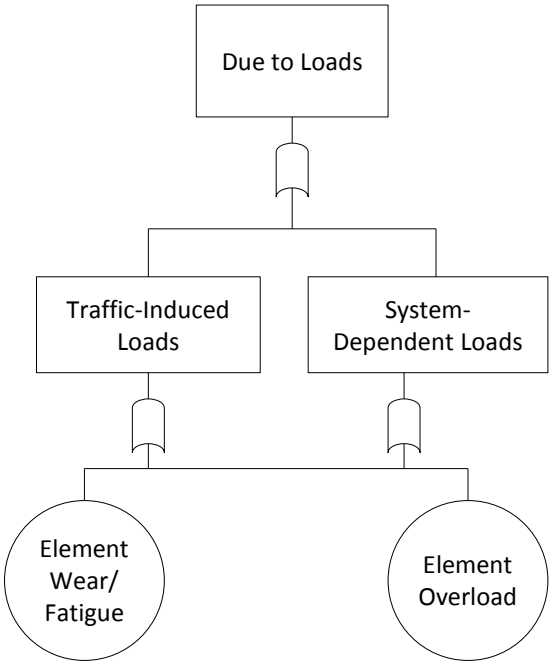


Figure 10.10. Factors affecting service life due to loads.

As illustrated, loads can be traffic loads (primarily truck loads), or system-dependent loads (primarily due to thermal activity). Each of these load types can result in element damage due to wear/fatigue or overload.

Truck traffic applies direct vertical load to bearings as well as superstructure end rotation and accompanying horizontal translation. Thermal activity applies horizontal and transverse translations and girder end rotation to bearings; however, depending on intended or unintended levels of restraint, thermal activity can also apply significant horizontal load to bearings.

Truck traffic produces high frequency, low amplitude cyclic movement at expansion bearings combined with vertical load. Cyclic movement from girder end rotation results in total cumulative movement that is significantly greater than total cumulative movement due to thermal activity. This behavior primarily affects wear of sliding surfaces.

Other loads such as those due to wind, longitudinal braking, or earthquakes can also apply various vertical and horizontal loads to bearings that must be resisted.

All of these loads can ultimately result in bearing element fatigue, wear, or overload at various levels depending on the specific bearing type and make up. For example, elastomeric bearings are subject to element fatigue, and PTFE sliding surfaces are subject to wear.

Overload, either from heavy truck loads or large thermal movement, in which the bearings experience greater loads than assumed in design, can also lead to reduced service life. Overload can result in various forms of damage depending on bearing type.

Incorrect assumptions during the design process can also significantly affect the service life of bearings because of system restraint. For example, as described in Chapter 8 on jointless bridges, in the case of curved girder bridges, there might not be a point within structure that can be designated as point of zero movement. Such an assumption can lead to use of a fixed bearing type, without any capability or allowance for transverse or longitudinal movements. The end result of such wrong assumption is that the bearing is subjected to actions that cannot be accommodated by bearing and the development of significant damage to bearing in the process.

Service life applications to specific bearing types are discussed later in this chapter.

10.3.1.2.2 Bearing Deficiency due to Natural or Man-Made Hazards

Figure 10.11 illustrates factors affecting service life due to natural or man-made hazards. For bearings, hazard-related deficiencies typically pertain to thermal climates, coastal climates, or chemical environments.

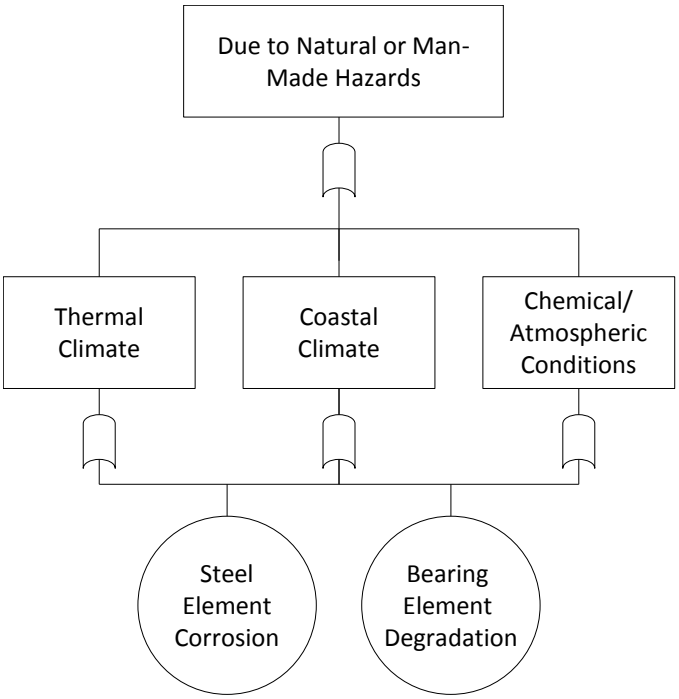


Figure 10.11. Factors affecting service life due to natural or man-made hazards.

Thermal climates pertain to cold and wet climates accompanied by snow and ice that result in high usage of deicing chemicals on roadways and bridge decks. Bridge bearings are affected by roadway drainage and salts leaking through expansion joints, or by salt spray rising up from crossed roadways.

Coastal climates are climates near the ocean or other salt water bodies where bridge bearing elements can be affected by airborne salt spray.

Chemical environments can include environments near chemical or industrial facilities where corrosive airborne chemicals can affect exposed bearing elements.

Ultimately, these climates or environments can result in exposed steel element corrosion or degradation of other bearing materials at various levels depending on specific bearing type and make up. More specific environmental factors are discussed as they relate to individual bearing types later in this chapter.

10.3.1.2.3 Bearing Deficiency due to Production/Operation Defects

Figure 10.12 illustrates factors affecting service life of bearings due to production/operation defects. For bearings, these defects can be in any one of four general subcategories:

- Design/Detail,

- Fabrication/Manufacturing,
- Construction, or
- Maintenance.

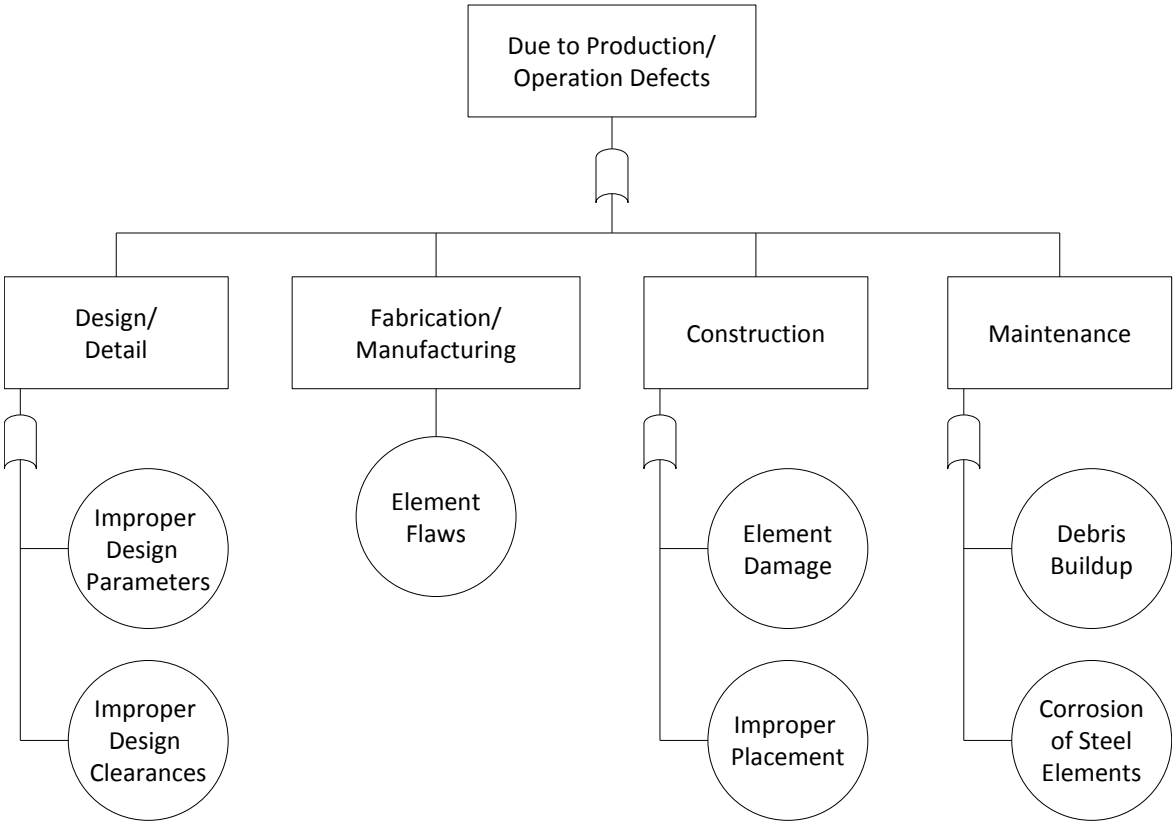


Figure 10.12. Factors affecting service life due to production/operation defects.

10.3.1.2.3a Design/Detail

Improper bearing design and accompanying details can result in significantly reduced service life. Major base factors include:

Improper Design Parameters—incorrect or improperly computed design values of vertical load, movement, and rotation, or combinations thereof. All bearings must be designed for proper superstructure loads, movements, and rotations. Improper calculation and application of these parameters at the design stage can result in bearings that are subject to excessive rotation, higher stresses, greater wear, and ultimately, reduced service life.

Improper Design Clearances—improper clearances within bearing details to permit proper movement and/or rotation. Details with inadequate clearances can cause binding that limits proper movement or rotation, which results in higher stress, damage, wear, and reduced service life.

10.3.1.2.3b Fabrication/Manufacturing

Material or fabrication flaws can lead to performance failure and reduced service life. Proper quality assurance (QA) and quality control (QC) procedures in accordance with current *AASHTO LRFD Construction Specifications* must be implemented in the fabrication or manufacturing of bridge bearings to ensure that the completed bearings meet specifications and provide required levels of performance (AASHTO 2010). See Section 10.4.1.2.3b for additional discussion.

10.3.1.2.3c Construction

Production/operation defects at the construction stage can be due to either element damage or improper placement. Protective care must be taken during field construction to prevent damage or contamination to sensitive bearing parts such as sliding surfaces or elastomeric pads or discs. Bearings must be set in the field at proper positions to accommodate installation temperatures and rotations.

10.3.1.2.3d Maintenance

Lack of or inadequate bearing maintenance can also lead to reduced service life. Issues typically relate to lack of cleaning, which results in debris build up below deck expansion joints, and steel element corrosion. Other types of maintenance related issues are discussed later for specific bearing types. Dirt and debris build up prevents proper bearing movement and rotation. Debris build up also retains and holds moisture and salt against exposed steel elements, which leads to corrosion if not cleaned.

10.3.2 Factors Affecting Service Life Unique To Each Bearing Type

This section looks at specific service life issues pertaining to bearing types described in Section 10.2 and addressed within the general fault tree categories described in Section 10.3.1.

10.3.2.1 Steel-Reinforced Elastomeric (SRE) Bearings

Steel reinforced elastomeric bearings have been in service in the United States for over 50 years, and longer elsewhere in the world, with very good results. They are typically very robust, and of all bearing types they likely

have the best chance for achieving a service life greater than 100 years. When properly designed, manufactured, and installed, there is very little that can go wrong, and long-term maintenance requirements are minimal. In rare instances however, certain problems have been observed, most often associated with production/operation defects relating to design and manufacturing.

10.3.2.1.1 Improper Design

In rare occurrences, improper design has led to overloaded pads, or pads subjected to excessive lateral movement, causing excessive bulging, splitting, or delamination. The following describes various potential failure modes and other issues with elastomeric bearings, and how they are addressed within current AASHTO design provisions. A significant amount of research has been performed on SRE bearings, and designs that follow recent provisions in the *LRFD Specifications* should adequately avoid these issues as they would affect service life.

10.3.2.1.1a Shear Deformation

Elastomeric bearings accommodate longitudinal and transverse expansion and contraction by shear deformation within the elastomer itself. If the shear displacements of the bearing are large, they may cause some rollover at the acute ends of the layer, which leads to cracking in the elastomer at the end of the top and bottom reinforcement plates. This condition is exacerbated by cyclic loading. Fatigue tests simulating temperature movement, which is low-cycle, high-amplitude movement, indicate that keeping the shear strain below 0.5 will prevent this. This is not conservative, however, if the deformation is due to high-cycle loading caused by braking forces or end rotation. It was found that high cycle fatigue was more damaging, and in those cases the maximum shear strain should be limited to 0.10. (Roeder et al. 1990)

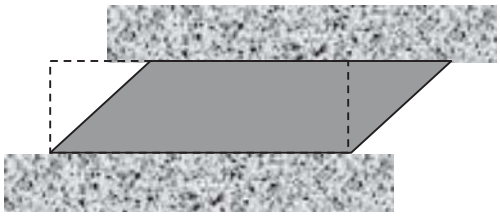


Figure 10.13. Shear deformation in elastomeric pad. (Stanton et al. 2008)

10.3.2.1.1b Plate Delamination and Debonding:

Shear delamination between layers of elastomer and steel reinforcing plates is the most significant potential mode of failure. Large shear strains occur in elastomeric bearings due to combined axial load, rotation, and shear,

and are illustrated in Figure 10.14 (Stanton et al. 2008). Each of these shear strains reaches its maximum value at the same location, namely the very edge of the layer at which the elastomer is bonded to the reinforcing plate. Under severe loading, this condition leads to local detachment and debonding of the elastomer from the plate. Once debonding occurs, the elastomer starts to extrude from the bearing, which in turn causes significant vertical deflection. Further, cyclic shear strain causes additional debonding damage than static shear strain of the same magnitude. Section 14.7.5.3.3 of the *LRFD Specifications* provides updated (2010) requirements for considering shear strain caused by combined axial load, rotation, and shear displacement, and considers an amplification factor of 1.75 for combined shear strains due to cyclic loading caused by traffic.

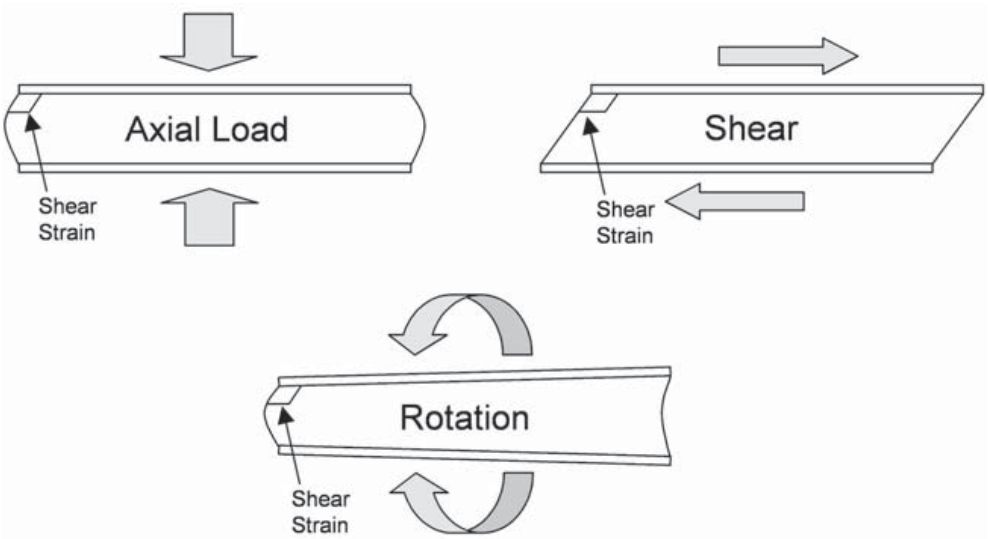


Figure 10.14. Deformations of a laminated elastomeric bearing layer. (Stanton et al. 2008)

10.3.2.1.1c Instability

Large lateral movement combined with large end rotations results in thick bearing designs. If the bearing becomes thick enough, instability can affect its performance (Stanton et al. 2008). The layered construction and the very low shear modulus of the rubber combine to cause this potential problem. Quite often the bearing is made as wide as possible (transverse to the girder axis) and then only a short length is needed to provide sufficient bearing area for supporting the axial load. However, too short a length would again risk instability. In such bearings, the axial stress may therefore be significantly lower than the limit because of the indirect influence of stability requirements. Section 14.7.5.3.4 of the *LRFD Specifications* provides updated (2010) requirements for stability, and limits the average compression stress to half the predicted buckling stress.

10.3.2.1.1d Plate Fracture

Lateral expansion of the elastomeric layers causes tension in the steel reinforcing plates (Stanton et al. 2008). At extreme loads the plates could fracture, typically splitting along the longitudinal axis of the bearing. However, in plates of the thickness currently used, this behavior does not occur until the load has reached five to 10 times its design value, so plate fracture seldom controls design. Plates could actually be thinner but they are typically sized in practice to keep them from bending during the molding process. Section 14.7.5.3.5 of the *LRFD Specifications* provides requirements for the thickness of steel reinforcement, and considers both strength and fatigue.

10.3.2.1.1e Compressive Deflection

Instantaneous compressive deflection is limited by the Specifications in order to avoid damage to deck joints and seals, and to avoid additional impact when traffic passes from one girder to the other across a joint. A maximum relative live load deflection across a joint of 1/8 in. is recommended. Section 14.7.5.3.6 of the *LRFD Specifications* provides requirements for compressive deflection.

10.3.2.1.2 Improper Fabrication/Manufacturing and Installation

In addition to proper design, it is imperative that proper manufacturing and installation also be performed, and effective quality control during these processes is essential for successful performance.

One of the most commonly reported field problems associated with elastomeric bearings, albeit in only few instances, has been walking out, or slipping from the original position under the girder. Slippage was occurring primarily in elastomeric pads made of natural rubber because of paraffin that was added during the manufacturing process in order to meet required ozone degradation requirements established by AASHTO. Neoprene pads have inherently greater ozone resistance and do not need wax additives. When used, these waxes over time will bleed to the bearing surfaces and drastically reduce the coefficient of friction between the bearing and its contact surface, which leads to slipping (Chen and Yura 1995). Use of positive attachment to the substructure or superstructure, such as bonding, is recommended to prevent this occurrence. However, caution needs to be exercised in bonding elastomeric pads and contact surface in highly skewed steel bridges because of layovers.

10.3.2.2 Cotton Duck Pads (CDP)

Cotton duck pads are only used by a few states so their performance history is limited. Like SRE bearings, potential service life issues for CDP are most often associated with production/operation defects and CDPs require proper design, manufacturing, installation, and maintenance.

For SRE bearings, longitudinal and transverse movements of bridge are accommodated by shear deformation of the elastomer and the reinforcement has little influence on the shear stiffness of the bearing. CDPs behave differently. The fabric layers of CDP are many and closely spaced, and result in significantly larger shear stiffness and smaller shear deformation capacity than in other elastomeric bearing types. As a consequence, CDP tolerate large compressive stresses, but limited shear deformation because interlayer splitting occurs at much smaller shear strains. Also, the large shear stiffness of CDP can result in slip of the girder on the CDP, which may result in abrasion and deterioration of the CDP. As a consequence, translational shear strain is limited to only about 10%. That is, the shear deflection is limited to only 1/10 of the total CDP thickness. Larger movement requirements with CDP must be accommodated by the addition of a sliding surface such as PTFE sliding surface (Lehman et al. 2003).

The stiffness and deformation capacity of CDP varies from manufacturer to manufacturer. Proper quality control testing is necessary to assure that the bearing pad provides adequate performance.

Delamination of elastomer layers or secretion of oil and wax from the CDP are the common serviceability limit states for CDP. To control delamination, compressive stress limits of 3000 psi for total dead plus live load and 2000 psi for live load are recommended. Dynamic or cyclic rotation, which induces uplift or partial separation of the pad from the load surface, may cause delamination and reduced service life. Uplift damage depends on the maximum total rotation, as well as the rotation range caused by the live load variation, and separate rotation limits are provided. *LRFD Specifications* now include design provisions to ensure the serviceability and durability of CDP.

10.3.2.3 Sliding Surfaces (Primary Sliding Surfaces, Not Guide Bars)

Factors affecting service life of bearing sliding surfaces most often relate to deficiencies caused by loads, including both traffic (truck load) and system-induced (thermal movement). These can result in sliding element wear (from cyclic movement) and creep or cold flow (from compressive overload).

Deficiencies caused by production/operation defects can also occur, primarily during manufacturing and construction. They typically relate to surface scratching and damage due to inadequate protection during shipping or installation. Surface damage leads to increased wear. These problems can be mitigated by proper protection and inspection during shipping and installation.

Inadequate maintenance, particularly below open or leaking deck expansion joints, can allow buildup of dirt and other debris that can cause contamination of sliding surfaces and increased wear. Periodic maintenance and cleaning in these areas can mitigate these problems.

10.3.2.3.1 Wear of Sliding Surfaces

Plain PTFE is most commonly used for bearing sliding surfaces. It wears under service conditions and may require replacement after a period of time. Low temperatures, fast sliding speeds, high contact pressures, rough mating surfaces, and contamination of the sliding interface increase the wear rate. However, fast sliding speed has been shown to be the more dominant parameter (Stanton, et al. 1999). Movement due to temperature change is low-cycle, high-amplitude movement, with a slow movement rate, and produces the least amount of wear. However, movement due to truck load and associated dynamic effects is high-cycle, low-amplitude movement and has a much faster sliding speed, by as much as a factor of 10. Wear rates associated with high sliding speeds can be as much as 150+ times greater than wear rates at lower sliding speeds. Thus, plain PTFE should not be used as a sliding surface for bearings subject to relatively high sliding speeds and low temperatures.

Relatively thin layers of PTFE, from 1/16 in. to 3/16 in., are commonly used in the United States, but engineers in other countries often use thicker PTFE layers to accommodate wear.

Woven or glass filled PTFE surfaces provide much higher overall wear resistance, especially at higher sliding speeds, but these surfaces have higher friction coefficients that must be taken into account in the bridge system design.

Dimpled and lubricated PTFE also provides exceptional wear resistance and low friction, but the long-term effectiveness of lubrication is questionable (Stanton et al. 1999).

MSM® has been shown to provide exceptional wear resistance, but when used in a dry condition (without lubrication) it has a much higher friction coefficient than plain PTFE. When used in a dimpled and lubricated condition however, its friction coefficient reduces considerably and is more comparable to lubricated PTFE.

10.3.2.3.2 Creep or Cold Flow

PTFE may creep (or cold flow) laterally when subjected to high compressive stress and shorten the life of the bearing. The reduction in PTFE thickness may also allow hard contact between metal components. Thus, while the compressive stress should be high to reduce friction, it must also be limited to control creep. PTFE is frequently recessed for one half its thickness and bonded to control creep and permit larger compressive stress.

Filled PTFE, which is reinforced with fiberglass or carbon fibers, has significantly greater resistance to creep, and is sometimes used to resist creep or cold flow.

10.3.2.4 HLMR Pot Bearings

In past years, pot bearings have had service life problems most often associated with production/operation defects relating to design and manufacturing. These issues have ultimately resulted in broken internal seals, leakage or extrusion of the elastomer, abraded elastomeric pads, and metal-to-metal contact, which have led some state departments of transportation (DOT) to recommend avoiding pot bearings altogether. However, improved design specifications and tighter manufacturing tolerances developed in the late 1990s have greatly improved overall performance. Current *LRFD Specifications* (2012) now incorporate research findings reported in *NCHRP Report 432, High-Load Multi-Rotational Bridge Bearings, 1999,* by Stanton, Roeder, and Campbell (Stanton et al. 1999), and address specific past service life issues. The following items have been recommended to mitigate certain past deficiencies, and are now part of standard design and manufacturing practice:

- Rotational capacity of pot bearings is limited by the clearance between various elements of the pot, piston, sliding surface, guides, and restraints (Stanton et al. 1999). Inadequate clearances can cause binding between metal components. See Figure 10.15. Clearance requirements are now included in the specifications.

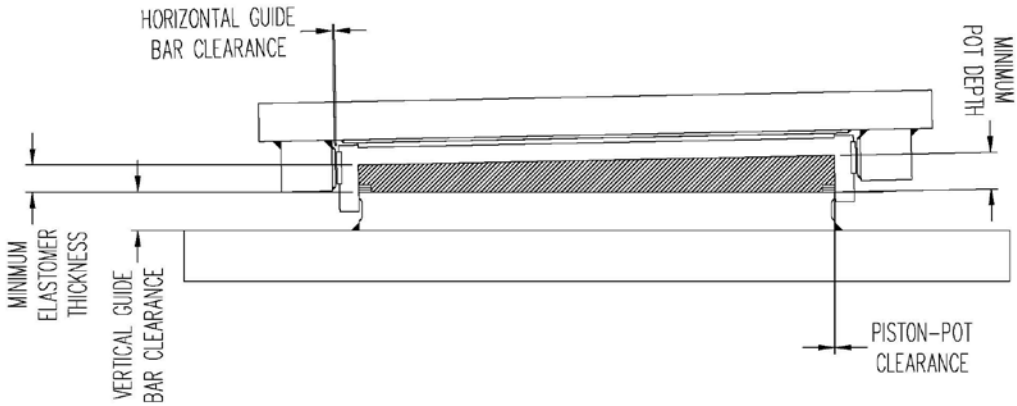


Figure 10.15. Critical clearances affecting rotation capacity.

(Courtesy D.S. Brown)

- Thickness of the elastomeric pad also affects rotational capacity, and is currently controlled by a 15% strain limit on the pad edge deflection under rotation. This strain value is based more on past practice than research results, but it is believed to be a reasonable value.

- Pot bearings are able to sustain many cycles of small rotation better than a smaller number of large rotation cycles. This is believed to be true because smaller rotations cause deformation of the elastomer but little slip. Slip caused by larger rotations abrades the surface of the elastomer. In an attempt to account for this, pot bearings should be designed for larger minimum rotations that reduce the potential for over-rotation. This minimum rotation should reflect increased rotation caused by construction tolerances expected in practice. Further, greater emphasis should be placed on calculation of rotations due to service loads, construction loads, and environmental conditions.

- Rotational resistance, wear, and abrasion are significantly reduced with a smooth surface finish inside the pot and on the piston. Metalizing of these interior surfaces for corrosion protection causes a rougher surface that leads to increased damage under cyclic rotation, and should not be used unless buffed to a smooth finish. In extreme corrosive environments, stainless steel could be considered.

- Failure of sealing rings causing escape of elastomer has been one of the major service life issues with pot bearings in past years. Solid circular cross section brass rings and multiple flat brass rings have both been used, and each has advantages and limitations. Circular cross section rings provide a tight seal but are susceptible to wear under cyclic rotation. Flat rings appear to be more susceptible to leakage and ring fracture, but they experience less wear. Heavier flat brass rings have been suggested as a means of improving their performance. The performance of circular rings could also be improved if the friction and wear were reduced. Currently, both circular cross section and flat brass rings are permitted in the specifications. Current design and manufacturing in accordance with *LRFD Specifications* have greatly resolved past issues with sealing rings and leakage of elastomer.

- Silicon grease lubrication appears to reduce the wear noted on rings, pot walls, and piston, and is recommended for use. This lubrication does not reduce the ultimate wear of the elastomeric disc.

- Relatively small lateral load (5% of gravity load) when combined with cyclic rotation can dramatically increase the rotational resistance, wear, and leakage of pot bearings. This damage is caused by the piston rim dragging against the pot wall during rotation. Alternate methods of external restraint are recommended for large lateral loads to mitigate this damage potential.

- Dirt or contamination in the pot increases wear and abrasion of the rings, pot, and elastomeric disc, and increases the potential of elastomer leakage. To mitigate this potential, pot bearings need to be sealed and protected during shipping and installation.

Although newer design and manufacturing criteria have improved the overall performance of pot bearings in recent years, it is still recognized that this bearing type has internal moving parts subject to wear and abrasion that can lead to reduced service life. Newer requirements for long-term deterioration testing in accordance with current *AASHTO Construction Specifications* provide greater assurance of proper performance.

10.3.2.5 HLMR Disc Bearings

Since their first use in the early 1970s, these bearings have had good performance and few reported field problems. Potential service life problems (albeit few) would most likely be associated with production/operation

defects relating to design and manufacturing. Current specifications are minimal regarding design, and are generally performance related. Therefore, performance testing and manufacturing QC measures are necessary to verify compliance.

Previous research testing of disc bearings for combined load and rotation (Stanton et al. 1999) made several performance conclusions:

- Tests showed that rotation of disc bearings is partly accommodated by uplift of the steel plates from the urethane disc, especially if the compressive load is light. This should not result in any problems with fixed bearings, but could be a concern with sliding bearings, since uplift of the disc produces edge loading on the PTFE sliding surface. To mitigate this potential problem, *LRFD Specifications* limit the edge contact stress on PTFE surfaces and *AASHTO Construction Specifications* require proof load testing.

- Tests showed the urethane disc to be somewhat deformed and abraded by cyclic rotations, but the damage was not so severe as to affect the performance of the bearing. *AASHTO Construction Specifications* require cyclic deterioration testing to confirm long-term performance.

10.3.2.6 HLMR Spherical Bearings

Although a very robust system, these types have had service life problems, most often associated with production/operation defects relating to design and manufacturing. The difficulties appeared to be attributable to faulty fabrication by manufacturers that were not necessarily first-tier suppliers. These bearing types require very precise manufacturing tolerances to assure proper fit of the curved mating surfaces.

Rotation capacity can be set at almost any desired level provided adequate clearances are provided.

PTFE surfaces may also eventually wear out. Variations in friction with different types of PTFE and under different temperature and load conditions cause variations in behavior that can lead to performance issues. Woven PTFE has often been used with spherical bearings in the United States, while dimpled and lubricated PTFE is often used in Canada and Europe.

These types of bearings are typically larger than other types and generally require additional space. Spherical bearings are traditionally considered to be the most expensive HLMR bearing type, but are also traditionally considered the most reliable.

10.3.2.7 Fabricated Mechanical Steel Bearings

Fabricated mechanical steel bearings have been used for the longest time of any bearing type and have the potential for extended service life if properly protected and maintained. Factors affecting service life relate to several categories including loads, primarily overload, which results in binding or over-rotation of rocker bearings; natural or manmade hazards, which result in steel element corrosion; and production/operation defects, specifically due to lack of maintenance.

10.3.2.7.1 Loads

Rocker expansion bearings can be designed for wide ranges of movement, but can have limited potential for accommodating overload. Excessive translation can lead to rockers tipping over, which has been reported in a few instances.

10.3.2.7.2 Natural or Man-Made Hazards

Corrosion of steel bearings, particularly those located below open or leaking deck joints in thermal environments, is the highest cause of reduced service life for these bearing types. In these locations, steel bearings are highly susceptible to corrosion due to roadway drainage with deicing salts and other dirt accumulation. Coastal climates and other chemical environments are also catalysts for steel element corrosion.

Pin, roller, and rocker bearings have direct metal-to-metal contact, which creates an environment of high stress concentration and accumulation of moisture between surfaces. In the areas of contact, any protective coating on the steel is inevitably damaged by the relative movement. All of these conditions contribute to corrosion.

Corroded expansion bearings can lockup or freeze, subjecting beams and substructure elements below to additional load and potential damage. Fyfe et al. 2006, reported that steel rocker and roller bearings are the most susceptible to freezing in position along with older type metal sliding plates. When bearings are frozen, bridges develop their own provisions for contraction and expansion, such as pier cap cracking or rocking of piers or abutments.

Dirt and debris accumulation has caused some rocker-type expansion bearings to move beyond their limits and actually roll over causing the superstructure to drop several in. or more. Figure 10.16 illustrates a ratcheting effect reported by Modjeski and Masters in 2008, whereby rocker bearings at one pier on the Birmingham Bridge in

Pittsburgh, Pennsylvania tipped over because of corrosion and debris accumulation. The rockers had an initial lean, and a leaking expansion joint above caused corrosion and increasing accumulation of debris on the bearings and under one side of the rockers. A ratcheting effect followed whereby additional debris and corrosion material kept accumulating under the rockers causing additional and increasing tilt. This continued until a lateral force or kick developed against the pier cap, which caused the pier to move just enough to initiate the final tipping of the rocker bearings.

Steel rocker bearings have also performed poorly in seismic events and have been replaced as part of seismic retrofit in many instances.

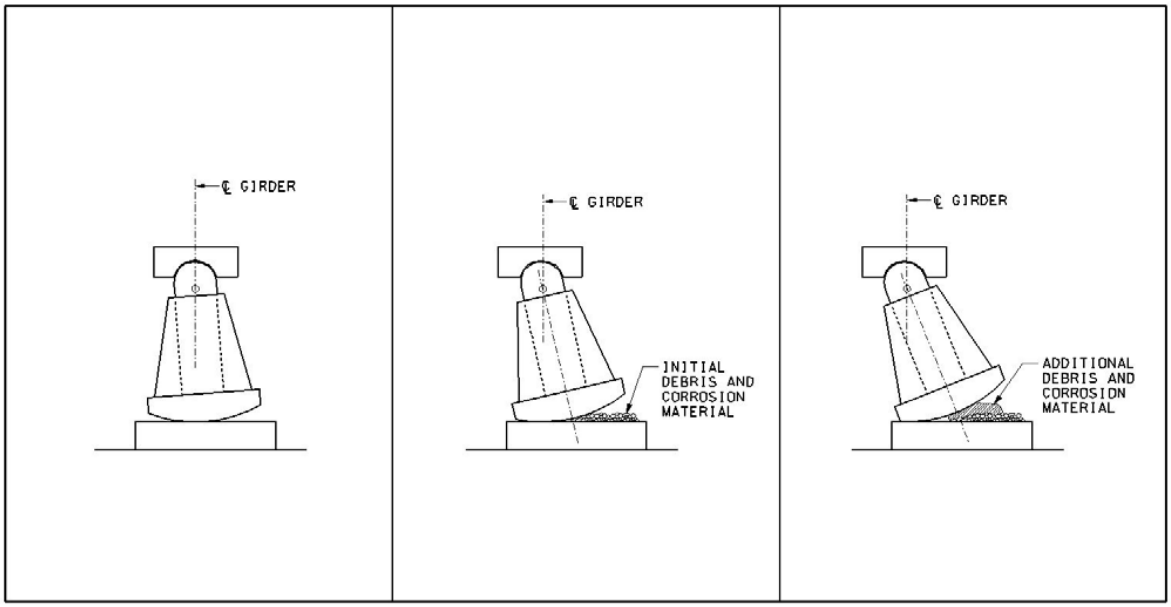


Figure 10.16. Ratcheting effect caused by debris and corrosion. (Modjeski and Masters 2008)

10.3.2.7.3 11.3.2.7.3 Production/Operation Defects

Lack of maintenance has been a contributing factor to reduced service life for steel bearings in many instances. Coating loss, surface corrosion, and debris buildup, all of which lead to reduced service life, can be effectively mitigated by periodic maintenance if performed. In locations below open deck expansion joints, particularly in thermal environments where bearings are highly susceptible to roadway drainage, salt, and debris, periodic cleaning to prevent buildup of deleterious materials is essential.

10.4 OPTIONS FOR ENHANCING SERVICE LIFE OF BEARINGS

This section presents available options for mitigating various bearing service life issues that were identified in Section 10.3. A procedure to select the optimal bearing type for given loads, movements, and environmental conditions is presented in Section 10.5.

Options for enhancing service life of bearings are presented in a way that addresses the issues discussed in the fault tree analysis and the specific categories and sub-categories of deficiencies that were shown to cause damage or deterioration. As previously illustrated, bearing deficiencies can be either related to bridge system deficiencies or related directly to bearing element deficiencies. Deficiencies can then be attributed to those caused by loads, natural or man-made hazards, or production/operation defects.

In the discussion of available options for enhancing service life of individual bearing types, the following general solution categories are included and addressed as applicable:

- Avoidance eliminates or bypasses a particular service life issue. This is a primary consideration, and should be incorporated where possible.

- Mitigation improves performance through enhanced materials, greater protection, or effective maintenance. Where avoidance is not practicable, mitigation techniques should be implemented to improve service life.

- Acceptance considers that a particular bearing or bearing component may not be able to achieve a service life equal to the bridge system design service life, even after mitigation to improve its service life, and may eventually need to be replaced.

In many instances, the need to provide capabilities and details for easy bearing replacement may still be a necessary consideration because of uncertainties regarding loads, hazards, or production/operation defects that can cause premature service life issues.

10.4.1 General Options for All Bearing Types

10.4.1.1 Solutions for Deficiencies of System-Related Items

The following table summarizes solutions for various service life issues identified in Section 10.3.1.1, and briefly identifies what each solution provides and what other considerations may still be needed. System-related issues were

primarily due to production/operational defects and, more specifically, due to design and details of components within the overall bridge system. Further discussion relating to each issue follows this table.

Table 10.2. Solutions for Service Life Problems–All Bearing Types.

General Options for All Bearing Types			
Bridge System-Related Deficiencies			
Service Life Problems	**Solutions**	**Advantages**	**Disadvantages**
Leaking deck joints Bridge systems with deck joints can have deterioration of bridge elements below deck caused by drainage through leaking deck joints carrying deicing salts and other debris.	Avoid by using integral or semi-integral construction at abutments	Fully integral abutment eliminates joints and bearings. Semi-integral eliminates joints. Prevents deterioration due to drainage and chlorides to elements below deck.	Requires an understanding of integral abutment behavior and limitations. See Chapter 8 on jointless bridges for detail design provisions.
	Avoid by using integral construction between superstructure and substructure at piers	Eliminates deck joints, bearings, and the need for future bearing maintenance. At piers, can also avoid sharp skews, or longer spans, or higher profiles.	Requires system design approach to accommodate bridge movements. Some systems require posttensioning for integral cap design and construction. May be more costly than isolated construction with bearings.
	Avoid by using continuous superstructure over piers	Eliminates deck joints and protects bearings below	None
	Mitigate by protecting bearings with coatings and maintenance	Coatings on steel surfaces protect against corrosion. Maintenance improves coating life and prevents debris buildup	Requires continual maintenance
	Mitigate by repair or replacing leaking joints	Cost-effective solution for existing conditions	Requires continual maintenance
Mis-orientation of expansion bearings on complex alignments	Align bearings along girder chords for curved bridges, in keeping with current recommendations Align bearings along long diagonal line on deck for skewed bridges. See Chapter 8 on jointless bridges for additional detail discussions and recommendations.	Proper orientation of guided expansion bearings prevents wear and binding against restraint mechanisms.	Requires understanding of bridge system behavior
Limited access for inspection	Provide details and clearances that facilitate bearing inspection and maintenance	Early detection of problems avoids major deterioration or distress. Reduces risk	Possible additional initial cost
Difficulty in replacing deteriorated bearings	Provide details that facilitate jacking and replacement under traffic	Minimizes impact to traveling public during rehabilitaion. Facilitates speed of replacement.	Additional initial cost is higher but saves greater future cost when replacing bearings

10.4.1.1.1 Leaking Deck Joints

The following are possible solutions for avoiding or mitigating bearing damage or deterioration caused by leaking deck joints. In considering options for improving service life, avoiding deck expansion joints by using continuous systems and/or integral systems offers the best opportunity. Where possible, integral systems that eliminate bearings entirely should also be considered, but in most cases this will not be feasible or practical, and use of bearings cannot be avoided.

- Integral abutment systems that avoid deck joints and bearings. Refer to Chapter 8 on jointless bridges, for general design guidelines. This has become a popular approach by many states for improving bridge service life. Fully-integral abutment construction eliminates both deck joints and bearings at abutments, and is also lower in initial cost. Semi-integral construction eliminates deck joints.

- Proper use of integral abutments requires an understanding of pile and cap behavior and limitations regarding bridge length and geometrics such as curvature and skew.

- Integral pier systems that avoid deck joints and bearings. See Chapter 2 on system selection, for more detailed discussion of integral pier options.

- Bridge systems using integral girder/pier cap construction have been used occasionally by some states to accommodate vertical clearance issues, to avoid sharp skews, or to develop frame action for seismic design. But they can also serve to eliminate joints, bearings, and associated future maintenance. Although there is a higher initial structure cost, savings are realized with integral pier caps in lower approach fills and lower long-term maintenance.

- Cast-in-place, post-tensioned integral bent caps have been used for many years, but are not widely employed. The Tennessee Department of Transportation (DOT) had its first application in 1978, with several others since, and all have performed well. The concept allows main longitudinal girders to pass directly through the pier's cap, rather than over the top in the traditional manner.

- Integral caps can also maximize column efficiency. Frame action in the longitudinal direction reduces column design moments at column bases compared to conventional cantilever columns, and can also enhance

seismic performance. However, it must also be considered that integral pier columns may also need to resist additional longitudinal moments due to live load, and a system analysis is required.

- When multiple piers are made integral with the superstructure, expansion must be accommodated by flexure of the pier columns. Tall, slender columns are best suited for this type of construction because their greater flexibility can accommodate temperature movement with less force developed.

- Continuous superstructure systems that avoid or minimize number of deck joints. These systems include fully-continuous, continuous for live load, or continuous deck slabs (link slab). These continuous systems can be combined with either integral abutments or conventional abutments. Bearings are still utilized at piers, but are protected by continuous superstructure.

- Mitigate for bearings below deck joints. When bearings must be located below deck expansion joints, mitigation procedures should be implemented. Depending on the environmental severity, all exposed steel bearing parts, which also includes sole plates, masonry plates, guides, and anchor bolts, should be stainless steel, galvanized, or metalized. These areas should also be cleaned periodically as part of a regular maintenance program to prevent salt and debris build up.

- Mitigate by repairing or replacing leaking joints. In existing bridges with deck joints, a cost effective strategy is to repair or replace joints when they start to leak, which would prevent any deterioration below the joint. This proactive approach to preventative maintenance requires periodic inspection with immediate repair or replacement when necessary.

10.4.1.1.2 Improper Expansion-Bearing Orientation

Proper orientation of bearings in curved and skewed bridges, which are subjected to multi-directional movements and/or rotations should be provided. Improper bearing orientation and/or multi-directional movement demand can lead to higher stresses, damage, wear, and reduced service life.

- Curved girder bridges. These bridges do not expand and contract along girder lines. A typical approach is to assume movement to occur along chord lines from the fixed point to expansion points. See Figure 10.17.

See Chapter 8 on jointless bridges for additional discussion on determining the point of zero movement or fixed point in integral abutment bridges.

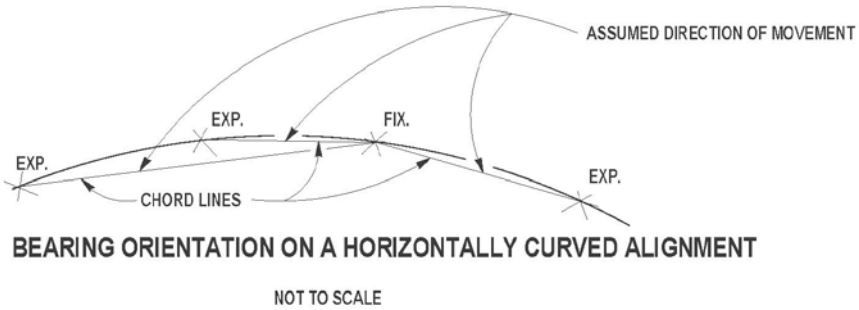


Figure 10.17. Recommended bearing orientation on a horizontally curved alignment. (Courtesy NSBA)

- Large skew bridges. For these bridges, one approximate solution is to consider the major axis of thermal movement along the diagonal from the acute deck corners due to thermal movement of the bridge deck. The alignment of bearings should be parallel to that axis. See Figure 10.18. It is necessary for expansion bearings to have multi-directional capabilities.

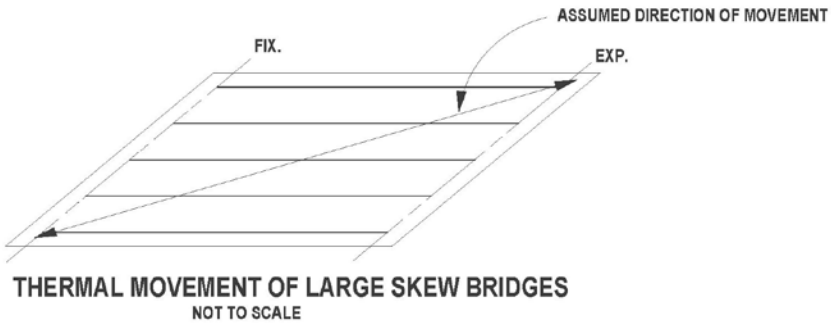


Figure 10.18. Recommended bearing orientation for large skew bridges. (Courtesy NSBA)

- Wide Bridges. Bridges wider than three lanes can experience significant transverse thermal movement. Guides and keeper assemblies should be limited to the interior portions of the bridge that do not experience large transverse movements. Bearing details for outer portions on wide bridges should be designed to accommodate transverse movement combined with longitudinal movement.

10.4.1.1.3 Proper Access for Bearing Inspection

Details, particularly sufficient room and access at tops of piers and abutments to permit proper bearing inspection, should be provided. Early detection of bearing related issues can allow maintenance or repair before major service life-reducing events can develop. When possible, shallow bearings should be placed on pedestals at

tops of piers and abutments in order to provide greater clearance for inspection. Use of thicker sole plates can also help provide greater clearance.

10.4.1.1.4 Proper Capability for Bearing Replacement

Details that will accommodate the potential need for replacement of all or part of the bearing system should be provided with the following considerations:

- Jacking locations should be provided at every girder. An alternative is to provide for jacking under a diaphragm that lifts adjacent girders simultaneously.

- Bearing attachment details should allow ease of replacement. If the bearing is unattached, it can easily be pulled from its position when the load is removed. Welds can be cut, but doing so requires equipment that may be cumbersome in the space available. Grinding may also be needed to produce a flat enough surface for installing the new bearing. Any anchor bolts should be placed so that they do not impede the removal of the bearing.

- Bearings should be detailed in a way that they can be replaced with only 1/4 in. of jacking to avoid causing a bump at the top of the deck that would affect traffic. The details also need to provide adequate vertical clearance for jacking. Jacking points on the structure should be designed to accommodate both dead load and live load in order to be able to maintain traffic during bearing replacement. (Actual jacking loads may have to include a factor applied to the design load to break loose the component before lifting.) Jacking for elastomeric bearings needs to consider the effect of compressive dead load strain in determining the amount of lift.

10.4.1.2 Solutions for Bearing Related Deficiencies

10.4.1.2.1 Due to Loads—Traffic- and System-Dependent

Service life issues can be avoided by:

- Determining truck traffic-induced cyclic movement for consideration in particular bearing type design, or

- Determining proper levels and combinations of load, movement and rotation.

10.4.1.2.2 Due to Natural or Man-Made Hazards

The potential for steel element corrosion should be avoided by using stainless steel, or mitigated by high performance protective coatings (galvanizing or metalizing).

10.4.1.2.3 Due to Production/Operation Defects

10.4.1.2.3a Design/Detail Related

Proper Loads, Movement, and Rotation

Design should take into account proper levels and possible combinations of load, movement, and rotation in order to avoid problems due to excessive stress, translation or over-rotation. A considerable number of bearing failures are attributed to improper allowance for displacements.

Further, construction tolerances and possible construction loadings, and how they might affect bearing loads and movements should also be considered. It is recommended that the worst possible combination of loads and displacements due to construction tolerances be considered. In other words, design displacements should include the largest sum of displacements caused by the worst out of level placement of piers and abutments, the largest camber and deflection, and the most adverse tolerances permissible in the construction of the bridge and bearing.

In *AASHTO/NSBA Steel Bridge Collaboration—Steel Bridge Bearing Design and Detailing Guidelines*, published in 2004, Appendix A provides recommendations for calculating beam rotations for dead load and live load conditions. There is great variation in the methods used in the industry for calculating live load rotations, and the guide was developed based on methods used in several states. A realistic approach for computing beam rotations is presented in the NSBA guideline.

Proper Clearances

Adequate clearances should be provided for horizontal movement and rotation to prevent binding, wear, or damage to restraining devices, anchor bolts, or internal bearing elements. Adequate widths and clearances at pier and abutment bridge seats should be provided to allow for required movement and rotation.

10.4.1.2.3b Fabrication/Manufacturing Related

Proper QA/QC procedures in the fabrication or manufacturing of bridge bearings are essential to assure that the completed bearings meet specifications and provide the required levels of performance. Current *AASHTO LRFD*

Construction Specifications provide minimum requirements for packaging, handling and storage, fabrication tolerances, materials and general performance. Testing requirements include: material certification tests, material friction tests, dimension checks, clearance tests, bearing friction tests, long term deterioration tests, proof load tests, and horizontal force capacity tests. The specifications require that manufacturers provide certification that each bearing satisfies the requirements of the contract drawings and construction specifications.

10.4.1.2.3c Construction Related

Care and protection during field construction should be provided in order to prevent damage to protective coatings or damage/contamination to sensitive bearing parts such as sliding surfaces or elastomeric pads or discs.

Proper QA/QC procedures should be provided during construction to assure that bearings are initially set with specified position, clearances and rotation.

10.4.1.2.3d Maintenance Related

Periodic maintenance cleaning should be provided at intervals depending on the location within the bridge system and environmental hazard conditions to avoid debris build up that could affect movement and rotation performance.

Steel surface cleaning and coating maintenance should be provided in order to prevent corrosion of exposed steel elements.

10.4.2 Options Related to Specific Bearing Types

10.4.2.1 Elastomeric Bearings—Plain and Steel Reinforced

The following technology table summarizes solutions for various service life issues identified in Section 10.3.2.1 for elastomeric bearings, and briefly identifies the advantages and disadvantages of each solution. Problems with elastomeric bearings, albeit rare, have been associated with production/operational defects relating to design, manufacturing, and installation. Following this table is further discussion relating to each issue.

Table 10.3. Solutions for Service Life Problems—Elastomeric Bearings.

Plain and Steel Reinforced Elastomeric Bearings			
Service Life Problems	**Solutions**	**Advantages**	**Disadvantages**
Improper element design resulting in excessive shear deformation, and excessive bulging, splitting, or delamination	Avoid by following current AASHTO LRFD design criteria for Methods A or B	Very robust bearing with minimal problems and need for maintenance when properly designed, manufactured, and installed AASHTO design criteria have incorporated recent research to avoid failure modes including combined axial load, shear, and rotation.	None
Need for greater longitudinal movement capacity	Combine with high performing sliding surfaces. Consider.	Can provide unlimited movement capability and longer service life	Additional initial cost
Need to accommodate multi-direction complex movement (skewed or curved bridges)	Use circular bearing pads	Accommodates multi-directional movement and rotation. Well suited for cases in which transverse movements are uncertain.	May require wider bridge seat Not as efficient as rectangular pad with proper orientation
Slipping or "walking out" (This has been more of a problem with plain pads.)	Control the use of wax additives in elastomer	Prevents reduction in coefficient of friction between pad and bearing surface	Less resistance to ozone degradation, but should not be an issue with neoprene
	Provide positive attachment to sole plates	Restrains pad from walking out	Small additional initial cost
Improper fabrication/ manufacturing	Provide effective QA/QC	Assures adherence to quality and performance requirements	None

10.4.2.1.1 Improper Element Design Resulting in Excessive Shear Deformation and Excessive Bulging, Splitting, or Delamination

These problems can be avoided by following recent provisions (2010) in the *LRFD Specifications* for either Method A or B. As previously discussed, improper design has, in some instances, resulted in deficiencies. Extensive research has identified various failure modes when pads are subjected to excessive shear deformation or overload. See Section 10.3.2.1.1 for further discussion of failure modes.

10.4.2.1.2 Need for Greater Longitudinal Movement Capacity

When there is a need to accommodate large displacements along with smaller axial loads, such as might be found at the end spans of a long continuous bridge, the use of a low-profile elastomeric bearing combined with a PTFE or

higher performing sliding surface offers a practical solution. In these cases, cyclic end movement due to girder rotation caused by truck load can cause PTFE surface wear. This can be avoided by designing the elastomeric pad to accommodate rotation and the smaller cyclic movement due to truck loads, and the sliding surface to accommodate the larger longitudinal movement due to thermal load.

10.4.2.1.3 Need to Accommodate Multi-Direction Complex Movement

When movement direction is not readily determined, such as with curved or skewed bridges, use of circular pads can more easily accommodate multi-directional requirements. Circular pads typically offer an advantage by requiring narrower bridge seats than rectangular pads when placed on a skew; however, circular pads are not as efficient from a design standpoint as properly oriented rectangular pads.

10.4.2.1.4 Slipping or Walking Out

This can be avoided with the following solutions:

- Limiting use of wax additives in the manufacturing of the elastomer. This prevents or reduces future bleeding of the wax to the contact surface, which results in a much lower coefficient of friction and bearing slipping.

- Providing positive attachment of the elastomeric pad to the bearing sole plates.

10.4.2.1.5 Improper Fabrication/Manufacturing

Effective quality control during manufacture is essential for successful performance. Proper cleanliness and care during fabrication helps prevent debonding of reinforcing plates with elastomer.

10.4.2.2 Cotton Duck Pads (CDP)

The following technology table summarizes solutions for various service life issues identified in Section 10.3.2.2 for CDPs. These bearing types have performed well, but have had limited usage. Research testing has identified potential failure modes that could result from improper design and/or manufacturing and installation.

Table 10.4. Solutions for Service Life Problems—CDPs.

Cotton Duck Pads (CDP)			
Service Life Problems	**Solutions**	**Advantages**	**Disadvantages**
Improper pad design resulting in interlayer splitting and delamination	Avoid with proper design, manufacturing, and installation	CDP are economical and very stiff in supporting vertical load. Provide greater stability during construction	High stiffness reduces capacity for rotation and horizontal translation Service life experience and use is limited
	Limit shear strain to 10% of total pad thickness		
	Limit compressive stress to 3,000psi for total load, and 2,000psi for live load		
Improper pad design resulting in over rotation	Use narrow pads in direction of movement	Improves expansion and rotation capacity	
Need for greater longitudinal movement capacity	Combine with high performing sliding surfaces	Can provide unlimited movement capability	Use of PTFE surfaces can limit service life due to wear Additional initial cost
Improper fabrication/ manufacturing	Provide effective QA/QC	Assures adherence to quality and performance requirements	None

10.4.2.2.1 Improper Pad Design Resulting in Interlayer Splitting and Delamination

This damage is caused by excessive horizontal movement, axial load, and rotation, and can be avoided by following current *LRFD Specifications*. Key criteria include:

- Limiting shear strain to 10% of total pad thickness, and
- Limiting compressive stress to 3,000 psi for total load, and 2,000 psi for live load.

10.4.2.2.2 Improper Pad Design Resulting in Over-Rotation

This can be avoided by implementing rotation limits provided in current *LRFD Specifications*. Use of narrow pads in the direction of movement and rotation can improve expansion and rotation capacity.

10.4.2.2.3 Need for Greater Longitudinal Movement Capacity

The high shear stiffness of these bearing types reduces their capacity for accommodating horizontal translation. This can be accommodated however, by combining CDP with PTFE sliding surfaces. PTFE can be subject to wear, which can reduce service life. See further discussion of solutions for sliding surface wear in Section 10.4.2.3.

10.4.2.2.4 Improper Fabrication/Manufacturing

Proper QA/QC and testing during manufacturing is necessary to confirm required performance. Stiffness and other performance characteristics can vary between manufacturers, so QC testing measures have been developed.

10.4.2.3 Sliding Surface Bearings

The following technology table summarizes solutions for various service life issues identified in Section 10.3.2.3 for sliding surfaces. PTFE is most commonly used for sliding surfaces. Its service life is affected by deficiencies caused by loads, which result in wear and creep or cold flow, and production/operation defects, which result in damage due to contamination. It is recommended to use higher performing sliding surfaces where possible. The higher initial cost of these surfaces are a very small fraction of total bridge cost and cost by itself is not a justifiable reason for not using higher performing sliding surfaces in place of PTFE, which is used exclusively in United States.

Table 10.5. Solutions for Service Life Problems—PTFE Sliding Surfaces.

PTFE Sliding Surfaces			
Service Life Problems	**Solutions**	**Advantages**	**Disadvantages**
Wear due to cyclic truck load	Possible thicker PTFE surface	Increases time for material to wear out	Thickness may be limited by other factors. Lower service life as compared to higher performing sliding surfaces
	Dimpled and lubricated PTFE	Improves wear resistance and reduces coefficient of friction	Long-term effectiveness of lubrication is uncertain. Further PTFE has much lower service life as compared to higher performing sliding surfaces.
	Woven or glass reinforced PTFE	Greatly improves wear resistance	Has slightly increased coefficient of friction
	Improved sliding material, MSM®	Greatly improves wear resistance	Proprietary product. Possibly higher initial cost Has significantly increased coefficient of friction without lubrication
	Combined performance of high performing sliding surfaces with elastomeric pad	Elastomeric pad can be designed to accommodate cyclic truck load	Requires special design considerations
Creep or cold flow due to high compressive load	Compressive overload prevention PTFE material set in recess, and epoxy bond	Recess and bonding holds PTFE in place	Has a possibly higher initial cost but prevents higher future maintenance cost
Surface damage due to contamination	Protection during shipment and installation Periodic maintenance cleaning	Prevents damage to sliding surface that increases coefficient of friction and wear	Maintenance needs to be scheduled and actually performed

10.4.2.3.1 PTFE Wear due to Cyclic Truck Load

As mentioned throughout this section, it is recommended to use higher wear-resistant sliding surface materials where possible when subject to fast sliding speeds associated with cyclic truck load. The reference to PTFE in this section reflects its widespread use in practice and providing solutions where they are needed. However, this approach is not meant to recommend the use of PTFE as a material of choice for sliding surfaces, except in instances where wear is not a factor.

The use of PTFE can be improved with the following solutions:

- Thicker PTFE. Other countries use thicker PTFE than used in the United States, but plain PTFE wears very rapidly, and very thick surfaces would be required to achieve long service life when subjected to cyclic truck load.

- Dimpled and lubricated PTFE. This has been shown to provide both improved wear resistance and reduced coefficient of friction; however, the long-term effectiveness of lubrication is uncertain.

- Woven or glass filled PTFE. This greatly improves wear resistance, but has higher coefficient of friction.

- Improved sliding material (MSM®). This has greatly improved wear resistance, and has reduced coefficient of friction when used in dimpled and lubricated condition. Coefficient of friction is greatly increased when unlubricated. This material is a proprietary product.

- Design of combined elastomeric/PTFE bearing. This design considers the combined performance of both materials. The elastomeric pad can be designed to accommodate rotation along with the low amplitude/high cycle movement due to truck load, and the PTFE can be designed to accommodate the high amplitude/low cycle movement due to temperature load, which produces the least amount of wear. This requires some additional design consideration.

10.4.2.3.2 Creep or Cold Flow due to High Compressive Stress

This can be avoided with the following solutions:

- Limiting compressive stress. This prevents cold flow, but lower compressive stress results in higher coefficient of friction.

- Recessing PTFE. Recessing PTFE for .5 of its thickness provides the best solution because it prevents cold flow and also allows higher compressive stress that has lower coefficient of friction.

10.4.2.3.3 PTFE Wear due to Surface Contamination

This can be avoided with the following solutions:

- Protection during shipment and installation. Bearing assemblies should be shipped together and protected to avoid PTFE surface damage.

- Periodic maintenance. Periodic maintenance and cleaning can mitigate the potential for sliding surface damage by preventing buildup of dirt and other debris.

10.4.2.4 HLMR Pot Bearings

The following technology table summarizes solutions for various service life issues identified in Section 10.3.2.4 for HLMR pot bearings. Past issues have most often been related to production/operation defects relating to design and manufacturing. Also, expansion pot bearings use PTFE sliding surfaces, which are affected by loads resulting in wear. External steel surfaces are exposed to environmental hazards, which can cause corrosion; however, corrosion of external surfaces typically does not affect operation. Current design and manufacturing procedures have greatly reduced past issues.

Table 10.6. Solutions for Service Life Problems—HLMR Pot Bearings.

HLMR Pot Bearings			
Service Life Problems	**Solutions**	**Advantages**	**Disadvantages**
Improper design and manufacturing resulting in leakage of elastomer, broken sealing rings, abraded elastomeric pads, and internal metal-to-metal contact	Proper clearances between pot elements	This has been the most common and economical HLMR bearing for many years. Recent sealing ring and pot manufacturing improvements have eliminated most of the earlier failures. Long-term deterioration testing assures greater performance.	This has internal moving parts and requires a high degree of quality control in manufacturing. Internal sealing ring wear could always be a concern. This should provide for bearing replacement in design details.
	Proper pad rotational capacity		
	Proper calculation of movements and rotations		
	Smooth surfaces on pot rim and inside piston		
	Improved sealing ring design and manufacturing tolerances		
	Silicon grease lubrication		
	Alternate methods of external restraint for lateral loads		
	Sealing and protecting pot bearings during shipping and installation		
	Long-term deterioration testing as per current AASHTO procedures		
Load-induced PTFE sliding surface wear	See table for PTFE sliding surfaces		

10.4.2.4.1 Improper Design and Manufacturing Causing Various Deficiencies

Various recommendations were made by Stanton, Roeder, and Campbell (Stanton et al. 1999), to mitigate or avoid previous deficiencies, and are now incorporated in current *LRFD Specifications.* These include:

- Providing proper clearances between various elements of the pot, piston, sliding surface, guides, and restraints. This can avoid binding between metal components.

- Providing proper rotational capacity with proper elastomeric pad thickness design, controlled by a 15% strain limit on the pad edge deflection under rotation.

- Protecting against over-rotation by designing for larger rotations that include possible rotation due to construction tolerances, and placing greater emphasis upon calculation of rotations due to service loads, construction loads and environmental conditions.

- Providing smooth surfaces on the piston and inside the pot to reduce rotational resistance, wear, and abrasion. Metalizing of these interior surfaces for corrosion protection should be avoided because it produces a rough surface that leads to increased damage under cyclic rotation. In highly corrosive environments, stainless steel should be considered, but this affects cost greatly.

- Providing tight control on design and manufacturing of sealing rings to avoid escape of elastomer. Circular cross section rings provide a tight seal but are susceptible to wear under cyclic rotation. Flat rings appear to be more susceptible to leakage and ring fracture, but they experience less wear. Heavier flat brass rings have been suggested as a means of improving performance. The performance of circular rings could also be improved if the internal friction and wear were reduced. Multiple flat brass sealing rings have been the most frequently used system since the mid to late 1990s, and have had good results.

- Using silicon grease lubrication to reduce potential wear on rings, pot walls and piston.

- Providing alternate methods of external restraint for lateral loads to avoid having these loads resisted by the piston rim bearing against the pot wall. Relatively small lateral load (5% of gravity load) when combined with cyclic rotation can dramatically increase the rotational resistance caused by the piston rim dragging against the pot wall during rotation, and cause wear.

- Sealing and protecting pot bearings during shipping and installation to prevent dirt or contamination from getting inside the pot, which can lead to increased wear and abrasion of the rings, pot, and elastomeric disc.

- Providing long-term deterioration testing as per current *AASHTO Construction Specifications* to assure required performance.

Even with recent improvements in serviceability, this bearing type still has internal moving parts that are subject to wear and abrasion. This behavior can still lead to reduced element service life and the potential need for bearing replacement before the service life of the bridge system is realized.

10.4.2.4.2 Load-Induced Sliding Surface Wear

Expansion bearings using PTFE sliding surfaces are subject to wear due to truck loads or thermal loads. See further discussion of solutions for sliding surface wear in Section 10.4.2.3.

10.4.2.5 HLMR Disc Bearings

The following technology table summarizes solutions for various service life issues identified in Section 10.3.2.5 for HLMR disc bearings. Research testing has identified potential issues that would most often be related to production/operation defects relating to design and manufacturing. Also, expansion disc bearings use PTFE sliding surfaces, which are affected by loads resulting in wear. External steel surfaces are exposed to environmental hazards, which can cause corrosion.

Table 10.7. Solutions for Service Life Problems—HLMR Disc Bearings.

HLMR Disc Bearings			
Service Life Problems	**Solutions**	**Advantages**	**Disadvantages**
Improper design and manufacturing causing fatigue deformation and abrasion of urethane disc	Proper design, manufacturing and installation. *AASHTO Construction Specifications* require cyclic testing to confirm performance.	This is a simple concept using polyether urethane disc, which provides multi-directional rotation capability. Good performance has been experienced over the last 40 years without any known field problems.	They have not been extensively used because of proprietary status until recent years. Service life experience and fatigue testing experience is limited. *AASHTO Design Specifications* are limited. There is concern that rotational stiffness can cause high stresses on sliding surfaces. Additional research and experience is required to determine if 100-year service life is achievable.
Improper Design causing over rotation and high edge pressure and damage to PTFE surfaces on expansion bearings Over rotation can also cause binding on center pin.	Proper analysis and design to identify rotations *AASHTO Design Specifications* limit edge contact stress on PTFE surfaces.		
Load-induced PTFE sliding surface wear	See Table for PTFE sliding surfaces.		

10.4.2.5.1 Improper Design and Manufacturing Causing Fatigue, Abrasion, and Over-Rotation

Various performance conclusions and recommendations were made by Stanton, Roeder, and Campbell, (Stanton et. al 1999), based on testing of disc bearings subjected to combined axial load and rotation. These include:

- Following *AASHTO Construction Specifications* requirements for cyclic load testing to confirm long-term performance. Cyclic rotation tests have shown slight disc deformation and abrasion, but did not affect performance.

- Following *LRFD Specifications* in limiting edge contact stress on PTFE surfaces in sliding expansion bearings. Tests showed that rotation of disc bearings is partly accompanied by uplift, which can produce high edge loading on sliding surfaces that affects service life.

- Over-rotation also causes binding on the center shear pin which can be mitigated by limiting rotation or providing adequate clearance.

10.4.2.5.2 Load-Induced Sliding Surface Wear

Expansion bearings using PTFE sliding surfaces are subject to wear due to truck loads or thermal loads. See further discussion of solutions for sliding surface wear in Section 10.4.2.3.

10.4.2.6 HLMR Spherical/Cylindrical Bearings

The following technology table summarizes solutions for various service life issues identified in Section 10.3.2.6 for HLMR spherical/cylindrical bearings. Issues have most often been related to production/operation defects relating to design and manufacturing.

Table 10.8. Solutions for Service Life Problems—HLMR Spherical/Cylindrical Bearings.

HLMR Spherical/Cylindrical Bearings			
Service Life Problems	**Solutions**	**Advantages**	**Disadvantages**
Improper design and manufacturing causing binding of steel components reducing rotational capacity	Ensure proper clearances	This is a very robust bearing system that is traditionally considered to be the most reliable HLMR type. It can be designed to accommodate large loads and rotations. Behavior is close to what is assumed in design.	This is the most expensive of the HLMR types. It requires high degree of manufacturing quality control.
Improper design causing overload of spherical surface due to high lateral loads	Provide external restraint system		
Improper manufacturing causing surfaces to not mate properly causing excessive localized stress	Ensure proper manufacturing tolerances		
Load-induced PTFE sliding surface wear	See Table for PTFE sliding surfaces		

10.4.2.6.1 Improper Design and Manufacturing Causing Binding of Steel Components

Adequate clearances between moving bearing components should be provided to accommodate full rotation demand without premature binding.

10.4.2.6.2 Improper Design Causing Overload of Spherical Surface due to High Lateral Loads

Additional external restraint system should be provided to accommodate large lateral loads to avoid generating excessive localized bearing stresses that can damage PTFE surfaces.

10.4.2.6.3 Improper Manufacturing Causing Surfaces to Not Mate Properly

Tight manufacturing tolerances on curved mating surfaces should be maintained to avoid excessive localized stresses that can damage PTFE surfaces.

10.4.2.6.4 Load-Induced Sliding Surface Wear

PTFE sliding surfaces are subject to wear due to truck loads or thermal loads. See further discussion of solutions for sliding surface wear in Section 10.4.2.3.

10.4.2.7 Fabricated Mechanical Steel Bearings

The following technology table summarizes solutions for various service life issues identified in Section 10.3.2.7 for fabricated mechanical steel bearings. As previously discussed, factors affecting service life relate to several categories including loads, primarily overload, which results in binding or over-rotation of rocker bearings; natural or manmade hazards, which result in steel element corrosion; and production/operation defects, specifically due to lack of maintenance.

Table 10.9. Solutions for Service Life Problems—Mechanical Steel Bearings.

Fabricated Mechanical Steel Bearings			
Service Life Problems	**Solutions**	**Advantages**	**Disadvantages**
Overload causing binding or excessive rotation or tipping	Use proper demand parameters, proper clearances, and proper initial setting.	This has been the longest used bearing type. This bearing type has potential for 100-plus years of service life if maintained and protected from corrosion, freezing, and over-rotation.	It is expensive to fabricate and install. It requires additional maintenance than other bearings.
Corrosive environment causing steel surface corrosion	Use stainless steel in extreme environments. Use galvanizing or metalizing on bearing assembly components (fixed or rocker) and accompanying plates and anchor bolts.		
Improper maintenance causing debris buildup affecting movement capacity	Maintain		
Improper maintenance causing freezing affecting movement and/or rotation capacity	Maintain		

10.4.2.7.1 Overload

Special emphasis should be placed on determining load, rotation, and movement demands to avoid over-rotation that can lead to excessive tilting and possible tipping. Adequate clearances between moving components should be provided in order to avoid premature binding. Set rocker bearings for proper temperature alignment.

10.4.2.7.2 Corrosive Environment

For bearings in corrosive environments, use of stainless steel can avoid the potential of corrosion on exposed surfaces and on contact surfaces. Use of galvanizing, metalizing, or high performance paint systems can serve to mitigate the potential for surface corrosion, but a maintenance plan for the longer term will still be required.

10.4.2.7.3 Improper Maintenance Causing Debris Buildup

Proper periodic maintenance is required for bearings located below deck expansion joints to clean bearing areas and prevent excessive buildup of debris that can affect horizontal movement and rotation. Dirt and debris buildup against steel surfaces can also hold moisture and accelerate coating deterioration and steel corrosion.

10.4.2.7.4 Improper Maintenance Causing Freezing

Proper maintenance is also required in corrosive environments to prevent or inhibit corrosion of steel surfaces that can cause freezing and reduce rotation and movement capacity. Proper field cleaning and recoating of existing steel bearings with zinc-rich paint systems, followed by maintenance touchup as required, will provide extended service life. Lubricating pins and knuckles that have metal to metal contact can also help prevent freezing.

10.5 STRATEGIES FOR BEARING SELECTION AND DESIGN

10.5.1 Available Service Life Design Philosophies

Currently there are no deterioration models for predicting the service life of bearings. As a result, measures should be taken to avoid the deterioration and provide long service life.

Previous bearing research has studied the behavior of various bearing types under static and cyclic loading, and has identified potential damage and deterioration modes that have been addressed by improved AASHTO design and construction specifications. This research has been performed primarily to address observed field problems or to provide improved understanding and design methodology. However, these studies have focused primarily on developing criteria that will avoid observed problems or improve performance, and have not developed models for determining how various bearing types will deteriorate over time under given loading and environmental conditions. Research on sliding surfaces has determined the potential for a deterioration model for wear based on various factors including pressure and sliding velocity, but this has not yet been fully developed.

With the lack of deterioration models, experience and expert opinion are the only methods for predicting service life. However, previous experience with certain bearing types may not be a good indicator of newer design performance, particularly with recently updated and improved design and construction requirements. Future long-term data collection regarding bearing performance will be necessary for developing more accurate service life predictions.

10.5.2 Bearing Selection and Design for Service Life

Selecting the proper bearing should always be done as part of the overall bridge system development, and should consider the expected bridge system behavior and optimal superstructure/substructure interaction. Bearings should be selected and designed as an integral part of the overall system and should not be designed as an afterthought, which increases chances for problems. It is recommended to use elastomeric bearing and higher performing sliding surfaces where possible. A combination of rectangular or circular elastomeric bearing pad and higher performing sliding surfaces can meet the demands of most bridges and result in a very long service life.

10.5.2.1 Selection Process

The process for selecting the proper bearing type involves four main steps:

1. Determine **demand**, which identifies operational and service life requirements that the bearing must accommodate;
2. Determine suitable **options**, which identifies bearing types that have the potential to accommodate demand requirements; and perform preliminary design(s) to confirm;
3. Evaluate service life **mitigation** and replacement requirements and evaluate life cycle costs; and
4. **Select** optimal bearing type considering all factors.

When considering service life, the demand and options steps need to address additional issues beyond loads and movements, and consider all hazards that can have adverse effects. Figure 10.19 illustrates the overall selection process. When considering multiple options, the selection process should take into account the various levels of bearing performance, the initial cost and maintenance requirements, and the reliability of the bearings and their potential to achieve long service life.

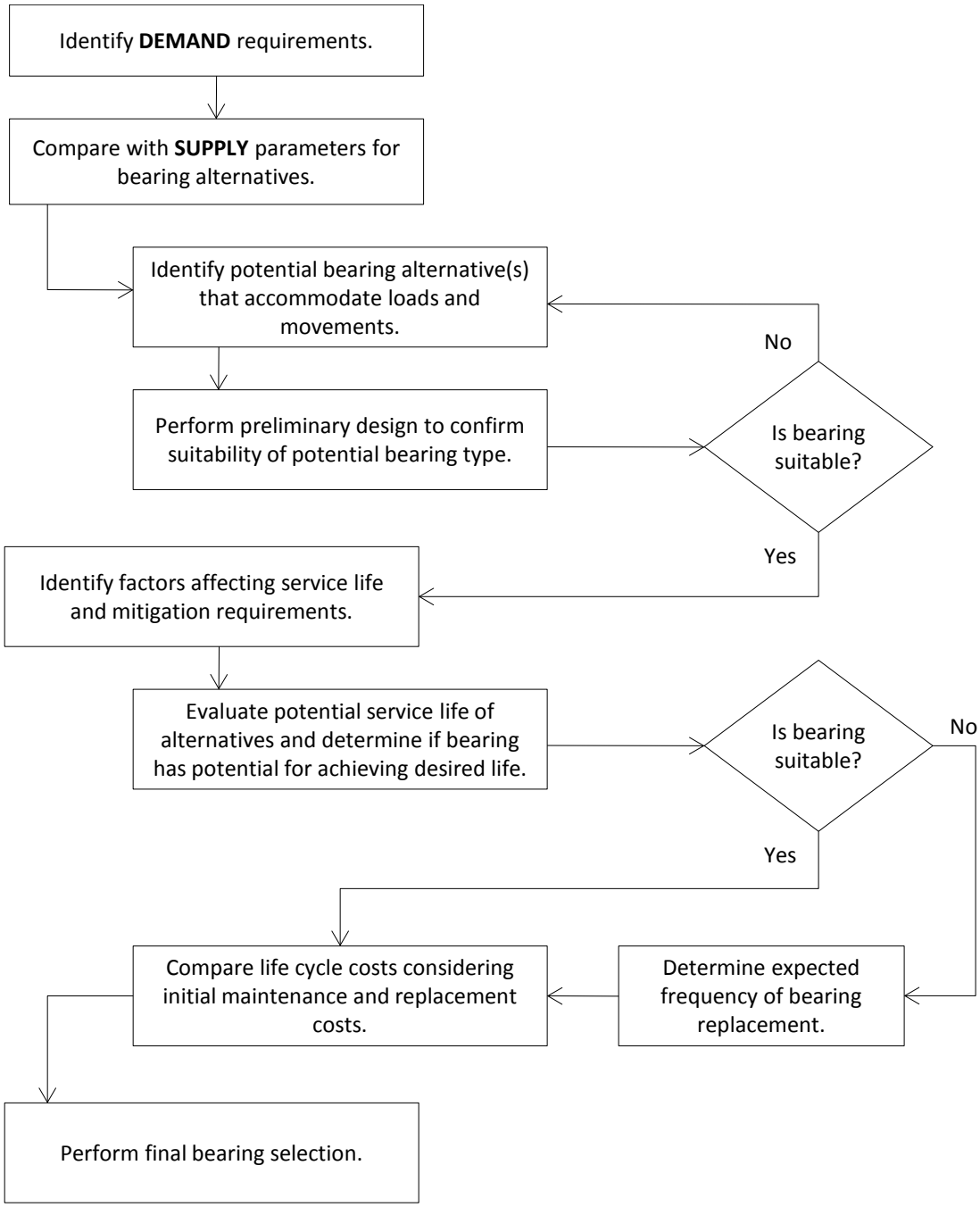


Figure 10.19. Process flow chart for bearing selection and design for service life.

10.5.2.2 Detailed Steps in Selection Process

Step 1. Identify Demand Requirements

The demand step identifies the desired service life and what requirements the bearings must accommodate from an operational and environmental standpoint throughout their service life. Operational requirements have typically included proper determination of gravity loads, rotations, and translational movements. However, for service life, this needs to further consider cyclic movements and cumulative movement due to truck load, which can have a more severe effect on service life in some cases. Environmental demand has typically involved determination of thermal climate and corresponding temperature ranges. But for service life, this also needs to identify specific local environmental hazards and their consequences that need to be avoided or mitigated in the design. Table 10.10 provides a format for identifying various demand requirements. The following steps summarize the process:

Step 1.1. Determine operational and service life requirements:

- Review the targeted bridge design service life. This should be done at the bridge system selection stage where the targeted bridge design service life is identified based on a number of factors. See Chapter 2 on bridge system selection.
- Identify total traffic and truck volumes.
- Identify local environmental factors that can affect bearing service life or performance:
  - Thermal movement: design temperature ranges, and
  - Environmental hazards, including:
    - Severe corrosive environment—location below deck expansion joints in northern wet climates;
    - Coastal environment—potential for salt spray; and
    - Chemical environment—potential for other deleterious atmospheric or corrosive activity.

Step 1.2. Determine general bearing requirements based on bridge system alternatives and superstructure/substructure interaction.

- Consider integral system options that eliminate bearings (at abutments and/or at piers).
- Consider continuous system options that eliminate deck joints at interior piers.
- Determine optimal fixity/expansion options at piers and abutments based on bridge system evaluation. Consider flexibility of piers in determining options for multiple pier fixity.
- For curved or skewed bridge systems, determine proper direction of movement, and point of fixity for curved systems.

Step 1.3. Determine superstructure loads and movements for given bridge system(s) being considered:

- Gravity Loads—dead and live load;
- End rotations—due do all sources, including construction tolerances;
- Longitudinal movements:
  - Maximum movement due to temperature change,
  - Cyclic movement due to truck load, and
  - Movement due to posttensioning, creep and shrinkage;
- Requirements for multi-directional movement;
- Longitudinal and transverse loads to be resisted by bearings; and
- Ensure that loads are distributed to bearings in accordance with system analysis.

Table 10.10. Demand Requirements for Bearing Selection.

Demand Description				Qualitative Value: N/A, Low, Medium, High	Quantitative Value: if Applicable	How Values are Used
General Requirements	Bridge Importance and System Design Service Life			Example: High	Example: 100 years	To Identify bearing design service life and replacement needs
General Requirements	Traffic			Example: High	Indicate total traffic volumes and truck volumes	To determine cyclic translation due to truck load
General Requirements	Environmental Factors	Temp Range for thermal climate and superstructure type		Moderate or Cold	Indicate design temperature range	To compute superstructure thermal movement
General Requirements	Environmental Factors	Environmental Hazards	Cold Climate w/deck deicing chemicals and subject to open deck joints	Example: Severe	N/A	To determine severity of environmental hazards to which bearings will be subjected, and to determine mitigation needs to achieve optimal service life
General Requirements	Environmental Factors	Environmental Hazards	Coastal Climate subject to salt spray	Example: High	N/A	
General Requirements	Environmental Factors	Environmental Hazards	Other Chemical Corrosive Environment	Example: N/A	Identify type if applicable	
General Requirements	Environmental Factors	Environmental Hazards	Other Atmospheric Environmental Factors	Example: N/A	Identify type if applicable	
Bridge System Loads and Movements	Design Loads (Kips)	Vertical	Max			For fixed and expansion bearing design
Bridge System Loads and Movements	Design Loads (Kips)	Vertical	Permanent			
Bridge System Loads and Movements	Design Loads (Kips)	Vertical	Min			
Bridge System Loads and Movements	Design Loads (Kips)	Transverse				
Bridge System Loads and Movements	Design Loads (Kips)	Longitudinal				
Bridge System Loads and Movements	Rotation (Rad)	Longitudinal	Permanent			
Bridge System Loads and Movements	Rotation (Rad)	Longitudinal	Cyclic—Max			
Bridge System Loads and Movements	Rotation (Rad)	Longitudinal	Cyclic—Min			
Bridge System Loads and Movements	Rotation (Rad)	Transverse	Permanent			
Bridge System Loads and Movements	Rotation (Rad)	Transverse	Cyclic			
Bridge System Loads and Movements	Translation (In)	Longitudinal	Cyclic thermal			For expansion bearing design
Bridge System Loads and Movements	Translation (In)	Longitudinal	Cyclic ruck			
Bridge System Loads and Movements	Translation (In)	Longitudinal	Irreversible			
Bridge System Loads and Movements	Translation (In)	Transverse	Cyclic thermal			
Bridge System Loads and Movements	Translation (In)	Transverse	Cyclic truck			
Bridge System Loads and Movements	Translation (In)	Transverse	Irreversible			

Step 2. Identify Suitable Bearing Options

The options step involves comparing demand requirements with supply parameters for various bearing types, and determining which types are suitable. Often, there may be more than one option that can meet load and movement performance requirements. For shorter spans with lighter loads, which represent the largest population of bridges, plain or reinforced elastomeric pads will typically provide the best overall service.

CDPs with greater load capacity can be a suitable alternative for plain elastomeric pads when movement demand is light; however, when movement demand increases, CDP will require sliding surfaces, which affect service life.

When loads and rotational demands increase, SRE bearings will have to be evaluated against HLMR pot or disc bearings for accommodating the required loads. As long as an SRE bearing can be designed for the combined load, movement, and rotation, it will also have the greatest potential for achieving the desired service life. Greater movement capacity can be provided with SRE bearings by combining them with sliding surfaces, but this typically would be considered only where movement demand is large and vertical loads are relatively light, such as at the ends of a long, multi-span continuous unit.

When load demand is beyond the capacity of SRE bearings, other bearings such as HLMR pot, disc, or spherical must be considered. Fabricated steel bearings can also be considered, but cost and service life mitigation issues have to be weighed.

The preliminary design step involves performing preliminary design in accordance with AASHTO specifications to determine if potential bearing type(s) can actually provide the required capacities depending on actual bridge layout, and to evaluate further size and geometric requirements.

The following steps summarize the process for identifying viable options:

Step 2.1. Using Table 10.11, or other applicable agency or industry guidelines, or professional experience, identify potential bearing alternatives that accommodate load and movement requirements.

Step 2.2. Perform preliminary design on potential bearing types following AASHTO design requirements. Determine whether selected alternatives can actually accommodate load and movement demand. Determine bearing size requirements.

Table 10.11. Supply Parameters for Bearing Selection.

Bearing Type	Load and Movement Performance Values				Durability Factors			Avoidance or Mitigation Requirements	Life-Cycle Costs		Service Life Potential
	Load (Kips)	Rotation (Radians)	Movement (Inches)	Multi-Directional Rotation/ Movement Capability	Relative Ability to Accommodate Cyclic Truck Movement	Resistance to Corrosive Environment	Resistance to Production/ Operation Defects		Relative Initial Cost A = Lowest	Relative Maintenance	
Plain Elastomeric Pad	Low 0 to 100	Low 0.01	Low 0.5	Yes	High	High—pad not affected	High	Avoid pad splitting by proper design	A	Low	High—potential for 100+ years
Steel Reinforced Elastomeric Pad	Low to Med 50 to 750	Med 0.02	Low to Med 4	Yes Circular pads	High	High—SRE pad not affected	High—SRE pad splitting or delamination	Avoid pad splitting by proper design	B	Low	High—potential for 100+ years
Elastomeric Pad with Sliding Surface	Low to Med 50 to 750	Same as above	High No Limit	Yes	High with high performing sliding surface. Low with PTFE	High	High	Mitigate PTFE wear with improved sliding surface	B	Low to Mod	Low to Moderate (moderate with improved sliding surface)
CDP	Low to Med 0 to 300	Low 0.005 max	Low 0.25	No	Low	High—pad not affected	High—CDP pad splitting or delamination	Avoid pad splitting by proper design	B	Low	Uncertain due to lack of data
CDP with Sliding Surface	Low to Med 0 to 300	Low 0.005 max	High No Limit	Yes	Low due to wear of sliding surface	High—pad not affected	High	Mitigate PTFE wear with improved sliding surface	B	Low to Mod	Low (moderate with improved sliding surface)
HLMR Pot (Except with sliding surface)	High 250 to 2500+	High 0.02 to 0.04	High No Limit w/ slider	Yes	Low due to wear of sliding surface	Moderate for exposed sides of pot and piston	Moderate—internal sealing ring wear and elastomer leakage	Ensure proper design and construction. improved sliding surfaces Metalized surfaces	E	Moderate	Moderate

Bearing Type	Load and Movement Performance Values				Durability Factors			Avoidance or Mitigation Requirements	Life-Cycle Costs		Service Life Potential
	Load (Kips)	Rotation (Radians)	Movement (Inches)	Multi-Directional Rotation/ Movement Capability	Relative Ability to Accommodate Cyclic Truck Movement	Resistance to Corrosive Environment	Resistance to Production/ Operation Defects		Relative Initial Cost A = Lowest	Relative Maintenance	
HLMR Disc (Exp with sliding surface)	High 250 to 2500+	High 0.02 to 0.03	High No Limit w/ slider	Yes	Low due to wear of sliding surface	High Disc not affected	Moderate PTFE wear	Proper design and construction Improved sliding surfaces	E	Moderate	Moderate
HLMR Spherical (Curved sliding surface; Exp with sliding surface)	High No Limit	High No Limit	High No Limit w/ slider	Yes	Low due to wear of sliding surface	Moderate for exposed sides of steel elements	Moderate PTFE wear Surfaces not mating properly	Proper design and construction Improved sliding surfaces Metalized surfaces	F	Moderate	Moderate
Fabricated Steel (Pin fixed, Rocker or roller exp.)	Low to Med 50 to 750+	High No Limit	High No Limit	No	High	Low for all elements	High	Use stainless steel or mitigate with galvanizing or metalizing	D	Moderate Low with SS	High with mitigation for corrosion potential
Steel sole plates, base plates and anchor bolts	N/A	N/A	N/A	N/A	N/A	Low	High	Use stainless steel or mitigate with galvanizing or metalizing	N/A	Moderate Low with SS	High with mitigation for corrosion potential

(Stanton, et al, 2008) Note: Performance condition limits are approximate, and may not occur simultaneously.

Legend:

SRE	Steel-reinforced Elastomeric Bearings	CDP	Cotton Duck Pads
HLMR	High Load Multi-Rotation	PTFE	Polytetrafluoroethylene (Teflon®)
MSM®	Maurer Sliding Material (MSM®)	SS	Stainless Steel

Step 3. Service Life Factors and Mitigation Requirements

After suitable options are identified in Step 2 based on satisfying vertical load and movement requirements, this step includes an evaluation of the bearing's ability to resist various factors that affect service life, and considers what mitigation requirements will be necessary. Included is an evaluation of whether the bearing type(s) has the potential for achieving the desired service life, and identification of possible replacement needs and maintenance requirements.

Table 10.11 also summarizes various supply parameters for individual bearing types related to service life. It lists relative durability factors for each bearing type and also identifies key avoidance or mitigation requirements. In addition, relative qualitative initial costs and maintenance requirements as part of a qualitative life cycle cost comparison are listed. The table then indicates relative qualitative service life potential.

The following steps summarize the process for evaluating service life requirements:

Step 3.1. For potential bearing alternatives, evaluate factors affecting service life and identify required avoidance, mitigation, or acceptance options. Consider following primary service life reduction categories, including:

- Due to loads (primarily cyclic truck load);
- Due to environmental hazards (primarily corrosive or deleterious environment); and
- Due to production/operational defects (primarily element damage or wear)

Step 3.2. Evaluate potential service life of identified bearing alternatives with consideration of any required mitigation and maintenance.

- Use deterioration models (currently not available for bearings):
  - Potential deterioration model for sliding surface resistance to wear.
- Use experience or expert opinion:
  - Bearing system and/or material resistance to wear or other deterioration, and
  - Steel element resistance/protection from corrosion.

Step 3.3. Relate potential service life of identified bearing alternatives to the target design life of bridge system.

If service life of bearing alternative is less than target bridge system design life, consider the need to replace bearing after service life is exhausted.

Step 3.4. Evaluate life cycle cost of bearing options considering initial cost, long-term maintenance cost and potential replacement cost. This can be done qualitatively at this stage.

Step 4. Optimal Bearing Type Selection

After determining suitable options based on load and movement requirements, and evaluating service life mitigation and replacement requirements, the optimal bearing type for the given application can be determined after considering and comparing all parameters.

The selection process should summarize final mitigation, maintenance and replacement requirements that will need to be incorporated into final design.

The final bearing design process should fine-tune the preliminary bearing design after the final bridge system analysis and final load distribution. Final design details would then be developed to accommodate required clearances and to include any service life mitigation measures or details for required bearing replacement.

10.6 BRIDGE MANAGEMENT RELATED TO BEARINGS

This section provides guidance related to inspection and maintenance of bearings, and needs for future data collection.

It is recommended that bridge inspections and inspection data collection for bearings be expanded to identify bearing types, specific conditions and other relevant data. These recommendations for more detailed bearing data collection can be used within *FHWA's Long-Term Bridge Performance (LTBP)* program, which is intended to study the deterioration and durability of bridges and the impacts of maintenance and repair. These recommendations can also be used to supplement the types of data collected for use within bridge management systems such as Pontis™. Improved data collection for bearings can be useful in determining and scheduling required maintenance and for developing more accurate deterioration models.

10.6.1 Bridge Owner's Manual

Chapter 1 provided detail description of a bridge Owner's Manual, which must be provided when requested by the owner or service life design of unique bridges are involved.

The following data regarding bearing design should be included in the bridge Owner's Manual. This information will be helpful in evaluating future bearing performance.

- Bearing type(s);
- Design movements considering temperature and truck;
- Design rotations considering DL, and expected construction rotation and live load;
- Expected bearing service life and expected replacement schedule;
- Summary of major factors considered that could affect bearing service life;
- Summary of mitigations included in design for factors affecting service life;
- Types and frequency of recommended maintenance;
- Basis for designed-in details/capabilities for future bearing replacement; and
- Special features that should be monitored with future NBIS inspections (see Section 10.6.2).

10.6.2 Recommendations for Inspection

The following list provides inspection recommendations that can provide early indications of bearing problems for various bearing types. Early indication can facilitate maintenance scheduling, which can prevent more serious problems leading to bearing replacement.

- All bearings should be checked for misalignment. All guided sliding bearings should be checked for binding against the guides. All expansion bearings should be checked for movement position at respective temperature and compared against initial settings as identified in the Owner's Manual.
- Elastomeric bearings should be checked for:

- Over-rotation;
- Excessive shear deformation;
- Splitting or tearing;
- Excessive bulging; and
- Sliding or walking out.

- PTFE/SS surfaces should be checked for:
  - PTFE fragments indicating wear;
  - Migration of PTFE surface;
  - Exposed SS surface should be checked for scratching, paint or other contamination; and
  - Proper position of SS surface on PTFE surface.
- Pot bearings should be checked for:
  - Leakage of elastomer from pot;
  - PTFE/SS surfaces for expansion bearings;
  - Steel surface corrosion on exposed surfaces of pot, piston and plates; and
  - Adequate rotational clearances or binding of pot elements due to rotation.
- Disc bearings should be checked for:
  - Splitting, cracking or bulging of urethane disc;
  - PTFE/SS surfaces for expansion bearings; and
  - Steel surface corrosion on plates.

- Mechanical steel bearings should be checked for:
    - Surface corrosion;
    - Debris buildup that could prevent movement/rotation;
    - Over rotation; and
    - Freezing.
- Miscellaneous items
    - Bent or misaligned anchor bolts;
    - Loose or missing nuts on anchor bolts;
    - Voids under bearing plates; and
    - General debris buildup.

10.6.3 Future Data Needs

There is a need to study the performance of bridge bearings in actual service conditions, and to accumulate data that would be helpful in determining more accurate life predictions for various bearing types given those service conditions. For example, measuring actual longitudinal bearing movements due to girder end rotations under traffic load can be useful in understanding and predicting service life of sliding surfaces. This type of data should be collected by the FHWA LTBP program and other research for enhancing the life prediction capability and development of deterioration models for various bearing types.

CHAPTER 11

LIFE-CYCLE COST ANALYSIS

11.1 INTRODUCTION

This chapter introduces life-cycle cost analysis (LCCA) and its use in the decision making process for selecting optimum cost effective bridge systems, subsystems, and elements that can achieve long term service life. It includes general guidelines and best practices for application of LCCA, and outlines the steps in the process, along with a brief discussion of the economic principals involved. References to available models are made without recommending the use of a specific software or model.

The use of LCCA, in lieu of purely initial construction cost evaluation, is essential in evaluating the long-term benefit of many strategies that can achieve extended service life (in excess of 100 years) yet require additional and sometimes considerable initial investment. LCCA can assist agencies with investment decisions by considering initial costs and relevant future costs associated with required inspection, maintenance, rehabilitation, and possible component replacement, including associated demolition, disposal, and user costs.

In its broadest form, LCCA can be used in the evaluation of alternative bridge systems. In its more simplified forms, it aids in evaluating alternatives for bridge components such as decks, superstructures, substructures, or more specialized bridge element applications, such as comparing alternatives for deck joints or bearings.

This chapter is intended to provide only a brief discussion of the benefits, principles and methodologies involved in LCCA. Additional detail on this entire process and its application is provided in the attached references, primarily in the *Life-Cycle Cost Analysis Primer* (FHWA 2002), and in *NCHRP Report 483, Bridge Life-Cycle Cost Analysis* (Hawk 2003).

11.2 LCCA DEFINED

LCCA is an analysis methodology that assists in comparing and choosing alternative strategies for achieving long-term service life for bridge systems, subsystems or elements. It considers not only the initial construction cost, but also all of the costs that are expected to occur over the entire service life of the bridge, typically maintenance, major rehabilitation, and component or element replacement, including relevant demolition and disposal costs, and

user costs. Economic methods are used to convert anticipated future costs to present dollar values so that lifetime costs of various alternatives can be directly compared.

11.2.1 Steps in LCCA

There are five basic steps in the LCCA process, which are described in the following sections (FHWA 2002).

Step 1. Establish Design Alternatives

This step involves establishing the elements of initial design and identifying the associated activities that will be required throughout its service life for maintenance, rehabilitation, or element replacement within a system or subsystem—for each alternative being considered.

Step 2. Determine Activity Timing

The timing of associated activities throughout the period of comparison must be determined as part of the identification process. Estimating when and how often certain activities must be performed is important in making realistic comparisons. This process might involve identifying certain required maintenance on a yearly basis, or certain levels of potential rehabilitation due to expected wear after a specified period of time, or when individual components or elements such as decks or bearings may have to be replaced. Agency data is important in establishing when various levels of maintenance, rehabilitation or replacement may be required. In the absence of data, expert opinion can be used.

Step 3. Estimate Costs

This step involves estimating the initial construction cost associated with each design alternative and the costs associated with the various identified future maintenance, rehabilitation, and replacement activities. Costs are computed on the basis of current cost data. It is recommended as a best practice that costs include both agency costs and user costs. This is discussed further in Section 11.2.2.

Step 4. Compute Life-Cycle Costs

This step involves computing the present value (PV) of all costs identified for a given alternative. The concepts of PV are further discussed in Section 11.3.1. As part of this computational process, there are two approaches, either deterministic or stochastic (probabilistic), that address the variability and uncertainty associated with input factors.

The deterministic approach (most used) uses fixed discreet values while the stochastic approach defines input variables by a probability distribution. These computational approaches are further discussed in Section 11.3.4.

Step 5. Analyze Results

This step involves comparing the initial and life-cycle costs associated with the various alternatives and determining the optimal cost-effective solution. If the alternatives provide different levels of service, then the alternatives that provide the best overall long-term benefit can also be compared.

11.2.2 LCCA Cost Types

11.2.2.1 Agency Costs

These costs include all the costs born by the agency or owner of the bridge, including design, initial construction, inspection, maintenance, rehabilitation, and element replacement, and have been the primary elements for consideration in LCCA. Some costs such as initial construction, rehabilitation, and replacement are more easily estimated based on current industry cost data. Other costs, such as maintenance, are more difficult and rely on the existence and accuracy of agency historical cost data. In the absence of this data, expert opinion can be used.

11.2.2.2 User Costs

These costs are primarily associated with reduced traffic capacity in work zones, and involve costs to the users because of delays, vehicle operating costs, and accidents. Estimating these costs may be the greatest challenge to LCCA implementation. Many agencies have been reluctant to incorporate user costs in LCCA because of difficulty and uncertainty in assigning value to user delay time, or because user costs are not factored into agency budgets. These issues have led many agencies to give lesser credence to user costs in evaluating overall lowest cost solutions; however, minimizing user impacts and associated costs is a major concern today, especially with implementation of accelerated bridge construction (ABC). A recent pooled-fund study led by the Oregon Department of Transportation (DOT) (Doolen, et al. 2011) developed a set of decision making tools to determine if ABC techniques are more effective that traditional construction for a given bridge replacement or rehabilitation project. These tools incorporate quantified user costs as part of a life-cycle cost analysis evaluation.

User costs can play an important factor in evaluating various options for long-term service life. Section 11.3.3 further discusses the approach for estimating user costs based on traffic volumes and user delays.

11.2.2.3 Vulnerability Costs

These costs are associated with extraordinary circumstances and risks, such as overload, collision, blast, fire, floods, scour, or earthquake, and typically would not be included in LCCA for comparison of service life strategies. They are useful, however, in evaluating vulnerability of existing bridges that might have high probability for one or more of these extraordinary events.

11.2.3 LCCA Versus Benefit-Cost Analysis (BCA)

LCCA is a subset of benefit cost analysis, which compares benefits as well as costs in selecting optimal alternatives. BCA is useful in comparing alternatives that do not achieve the same level of service or benefit. For example, BCA can be useful in comparing bridge replacement options that provide either three or four lanes of traffic with corresponding different levels of service. Clearly there are both cost and benefit variations between each option. LCCA is typically considered alone for service life evaluation of various alternatives that can ultimately provide the same level of service.

11.3 ELEMENTS OF LCCA

This section discusses methodologies for determining net present value and discount rates. It further discusses other elements considered in LCCA such as activity timing and service life, user costs, computational approaches, and analysis tools.

11.3.1 Net Present Value (NPV)

The net present value concept in LCCA is an economic method for combining initial costs and present dollar values of future expected costs so that lifetime costs for various alternatives can be directly compared. Dollars spent at different times within a structures life have different present values, so the projected activity costs for an alternative cannot simply be added together to calculate the total life cycle cost for that alternative. A dollar today is worth more than a dollar five years from now, even if there is no inflation because today's dollar can be used productively in the ensuing five years, yielding a value greater than the initial dollar. Future benefits and costs are discounted to reflect this fact.

The purpose of discounting is to put all present and future costs into a common metric, their net present value.

The formula to convert the sum of the initial cost and the present value of future repair and renewal costs into net present value is given by Equation 11.1.

$$NPV = initial\ cost + \sum_{k=1}^{N} rehab\ cost_k \left[\frac{1}{(1+r)^{n_k}}\right], \qquad \textbf{EQ 11.1}$$

Where:

r = real discount rate;
k = order number of a rehabilitation activity undertaken in the future
N = total number of rehabilitation activities
n_k = year in the future when the cost will be incurred

The term $\left[\frac{1}{(1+r)^{n_k}}\right]$ is called the discount factor.

Input factors in computing NPV are the initial cost (usually the cost of planning, design and construction of a new structure), the cost and timing of rehabilitation activities, and the real discount rate.

Discounting is a method of considering the opportunity cost of money as it applies to current versus future funds. It can be thought of in terms of the alternative economic return that could be gained on funds such as earning interest. The computed potential amount of interest is based on what is referred to as the discount rate, which is generally described as having three components:

1. The real opportunity cost of capital to account for productive value of funds;
2. The premium to account for financial risk (i.e. that the loan will not be repaid); and
3. The anticipated rate of inflation.

Life-cycle analyses typically ignore inflation because the prediction of future prices introduces unnecessary uncertainty into the analysis. Therefore, discount rates are typically based on interest rates for government borrowing, which have little risk, with the inflation component removed, yielding the "real" interest rate. This rate is typically calculated by subtracting the rate of inflation (consumer price index) from the interest rate of an investment such as a 10-year U.S. Treasury bill. For example, if the interest on a 10-year Treasury bill is 5.5% and the inflation rate is 3%, then the discount rate would be 2.5%.

OMB Circular No. A94 (OMB 1992) provides general guidance for conducting cost-effectiveness analyses, and provides specific guidance on the discount rates to be used in evaluating programs whose benefits and costs are

distributed over time. It provides standard criterion for deciding whether programs can be justified on economic principles, and the OMB publishes real interest rates for net present value analyses on its website.

The discount rate can have a significant impact on the analysis, as can be seen in Figure 11.1. A low discount rate favors projects with long-term benefits and near-term costs. When evaluating alternative projects, a sensitivity analysis using a range of discount rates can be used to determine the importance or impact of the discount rate in the relative project performance. Even with a low discount rate, values far in the future have a relatively low present value.

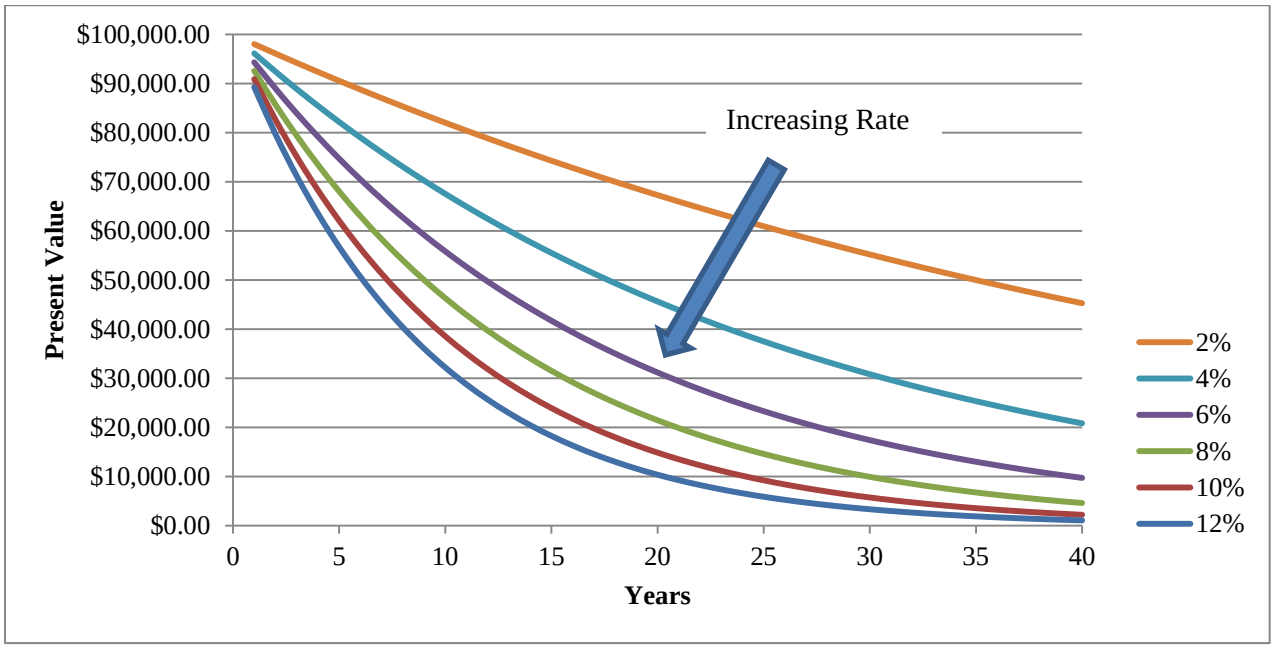


Figure 11.1. Effect of discount rates on present value.

11.3.2 Activity Timing, Service Life, and Life Cycle

11.3.2.1 Deterioration Models

Deterioration models describe the relationship between the condition of the bridge (or its element) and time, showing how the bridge deteriorates. It assumes that no replacements or major repairs are made, but it usually implies that scheduled maintenance actions are performed as planned. The basic model applies either to a bridge as a whole, or to any of its elements (e.g., deck, substructure, bearings, columns). The shape of a deterioration curve depends on the type of the element and the definitions of condition states.

An example of a deterioration curve is presented in Figure 11.2. If the bridge is placed in service at period T_0, its condition gradually declines, and the deterioration curve represents its condition over time. Initially the condition is good, but after a period of wear and aging, it eventually (at time T_f) reaches an unacceptably low condition C_f. The time period between T_0 and T_f is called the service life (SL) of the bridge.

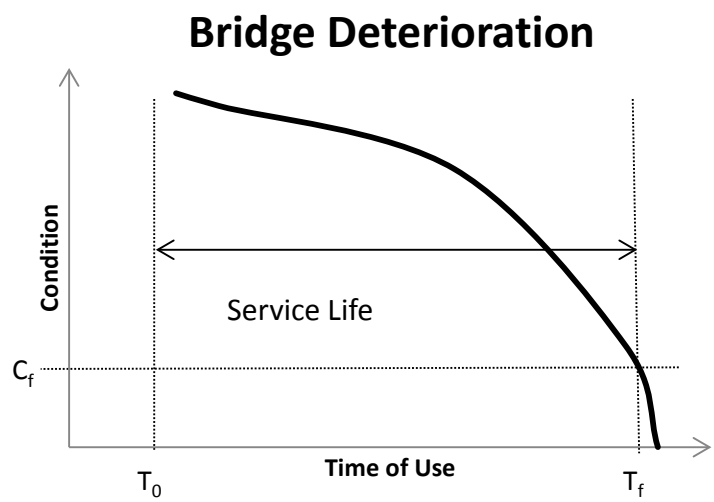


Figure 11.2. Bridge deterioration curve.

In practice however, realistic deterioration models that are based on actual physical and chemical deterioration processes are generally not available to accurately predict service life. The most acceptable deterioration model is in the form of the solution to Fick's second law, used to predict the rate of chloride ingress through concrete cover. This model, including its limitations, is described in Chapter 5 of the *Guide*. It is expected that with time additional deterioration models will become available and will greatly enhance quantification of the service life of bridge elements, components, subsystems or systems.

Developing deterioration models is a data-intensive procedure and is complicated by the lack of knowledge of the underlying processes that foster deterioration as well as by data availability. In lieu of deterioration models based on actual physical and chemical deterioration processes, other approximate methods must be used. Bridge management software programs such as Pontis and BRIDGIT, which are used in nearly all 50 states, have deterioration models contained within them that are typically based on expert opinion and analysis of available historical data.

Recent studies by the New York State DOT (Agrawal and Kawaguchi 2009) and the Florida DOT (Sobanjo 2011) have further attempted to develop bridge element deterioration models based on state DOT bridge inspection

databases along with expert opinion. The New York State DOT study applied computerized statistical methods to develop deterioration curves using inspection data going back to 1981. The study included the influence of various factors such as average daily truck traffic (ADTT) and climate, among others.

The New York State DOT study further implemented a stochastic approach to account for the uncertainty and randomness of factors affecting the deterioration process. In the stochastic approach, the ratings of bridge elements (reflective of their condition at a particular time) and the durations that elements will stay at a particular rating were assumed to be random variables and were modeled by probability distributions. The study developed and compared deterioration curves using both Markov chain and Weibull-distribution based stochastic models. Markov chain is the most commonly used model for developing deterioration rates for infrastructure facilities, and is used in advanced bridge management systems such as Pontis and BRIDGIT. It models the deterioration process by considering the probability of transition from one condition state to another in a discrete time, and accounts for the current element condition in predicting the future condition. A Weibull-based model considers the probability of how long a bridge element will remain at a particular state, and also considers past conditions. The New York State DOT study found that the Weibul-based models generally provided the best overall fit with historical bridge inspection data.

It should be noted that methods that use historical data to develop long term bridge element deterioration models have certain limitations. Aside from not considering the actual physical deterioration process, older historical data does not consider more recent improvements in construction materials or methods, which can greatly affect future service life. Further, there is some uncertainty about extrapolating the data beyond the duration for which the data was collected.

11.3.2.2 Expenditure Stream

As shown on Figure 11.3, a bridge will deteriorate over the period of its service life if left unattended. However, in most cases a bridge is not left to follow the basic deterioration path and reach an unacceptable condition without interruption. The agency responsible for the bridge will, from time to time, undertake repairs, rehabilitations, and renewals that return conditions to higher levels and extend its service life.

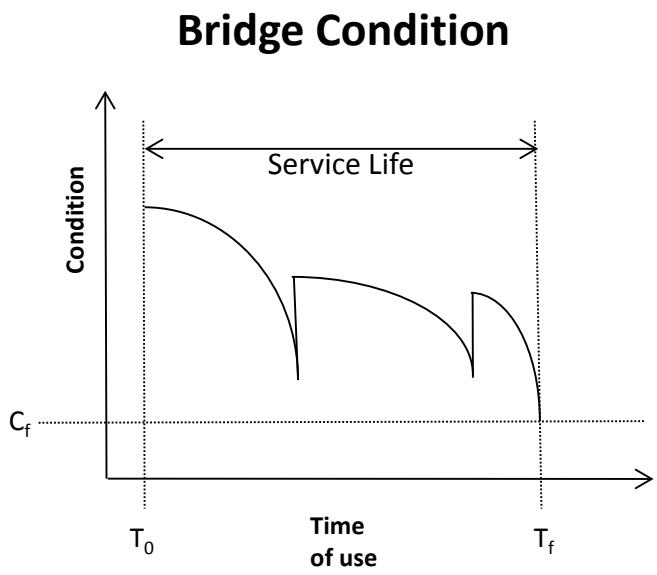


Figure 11.3. Bridge condition life cycle.

The sequence of events and actions that determine the bridge condition throughout its life cycle is called Life Cycle Activity Profile. Actions are usually associated with expenditures that have to be incurred when repair, rehabilitation, and renewal activities are taking place. These expenditures may be plotted on a separate diagram that represents the stream of expenditures associated with construction and repair activities. Such a diagram is sometimes called a cash flow diagram. An example of such diagram is shown on Figure 11.4.

Usually on cash flow diagrams, all resource flows are attributed to either the beginning or the end of the time period in which they actually occur. In cases in which a resource flow is extended over several periods, the expenses are represented as a series of lines. It is important to note that cash flow diagram expenses are shown as lines graphed in the positive domain, while revenues and other returns (e.g., the terminal value of the bridge) are shown as negative values. For simplicity, only Agency costs have been plotted on the example cash flow diagram shown in Figure 11.4. When other types of costs (e.g., user costs and vulnerability costs) are included, the cash flow diagram serves as a graphic representation of the NPV computation.

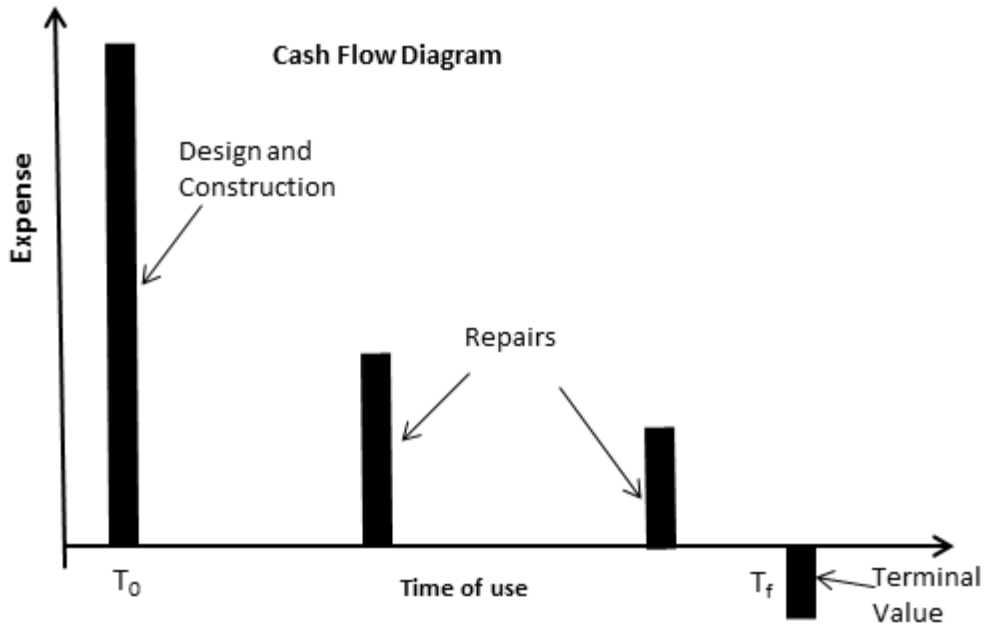


Figure 11.4. Cash flow diagram.

11.3.3 Estimating User Costs

Work zone user costs are the increased vehicle operating costs (VOC), delay, and crash costs incurred by highway users as a result of construction, maintenance, or rehabilitation work zones. User costs may represent the greatest data challenge for its consideration in LCCA. When calculated, user costs are often so large that they may substantially exceed agency costs, particularly for transportation investments being considered for high traffic areas. Congestion statistics and cost can be obtained from the *Annual Urban Mobility Report* prepared by the Texas Transportation Institute (Schrank et al. 2011).

The publication *Life-Cycle Cost Analysis in Pavement Design*, by the FHWA (Walls and Smith 1998), includes a rational step-by-step procedure for determining user costs associated with work zones. Work Zone is defined in the Highway Capacity Manual (TRB 2010) as an area of a highway where maintenance and construction operations impinge on the number of lanes available to traffic or affect the operational characteristics of traffic flowing through the area. In order to analyze work zone user costs, work zone characteristics associated with alternative designs and supporting maintenance and rehabilitation strategies must be defined as part of the development of alternative designs.

Key work zone characteristics include such factors as work zone length, number and capacity of lanes open, duration of lane closure timing (e.g., hours of the day, days of the week, season of the year, etc.), posted speed, and the availability of, and the physical and traffic characteristics of alternative routes. The strategy for maintaining traffic should include any anticipated restrictions on the contractor's or maintenance force's hours of operation or ability to establish lane closures.

Specific details in an LCCA should include:

1. Projected year work zones occur (years 5, 8, 12, etc.);
2. Number of days the work zone will be in place (construction period);
3. Specific hours of each day, as well as the days of the week the work zone will be in place; and
4. Work zone length and posted speed.

The duration of a work zone (the overall length of time a facility or portion of a facility is out of service or traffic is restricted) can range from sporadic daily lane closures for maintenance to several months for bridge deck replacements. In many cases, the differential routine maintenance cost between alternatives tends to be insignificant when compared to initial construction and rehabilitation costs. To a great extent, the same is true of user costs resulting from routine maintenance activities. Routine maintenance work zones tend to be relatively infrequent, of short duration, and outside of peak traffic flow periods. As such, attention should focus on user costs associated with major work zones.

User costs are directly dependent on the effects of the volume and operating characteristics of the traffic on the facility. Each construction, maintenance, and rehabilitation activity generally involves some temporary effect on traffic using the facility, varying from insignificant for minor work zone restrictions on low-volume facilities to highly significant for major lane closures on high-volume facilities.

The major traffic characteristics of interest for each year that a work zone will be established include:

1. Overall projected average annual daily traffic (AADT) volumes on both the facility and possibly alternate routes;
2. Associated 24-hour directional hourly demand distributions; and
3. Vehicle classification distribution of the projected traffic streams.

On high-volume routes, distinctions between weekday and weekend traffic demand and hourly distributions become important. Further, seasonal AADT traffic distribution also becomes important when work zones are proposed on recreational routes during seasonal peak periods.

Once the individual work zones have been identified, each is evaluated separately. This is the point at which individual user cost components are quantified and converted to dollar cost values. A detailed example of the user cost calculation is given in the 1998 FHWA publication, *Life Cycle Cost Analysis in Pavement Design* (Walls and Smith 1998).

As mentioned in Section 11.2.2.2, the recent pooled-fund study led by the Oregon DOT (Doolen, et al. 2011) developed a set of decision making tools to evaluate the cost effectiveness of using ABC techniques versus. conventional construction, and incorporated user costs as part of the life-cycle cost analysis comparison. The tools also incorporated an analytical hierarchy process (AHP), which is a technique that aids decision makers in prioritizing multiple criteria, and uses a multi-level hierarchical structure of objectives, criteria and alternatives. It considers both quantitative and qualitative criteria and quantifies the qualitative trade-offs and relationship between criteria using a hierarchy of criteria. This was important for ABC because it quantified various qualitative factors contributing to user costs, such as user delay from a long detour, and could show the economic benefit resulting from reduced construction duration.

11.3.4 Computational Approaches

The two approaches used in preparing an LCCA differ dramatically in how they address the variability and uncertainty associated with various input factors, and with the risk associated with the various uncertainties. Often there is some level of possible variability and uncertainty in regard to the values identified for each input parameter. This possible variation can often have significant effects on the LCCA outcome.

11.3.4.1 Deterministic Approach

Traditionally in the deterministic approach, input variables are treated as fixed values as if those values were certain. This approach assigns each LCCA input variable with a fixed (base case) value based on statistics and non-linear regression of actually-occurring data, or professional judgment.

This method does not specifically address the degree of variability or uncertainty with input values. In order to incorporate uncertainty about input values into the analysis, a sensitivity analysis can be performed to see the effect of variation on any one parameter. However, the deterministic approach combined with the sensitivity analysis has two drawbacks. First, it can only be applied to input variables one-by-one, while in reality the real question of interest is how the variation in several variables simultaneously can affect the result. Even more importantly, sensitivity analysis alone does not provide any information on the relative likelihood of different outcomes. For example, the sensitivity analysis may suggest that if the initial construction cost is 10% higher than is assumed in the base case, the corresponding NPV of all costs would be 7% higher than in the base case. However, it will provide no information on whether this scenario is likely to occur. In order to characterize relative likelihood of various potential outcomes, a stochastic approach should be adopted.

11.3.4.2 Stochastic Approach

Stochastic approach (sometimes referred to as probabilistic approach, or risk analysis approach) defines the value of input variables by a frequency (probability) distribution. For a given project alternative, the uncertain input parameters are identified. Then, for each uncertain parameter, a sampling distribution of possible values is developed. Simulation programming randomly draws values from the stochastic description of each input variable and uses these values to compute a forecasted NPV. This sampling process is repeated through thousands of iterations, and through the process, an entire probability distribution of NPVs is generated for the project alternative along with the mean or average NPV for that alternative. The resulting NPV distribution can then be compared with the projected NPVs for alternatives, and the most economical option for implementing the project can be determined for any given risk level.

The concept of risk arises from the uncertainty associated with future events and the inability to know what outcomes will result from particular actions taken today. Risk can be objective or subjective. Subjective risk is a person’s perception of the likelihood of a particular event; this perception of risk may include the ability to avoid the risk and an assessment of the consequences of a particular outcome. Objective risk, on the other hand, incorporates theory, experiments, observation, and other unbiased information. Ultimately, decision makers who are characterized by different degrees of risk tolerance and whose perception of risk is intrinsically subjective make the decisions. . However, the goal of risk analysis is to provide the best-unbiased risk estimates to arm the decision makers with the most accurate representation of objective risk.

Figure 11.5 depicts the stochastic approach as a whole. Stated succinctly, it reflects uncertainty in input factors (e.g., construction and repair costs and the timing of those activities) in the probability distribution of the results. Description of the specific steps involved in stochastic approach follow.

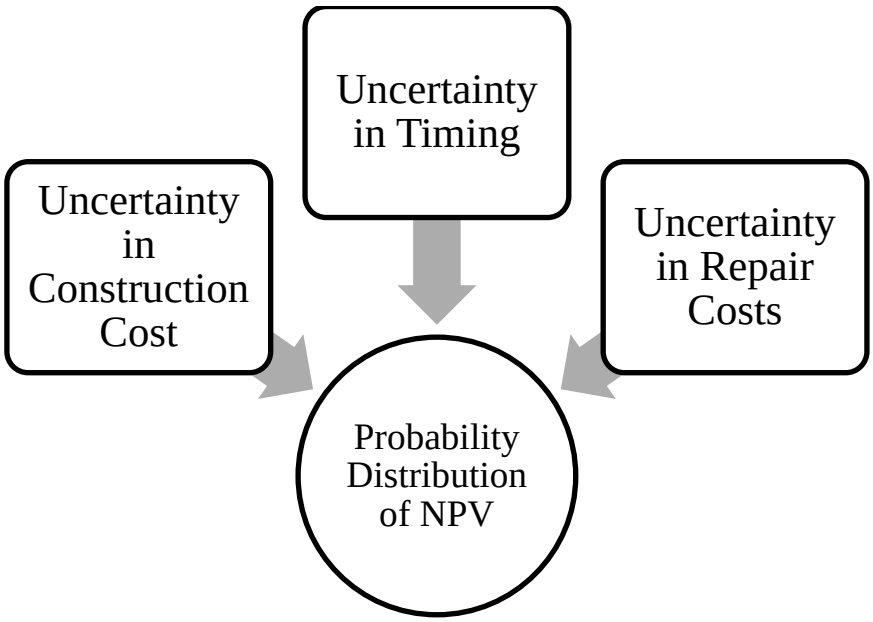


Figure 11.5. Stochastic approach to LCCA.

Develop Structure and Layout of the Problem. The first step in conducting risk analysis consists of reducing the problem to its most basic elements and describing it in the form of an analytical model. The models for LCCA problems typically include NPV computation, definition of cost categories, and determination of other functional relationships such as bridge condition curves. Project alternatives are also identified and described. Cash flow diagrams and life-cycle curves are convenient tools to clearly present the project details and the features of alternative implementations.

Once the structure of the problem is fully determined, a list of inputs can be developed. An example of input variables for a LCCA project is presented in Table 11.1. For each of the input variables, the general basis used to determine their values is established, and a subset of input variables for which a stochastic distribution will be used is specified.

Table 11.1. LCCA Input Variables.

LCCA Component	Input Variable	Source
Initial and Future Agency Costs	Preliminary Engineering	Estimate
	Construction Management	Estimate
	Construction	Estimate
	Maintenance	Assumption
Timing of Costs	Bridge Deterioration	Projection
User Costs	Current Traffic	Estimate
	Future Traffic	Projection
	Hourly Demand	Estimate
	Vehicle Distributions	Estimate
	Value of Delay Time	Assumption
	Work Zone Configuration	Assumption
	Work Zone Hours of Operation	Assumption
	Work Zone Duration	Assumption
	Work Zone Activity Years	Projection
	Crash Rates	Estimate
	Crash Cost Rates	Assumption
Vulnerability Costs	Flood Probability	Estimate
	Flood Damage Distribution	Estimate
	Earthquake Probability	Estimate
	Earthquake Damage	Estimate
	Load Distribution Probability	Estimate
	Load-Related Structural Damage	Estimate
Other Parameters	Discount Rate	Assumption

Develop Input Data. The next step in conducting LCCA is developing probability distributions for the uncertain variables identified in the next step. A probability distribution describes the complete range of values that a variable may assume and weighs the likelihood of the occurrence. Figure 11.6 illustrates some of the most common probability distributions shown in a histogram format—uniform, triangular, and normal distributions. The horizontal axis provides a range of all possible values that the variable assumes and the vertical axis shows the relative frequency weighting of the occurrence of any particular value. For the distributions on Figure 11.6, the probability of a range of values is equal to the area under the curve, and the total area under the curve is equal to one.

The choice of a particular distribution depends on the type of input and the amount of data available. Triangular distribution (see Figure 11.6) is the most common distribution used to represent various variables using expert

elicitation. Expert opinions are used to determine the minimum, the maximum, and the most likely value, and the triangle is constructed using those three points. This method is most appropriate for modeling such input variables as service life, discount rate, work zone delay, etc. Normal distribution is the most common continuous distribution used to represent random variables symmetrically distributed around the mean value. It is usually a good candidate to represent cost-like variables, such as construction cost, maintenance cost, etc. Normal distribution can either be used to represent the information obtained from the expert elicitation or to use data collected from other sources when the amount of data is sufficiently large. On the other hand, when little information about the input variable is available, a uniform distribution might be used as a rough approximation. Uniform distribution assumes that outside of a certain range, the probability of outcomes drops to zero, but within the range we have no information as to which outcomes are more likely, so it is assumed uniform over that range.

While normal and uniform distributions are symmetric around their mean values, the triangular distribution can be either symmetric or asymmetric. Other types of common asymmetric distributions are exponential and lognormal. When a large amount of hard data is available, the best distribution can be determined by using statistical techniques to establish goodness of fit for each of them. However, when data availability is limited, triangular distribution can be used as a rough estimate of the distribution shape.

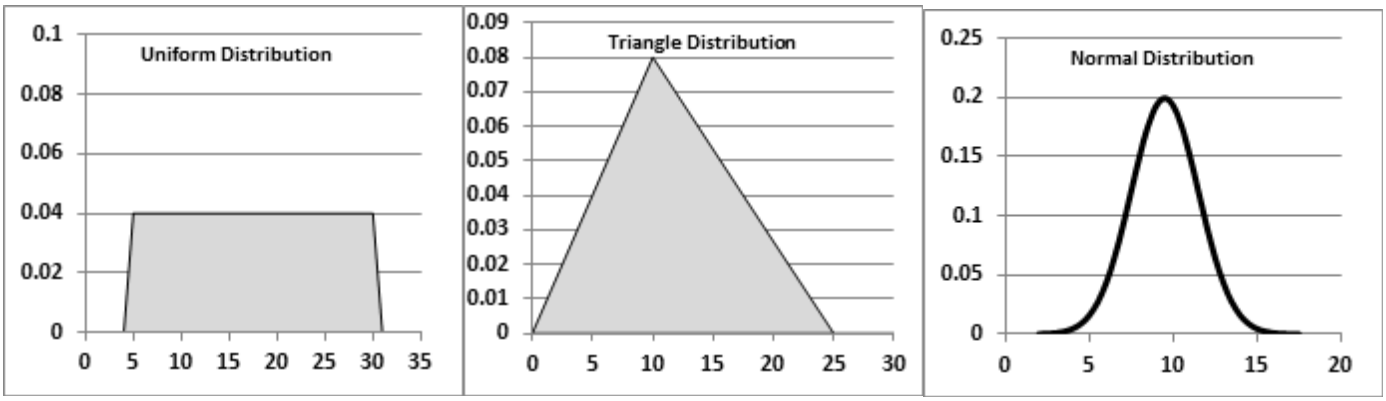


Figure 11.6. Example probability distributions.

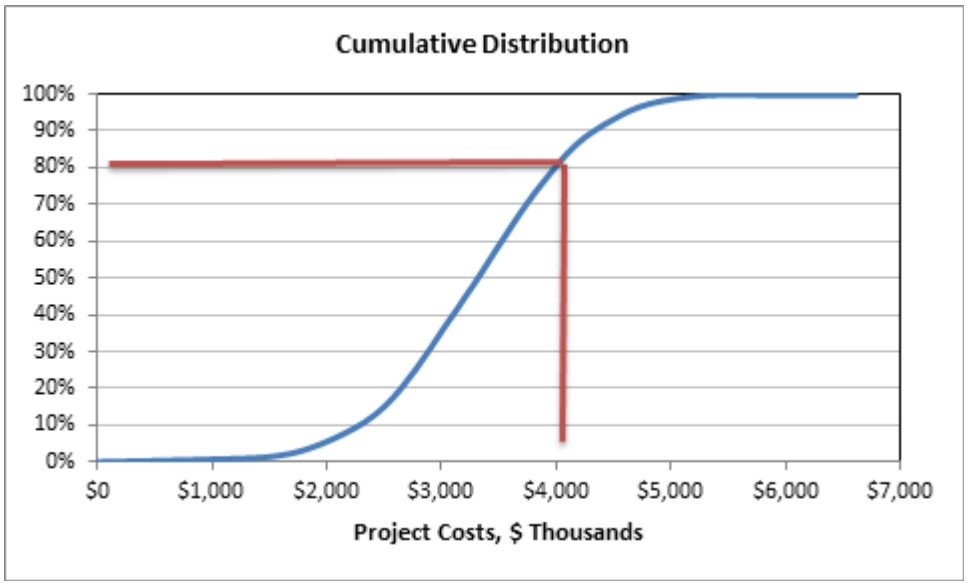


Figure 11.7. Ascending cumulative probability distribution.

Figure 11.7 shows a cumulative probability distribution in ascending order that portrays a cumulative probability of a group of possible events. (Sometimes cumulative distributions are shown in descending order and then the points on the curve show the probability of exceeding a particular value.) For example, there is an 80% probability that the project cost will not exceed $4 million. Each probability distribution has a corresponding cumulative distribution. Cumulative distributions present the data in a form that is easy to interpret for the purposes of risk analysis.

Develop Probability Distributions. When existing data is available, the standard method is to use the data to choose a functional form of a probability distribution that best fits the available data. Many statistical methods and software packages can be used to compare common distribution types with the available data and to determine goodness of fit, in other words, to indicate how well a particular probability distribution fits the data.

However, when sufficient relevant data is not readily available, group interviews are often used to develop probabilities of uncertain variables. Expert panels are convened to establish the boundaries and general shape of input distributions. The process of eliciting information from experts may include group meetings, individual interviews, and formal surveys and questionnaires. The goal of expert elicitation is to establish the general shape of input distribution, and to determine if there are any interdependencies among input variables. The process of expert elicitation is shown on Figure 11.8.

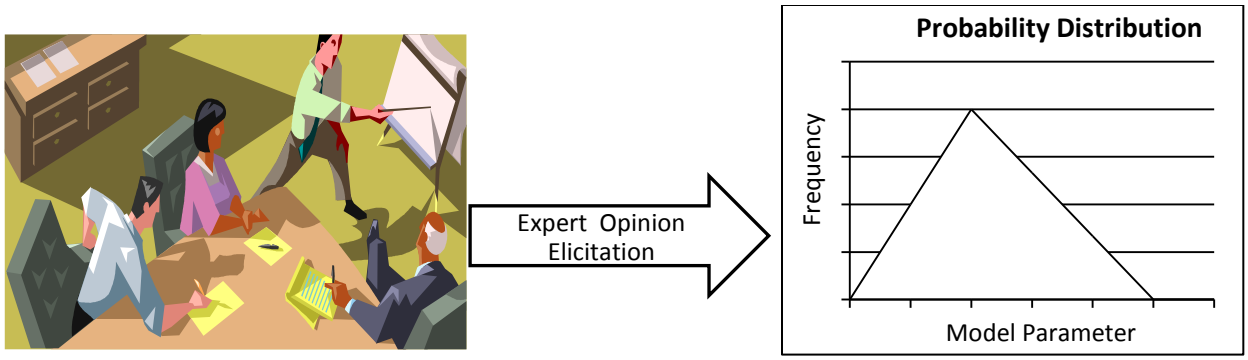


Figure 11.8. Using expert opinion to develop probability distributions.

Perform Simulations. The next step in the LCCA process involving risk analysis is to run a computer simulation of the model in order to obtain results. A process of using random numbers to sample from a probability distribution is known as *Monte Carlo* sampling. In the Monte Carlo simulation process, a series of random numbers are generated by the computer along the cumulative probability scale of input distribution (see Figure 11.7). Values corresponding to each random number are sampled along the x-scale. The sampled value for one input is then combined with sampled values for all other inputs to compute the single result. This process is repeated hundreds or thousands of times to generate a cumulative distribution of the outcomes. The stopping rule involves either a pre-specified number of iterations or a convergence rule, in other words, a situation in which additional iterations do not significantly affect the distribution of the results.

Monte Carlo simulations require a large number of iterations to assure that values with low probabilities are sufficiently sampled and represented in the results. This is especially important when the input distributions are highly unsymmetric. When the number of iterations is insufficient, the low probability outcomes may be underepresented and not adequately accounted for in simulations. This is especially significant when a low-probability outcome can have a particularly strong effect on the results. In order to avoid such problems, different sampling methods can be employed. For example, Latin Hypercube sampling uses special techniques to generate samples from all probability ranges with a relatively low total number of iterations.

Interpret Results. If the analysis uses the traditional deterministic approach, the only data available to decision-makers would be the means of the output distributions for the alternatives investigated. Based on the comparisons of the means (50% probability), the difference between Alternative 1 and Alternative 2 in Figure 11.9 is small and may be assumed to be negligible. The risk analysis allows one to evaluate a much more nuanced picture. When

interpreting the risk profile on Figure 11.9, it is important to distinguish the upside risk from the downside risk. Downside risk for project costs implies a cost overrun, a chance that the costs will be much higher than anticipated. On the other hand, the upside risk presents an opportunity for low cost, a cost underrun. From Figure 11.9, we can see that Alternative 1 has greater upside risk than Alternative 2, in other words, it has a better chance that the cost will be very low. At the same time, Alternative 1 is a preferred choice because it reduces the downside risk of cost overruns. Another consideration is to compare the two alternatives at the 80% cumulative probability level. Alternative 1 is about $100,000 while Alternative 2 is $130,000. While the means have a negligable risk difference, Alternative 1 exhibits far less of a financial risk.

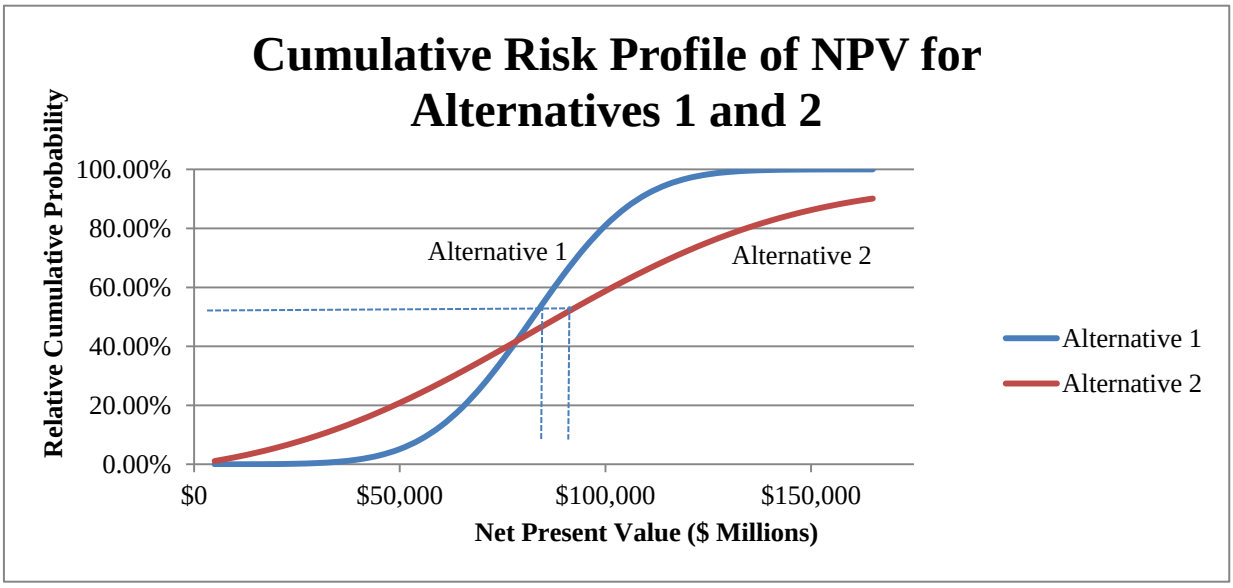


Figure 11.9. Cumulative risk profile of NPV for Alternatives 1 and 2.

As a part of the risk assessment, a sensitivity analysis of simulation results can be performed to identify the key input variables that have the most influence on the output distributions. Typically, this is done by computing the degree of correlation between inputs and outputs: the higher the degree of correlation, the more significant a particular input variable is for determining the results.

From the perspective of most transportation agencies, the application of stochastic LCCA is relatively new. Stochastic LCCA has become more practical due to the dramatic increases in computer processing capabilities. Simulating and accounting for simultaneous changes in LCCA input parameters can now be accomplished easily and quickly and provides invaluable information for making informed decisions.

11.3.5 LCCA Analysis Tools

11.3.5.1 Simplified LCCA Applications

Various tools and software are available for determining LCCA. All steps of a simplified deterministic LCCA can be performed using generic spreadsheet software such as Microsoft Excel.

For conducting stochastic or risk based LCCA, there are several commercially-available, microcomputer-based risk and analysis software programs that are either spreadsheet based or work as add-ons to other generic spreadsheet software. These tools incorporate Monte Carlo simulation capabilities for stochastic analyses. Two common applications are presented in Table 11.2.

Table 11.2. Risk Based LCCA software.

Software Name	Producer
@Risk	Palisade Corporation www.palisade.com
Oracle Crystal Ball	Decisioneering Corporation www.decisioneering.com

In using simple generic spreadsheets, the economic life-cycle analysis model has to be programmed into the spreadsheet, and other considerations such as incorporating user costs also have to be computed separately. However, the user has much more control over the process and how data is used and presented. For simple applications, generic spreadsheet solutions are common and easily applied.

11.3.5.2 Comprehensive LCCA Applications

Specialized comprehensive software tools for performing LCCA have recently been developed by federal agencies and are available from governing public websites. The available software tools described are capable of calculating comprehensive life-cycle costs including agency, user, operations and maintenance, disposal, and remaining service life costs. They are also capable of performing both deterministic and stochastic analyses.

BridgeLCC Software from the National Institute of Standards and Technology (NIST)

BridgeLCC 2.0 is comprehensive LCCA software developed by the National Institute of Standards and Technology to help bridge designers determine the cost-effectiveness of alternative bridge designs, construction and repair strategies, and construction materials (Ehlen 2003). The software uses a life-cycle costing methodology based

on the ASTM E917 standard practice for life-cycle costing and a cost classification scheme developed by NIST (Ehlen 2003). This software is specifically tailored to highway bridges.

BridgeLCC 2.0 can segregate costs by bearer (agency, user and third party), by timing (initial construction, operations, maintenance and repair [O, M and R], and Disposal), and by component (deck, superstructure, substructure, other, non-elemental and new technology introduction). The program also includes advanced features such as calculation of user delay and cost, has capabilities for both deterministic and stochastic analyses, and includes sensitivity analysis and risk analysis using Monte Carlo simulation (Ehlen 2003).

RealCost Life-Cycle Cost Analysis Software

RealCost was developed by the FHWA to support the application of LCCA in the pavement project-level decision-making process (FHWA 2004). RealCost automates FHWA's LCCA methodology as it applies to pavements. Work is being considered to make it applicable to bridges but currently, bridges have not been included in this document.

The software calculates life-cycle values for both agency and user costs associated with construction and rehabilitation. The software can perform both deterministic and stochastic modeling (Monte Carlo Simulation), and can also include user costs. Outputs are provided in tabular and graphic format.

RealCost is an add-in for Microsoft Excel providing a graphical user interface that facilitates the creation an Excel Workbook containing the input data and results of the life-cycle cost analysis. RealCost can only evaluate two alternatives at a time.

APPENDIX A

DESIGN PROVISIONS FOR SELF-STRESSING SYSTEM FOR BRIDGE APPLICATION WITH EMPHASIS ON PRECAST PANEL DECK SYSTEM

Steel girder bridges often use continuity over the interior supports to reduce interior forces on the spans. In continuous structures with composite concrete decks, the location of maximum negative bending moment is over the interior supports. This moment produces tensile stresses in the concrete deck and compressive stress in the bottom flanges of the girders. The tensile stress in the deck leads to cracking, which allows intrusion of moisture and road salt, causing corrosion of the reinforcement, and supporting girders. Continued maintenance is required to forestall the deterioration; however, replacement of the deck is eventually required.

To help alleviate this problem, a self-stressing system was developed as part of the *SHRP 2 R19A* project. Additional details of the development can be found in the forthcoming final report. The method induces a compressive force in the deck accomplished by raising the interior supports above their final elevation while the deck is cast (cast-in-place construction) or placed (precast construction). Once the concrete has cured, the supports are lowered to their final elevation. Continuity of the steel member and the composite action with the deck produce a compressive stress in the concrete slab, which is balanced by tensile stresses in the bottom of the steel member. As a result, the cracking over interior support is reduced, increasing durability. Additionally, the need for girder splices may be eliminated, making the overall bridge design more efficient and less expensive when compared to conventional design.

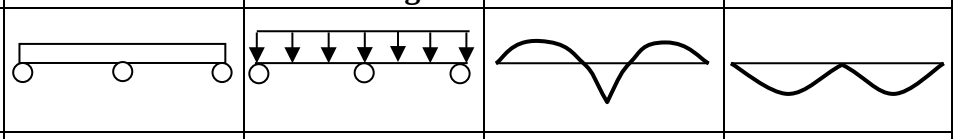
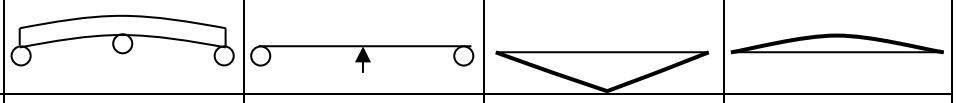
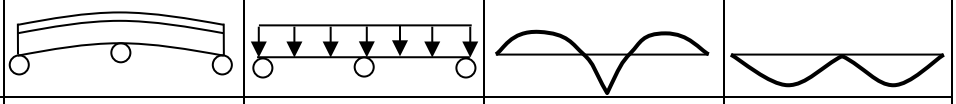
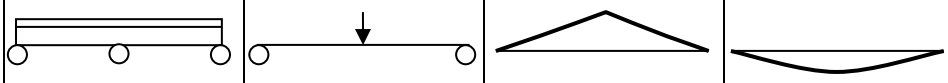
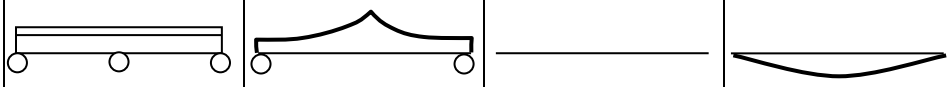
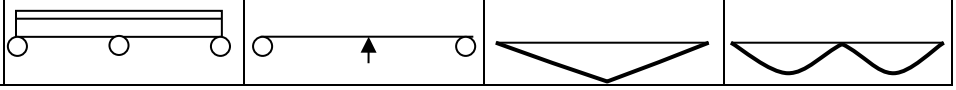
This appendix describes the construction procedure, design considerations, and implementation details for using the self-stressing method. A flow chart is provided to aid in implementation. Simplified formulas applicable to two-span bridges, which represent the most likely use of the method, are also included.

A.1. CONSTRUCTION PROCEDURE OVERVIEW

This section provides a brief description of the major steps in the construction procedure, in order to establish a frame of reference and to introduce vocabulary used throughout the appendix. Table A.1 illustrates the major steps

required for constructing a bridge using the self-stressing method. These steps will be used as points of reference in the remaining discussion.

Table A.1. Self-Stressing Method Major Steps.

	Stage	Structure	Loading	Moment	Deflection
1	Place Girder on Level Supports				
2	Raise Interior Support				
3	Cast Concrete				
4	Lower Interior Support				
5a	Relaxation				
5b	Restoring Force				

The first stage consists of simply placing the girder onto the level supports. The resulting moments and deflections are those obtained from a continuous beam analysis.

During the second stage, the interior support is raised. The bare steel girder responds as a simply supported beam subjected to an upward directed point load at the location of the interior support. Note that the supports could be in the raised position prior to placing the girder. However, due to superposition, the analysis would be the same as described.

Next the concrete deck is cast, or precast panels are placed and grouted. The structure responds like a continuous bare steel beam, just as it would be for conventional construction.

During the fourth stage, the interior support is lowered to its final position. Just as in Stage 2, the response is that of a beam supported at the exterior supports only and subjected to a point load. However, the structure is now composite and the point load is directed downward. This action places the concrete deck over the supports into compression.

Over time, creep and shrinkage occur in the concrete deck. This may be accounted for in two stages. First, the creep and shrinkage are seen as an applied curvature on the structure. If the beam were simply supported by the exterior supports, this applied curvature would result in additional deflection without inducing additional load. However, due to the continuity, a restoring force is generated that prevents the displacement and results in additional stresses.

A.2. DESIGN CONSIDERATIONS

This section provides a discussion of the design issues specific to the use of the self-stressing method.

Design of bridges using the self-stressing method should follow the provisions for I-Section and Box-Section flexural members contained in Section 6.10 and 6.11 respectively, of the *AASHTO LRFD Bridge Design Specifications* (*LRFD Specifications*), except as modified herein (AASHTO 2012).

A.2.1. General

The use of the self-stressing method is limited to straight I- and Box-Section steel girders and is applicable only to continuous multi-span structures with a composite deck. Practical limitations dictate that the method is most likely to be used in two-span structures. Simplified design aids are provided in Section A.5 for structures with two spans.

A.2.2. Analysis

Two options provided for the analysis of the structure are described in the following section. Note that the analysis methods should only be used when analyzing the construction steps associated with the self-stressing method and not the overall analysis procedures as covered in Chapter 4 of the *LRFD Specifications*.

A.2.2.1. Simplified

The simplified analysis method relies on first order techniques that disregard time effects in the concrete. These effects are accounted for using conservative correction factors presented in the implementation details portion of the provisions (Section A.3). The correction factors account for the effects of creep and shrinkage in the evaluation of stresses and deflections. As an alternative, advanced methods of analysis may be used that directly evaluate these effects.

A.2.2.2. Advanced

Advanced methods explicitly consider the effects of creep and shrinkage to evaluate the stresses and deflections.

Several examples are the effective modulus method (EMM), adjusted effective modulus method (AEMM), step-by-step method (SSM), and the rate of creep method (RCM).

When the creep and shrinkage strains are known, or otherwise assumed, then the *LRFD Specifications*, Section C4.6.6 can be used for calculating the resulting stresses and deformations.

A.2.3. Forces

The forces and stresses in all components that arise due to the self-stressing construction procedure should be considered in evaluating the load effects during design. For the purpose of design, the locked-in prestressing force shall be considered a dead load force applied to the composite long-term section (DC2).

The *LRFD Specifications*, Section 3.4.1 states that where prestressed components are used in conjunction with steel girders, the force effect should be considered as locked-in construction loads (EL). However, in this situation the prestressing forces are being developed by gravity effects rather than applied by prestressing devices. As such, the variability in the resulting stresses will be of the same magnitude as the variability of the dead load effects, which leads to the decision of considering the prestress stress as DC2 loading.

Note that the self-stressing procedure will generate tensile stresses in the bottom of the steel girders that will serve to offset some of the compressive dead and live load stresses. As such, the stresses due to the self-stressing procedure should be kept separate from other dead load stress sources and the minimum load factor for dead load should be used (0.9).

A.2.4. Deflections

The final deflected shape is necessary for determining the camber requirements of the girders and can be obtained by summing the deflections from the various construction stages.

A.3. DESIGN PROCEDURE AND IMPLEMENTATION DETAILS

This section provides a step-by-step procedure for designing a bridge incorporating the self-stressing method. All grout, and/or adhesives must be adequately cured prior to lowering the interior support. The creep and shrinkage properties of the materials must be compatible with the intended use and properties assumed during analysis.

Step 1. Determine Required Amount of Prestress

The self-stressing method is a way to introduce compressive stresses in the concrete deck of a multi-span continuous beam. The compressive stresses are generally located near the interior supports and therefore work to counter the tensile stresses that arise in this vicinity due to gravity and live loading. The result is a reduction in cracking and an accompanying increase in service life. The magnitude of the prestress that must be applied in order to achieve the desired effects has been determined based on past experience with decks that have been prestressed using traditional mechanical methods.

Minimum Final Prestress

The recommended minimum level of prestress at the top fiber of the concrete deck over an interior support, after all losses, is 750 psi.

The simplified (Bernoulli assumption) analysis methods predict a linear variation of stresses through the thickness of the deck, which produces a maximum stress value at the face of the concrete. In practice, creep effects quickly blunt this maximum stress value, resulting in a more uniform stress profile through the depth of the concrete. The prescribed minimum prestress value at the face of the slab is intended to provide a final uniform value over the top half of the slab of 250 psi, which is the value recommended in Section 9.7.5.3 of the *LRFD Specifications* for longitudinal prestressing of concrete slabs.

Figure A.1 shows the initial stress distribution in the concrete deck and the distribution that develops after some period of time has elapsed.

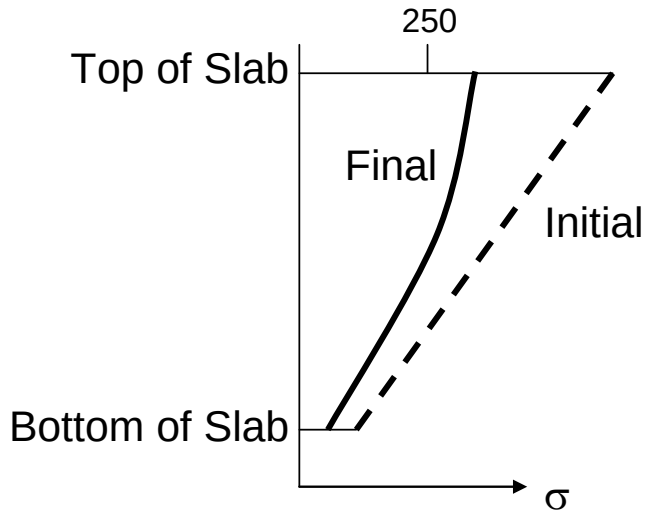


Figure A.1. Stress distribution in concrete deck.

Maximum Initial Prestress

The maximum initial prestress to be applied shall be no greater than 60 percent of the concrete compressive strength.

There is no upper limit recommendation in the literature because the material maximum strength is a natural upper bound. However, in order to maintain a safe margin, the upper limit shall not be greater than 60% of the concrete compressive strength (0.6*f'c), which is the compressive stress limit recommended in Section 5.9.4.1.1 of the *LRFD Specifications* for pretensioned and posttensioned concrete components, including segmentally constructed bridges.

Adjust Prestress for Losses

In lieu of an exact analysis, the prestress loss may be conservatively estimated as 20% when the initial prestress value is less than 40% of the concrete compressive strength, and 30 percent when the initial prestress value is greater than 40% of the concrete compressive strength.

Equation A.1 gives the initial prestress at the top fiber that is to be applied.

$$\sigma_{pi} = \frac{\sigma_{pf}}{(1 - r_s)} \qquad \textbf{EQ A.1}$$

Where:

σ_{pf} = final prestress stress
σ_{pi} = initial prestress stress
r_s = loss due to creep and shrinkage

Step 2. Calculate Amount of Deflection to Obtain Desired Prestress

Determine the height to which the interior support must be raised so that upon release it will provide the desired amount of prestress.

The problem at hand is essentially that of support settlement. How far must the interior support settle so that the stress in the top of the deck is the value chosen in the previous design step?

For the following steps, the structure to be considered is the composite structure being supported at the exterior supports only, which is shown in Figure A.2.

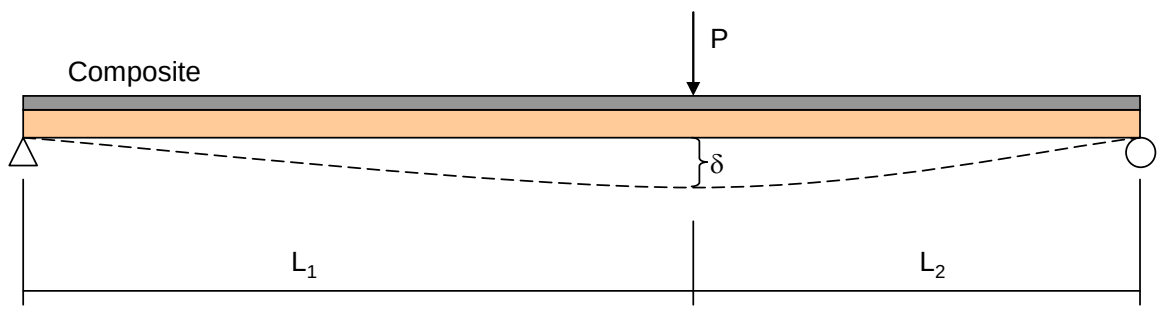


Figure A.2. Equivalent structure used for calculating stresses during lowering of support.

a) Determine the stress at the top fiber of the deck due to point loading applied at the interior support location.

b) Use the result from (a) to solve for the magnitude of the forces required to produce the desired prestress determined in **Step 1**.

c) Calculate the stiffness with respect to point load applied at the interior support location.

d) Use the stiffness from (c) to solve for displacement required to produce the necessary force. For the structure shown in Figure A.2, this displacement is given in Equation A.2.

$$\delta = \frac{\sigma_{ts} L_1 L_2}{3 E_{conc} c_{ts}} \qquad \textbf{EQ A.2}$$

Where:

δ	=	amount of displacement required
σ_{ts}	=	initial prestress stress
L1	=	length of span 1
L2	=	length of span 2
EConc	=	modulus of elasticity of concrete
Cts	=	distance from neutral axis to top fiber of slab

Step 3. Determine Forces Due to Lifting Bare Steel Beam

The results obtained from this step are used to complete the constructability check of the structure. For the following steps, the structure to be considered is the bare steel beam being supported at the exterior supports only, as shown in Figure A.3.

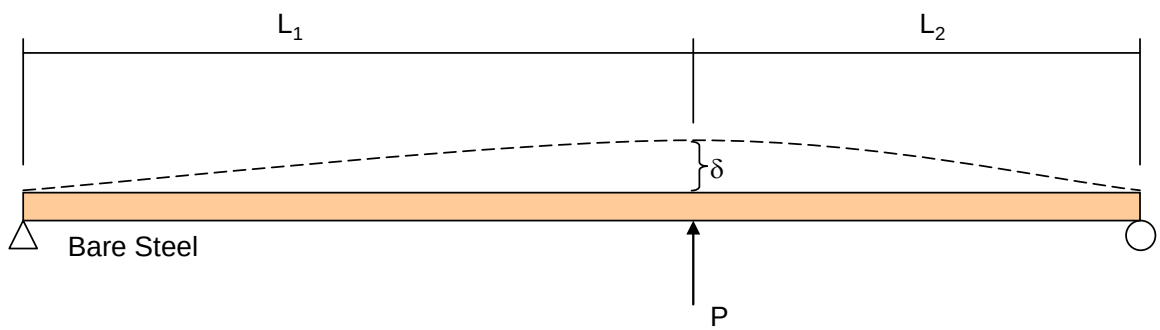


Figure A.3. Equivalent structure used for calculating stresses during the raising of support.

a) Calculate the stiffness with respect to point loads applied at the interior support locations.

b) Use the stiffness from (a) to calculate the force required to lift the interior supports to the height determined in the previous design step. For the structure shown in Figure A.3, this force is given in Equation A.3.

$$P = \frac{3E_{steel} I_{steel} (L_1 + L_2)}{L_1^2 L_2^2} \delta \qquad \textbf{EQ A.3}$$

Where:

P	=	reaction at support due to deflection of support
δ	=	deflection of support
L_1	=	length of span 1
L_2	=	length of span 2
E_{steel}	=	modulus of elasticity of steel
I_{steel}	=	moment of inertia of bare steel girder

c) Using the force given by (b), the reactions, moments, and stresses can be calculated as needed for design.

The steel girders, and any support structures, temporary or permanent, must be designed for the concentrated forces that are developed due to lifting the girders.

End Anchorages

The calculated vertical displacement may require a lifting force that is greater than the self-weight of the steel girder such that the girder would lift off of the end supports. In this situation, the exterior ends of the girder may be anchored to prevent uplift. Once the concrete deck is in place, the weight of the deck will replace this anchorage force.

Also note that loading within the spans can affect uplift at the end supports. Consider the structure shown in Figure A.4. Loading in the first span will create uplift at the end support of the opposite span. Therefore, the progression of deck casting or precast panel placement may affect the need for end anchorages. This possibility must be properly accounted for through either design or the specification of explicit procedures to avoid the condition described.

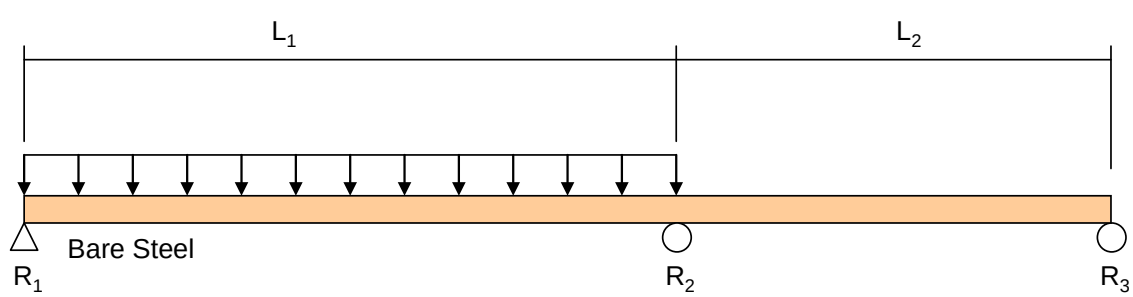


Figure A.4. Loading in Span 1 Producing Uplift at Support R3.

Equation A.4 gives the reaction at the end of Span 2 (unloaded span) due the following combination of loading:

- Self-weight of the steel girder (w_{steel})
- An upward displacement of the interior support (δ)
- Uniform load within Span 1 due to deck placement (w_{deck})

This equation will aid in evaluating the need and magnitude of end anchorages. The critical condition occurs when Span 1, the loaded span, is longer than Span 2. Therefore, when the spans are different lengths, the deck within the shorter span should be cast first.

$$\frac{w_{steel}(3L_2^2 + L_1L_2 - L_1^2)}{8L_2} - \frac{3E_{steel}I_{steel}}{L_1\,L_2^2}\delta - \frac{w_{deck}L_1^3}{8L_2\,(L_1 + L_2)} \qquad \textbf{EQ A.4}$$

Where:

w_{steel}	=	uniform load due to self-weight of the steel
w_{deck}	=	uniform load due to deck placement
δ	=	deflection of support (positive upward)
L_1	=	length of span 1
L_2	=	length of span 2
E_{steel}	=	modulus of elasticity of steel
I_{steel}	=	moment of inertia of bare steel girder

For the case of two equal spans (L_1=L_2=L), Equation A.4 can be simplified to:

$$\frac{3Lw_{steel}}{8} - \frac{3E_{steel}I_{steel}}{L^3}\delta - \frac{Lw_{deck}}{16} \qquad \textbf{EQ A.5}$$

Where:

L	=	length of spans 1 and 2 (equal)

End Anchorages, when necessary must be designed to withstand the concentrated force that is to be applied.

Step 4. Determine Forces and Stresses Due to Lowering Composite Bridge

The forces and stresses imparted on the structure due to lowering the composite bridge are obtained from a similar analysis to that performed when the amount of deflection was originally calculated in **Step 3**.

For the following steps, the structure to be considered is the composite structure being supported at the exterior supports only, as shown in Figure A.5.

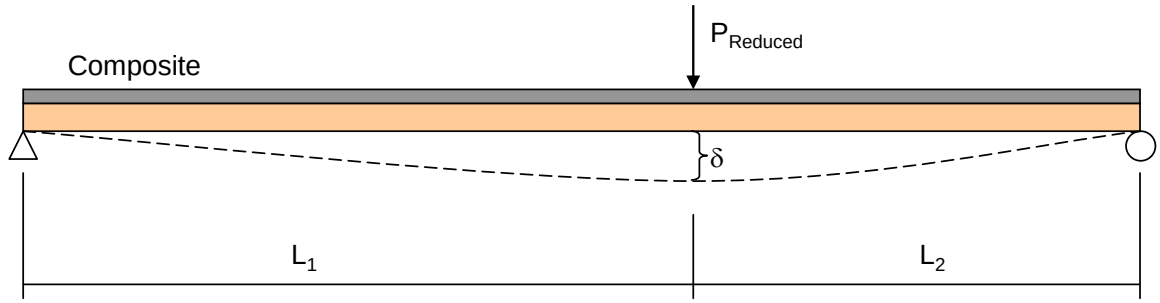


Figure A.5. Equivalent structure used for calculating stresses during lowering of support.

a) Calculate the stiffness with respect to a point load applied at the interior support location.

b) Use the stiffness from (a) to calculate the equivalent point force due to the lowering of the support.

c) Reduce the force calculated in (b) to account for the prestress loss due to creep and shrinkage, as determined in **Step 2**.

d) Using the reduced force applied to the composite structure supported at the exterior supports, calculate the internal forces and stresses necessary for design.

The resulting forces and stresses from this step should be considered dead load forces applied to the composite structure for the purpose of design.

Step 5. Determine Deflected Shape

The final deflected shape is necessary for determining the camber requirements of the girders. The final deflection is the summation of deflections from the various construction stages.

Bare Steel Deflection

Sources of deflection of the bare steel girder are:

- Self-weight of steel,
- Initial lift of interior supports, and
- Casting of wet concrete.

The deflection due to the self-weight of the steel and casting of the wet concrete is calculated in a conventional manner using the continuous bare steel structure, as shown in Figure A.6. Equations for calculating the deformation along the length of the beam can be found the Section A.5.

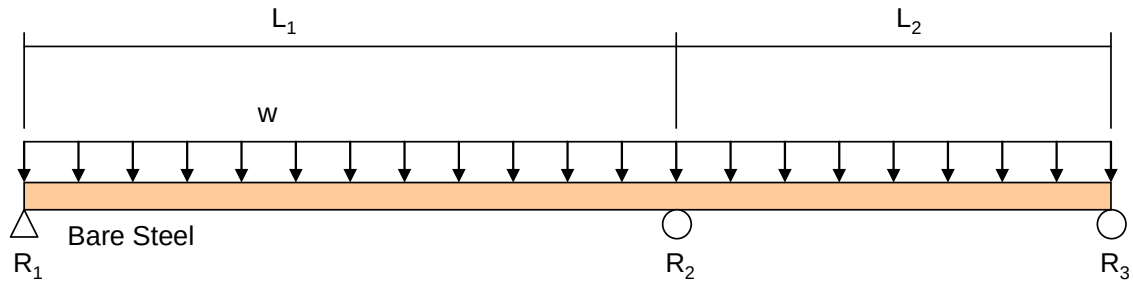


Figure A.6. Structure for calculation of bare steel deflections.

Calculation of the deflection due to the initial lift of the interior support is determined considering the bare steel girder supported at the exterior supports only, as shown in Figure A.7. The structure is subjected to point forces applied at the interior supports as determined in **Step 3**. Equations for calculating the deformation along the length of the beam can be found in Section A.5.

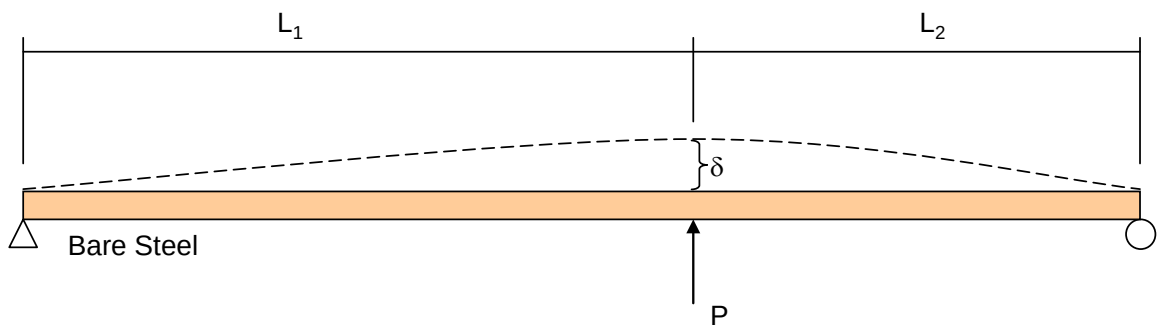


Figure A.7. Structure for calculation of composite deflections due to lowering the support.

Composite Deflection

Calculation of the deflection due to the lowering of the interior support is determined by considering the composite bridge girder supported at the exterior supports only, as shown in Figure A.8. The structure is subjected to point forces applied at the interior supports as determined in **Step 3** without the reduction in load meant to account for creep and shrinkage. Creep and shrinkage have the opposite effect, resulting in an increase of the total deflection. This effect is discussed in the following section. Equations for calculating the deformation along the length of the beam can be found in **Step 2**.

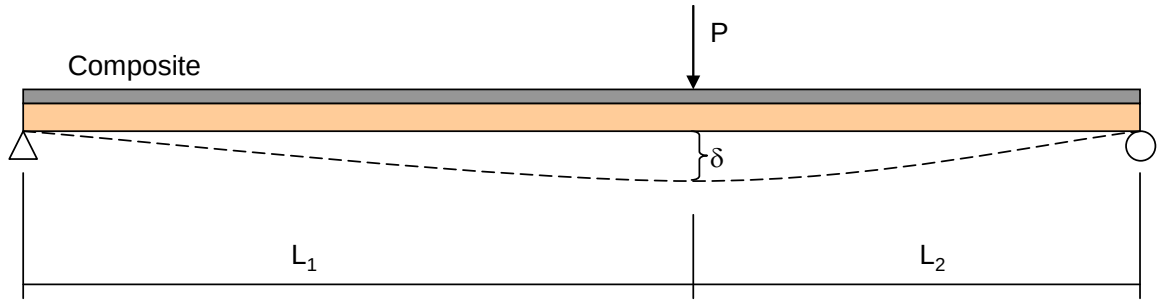


Figure A.8. Structure for calculation of bare steel deflections due to initial lifting of support.

Relaxation Deflection

Additional deflections arise due to curvature induced along the beam due to the effects of creep and shrinkage. The resulting loading can be seen in Figure A.9.

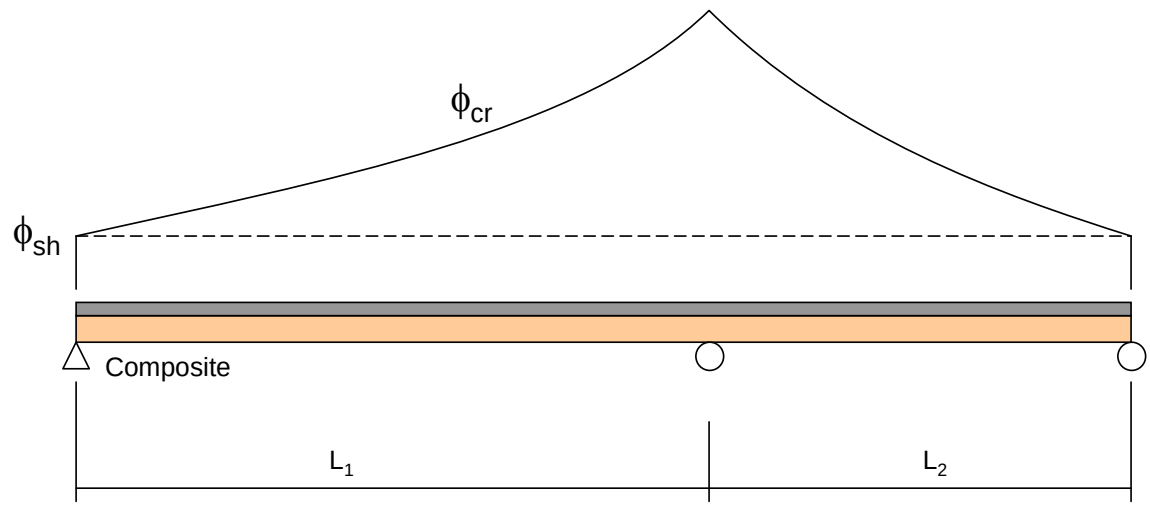


Figure A.9. Curvature applied to continuous structure due to creep and shrinkage.

The steps for calculating the deflected shape can be performed using the following steps, considering the structure supported at the exterior supports only, as shown in Figure A.10.

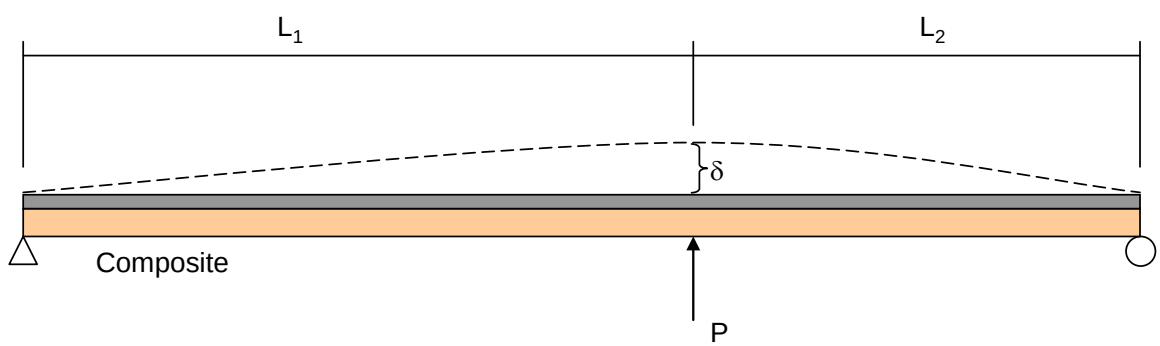


Figure A.10. Structure for determination of restoring force.

a) Calculate the stiffness with respect to a point load applied at the interior support location.

b) Determine the curvature along the length of the beam. The curvature at a section can be obtained from Equation A.6. *LRFD Specifications*, Section 5.4.2.3.1 provides methods for determining the values of ε_{sh} and ε_{cr}.

$$\varphi = \frac{1}{I_c}\int(\varepsilon_{sh} + \varepsilon_{cr})z\,dz \qquad \textbf{EQ A.6}$$

Where:

Φ = curvature of section
I_c = composite moment of inertia
ε_{sh} = strain due to shrinkage
ε_{cr} = strain due to creep
z = distance from neutral axis

c) Calculate the displaced shape of the structure due to the applied curvature, shown in Figure A.11. The displacement can be calculated using the integration given in Equation A.7.

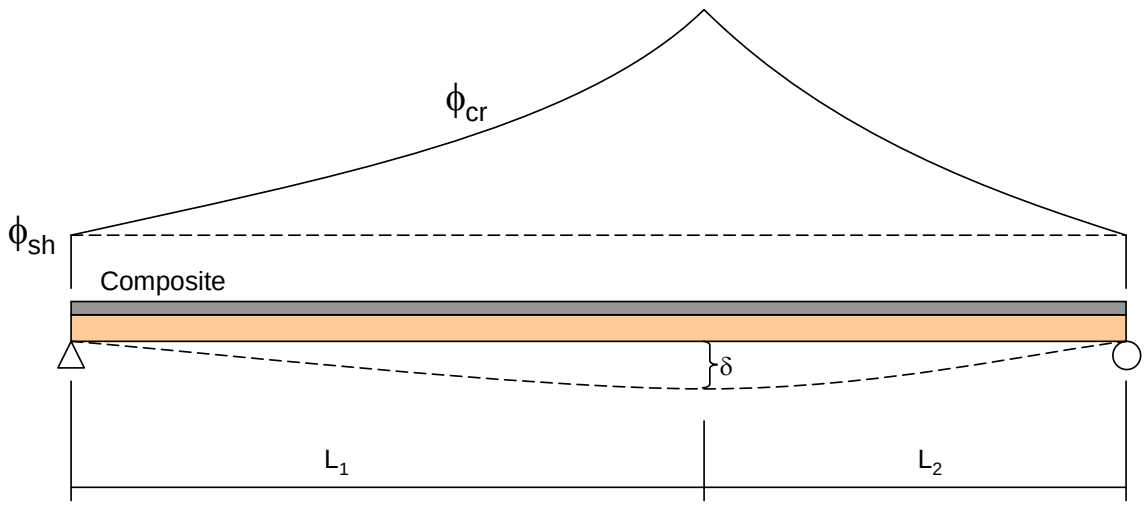


Figure A.11. Structure for determination of deflection due to curvature.

$$\delta(x) = \int_0^x \int_0^x \varphi(x)dx\,dx \qquad \textbf{EQ A.7}$$

Where:

$\Phi(x)$ = curvature along the length of the beam

d) Using the stiffness from (a), determine the force required to offset the displacement at the support location calculated in (c).

e) The resulting deflection due to the relaxation is the sum of the deflections obtained from the applied curvature (Equation A.6) and the application of the point load determined in (d) upon the structure shown in Figure A.10.

Step 6. Carry Out Remainder of Design

The remainder of the design proceeds as it would for a conventional steel girder bridge with concrete deck.

A.4. DESIGN FLOWCHART

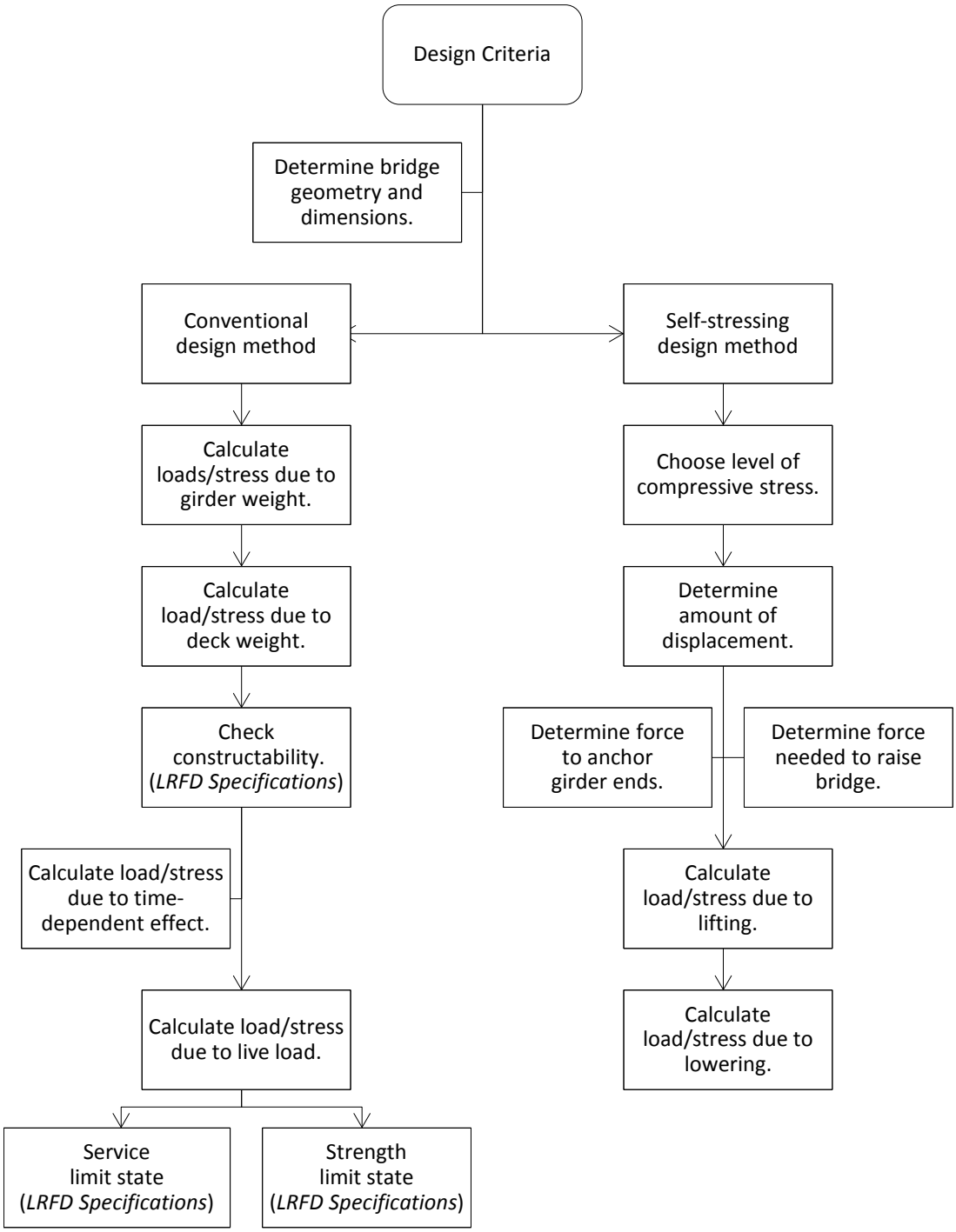
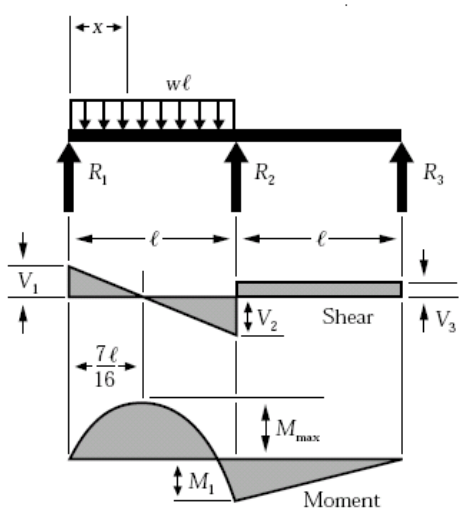


Figure A.12. Design flowchart.

A.5. DESIGN AIDS FOR TWO-SPAN BRIDGES



$$R_1 = V_1 \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots = \frac{7}{16} wl$$

$$R_2 = V_2 + V_3 \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots = \frac{5}{8} wl$$

$$R_3 = V_3 \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots = -\frac{1}{16} wl$$

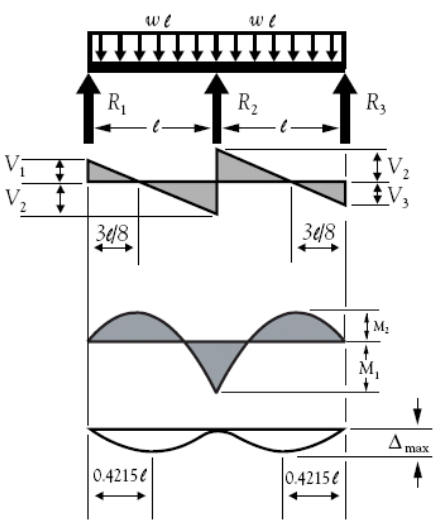
$$V_2 \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots = \frac{9}{16} wl$$

$$M_{\max}\left(\text{at } x = \frac{7}{16} l\right) \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots = \frac{49}{512} wl^2$$

$$M_1(\text{at support } R_2) \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots = \frac{1}{16} wl^2$$

$$M_x(\text{when } x < l) \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots = \frac{wl^2}{16}(7l - 8x)$$

Figure A.13. Continuous beam—two equal spans—uniform load on one span.



$$R_1 = V_1 = R_3 = V_3 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots = \frac{3wl}{8}$$

$$R_2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots = \frac{10wl}{8}$$

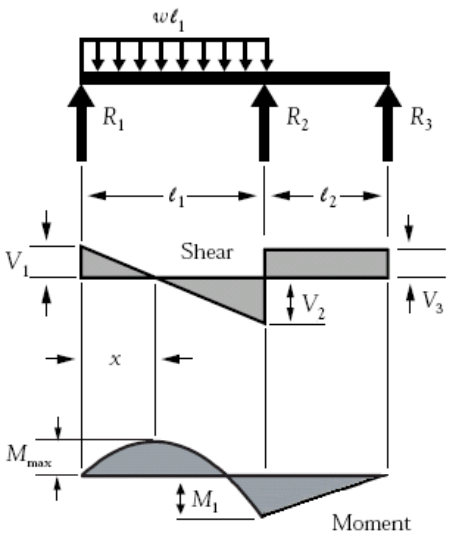
$$V_2 = V_{\max} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots = \frac{5wl}{8}$$

$$M_1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots = \frac{wl^2}{8}$$

$$M_2\left(\text{at } \frac{3l}{8}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots = \frac{9wl^2}{128}$$

$$\Delta_{\max}(\text{at } 0.4215l, \text{approx. from } R_1 \text{ and } R_3) .. = \frac{wl^4}{185EI}$$

Figure A.14. Continuous beam—two equal spans—uniformly distributed load.



$$R_1 = V_1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots = \frac{w\,l_1}{2} - \frac{M_1}{l_1}$$

$$R_2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots = w\,l_1 - R_1 - R_3$$

$$R_3 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots = -\frac{M_1}{l_2}$$

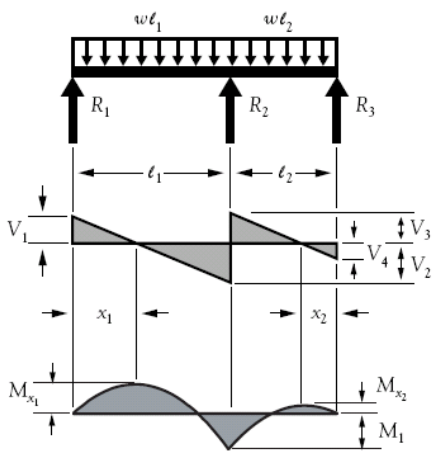
$$V_2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots = w\,l_1 - R_1$$

$$V_3 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots = R_3$$

$$M_1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots = \frac{w\,l_2^3}{8\,(l_1 + l_2)}$$

$$M_{max}\left(\text{when } x = \frac{R_1}{w}\right) \ldots\ldots\ldots\ldots = R_1\,x - \frac{w\,x^2}{2}$$

Figure A.15. Continuous beam—two unequal spans—uniformly distributed load on one span.



$$R_1 = V_1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots = \frac{M_1}{l_1} + \frac{w\,l_1}{2}$$

$$R_2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots = w\,l_1 + w\,l_2 - R_1 - R_3$$

$$R_3 = V_4 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots = \frac{M_1}{l_2} + \frac{w\,l_2}{2}$$

$$V_2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots = w\,l_1 - R_1$$

$$V_3 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots = w\,l_2 - R_3$$

$$M_1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots = \frac{w\,l_2^3 + w\,l_1^3}{8\,(l_1 + l_2)}$$

$$M_{x_1}\left(\text{when } x_1 = \frac{R_1}{w}\right) \ldots\ldots\ldots\ldots = R_1\,x_1 - \frac{w\,x_1^2}{2}$$

$$M_{x_2}\left(\text{when } x_2 = \frac{R_3}{w}\right) \ldots\ldots\ldots\ldots = R_3\,x_2 - \frac{w\,x_2^2}{2}$$

Figure A.16. Continuous beam—two unequal spans—uniformly distributed load.

APPENDIX B

DISPLACEMENT OF SKEWED BRIDGE

This appendix provides summary of the work for addressing the effect of skew on lateral movement of the bridge at the abutment.

B.1. BACKGROUND

A skewed bridge is a bridge with the longitudinal axis at an angle other than 90° with the piers and abutments. The skew angle, θ, is shown in Figure B.1. With skewed integral abutment bridges, the soil passive pressure developed in response to thermal elongation has a component in the transverse direction as illustrated in Figure B.1. Within certain limits of the skew angle, soil friction on the abutment will resist the transverse component of passive pressure. However, if the soil friction is insufficient, then, depending on the transverse stiffness of the abutment, either significant transverse forces or significant transverse movements could be generated.

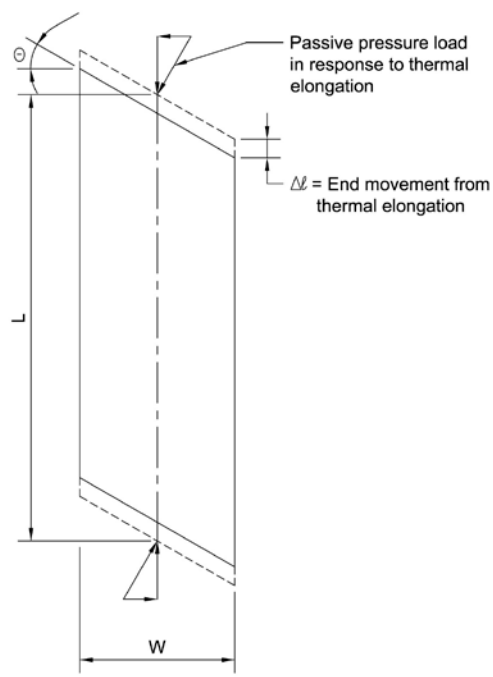


Figure B.1. Components of abutment soil passive pressure response to thermal elongation in skewed integral abutment bridges.

Figure B.2 shows a two-span bridge with a skew angle of 45° (Nicholson et al. 1997). This bridge was constructed in 1969 with semi-integral abutments. The semi-integral construction included an integral end

diaphragm that was designed to move with the superstructure which slides longitudinally, and is guided transversely by, relatively stiff abutments.

Figure B.2. Two-span semi-integral abutment bridge with an overall length of 89 m (259 ft), width of 11.6 m (38 ft), and a skew angle of 45°. (Nicholson et al. 1997)

Figure B.3 shows cracking in the abutment wall near an acute corner of the superstructure, presumably caused by transverse forces related to soil pressures.

Figure B.3. Cracking in the abutment wall near an acute corner of the superstructure for the bridge shown in Figure B.2. (Nicholson et al. 1997)

Figure B.4 shows distress in an asphalt overlay at the skewed end of an approach slab because of the transverse movement (Tabatabai et al. 2005).

Figure B.4. Asphalt overlay distress (west end). (Tabatabai et al. 2005)

Figure B.5 shows a closer view of the barrier wall joint from Figure B.4 at the end of the approach slab. The expansion joint in the barrier wall was made perpendicular to the longitudinal direction and could not accommodate the transverse movement.

Figure B.5. Barrier distress at west abutment. (Tabatabai et al. 2005)

Because of potential problems and uncertainty related to the response of skewed integral abutments, many State DOTs limit the skew angle. A typical limit for maximum skew angle for integral abutment bridges used by many States is 30 degrees. However, maximum skew angle limits in various States range from 0 degrees to no limit

(Chandra 1995). Therefore studies were conducted in the FHWA Jointless Bridge Project (Oesterle and Lotfi 2005) to:

1. Develop a relationship between skew angle and abutment soil friction for limiting skew.
2. Develop a relationship for the magnitude of forces required to restrain transverse movement in integral abutment bridges with large skew angles.
3. Develop a relationship between skew angle and expected transverse movement for a typical integral stub abutment with no special design features to restrain this movement.
4. Compare analytical results with field data for a skewed bridge that was monitored as part of the experimental portion of this project.
5. Perform a sensitivity study to demonstrate the relationship between transverse movement and longitudinal expansion for various skew angles and ratios of bridge length to width.

This work was accomplished by developing equilibrium and compatibility equations for end abutment forces and, for the case of a typical stub abutment, solving these relationships for various skew angles and bridge length-to-width ratios.

B.2. ANALYSES FOR TRANSVERSE RESPONSE TO THERMAL EXPANSION

B.2.1. Skew Angle Limit for Limiting Transverse Effects

Figure B.6 shows the passive soil pressure response, P_p, due to thermal expansion and soil/abutment interface friction, F_{af}, assuming no rotation in the plane of the bridge superstructure. For rotational equilibrium:

$$F_{af}\left(L\cos\theta\right) = P_p\left(L\sin\theta\right) \qquad \textbf{EQ B.1}$$

and from interface behavior:

$$F_{af} = P_p \tan\delta \qquad \textbf{EQ B.2}$$

Where:

$\tan\delta$ = friction coefficient for interface of formed concrete and soil.

Substituting Equation B.1 into Equation B.2:

$$\tan\delta = \frac{\sin\theta}{\cos\theta} = \tan\theta \quad or \qquad \textbf{EQ B.3}$$
$$\delta = \theta$$

Therefore, the bridge superstructure can be held in rotational equilibrium until the skew angle exceeds the angle of interface friction. Integral abutments are typically backfilled with granular material. *NCHRP Report 343* lists a friction angle of 22 degrees to 26 degrees for formed concrete against clean gravel, gravel sand mixtures, and well-graded rock fill (Arsoy et al. 2002). Based on these data, the angle of θ = 20 degrees represents a reasonably conservative skew angle limit below which special considerations for transverse forces or transverse movement are not needed.

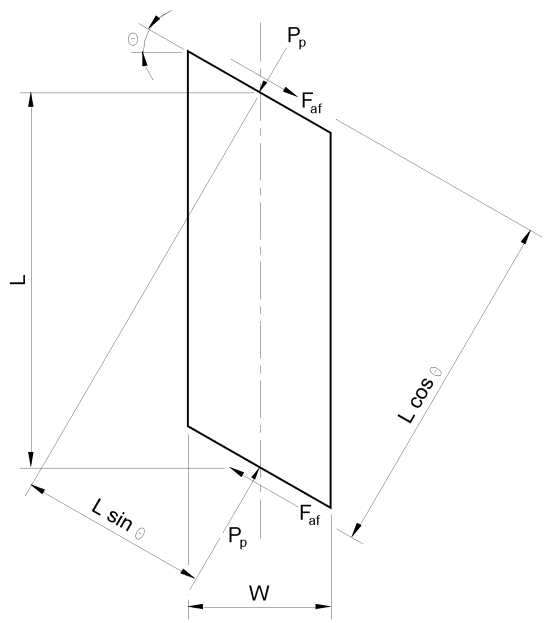


Figure B.6. Soil pressure load, P_p, and soil abutment interface friction, F_{af}.

With larger skew angles, the integral abutment can be either designed to resist the transverse force generated by the soil passive pressure in an attempt to guide the abutment movement to be predominantly longitudinal, or detailed to accommodate the transverse movement.

B.2.2. Forces Required to Resist Transverse Movement

Adding lateral resistance of the abutment, F_a, to wall/soil interface friction, F_{af} ., in Figure B.6, rotational equilibrium is found by:

$$\left(F_a + F_{af}\right)\left(L\cos\theta\right) = P_p\left(L\sin\theta\right)$$ **EQ B.4**

Substituting from Equation B.2 into Equation B.4:

$$F_a = P_p\left(\tan\theta - \tan\delta\right)$$ **EQ B.5**

F_a is the summation of abutment lateral resistance from pile and passive pressure on the substructure surface perpendicular to the abutment.

Figure B.7 shows the relationship between F_a and P_p, assuming the interface friction angle, δ, to be 20º. As shown in Figure B.7, the force required to resist transverse movement is a significant portion of the soil passive pressure, P_p. It should be noted that P_p is not necessarily full passive pressure, but can be determined for the end movement using relationships calculated by Clough and Duncan (Clough and Duncan 1991; Barker et al. 1991) shown in Figure B.6. The end movement to consider in calculating passive pressure is the end movement normal to the abutment, Δl_n.

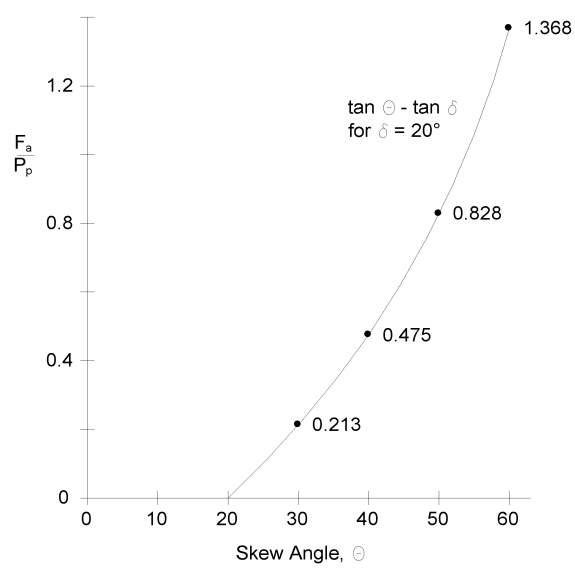


Figure B.7. Relationship between force required for abutment lateral resistance, F_a, and passive pressure response, P_p, to restrain lateral movement.

As illustrated in Figure B.8, this end movement is:

$$\Delta l_n = \Delta l \cos\theta \qquad \textbf{EQ B.6}$$

Where:

Δl = maximum expected end movement for thermal re-expansion from the starting point of full contraction for the full range of effective bridge temperatures as discussed in Section 8.6.2.3.1.

From Figure B.8, it can be seen that Δl_n is reduced with respect to Δl as the skew angle, θ, increases. This relationship helps offset the increase in F_a / P_p with increasing θ. However, F_a will still be a sizeable portion of P_p.

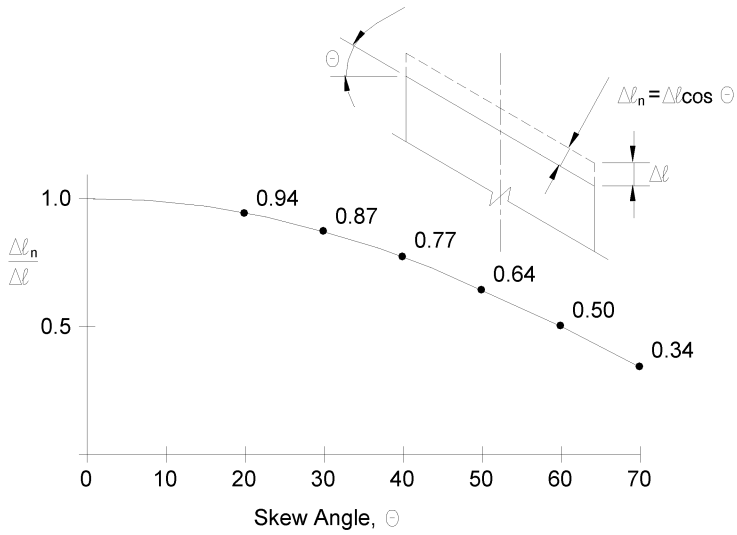


Figure B.8. Relationship between end normal movement, $\Delta\ell_n$, and end thermal expansion, $\Delta\ell$.

For relatively short bridges and/or bridges in locations with small effective temperature ranges, it may be feasible to design the abutment substructure to resist F_a. However, it should be understood that for whatever means used to develop F_a (battered pile and/or lateral passive soil resistance), lateral movements are required to develop the resistance. Therefore, details anticipating some transverse movement should be used. The expected movements are a function of the relative stiffnesses of response for P_p and F_a. It should also be noted that adding battered piles to an integral abutment for lateral loading will also increase the stiffness in the longitudinal direction, which induces more demand on the superstructure and connections between the girders and abutments.

B.3. EXPECTED TRANSVERSE MOVEMENT WITH TYPICAL INTEGRAL ABUTMENT

B.3.1. Method of Analyses

To investigate the relationship between skew angle and expected transverse movement for a typical integral stub abutment, a set of relationships were derived based on equilibrium and compatibility of end abutment forces in the plane of the bridge superstructure. For this analysis, the superstructure is assumed to act as a rigid body with rotation, β, about the center of the deck (for a longitudinally symmetrical bridge). The rotation occurs to accommodate the thermal end movement, Δl . Forces considered in response to this movement include soil pressure on the abutment and wingwalls, wall/soil interface friction on the abutment, and pile forces normal to and in line with the abutment and wingwalls. Details of the forces, stiffness, and equations of compatibility and equilibrium are provided in the report on the analytical work for the *FHWA Jointless Bridge Project* (Oesterle 2005).

A spreadsheet program was used for solving rotational equilibrium in the plane of the deck. For a given end thermal movement, Δl , the equilibrium position can be found using an iterative analysis by progressively increasing the rotation angle, β , until the sum of the in-plane moments is zero.

B.3.2. Results of Analyses for Instrumented Bridge

As part of the experimental program for the *Jointless Bridge Project* (Tabatabai et al. 2005), a heavily skewed bridge in Tennessee was instrumented and monitored for one year. This bridge carries U.S. Interstate 40 (I-40) over Ramp 2B in Knox County, Tennessee. It has a three-span steel plate girder superstructure with an overall length of 415.92 ft and integral abutments. This structure is sharply skewed with a skew angle of 59.09º. The three span lengths are 139.83 ft, 208 ft, and 68.08 ft. The bridge was instrumented to monitor the longitudinal and transverse movements of the east abutment, and obtain an indication of restraint to the longitudinal expansion.

The east abutment was analyzed using the spreadsheet program developed to solve for rotational equilibrium (Oesterle 2005; Oesterle and Lotfi 2005). Based on the experimental data, an end movement of Δl = 0.781 in. was used in the analysis. A measured superstructure rotation angle of β = 0.000224 radians corresponded with the Δl = 0.781 in. Using the spreadsheet to determine rotational equilibrium, an angle of β = 0.000226 radians was calculated. The calculated value indicated very good agreement with the measured data.

B.3.3. Sensitivity Analyses for the Effects of Skew Angle on Transverse Movement and Longitudinal Restraint

To demonstrate the effects of skew angle on expected transverse movement and longitudinal restraint forces, further analyses were carried out using the spreadsheet program. Variables included skew angle and the length-to-width ratio for the bridge. The abutment for the instrumented bridge is a relatively typical type of stub abutment used in Tennessee and was used as the baseline abutment for the analyses (Oesterle 2005; Oesterle and Lotfi 2005).

The instrumented bridge is relatively wide compared to the length. The ratio of length to width (L/W) for this bridge is 3.15. To demonstrate the sensitivity to the bridge L/W ratio, the analyses were repeated for abutments reduced to 2/3W and 1/3W. The length of the wingwalls at each skew angle was kept constant.

Results of these analyses for the ratio of transverse movement to longitudinal movement, $\Delta_{t1}/\Delta l$ are shown in Figure B.9 for a Δl of 1 in. The transverse movement, Δ_{t1}, is the transverse movement of the acute corner of the bridge deck. This is the corner that experiences the greatest transverse movement because of the skew angle.

The results in Figure B.9 demonstrate the increase in the transverse movement with increasing skew angle. The data in Figure B.9 also demonstrates the increase in transverse movement with decreasing L/W ratio. It should be pointed out that the change in L/W was accomplished in the analyses by decreasing the width and keeping the length constant. The length of the wingwall at each skew angle was constant; therefore, the results in Figure B.9 demonstrate the effects of increasing the ratio of the length of the wingwalls to the length of the abutment wall. The data in Figure B.9 shows that increasing the wingwall length relative to the abutment wall length (which includes increasing the number of wingwall piles relative to the number of abutment wall piles) can significantly decrease the transverse movement. However, the wingwalls and abutment must be designed to transmit the wingwall forces into the superstructure.

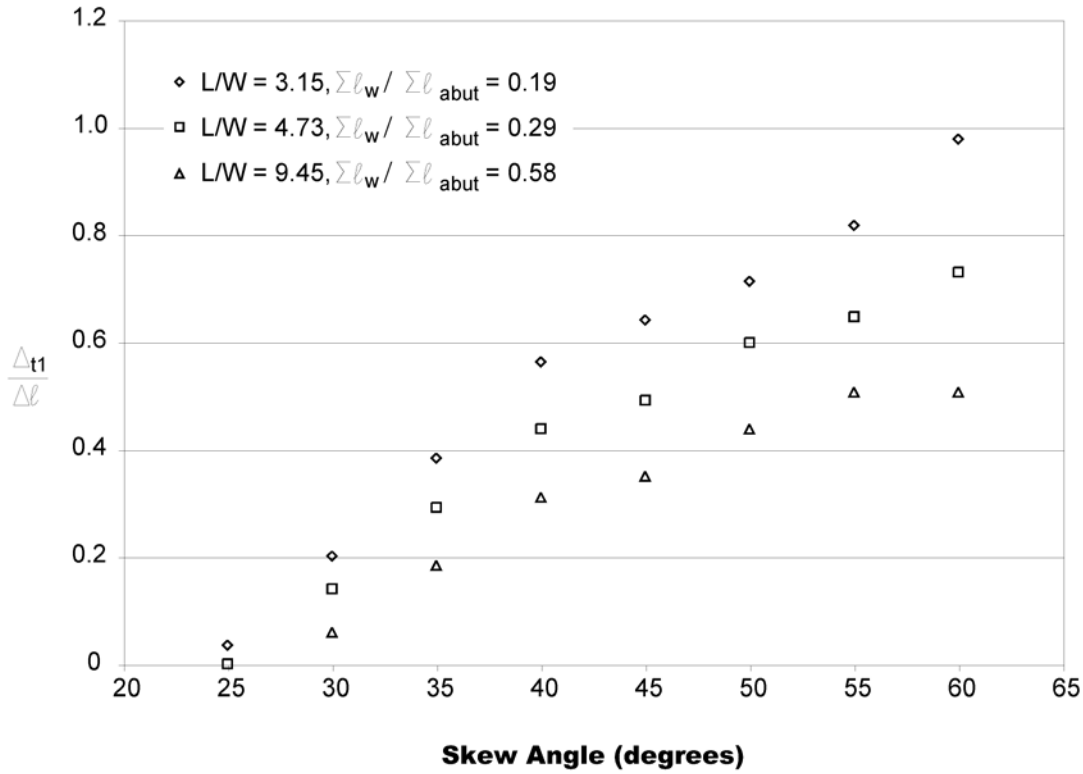


Figure B.9. Relationship between transverse movement at the acute corner, Δ_{t1}, and thermal expansion, $\Delta\ell$, for an expansion of 1 in. with constant length bridge, L = 415.92 ft, and varying L/W.

Figure B.10 shows the resulting total longitudinal restraint force for these analyses and demonstrates the decrease in longitudinal restraint with increasing skew angle. For the full-width bridge with L/W = 3.15, the longitudinal restraint at a skew angle of θ = 60 degrees is approximately 60% of the longitudinal restraint at θ = 25 degrees. For the larger L/W = 9.45, the ratio of longitudinal restraint at θ = 60 degrees is approximately 70 percent of the restraint at θ = 25 degrees. This demonstrates the increase in restraint resulting from the increase in resistance to lateral moment because of the larger ratio of wingwall length to abutment length.

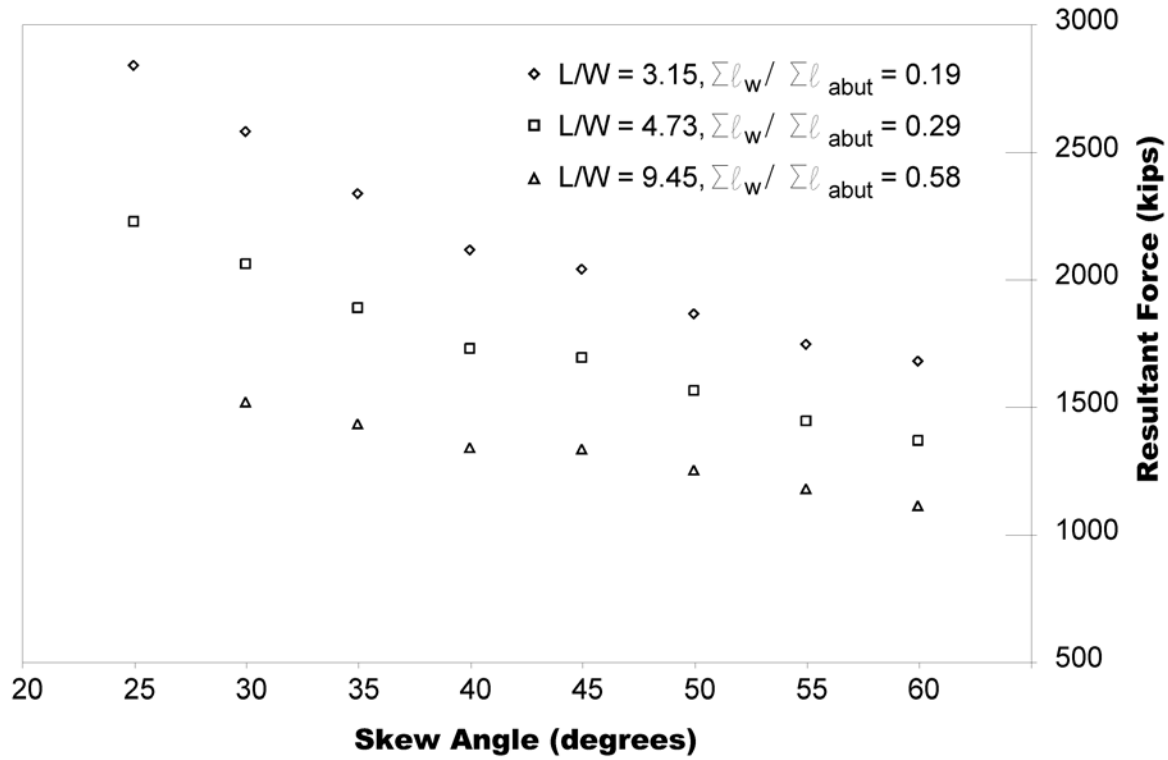


Figure B.10. Relationship between resultant longitudinal restraint force and skew angle for thermal expansion, $\Delta\ell$, of 1 in. with constant length bridge, L = 415.92 ft, and varying L/W.

B.3.4. Design Recommendations

Since the baseline abutment used in these analyses is a relatively typical stub abutment (but also relatively deep, with an abutment height of 13.0 ft and with strong axis pile bending for movement normal to the abutment versus weak axis bending for movement parallel to the abutment), the data in Figure B.9 represents a reasonably large estimate for the transverse movement of skewed abutments. Although there is significant uncertainty for actual soil and pile stiffness, the maximum expected end movement, Δl , discussed in Section 8.6.2.3.1 includes a multiplier to account for uncertainty. Therefore, it is suggested that the data in Figure B.9 can be used by designers to determine an approximate estimate for expected transverse movement in skewed integral abutments resulting from the restraint of longitudinal thermal expansion. In addition, the relationships between longitudinal restraint force and skew angle shown in Figure B.10 can be used to estimate the relative decrease in restraint forces in a skewed bridge. Also, the transverse movements can be used to estimate the transverse forces on the wingwall resulting from passive soil load and pile, and to estimate longitudinal and transverse movement for the abutment pile for biaxial bending

considerations. All of the other components of movement and forces can be determined from Δ_{t1} and Δl using equations presented in the full analytical report (Oesterle 2005).

APPENDIX C

DESIGN OF PILES FOR FATIGUE AND STABILITY

This appendix provides steps that could lead to development of design aid for piles subjected to axial load and lateral movement. The principal steps are explained and are customized for development of design aids for 50 ksi steel H piles.

C.1. ESTIMATION OF MAXIMUM ALLOWABLE STRAIN

Seasonal and daily temperature fluctuations subject steel H piles in jointless bridges to cyclic loading, which can result in fatigue failures of H piles. This is especially important as the magnitude of cyclic strain exceeds elastic limits. The seasonal and daily temperature fluctuations subject H piles, every year, to one large cycle, due to seasonal temperature change, and a number of smaller load cycles, due to daily temperature loading (Dicleli and Albhaisi 2004; Karalar and Dicleli 2010). Figure C.1 shows typical H-pile cyclic strain (Dicleli and Albhaisi 2004).

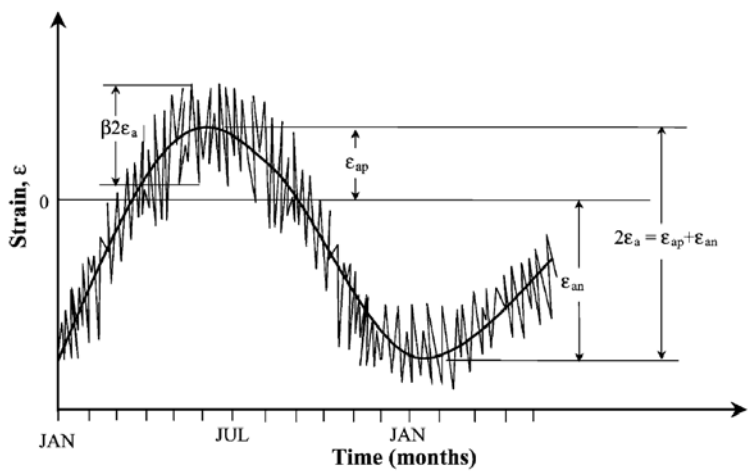


Figure C.1. Pile strain as a function of time. (Dicleli and Albhaisi 2004)

This strain is the maximum longitudinal strain in steel H piles, typically located at the point of fixity below the pile head.

The following is one alternative for predicting the fatigue life of steel elements subjected to variable amplitude cyclic loading. The steps involve concepts of cycle counting and use of damage models for keeping track of accumulated damages due to cycling loading (Gere and Goodno 2012).

1. Obtain the loading history to which the steel element is subjected.

2. Develop an S-N type curve for the material under consideration. In general, in the low cycle regime (yearly seasonal cycle when steel element is subjected to strain exceeding elastic limits), the data should be presented in terms of strain versus number of cycles to failure.

3. Use a cycle counting method, such as the rain flow method (ASTM 1049-85), to convert the variable amplitude loading into equivalent constant amplitude loading.

4. Use a damage model, such as Miner's rule, to determine the time to failure.

Dicleli and Albhaisi (2004) suggest using equation Equation C.1for relating strain amplitude to fatigue life.

$$\varepsilon_a = M\left(2N_f\right)^m \qquad \textbf{EQ C.1}$$

Where:

ε_a = constant strain amplitude

N_f = fatigue life (number of cycles to failure) corresponding to ε_a

M = factor determined from experimental testing (0.0795)

m = exponent determined from experimental testing (-0.448)

Dicleli and Albhaisi (2004) suggest using Miner's rule, as a damage model for steel H piles. Equation C.2 expresses Miner's rule.

$$\sum_{i=1}^{n} \frac{n_i}{N_i} \leq 1 \qquad \textbf{EQ C.2}$$

Where:

n_i = cycles associated with the loading number i ,

N_i = the number of cycles to failure for the same case

Dicleli and Albhaisi (2004) assume that H steel piles are subjected to two different constant amplitude loadings, one corresponding to seasonal temperature changes and another representing daily temperature changes. Therefore, Miner's rule can be expanded as shown by equation C-3.

$$\frac{n_s}{N_{fs}} + \frac{n_l}{N_{fl}} = 1 \qquad \textbf{EQ C-3}$$

In Equation C-3, n_s and n_l are the number of small and large amplitude strain cycles due to temperature changes during the service life of the bridge, respectively, and N_{fs} and N_{fl} are the total number of cycles to failure for the

corresponding small and large amplitude strain cycles, respectively. According to Dicleli and Albhaisi (2004), for 100 years of service life, the number of small amplitude cycles is $n_s = 14800$, and the number of large amplitude cycles is $n_l = 100$. These values are obtained by studying field performance of several jointless bridges and developing types of data shown in Figure C.1.

For large and small amplitude loading, Equation C.1 can be customized as follows (Dicleli and Albhaisi 2004):

$$\varepsilon_{as} = M\left(2N_{fs}\right)^m \qquad \textbf{EQ C.4}$$

$$\varepsilon_{al} = M\left(2N_{fl}\right)^m \qquad \textbf{EQ C.5}$$

To facilitate development of an "allowable" strain to be used in selecting a steel pile capable of meeting the fatigue requirement, the small strain amplitude, ε_{as}, is related to large strain amplitude, ε_{al}, using a proportionality constant, β, resulting in the following relationship:

$$\varepsilon_{as} = \beta\ \varepsilon_{al} \qquad \textbf{EQ C.6}$$

In Equation C.6, β is estimated to be 0.25 (Karalar and Dicleli 2010). By substituting Equation C.6 into Equation C.4 and solving for constant amplitude life to failure, Equation C.4 and C.5 could then be used to determine the following (Dicleli and Albhaisi 2004):

$$N_{fs} = \frac{1}{2}\left(\frac{\beta\,\varepsilon_{al}}{M}\right)^{\frac{1}{m}} \qquad \textbf{EQ C.7}$$

$$N_{fl} = \frac{1}{2}\left(\frac{\varepsilon_{al}}{M}\right)^{\frac{1}{m}} \qquad \textbf{EQ C.8}$$

By substituting Equations C.7 and C.8 into Equation C-3 and solving for ε_{al}, the maximum large amplitude strains that the pile can sustain without fatigue failure can be obtained as follows (Dicleli and Albhaisi 2004):

$$\varepsilon_{al} = \left(\frac{2n_s}{\left(\frac{\beta}{M}\right)^{\frac{1}{m}}} + \frac{2n_l}{\left(\frac{1}{M}\right)^{\frac{1}{m}}} \right) \qquad \textbf{EQ C.9}$$

Substituting the previously stated values for the parameters in Equation C.9, which are: $n_s = 14800$, $n_l = 100$, $\beta = 0.25$, $M = 0.0795$, and $m = -0.448$, ε_{al} is then determined to be 0.002967.

Based on the calculated maximum large strain amplitude, $\varepsilon_{al} = 0.002967$, the maximum cyclic curvature amplitude, Ψ_f at fatigue failure of the pile is expressed as :

$$\psi_f = \frac{2\varepsilon_{al}}{d_p} \qquad \textbf{EQ C.10}$$

Where:

d_p = width of the pile in the direction of the cyclic displacement

Knowing the cross section of the steel pile to be used, complete non-linear moment curvature characteristics of the pile can be developed. From this relationship, the maximum moment that a steel pile can sustain without failure can be estimated using the maximum "allowable" curvature, as obtained from Equation C.10. The maximum moment that a steel pile can sustain can then be used to obtain the maximum lateral displacement that the steel pile can accommodate without fatigue failure. The maximum lateral displacement is obtained through a non-linear pushover analysis as described in the next section.

C.2. PUSHOVER ANALYSIS EXAMPLE

The development of the design aids require conducting static pushover analysis. Static nonlinear pushover analysis using the finite element software SAP2000 can be used to estimate the maximum lateral displacement capacity of steel H piles based on fatigue consideration. Only two sections (HP10x57 and HP12x84) meet the compact ductility requirements for A36 and A50 steels, as described in Section 8.6.2.4.2b. These two cross-sections were used for pushover analysis.

C.2.1. Soil-Pile Interaction Model

C.2.1.1. Piles Driven in Clay

For the purpose of a pushover analysis, the *p-y* curve (for piles driven in clay) can be simplified as a bilinear curve, as shown in (Figure C.2).

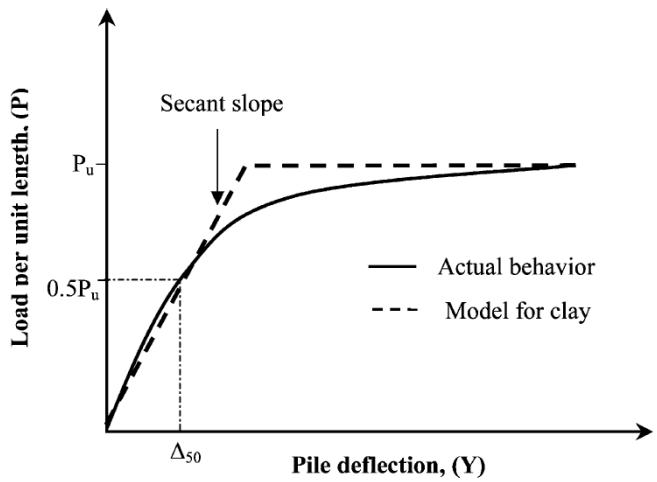


Figure C.2. Actual and modeled *p-y* curves for clay. (Dicleli and Albhaisi 2004)

In this figure, the ultimate response, P_u, is estimated as:

$$P_u = 9C_u d_p \qquad \textbf{EQ C.11}$$

Where:

C_u = un-drained shear strength of the clay

d_p = pile width

The elastic modulus of the clay soil can be estimated as:

$$E_s = \frac{9C_u}{5\varepsilon_{50}} \qquad \textbf{EQ C.12}$$

Where:

ε_{50} = soil strain at 50% of ultimate soil resistance

The following table lists the corresponding values of C_u and ε_{50} for different consistencies of clay soil:

Table C-1. Representative Values of C_u and ε_{50}.

Consistency of Clay	C_u (psi)	ε_{50}
Soft	2.9	0.020
Medium	5.8	0.010
Stiff	17.4	0.005

C.2.2. Description of the Model

To conduct a pushover analysis, the pile was modeled using SAP2000 and divided into small beam elements, each one ft in length. For purpose of the analysis, a 40 ft length of pile was modeled for soft and medium density clays. The models show that this length is sufficient to provide a relative fixed condition in the lower portion of the pile. The pile tip is restrained from movements in all directions.

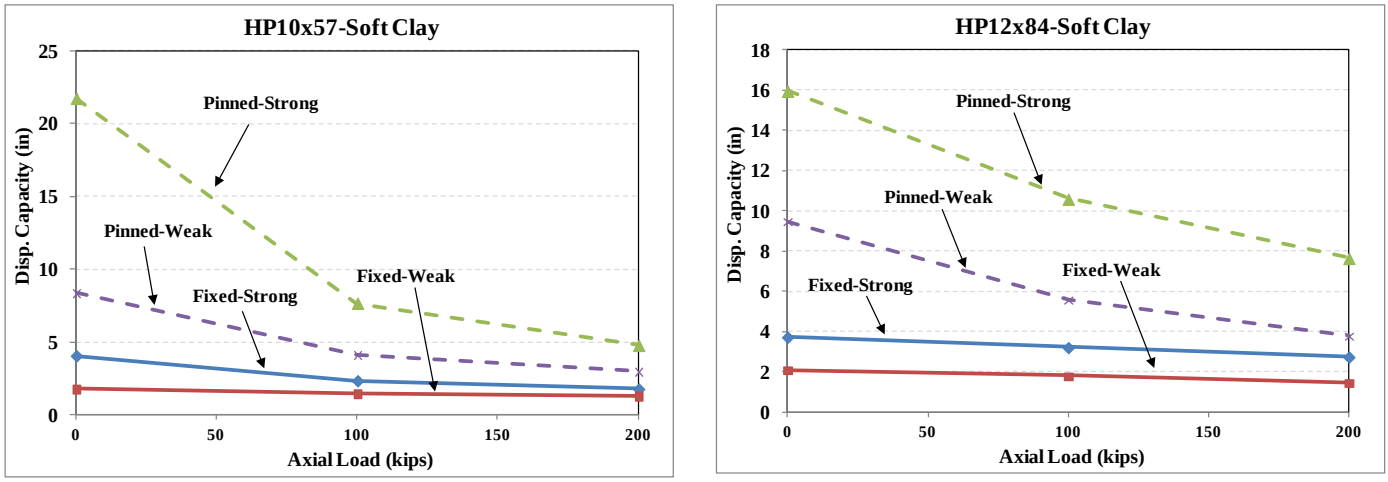
The soil response to lateral deflection was modeled using nonlinear link elements placed every foot. The load deflection properties of the link elements were defined based on the *p-y* curve, described in Chapter 8.

Material properties were assumed to be 36 ksi steel for the pile section. Non-linear beam elements with capability to develop hinges at both ends were used in the pushover analysis. The properties of these hinges are defined based on the orientation and the level of axial load on the pile.

For a given axial load in the pile, soil condition, and steel section, a pushover analysis is then performed to obtain the maximum lateral displacement, capable of meeting the fatigue limit. Based on the assumptions made, the maximum moment that a pile can sustain without fatigue failure was established. This maximum moment is used in pushover analysis to establish the maximum lateral displacement. Results of the pushover analyses for various axial loads are shown in Figure C.3 and Figure C.4.

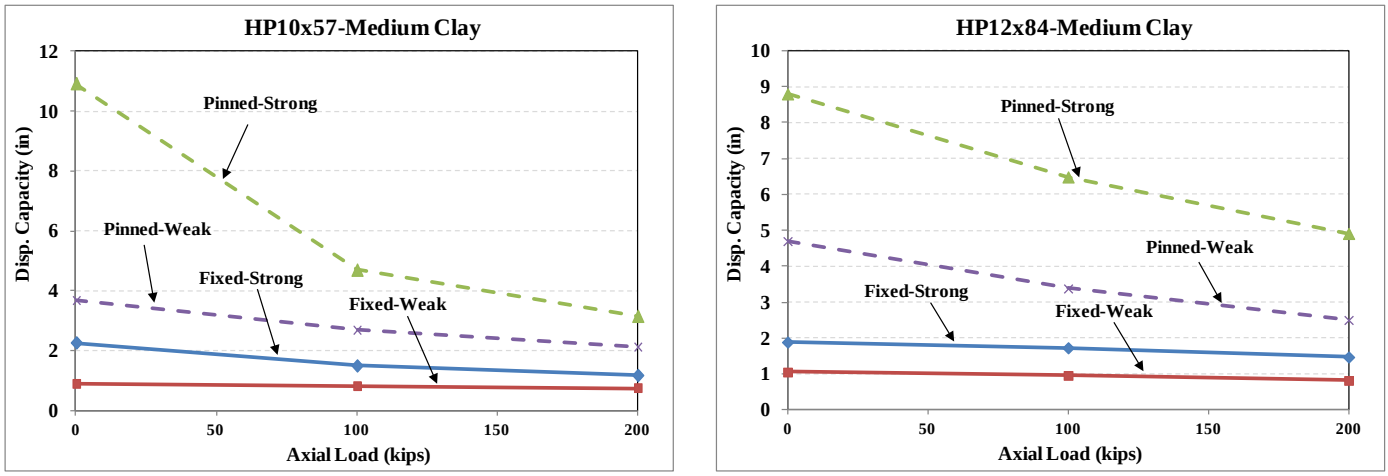
C.2.3. Results of the Analyses

Using the described method and by performing pushover analyses, the maximum displacement that steel H piles with a specified minimum yield strength of 50 ksi can accommodate has been estimated and is shown in the following figures.



(a) **(b)**

Figure C.3. Lateral displacement capacity of compact HP sections in soft clay (c = 2.9ksi) (a) HP10x57 (b) HP12x84.



(a) **(b)**

Figure C.4. Lateral displacement capacity of compact HP sections in medium clay (c = 5.8 ksi) (a) HP10x57 (b) HP12x84.

These figures can be used to determine maximum lateral displacement that a pile can sustain, based on fatigue considerations.

An interesting aspect of the data shown in these figures is that piles oriented with bending about the strong axis provide larger displacement (up to four times). Many DOTs orient the steel piles about weak axis of bending, based on the logic that it could provide larger lateral displacement. Results shown in the previous figures contradict this belief.

APPENDIX D

RESTRAINT MOMENTS

This appendix provides methods of estimating the restraining moment developed in prestressed girders when girders are made continuous over supports, as well as methods to mitigate the problem.

D.1. BACKGROUND

In simple-span non-composite bridges, time-dependent deformations result in little or no change in the distribution of forces and moments within the structure. However, continuous multiple-span composite bridges are statically indeterminate. As a result, inelastic deformations that occur following construction will generally induce statically indeterminate forces and restraining moments in the girders.

Sources of inelastic deformation include concrete creep and shrinkage, and temperature gradients. For example, a common type of jointless bridge construction consists of precast, prestressed girders connected with a continuous cast-in-place deck slab as illustrated in Figure D.1. The girders are simply supported for dead load but may be considered continuous for live load. Continuity is established with deck steel as negative moment reinforcement over the piers. Commonly, a positive moment connection is also provided in the diaphragms.

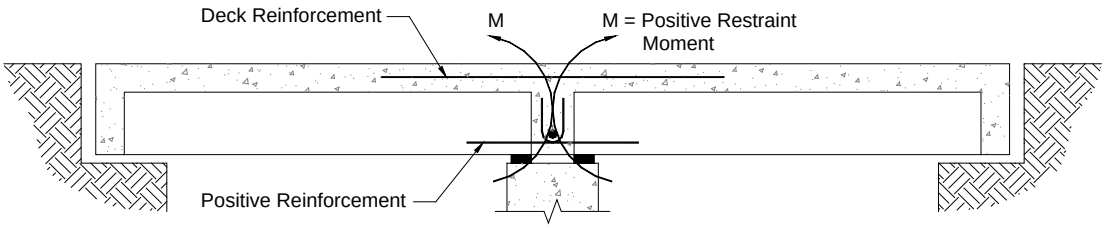


Figure D.1. A typical precast prestressed bridge simply supported for dead load and made continuous for live load.

It has long been recognized that positive secondary moments develop in the connection at piers of continuous prestressed concrete bridges when the deck is cast at a relatively young girder age (Freyermuth 1969). Creep of the girder concrete under the net effects of prestressing and self-weight will tend to produce additional upward camber with time. The piers prevent this upward movement. When girders are made continuous at a relatively young age, it is possible that positive moments will develop at the supports over time as shown in Figure D.2.

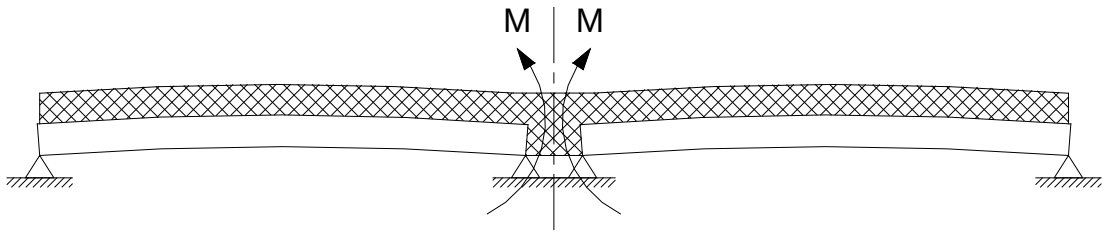


Figure D.2. Restraint against upward movement, positive secondary moment.

Conversely, differential shrinkage, with the newer deck slab concrete shrinking more than the girder concrete, causes the continuous structure to bow download. Therefore, differential shrinkage has a tendency to reduce the positive moment due to creep, or result in negative secondary moments at the supports.

In addition to creep and shrinkage of concrete, temperature gradients can play a major role if the girders are made continuous. Solar heating of the top deck will tend to produce upward camber adding to the positive restraint moment caused by creep. Large restraining positive moment can cause cracking in the bottom flange near the pier locations. Heat of hydration in the cast-in-place deck concrete can have a mitigating effect on the development of positive restraint moment. The cast-in-place deck may be heated to a temperature that is higher than the supporting girder temperature by heat of hydration during the initial hydration when the concrete is still plastic. Contraction of the deck concrete with subsequent cooling after the concrete has hardened results in a downward deflection, thereby reducing the positive restraint moment caused by creep and solar heating.

NCHRP and FHWA funded an experimental and analytical research program on the behavior of continuous and jointless integral abutment prestressed concrete bridges with cast-in-place deck slab (Oesterle et al. 1989; Oesterle et al. 2004a, Oesterle et al. 2004b). Results of analytical studies (Oesterle et al. 2004a) showed that the age of the girder when the deck was cast was the most significant factor in determining whether positive or negative restraint moments occurred at the interior transverse joints over the piers due to the interaction of creep and shrinkage. Results of analytical and experimental research (Oesterle et al. 1989; Oesterle et al. 2004a; Oesterle et al. 2004b) indicated that the live load continuity of the bridge may be reduced significantly with long-term and time dependent loading effects and with thermal effects.

In the experimental part of the jointless bridge research (Oesterle et al. 2004a, Oesterle et al. 2004b) testing of materials, bridge components, and a full scale girder indicated that:

1. Expected shrinkage of the deck concrete did not occur in the concrete in the outdoor environment of Skokie, Illinois. Thus, the effects of deck shrinkage to mitigate the effects of girder creep did not occur.

2. Heat of hydration effects in the cast-in-place deck concrete can have a mitigating effect on the development of positive restraint moment.

3. Daily temperature effects of heating and cooling of the deck with respect to the girder have a significant effect on restraint moments. Solar heating of the deck causes positive restraint moments of the same order of magnitude as the moments due to girder creep and are additive to the moments caused by creep.

4. Tests on a full scale girder that was monitored and loaded periodically with simulated live load on sunny days and on cloudy days during different seasons over an 18 month time frame, demonstrated that positive restraint moment and the resulting cracking at the transverse connection significantly reduced continuity for live load. Using change in beam reactions under application of live load to assess continuity, the lowest measured percentage of full live load continuity was 48% measured on a cloudy day in summer.

5. Continuity induces restraint moments, and effective continuity requires assessment considering all loads. Effective continuity in the test girder was assessed using the distribution of total reactions supporting the test girder that included effects of dead load, live-load, and restraint moments. Effective continuity is defined as 100% if the distribution of total reactions corresponds to the combination of simply supported dead load reactions plus fully continuous live load reactions. Effective continuity is 0% if the distribution of total reactions corresponds to the combination of simply supported dead load reactions and simply supported live load reactions. The measured effective continuity in two of the live load tests in the jointless bridge study was in fact negative (i.e., less than 0%). That is, the total midspan positive moment in the tested “continuous” girder was slightly higher than the anticipated positive moment if the girder was a simply supported girder for both dead and live load.

6. The positive moment due to combined creep and temperature effects in the test girder resulted in stresses in the positive moment reinforcement in the connection over the pier that reached or exceeded yield stress.

Results of this research indicated that use of a positive moment connection in the diaphragms is not beneficial in determining the net resultant midspan service level stresses under dead, live, and restraint loads. Without a positive moment connection at the supports, effects that would tend to produce a positive restraint moment (creep in the prestressed girders and solar heating of the deck) will likely cause a crack to form at the bottom of the diaphragm concrete between the ends of the girders. With application of live load that would tend to produce a negative moment at the support, the crack at the bottom of the diaphragm concrete has to close before full negative moment develops. The net effect is the loss of some live load continuity, which, depending on the parameters, can range from 0 to 100% of live load continuity. If effects that would tend to produce a positive restraint moment are large enough, the crack at the bottom of the diaphragm can remain open under live load and the girder acts as if it is simply supported.

If a positive moment connection is provided, a crack will still likely form at the bottom of the diaphragm concrete from effects that tend to cause positive restraint moment. The positive moment connection will decrease the crack width, but a positive restraint moment will develop. The positive restraint moment superimposed on live load negative moment will negate, at least in-part, the beneficial effects of the negative moment continuity connection over the piers (for service load stresses). Studies (Oesterle et al. 1989, 2004a, 2004b; Mirmiran et al. 2001) have shown that the effect of the crack at the bottom of the diaphragm that would form without the positive moment connection is essentially equivalent to superposition of a positive restraint moment that would form if a positive moment connection is provided (assuming the amount of positive moment reinforced provided is not excessive).

If effects that tend to cause negative restraint moments in the connection over the supports predominate, positive moment reinforcement is not needed. Therefore, these studies indicated that there is no net benefit, in terms of service level stresses in the prestressed girder, by providing positive moment reinforcement in the transverse connections. It is understood, however, that there may be benefit in terms of structural integrity for providing the positive moment reinforcement.

Recently, *NCHRP Project 12-53* was completed and results are included in *NCHRP Report 519* (Miller et al. 2004). This project was carried out to further examine the behavior of simple-span precast/prestressed girders made continuous by connections at the transverse joints over the piers. The focus was on the effectiveness of the positive moment connection and on design criteria for this connection. Results of analytical studies (Mirmiran et al. 2001) were similar to those reported in the previous NCHRP study (Oesterle et al. 1989). That is, if positive restraint

moments develop, these restraint moments must be added to the moments caused by dead and live load, and that the net positive moment at the midspan is essentially independent of the amount of positive moment reinforcement provided in the transverse connection (assuming the amount of positive moment reinforcement provided is not excessive). In addition, analytical studies indicated that cracking in the transverse joint decreases live load continuity.

NCHRP Project 12-53 also included experimental studies. Live load testing indicated that, contrary to analyses results, the continuity with application of live load was near 100% unless the positive moment crack at the connection became very large. The full scale testing result in the *NCHRP 12-53* study, with essentially no live-load continuity lost due to positive moment cracking, differed from the analytical results in the NCHRP studies (Oesterle et al. 1989; Mirmiran et al. 2001) and the result of full-scale testing in the jointless bridge study (Oesterle et al. 2004a; Oesterle et al. 2004b). However, live load continuity in the *NCHRP 12-53* study was assessed using change in reactions with application of live load. It is not clear how restraint moment present in the test specimen connection was considered.

Also, a reason provided in the *NCHRP Report 519* for the difference between the analytical studies and the experimental studies was that the observed positive moment cracks did not extend into the top flange until the crack was very large whereas, in the analytical model, the crack extents into the top flange as soon as it forms. In the *NCHRP 12-53* experimental beams however, effects of concrete creep were simulated by applying post-tensioning near the bottom flanges after the diaphragm concrete was cast. Post-tensioning rods were dead-headed at the ends of the girders on each side of the diaphragm and used to apply a relatively concentrated load near the bottom flanges at the end of the girders. The additional compressive strain due to the post-tensioning was intended to simulate the creep strain in the girders due to the pre-tensioned prestress and produce simulated positive moment cracks in the bottom of the diaphragm concrete. Applying the post-tensioning forces concentrated near the bottom at the ends of the girders, however, may have distorted the plane of the ends of the girders so that the change in crack width over girder depth did not simulate an expected positive moment crack in an actual bridge. Experimental tests in the jointless bridge study (Oesterle et al. 2004a; Oesterle et al. 2004b) were carried out with full scale girders with positive moment cracks in the diaphragm that were primarily the result of actual long-term creep in the girders due to the original pre-tensioned prestress combined with temperature gradient caused by actual solar heating.

Several results from the *NCHRP 12-53* full-scale tests were similar to those observed in the jointless bridge study including:

1. The shrinkage strains in the deck concrete were significantly less than expected,
2. The effects of heat of hydration in the deck concrete were significant, and
3. Daily thermal effects were significant.

Based on the analyses and testing, recommendations for the positive moment connection in *NCHRP Report 519* included:

1. The positive moment connection should be provided and designed for the calculated moment due to dead, live, and restraint moment. At least minimum reinforcement should be provided for a moment equal to 0.6 M_{cr} where M_{cr} is the cracking moment of the connection. Also, the design moment should not exceed 1.2 M_{cr} because providing additional reinforcement is not effective. If the design moment exceeds 1.2 M_{cr}, design parameters should be changed. The easiest change to reduce the positive moment is to specify a minimum age of the girder at the time of making the continuity connection.
2. If the contract documents specify that the girders are a minimum age of 90 days when continuity is established, the restraint moment does not have to be calculated. This is based on the observation from surveys and analytical work that, if the girders are more than 90 days old when continuity is formed, it is unlikely that time-dependent positive restraint moments from concrete creep and shrinkage will form.
3. The transverse connection can be considered fully effective if, "… the calculated stress at the bottom of the continuity diaphragm for the combination of super imposed permanent loads, settlement, creep, shrinkage, 50% live load and temperature gradient, if applicable, is compressive."

Results presented in *NCHRP Report 519* were used to provide extensive and comprehensive revisions and additions to the Fourth edition of the *LRFD Specifications* (2007) Article 5.14.1.4 for Bridges Composed of Simple Span Precast Girders Made Continuous. Based on Article 5.14.1.4.1, the connections between girders should be designed for all effects that cause moments at the connections, including restraint moments from time dependent effects. Note that although restraint moment due to thermal gradient is not specifically mentioned in Article

5.14.1.4.1, it should be included. However, Article 5.14.1.4 includes the following two exceptions regarding the need to design for the restraint moments:

1. Per Article 5.14.1.4.1, multispan bridges composed of precast girders with continuity diaphragms at interior supports that are designed as a series of simple spans are not required to satisfy Article 5.14.1.4.
2. Per Article 5.1.14.4.4, if contract documents require a minimum girder age of at least 90 days when continuity is established:
    a. Positive restraint moments cause by girder creep and shrinkage and deck slab shrinkage may be taken as zero,
    b. Computation of restraint moments shall not be required, and
    c. A positive moment connection shall be provided as specified in Article 5.1.14.4.9.

D.2. DESIGN RECOMMENDATIONS

This section provides various alternatives for handling the positive moment developed in continuous prestress girders.

D.2.1. Restraint Moments in Prestressed Concrete Girders

In general, it is recommended that *LRFD Specifications* Article 5.14.1.4 should be followed in the design of jointless bridges constructed with precast prestressed girders made continuous for live load. However, the following further considerations should be taken into account:

Thermal Effects. Calculated thermal gradient stress caused by the combined internal restraint and secondary continuity moments can be very high, particularly when combined with other secondary effects (Oesterle et al. 2004a, 2004b). *NCHRP Report 519* states that daily thermal effects were significant and mentions that they should be considered in design. However, results of analyses and example calculations included in the report to demonstrate that restraint moment is near zero if the girder age is at least 90 days when continuity is established did not include the effects of thermal gradient. Also, while the commentary to *LRFD Specifications* Article 5.14.1.4.2 mentions temperature variation as a cause of restraint moments, Article 5.14.1.4 does not specifically address design considerations for thermal effects. It is commonly considered that thermal effects are self-limiting

for strength limit states and can generally be disregarded. However, prestressed girders also have to be designed for service level and thermal stresses in continuous prestressed concrete bridges.

Differential Shrinkage Effects. Results of the FHWA jointless bridge project indicated that expected shrinkage based on theoretical shrinkage models and on laboratory shrinkage tests did not occur in the outside environment. *NCHRP Report 519* (Miller et al. 2004) included a similar observation; however, analyses and example calculations included in the *NCHRP Report 519* to demonstrate that restraint moment is near zero if the girder age is at least 90 days when continuity is established, did include the effects of differential shrinkage determined from a theoretical shrinkage model. Results of the analyses presented in the report show that early negative moment due to differential shrinkage between the deck and the girder essentially offset the longer-term positive moment that developed due to creep in the prestressed girder.

Combined Creep, Shrinkage, and Thermal Effects. The effects of creep in the prestressed girders and solar heating of the deck are additive with respect to inducing positive moment at the connection over the supports. When creep and solar heating are combined with an absence of differential shrinkage, it is not clear, even in bridges constructed with 90 day old girders, that positive moments will not be significant.

Potential Negative Moments. Limiting construction to use of girders with a minimum age of 90 days will increase the potential that factors that induce negative restraint moments over the supports may predominate. Increasing the potential for negative moment increases the risk of cracking in the deck over the support regions. Deck cracking over the support regions may have a more detrimental effect on long-term durability of a bridge than positive moment cracking in the diaphragm.

Uncertainties in Determining Restraint Moments. In addition to concrete creep, shrinkage, and solar heating of the deck, a number of other effects can contribute significantly to restraint moments. These include differential settlement of supports; heat of hydration of the deck concrete during construction; variation of the coefficient of thermal expansion between the girder and the deck; and seasonal moisture changes in the concrete, causing shrinkage reversals. In addition, in jointless bridges with integral abutments, additional forces may be imparted on the positive moment connection by the restraint of the abutment to longitudinal temperature movements. All of these factors contribute to restraint forces within a continuous jointless bridge structure. In

some instances, these factors are additive, while in others, they oppose one another. The magnitudes of these effects to be considered in design and the critical combinations are uncertain. Although there are methods available to estimate restraint moments due to all of these effects, the moments that actually occur may be significantly different than the estimated values.

Effects of Excessive Positive Moment Reinforcement. In spite of all the uncertainties regarding magnitudes and combinations of restraint moments, there have not been many cases of distress related to these secondary stresses. In general, concrete cracking and reinforcement yielding will diminish the stresses caused by the secondary effects. However, an overly strong connection combined with the effects of creep and thermal gradient may result in excessive positive restraint moment (ENR 1994; AL-DOT 1994, Telang and Mehrabi 2003). A strong positive moment connection increases the positive moment along the span and in some cases may result in cracking in the beams. Figure D.3 shows an example bridge (Telang and Mehrabi 2003) with significant flexural cracking of this type. The flexural crack occurred at the end of the embedment of the positive moment connection bars near the ends of the prestressed girders with a large quantity of positive moment reinforcement. On the other hand, Figure D.4 shows the end region of another girder in the same example bridge where cracking and spalling occurred within the diaphragm. This diaphragm cracking and spalling was associated with positive moment connection bars bent out of place during erection (because of constructability issues) for several girders in the bridge such that the connection bars became ineffective. However, no flexural cracking occurred within the span of these girders.

Figure D.3. Cracks near girder supports.

Figure D.4. Crack and spall at diaphragm over pier support.

Because of the uncertainty associated with calculations of positive continuity moments resulting from the variability of the creep and shrinkage effects, temperature gradient, differential coefficient of expansion effects, locked-in heat of hydration effects, settlement, and cracking, calculations to determine restraint moments are complex and probably unreliable. Therefore, in order to eliminate the need to attempt to calculate restraint moments and to simplify the design, the following recommendations for positive moment connections were developed based on the work in the NCHRP projects (Oesterle et at. 1989; Mirmiran et al. 2001; Miller et al. 2004), the FHWA jointless bridge project (Oesterle et al. 2004a, 2004b) and on the Fourth Edition of the *LRFD Specifications* (AASHTO 2007):

Option 1. Positive moment connection reinforcement at the piers should not be provided. This approach prevents the development of significant positive restraint moments in the pier diaphragms (and eliminates constructability issues with the overlapping reinforcement). The girders should be analyzed as simply supported for dead plus live loads at service levels. This is allowed by *LRFD Specifications* Article 5.14.1.4.1; eliminates the requirement to calculate restraint moments (without the need to age girders prior to construction); and, as stated in the commentary of the *LRFD Specifications*, has been used successfully by several state DOTs.

Option 2. If positive moment connections are used to improve structural integrity and to provide some crack control, as recommended in the commentary of *LRFD Specifications* Article 5.14.1.4.1, it is suggested that the positive moment capacity, ϕM_n, be limited to the minimum moment of 0.6 M_{cr} recommended in the *LRFD Specifications*. Note that M_{cr} should be determined using the properties of the diaphragm concrete. If additional reinforcement is used to increase crack control, the upper limit recommended by the *LRFD Specifications* of $\phi M_n = 1.2\ M_{cr}$ should not be exceeded. To eliminate the need for calculation of restraint moments, the girders should be analyzed as simply supported for dead plus live loads at service levels as allowed by *LRFD Specifications* Article 5.14.1.4.1. However, positive restraint moments are likely to occur. In spite of this, additional stresses in the girders due to positive restraint moment can be minimized by limiting the capacity of the connection ϕM_n so that the connection acts as a fuse to yield prior to development of detrimental stresses. Therefore, the girder service load stresses should be checked along the length of the girder under simple supported dead and live loads plus ϕM_n of the positive moment connections superimposed on the spans, such that the allowable tensile stress in the bottom of the beam of $0.19\sqrt{f'_c}$, ksi, ($6\sqrt{f'_c}$, psi) is not exceeded. Particular attention should be paid to the region of termination of the positive moment steel if mild reinforcement is used for the connection.

For both options 1 and 2, the girder/diaphragm interface should consider details to allow relative movement between the bottom of the girder and diaphragm concrete for girders partially embedded in the diaphragm concrete. For the exterior surface of fascia girders, providing a sealed crack control joint at the beam-to-diaphragm interface should be considered.

Negative moment reinforcement should be provided over the supports, and diaphragm concrete should be provided between the ends of the girder bottom flanges. Negative restraint moments may develop, for example when the deck and diaphragms are cast when the concrete girders are older. However, parametric studies carried out in the FHWA jointless bridge project indicate that, with high restraint moments, cracking occurs in the deck and sufficient moment redistribution occurs to prevent the deck reinforcement from becoming overstressed. Therefore, restraint moments do not have to be calculated. Negative moment reinforcement in the deck can be designed for applied dead and live load moments calculated based on uncracked section properties. It can be assumed tht the girder is simply supported for dead load and fully continuous for live and superimposed dead loads, because of the parapets, barrier walls, wear surface, etc.. Since the deck in the negative moment region is considered reinforced concrete, the negative moment connection is only designed for strength limit states.

D.2.2. Restraint Moments in Composite Steel Bridge Girders

Temperature gradients and differential coefficients of thermal expansion in continuous composite steel beams produce both positive and negative restraint moments, while the shrinkage of deck concrete and the heat of hydration locked-in strains produce negative restraint moments. Deck slab cracking partially relieves negative restraint moments.

The parametric studies in the FHWA jointless bridge project indicate that stresses in both the concrete deck slab and steel beams are not excessive under the combination of dead and live load forces combined with positive restraint moments. Therefore, explicit calculations considering positive restraint moments are not necessary.

The analyses for effects of negative restraint moments in composite steel beams indicated that, in general, if negative moments are high, deck cracking results in redistribution and calculated stresses are not excessive. However, analyses also included the effects of a negative temperature gradient, which produces negative restraint moments, combined with dead and live load and restraint of longitudinal expansion provided by passive pressure in backfill and the lateral force in the piles of integral abutments. These analyses indicated that, under certain circumstances, calculated compressive stresses in the bottom flange of the steel beams near interior supports may be excessive, even after allowance for redistribution of the stresses because of deck cracking. Based on the parametric studies, the combination of loads described above may become critical for larger beam spacing. Calculations indicate that, for stringer spacing equal to or greater than 7 ft for A36 beams, and 9 ft for A572, Grade 50 beams, an

explicit check of the effects of the combined load effects of dead and live loads, negative temperature gradient, and restraint of longitudinal expansion may be required to check for lateral torsional buckling of the bottom flange near interior supports.

APPENDIX E

DESIGN STEPS FOR SEAMLESS BRIDGE SYSTEM DEVELOPED BY *SHRP 2 R19A*

Expansion joints are one of the main causes for high maintenance costs in bridges. A new seamless bridge system was envisioned within the *SHRP 2 R19A* project that should result in bridges with long service lives by eliminating the joints over the entire length of the bridge, approach slab, and a segment of the roadway (Ala and Azizinamini 2013a; Ala and Azizinamini 2013b). The system is similar to a system developed in Australia for use with continuously reinforced concrete pavements (CRCP) (Bridge et al. 2000). Proposed modifications have been made to the Australian system to adapt it to United States practice in which most pavements are either jointed plain concrete or flexible pavement (Ala 2011). While pavement within a particular roadway may be jointed or flexible, the segment of roadway containing the bridge and the proposed seamless transition is similar in nature to CRCP. Therefore, transition details would be similar to those used when transitioning from CRCP to jointed or flexible pavements.

The key factor is establishing an effective longitudinal force transfer mechanism from the transition slab to the base soil that minimizes the length of the transition. The goal is achieving limited end movements, a predictable and controlled crack pattern, and controlled axial forces in the system.

The system that was developed to meet these needs is shown in Figure E.1. The transition slab is connected to a secondary slab that is embedded below. The two slabs are connected by a series of small piles. The secondary slab increases the stiffness of the transition region resulting in the desired short transition length. A similar system without the transition slab may lose its effectiveness after multiple cycles due to compaction of the soil surrounding the small piles.

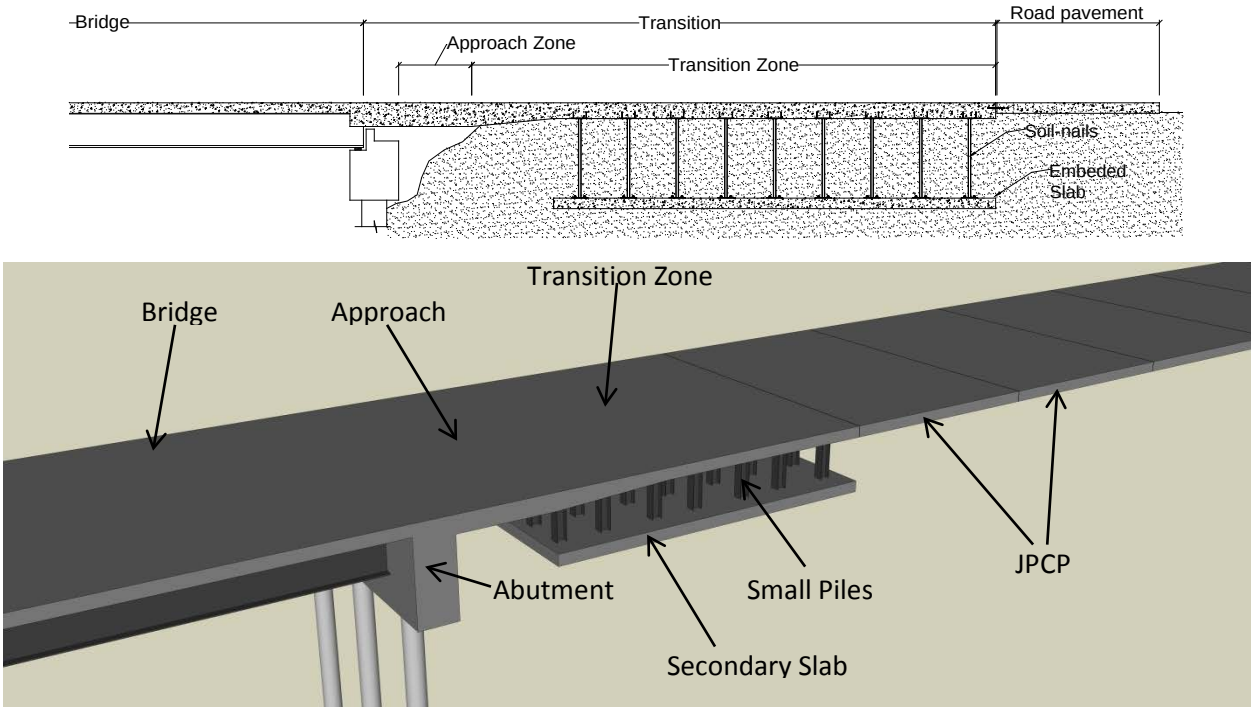


Figure E.1. Schematic and rendering of the recommended practice for bridge/roadway interface.

A special reinforcement reduction detail is used over the length of the transition zone to achieve a controlled crack pattern when the bridge system is in tension. The system behavior in tension (temperature reduction/bridge contraction) is an important factor since the crack pattern plays a major role in design life and maintenance costs. Figure E.2 shows a transition in which the reinforcement detail helps to maintain the desirable crack pattern (Jung et al. 2007). The reinforcement is reduced over the length of the transition region as the force is reduced.

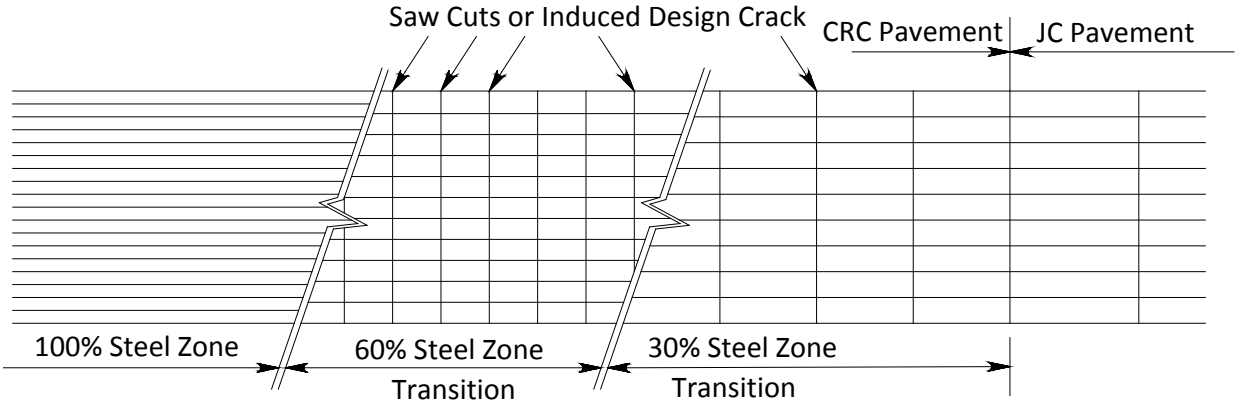
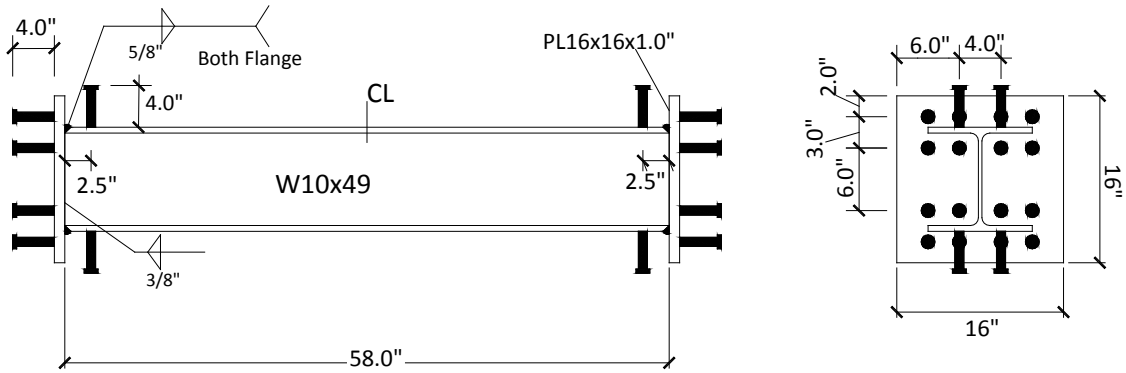


Figure E.2. Gradual transition continuously reinforced to jointed pavement. (Jung et al. 2007)

Analysis, design, and construction of a seamless bridge and approach slab system are similar to other bridge structures. However, there are some new components involved in the system that are not typically seen in other bridge systems. The new components include transition slab, secondary slab, small piles, and the connection of the small piles to the transition concrete slabs. Figure E.3 shows an example of the small piles used to connect the transition slab to the secondary slab during the developmental phase of the concept.



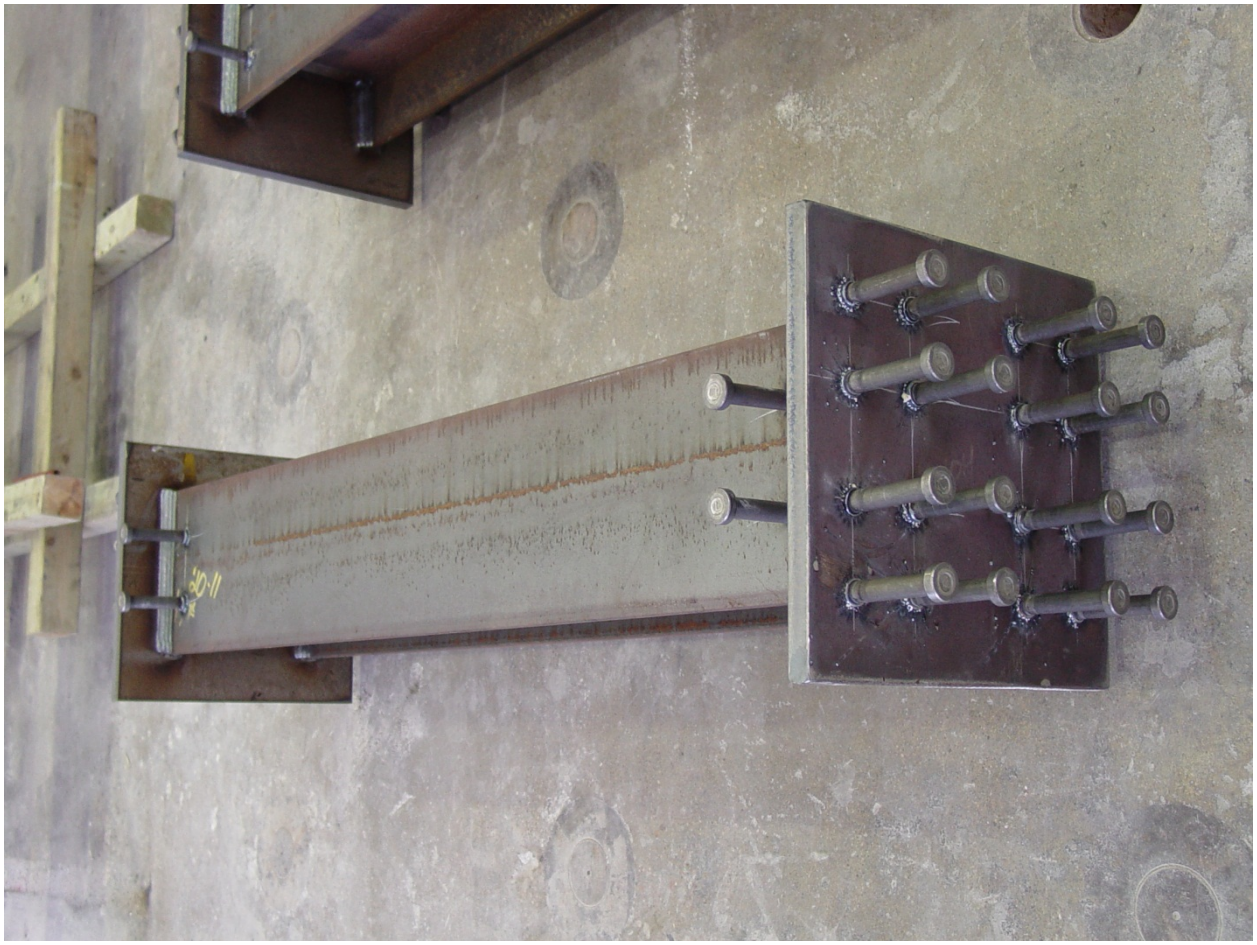


Figure E.3. Small piles to be used in the test to connect the upper and lower slabs.

The initial system design is an iterative process in which the length of the transition and secondary slabs; the shape, size, and spacing of small piles; and the embedment depth of the secondary slab are determined via structural analyses of various system configurations. Demand in all components is determined during the initial design phase.

The various parts of the system may then be designed according to the applicable *AASHTO LRFD Bridge Design Specifications* (*LRFD Specifications)*. The reduced cracked stiffness of the system in tension may be neglected in the initial design.

Relevant pavement design loadings are the longitudinal strains (thermal effects, creep, and shrinkage) and the out-of-plane effects (due to traffic wheel loads, settlement of approach embankments, and rotational effects transferred from the bridge deck).

E.1. STRUCTURAL ANALYSIS

Until further research is completed to develop a simplified analysis approach, the seamless bridge should be analyzed as a holistic system with all components incorporated in the analysis. To account for the effect of temperature changes in design of the transition region of the seamless bridge system, only the effect of uniform temperature change needs to be considered. The calculation of uniform temperature change should be in accordance with *LRFD Specifications* Article 3.12.2. The interaction between the soil and the small piles can be modeled in the structural analysis using springs. The spring stiffness around the small piles highly depends on the relative density of the compacted soil material (geomaterial) surrounding the small piles and the confinement pressure. Since the soil material is manually compacted, the relative density of the compacted soil needs to be measured during the compaction process and this compaction should be related to the soil stiffness. The connection of the small piles to the slabs can be assumed rigid for analysis purposes.

The structural analysis should take into account the effects of longitudinal stiffness reduction due to cracking of the transition slab in tension (temperature reduction/bridge contraction). Iterative structural analyses of the seamless bridge and roadway system in conjunction with cracked section analyses are required. For the first iteration, the tensile forces in the structure are assumed equal to the compressive forces due to thermal expansion. Cracked section analyses are carried out for various segments of the transition slab, and the axial stiffness of the slab segments are modified. The structure is analyzed with the modified in-plane stiffness to determine the in-plane tensile axial forces in the system. This process is repeated until convergence of the axial forces is achieved.

E.2. DESIGN OF THE SYSTEM COMPONENTS

Once the initial system design has been completed, design of the individual system components can be performed.

E.2.1. Approach Slab and Bridge Deck

Extra reinforcement may be required in the approach slab for crack control under the tensile in-plane forces due to thermal expansion. The bridge deck also has to be checked for cracking. The approach slab should be checked for the compressive thermal stresses to avoid concrete crushing. The approach slab should be designed for the differential settlement of the bridge abutment and the transition system. Embankment settlement is another important criterion to check for the approach slab. The bridge approach embankments should be designed to achieve a long-term settlement of less than 3/4 in. to minimize traffic comfort issues on the motorway pavement. To account for the probable geotechnical and construction variations, however, a more conservative approach embankment settlement of 1.5 in. should be assumed for the seamless pavement design (Thomas Telford Service Ltd. 1993).

E.2.2. Transition Slab

The main objective of providing a transition slab is to provide a means for controlling the movement of the system to the point in which no expansion devices are needed where the transition slab meets the pavement. The design items for transition slab include designing against compressive force created by thermal expansion; achieving a uniform cracking pattern in the transition zone during contraction, preventing punching shear failure at the pile to slab connection area; and ensuring adequate flexural capacity at these locations. The thickness of the transition slab should be determined based on a) the punching shear requirements, b) connection requirements for developing the moment introduced from the small piles, and c) the in-plane horizontal stiffness of the system to reduce the movement of the end joint. Reinforcement of the transition slab should be determined from cracked section analysis under tensile in-plane forces. The transition slab should be checked for the maximum bending moments between the rows of small piles. Stirrups (tie bars) may be required for the connection to the slab around the ends of the small piles. The transition slab should also be designed for the design truck axle load exerted at the midspan of the slab between the small piles. Both slabs should be designed for punching shear and one-way shear. Detailed design provisions are provided in VTrans (2009) and also Ala and Azizinamini (2013a).

E.2.3. Secondary Slab

The length of the secondary slab should be greater than or equal to the length of the transition slab. Likewise, the secondary slab thickness is designed for punching shear and requirements to develop the moment introduced from the small piles into the slab (the secondary slab thickness will most likely be equal to the transition slab). The secondary slab should also be designed for the bending moment due to the soil pressure underneath. This slab should be designed for punching and one-way shear.

E.2.4. Small Piles

The stiffness, number, and arrangement of small piles connecting the transition and secondary slabs should be determined to control the longitudinal movement of the transition slab at the end where it meets the pavement. This limit eliminates expansion joint devices at these locations. Increasing the stiffness of the small piles will reduce the longitudinal movement of the transition slab at the end of transition zone. However, piles with high flexural stiffness will also create high stresses (tension or compression) in the transition slab and bridge deck, in addition to the secondary slab. Therefore, the design of small piles should consider a balance between longitudinal movement at the end of transition slab, and maximum longitudinal force that can be accommodated in the transition slab and bridge deck. Further, small piles with high stiffness will demand more sophisticated connection details to the transition and secondary slabs. The maximum longitudinal movement at the end of the transition slab, where it meets pavement, should be limited to about 0.25 in.

E.2.5. Connection of Small Piles to the Slabs

The connection design for attaching the small piles to the top (transition) and bottom (secondary) slabs should use high factors of safety and ensure that they stay elastic, when the weak element of the entire system fails. This is similar to the philosophy used in seismic design where some of the bridge elements are protected and remain elastic while plastic hinges form in other parts of the structure. The connection should be designed for cyclic loading, as the system will be subjected to daily and seasonal temperature fluctuation. Figure E.4 shows one possible connection detail that was used during the experimental phase. Based on the experimental results, the area around the connection could have a larger thickness or alternatively could use advanced materials such as ultra-high performance concrete (UHPC). Research is needed to develop more economical connection details.

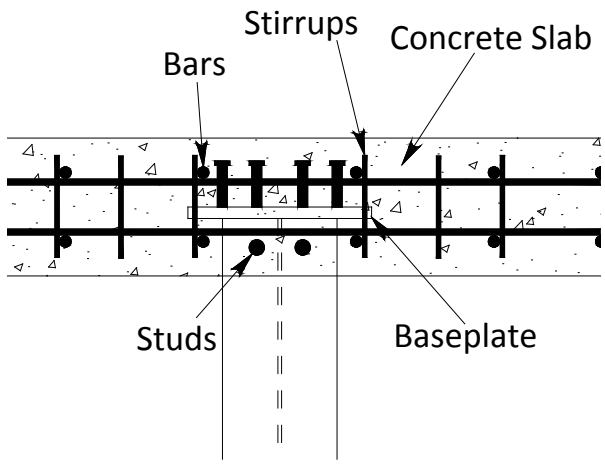


Figure E.4. Recommended small pile concrete slab connection.

E.2.6. Geomaterial

Design of the geomaterial consists of the selection of the geomaterial type and the compaction requirement. The required compaction depends on the stiffness requirement around the small piles. It is very important to achieve the required compaction (level and consistency) around the small piles so stringent quality control is required during soil compaction. It is highly recommended to use granular material in this region due to ease of compaction, resulting in smaller long-term settlement and smaller gap development around the piles caused by pile movements.

Moisture density relation (compaction) tests, maximum and minimum density (relative density) tests, and in-place moisture content and density determinations during placement of the backfill (using a nuclear moisture density meter) are the recommended soil mechanics tests.

E.3. CRACKED SECTION ANALYSIS

Methods of determining the maximum probable crack width and stiffness reduction for an axially tensioned concrete member are explained in *ACI Report 224.2R-92* (1997). The maximum probable crack width in a fully cracked member can be determined from;

$$W_{\max} = 0.10 \times 10^{-3} f_s \sqrt[3]{d_c A} \qquad \textbf{EQ E.1}$$

Where:

d_c = distance from center of bar to extreme tension fiber (in.),
f_s = service stress in the reinforcement (ksi)
A = effective tension area of concrete surrounding the tension reinforcement, having the same centroid as the reinforcement, divided by the number of bars (sq.in.)

Figure E.5 demonstrates the calculation of *A*. *S* is the bar spacing and *H* is the total thickness of the slab.

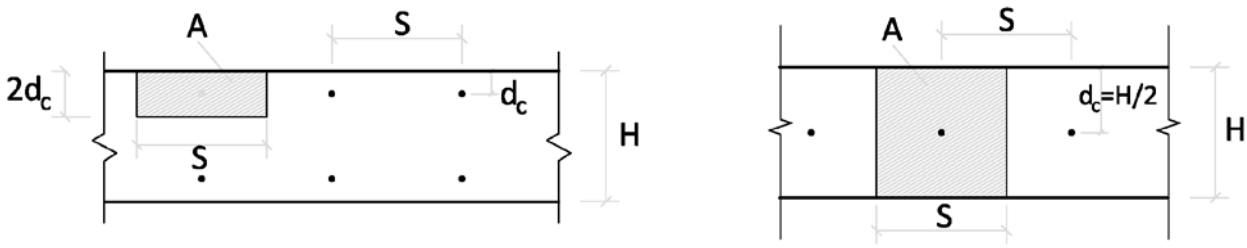


Figure E.5. Determination of effective tension area of concrete surrounding the tension reinforcement for an axially tensioned concrete member.

As can be seen in Figure E.5, the parameter $\sqrt[3]{d_c A}$ can be determined to be $d_c\sqrt[3]{2S/d_c}$ for both one and two layers of reinforcement.

The crack width allowed is inserted in Equation E.1 to obtain the service stress/strain in the reinforcement.

The *LRFD Specifications* define an exposure factor (γ_e) which is 1.00 for Class 1 exposure condition and 0.75 for Class 2 exposure condition. The crack width associated with Class 1 and 2 exposure conditions are 0.017 and 0.012 respectively.

The amount of required reinforcing steel can be determined from the axial tensile force (*P*) in the member (determined from the structural analysis).

$$P = f_s.A_s \quad \Rightarrow \quad A_s = \frac{P}{f_s} \qquad \textbf{EQ E.2}$$

Where:

A_s = reinforcement steel area

The axial force in various segments of the transition slab (*P*) is determined from structural analysis. The transition slab may crack when it is in tension. Cracking in the transition slab will result in reduction of axial stiffness. The reduced axial stiffness of the cracked transition slab should be used in structural analysis, requiring an iterative cracked section analysis. In this iterative analysis the section axial stiffness is modified based on the axial force determined from the previous analysis. Next, the structure is analyzed using the modified axial stiffness. This process is repeated until convergence. For the first iteration, the slab can be assumed un-cracked (the tensile force can be taken the same as the compressive force developed in the slab due to temperature increase).

Following is a description of the method for determining the reduced axial stiffness of the concrete member in tension.

The *ACI Report 224.2R-92* (1997) suggests the following equation for determining the direct tensile strength of the concrete (f_t').

$$f_t' = 0.33[\gamma_c . f_c']^{\frac{1}{2}} \qquad \textbf{EQ E.3}$$

Where:

From the *LRFD Specifications*, for a given f_c' , the unit weight can be determined from $\gamma_c = 0.14 + 0.001 f_c'$ and the E_c can be determined from $E_c = 33000 K_1 \gamma_c^{1.5} \sqrt{f_c'}$).

The stress in the reinforcing bars after the crack occurs ($f_{s,cr}'$) is determined from *ACI Report 224.2R-92* (1997).

$$f_{s,cr}' = f_t' \left(\frac{1}{\rho} - 1 + n \right) \qquad \textbf{EQ E.4}$$

Where:

ρ = reinforcing ratio (A_s/A_g)
n = modular ratio of steel to concrete

The axial load that causes first cracking in the axially tensioned member is;

$$P_{cr} = f_{s,cr}' \times A_s \qquad \textbf{EQ E.5}$$

During the cracked section analysis, if the force in a segment of the slab is smaller than P_{cr}, the slab will not crack and no modification will be required in the structural analysis. Otherwise, if the force in a segment of the transition slab exceeds the above P_{cr}, the segment will crack and the modified axial stiffness of the segment should be determined and used for the next iteration.

For a cracked section, the average strain in the tensile member can be calculated from the CEB Model Code (Thomas Telford Service Ltd. 1993) using following equation to determine the modified axial stiffness;

$$\varepsilon_m = \varepsilon_s \left[1 - k \left(\frac{f_{s,cr}}{f_s} \right)^2 \right] \qquad \textbf{EQ E.6}$$

In which $\varepsilon_s = f_s / E_s$, $k = 1.0$ for first loading and 0.5 for repeated or sustained loading. $k=0.5$ should be used for the seamless system.

The following equation provides the effective modulus of elasticity of steel bars;

$$E_{sm} = \frac{E_s}{\left[1 - k\left(\frac{f_{c,cr}}{f_s}\right)^2\right]} \qquad \textbf{EQ E.7}$$

The effective axial cross-sectional stiffness of the tensile concrete member can be written as $(EA)_{eff} = E_{sm} A_s$. The ratio term $(EA)_{eff}/(EA)$ is the section modification factor that should be used in the structural analysis to modify the axial stiffness of the member in tension. Figure E.6 shows the general flowchart for the cracked section analysis.

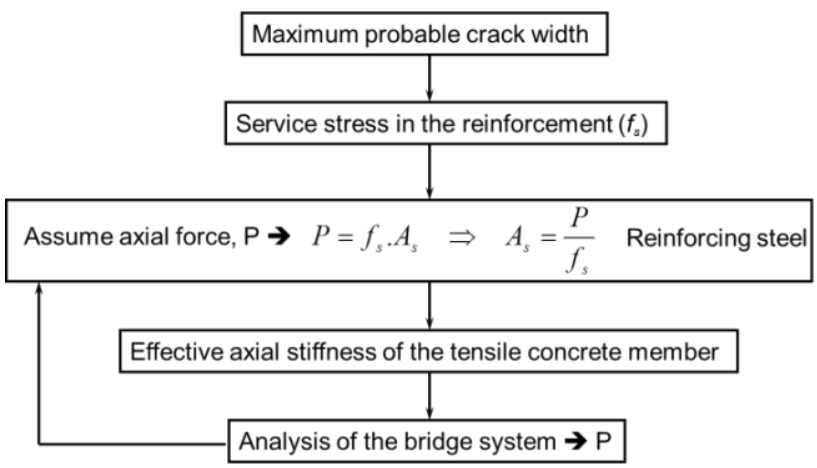


Figure E.6. Cracked section analysis flowchart.

APPENDIX F

CURVED GIRDER BRIDGES

F.1. BACKGROUND

This section contains a procedure developed (Doust 2011) to extend the application of jointless bridges to curved steel I-girder bridges. There are some limitations that must be observed when using the suggested approach that reflect the range of parameters considered in its development. The study considered several bridge configurations for which detailed finite element analyses were conducted. These analyses were then used to a) comprehend the performance of jointless curved girder bridges, and b) develop approximate solutions that are in reasonable agreement with results of detailed finite element analysis. Following is the list of assumptions and cases that were considered during the development of the suggested approach.

1. Steel I-girder superstructure made composite with concrete deck;
2. Concrete integral abutments at the bridge ends supported on steel H-piles;
3. One or more intermediate piers isolated from the bridge superstructure by elastomeric bearings;
4. Concrete parapets integrally connected to the concrete deck;
5. Superstructure super-elevation ranging between 0% and 6%;
6. Abutment wall height ranging between 9 ft and 13 ft;
7. Wingwalls separated from the abutment wall by means of joints;
8. Approach slab connected to abutment wall using a pinned connection detail;
9. Bridge plan symmetric with respect to the mid-length of the bridge;
10. Radial piers and abutments (i.e., the lines of all abutments and piers intersect at the bridge center of curvature);
11. Bridge arc length-over-width ratio larger than 3.0;
12. Ratio of the lengths of end spans to interior spans approximately equal to 0.8; and
13. All intermediate spans of approximately equal length.

The following two sections present step-by-step procedures to calculate the magnitude and direction of bridge end displacements and determine the optimum abutment piles orientation.

F.2. CALCULATING THE MAGNITUDE AND DIRECTION OF END DISPLACEMENT

For curved integral abutment bridges meeting the limitations described earlier, the following procedure can be employed to calculate the magnitude and direction of end displacements.

1. Determine the point of zero movement for the bridge and consequently the bridge length along the centerline of the bridge, L_o,that should be used in calculating the end displacement. For symmetric bridges supported on substructure with relatively symmetrical stiffness, it can be assumed that L_o is equal to half the bridge total arc length. Otherwise, a more detailed approach that takes into account the relative stiffnesses of the supports should be used to calculate the point of zero movement.

2. Determine the effective coefficient of thermal expansion using:

$$\alpha_{equivalent} = \frac{(EA\alpha)_{deck} + (EA\alpha)_{girder}}{(EA)_{deck} + (EA)_{girder}} \qquad \textbf{EQ F.1}$$

3. Calculate the bridge shortening due to contraction using:

$$\Delta_{contraction} = \alpha_{equivalent} . \Delta T . L_o \qquad \textbf{EQ F.2}$$

4. Find the modification factor for bridge shortening due to contraction using the information provided in Figure F.1, which provides the relationship between radius of curvature and the modification factor used in Equation F.5.

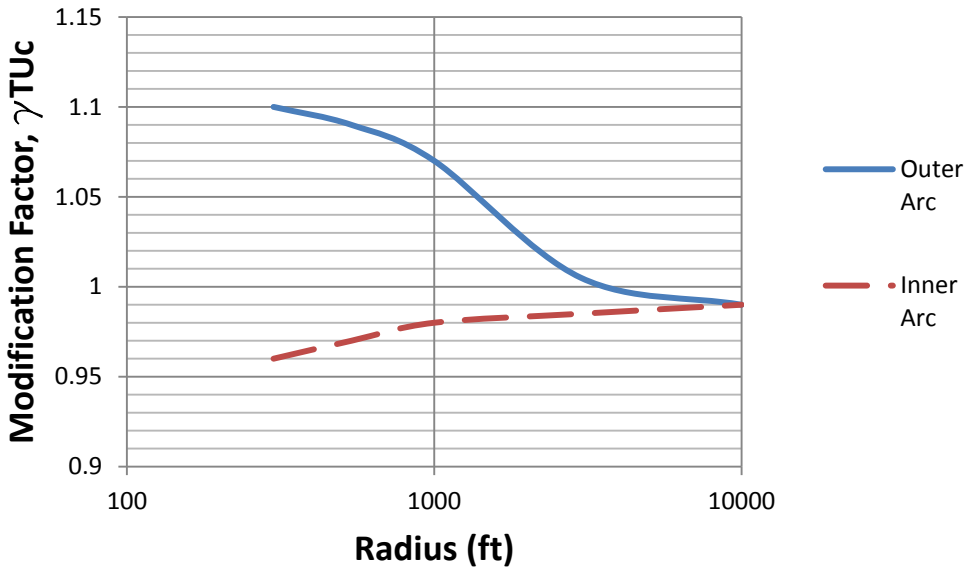


Figure F.1. Modification factor for bridge contraction.

5. Determine the equivalent shrinkage strain using:

$$\varepsilon_{sh,equivalent} = \varepsilon_{sh,girder} + (\varepsilon_{sh,deck} - \varepsilon_{sh,girder})\frac{(EA)_{deck}}{(EA)_{deck} + (EA)_{girder}} \qquad \textbf{EQ F.3}$$

6. Calculate the bridge shortening due to shrinkage using:

$$\Delta_{shrinkage} = \varepsilon_{sh,equivalent} \times L_o \qquad \textbf{EQ F.4}$$

7. Find the modification factor for bridge shortening due to shrinkage using Figure F.2.

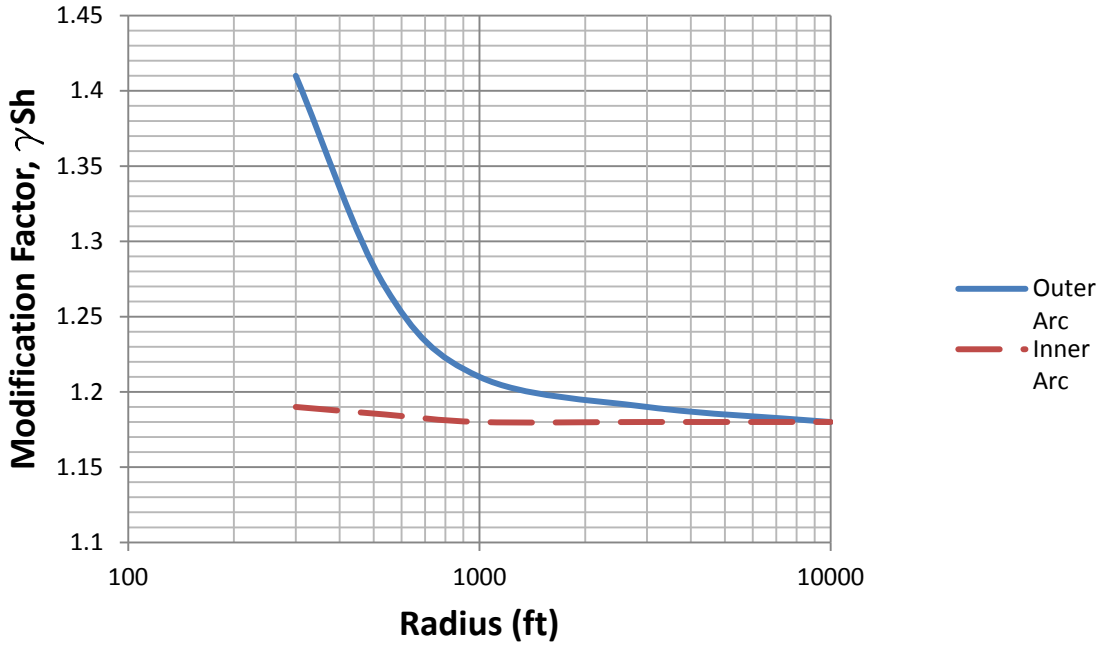


Figure F.2. Modification factor for bridge shrinkage.

8. Calculate the total factored bridge shortening using:

$$\Delta_{total} = 1.3\ (\gamma_{TUc}\ \Delta_{thermal} + \gamma_{Sh}\Delta_{shrinkage}) \qquad \textbf{EQ F.5}$$

9. Calculate the bridge width effect factor using the following equations. These factors are calculated for the inner and outer corners of the bridge separately. The purpose of these factors is to determine the direction of end displacement.

$$k_{in} = 1 + 0.84\ \frac{W}{L_c} \qquad \textbf{EQ F.6}$$

$$k_{out} = 1 - 0.84\ \frac{W}{L_c} \qquad \textbf{EQ F.7}$$

10. Find the direction of the bridge corners displacements using:

$$\alpha_{in} = k_{in}.\left[90 - 11\ \left(\frac{L}{R}\right)\right] \text{in degrees} \qquad \textbf{EQ F-8}$$

$$\alpha_{out} = k_{out}.\left[90 - 11\ \left(\frac{L}{R}\right)\right] \text{in degrees} \qquad \textbf{EQ F.9}$$

11. Knowing the total bridge shortening found in step 8 and the direction found in step 10, solve Equations F.10 through F.16 to find the new location of the bridge corner. The corner of the bridge is assumed to be originally located at the coordinates $x_A = R_A$ and $y_A = 0$ in which R_A is the radius of the bridge at that specific corner.

$$x_{A'} = (-ab + \sqrt{a^2b^2 - (b^2 - R'^2)(1 + a^2)})/(1 + a^2) \quad \textbf{EQ F.10}$$

$$y_{A'} = a\, x_{A'} + b \quad \textbf{EQ F.11}$$

Where,

$$a = -\tan\alpha \quad \textbf{EQ F.12}$$

$$b = R\tan\alpha \quad \textbf{EQ F.13}$$

$$\gamma = \tan^{-1}(\frac{y_{A'}}{x_{A'}}) \quad \textbf{EQ F.14}$$

$$L' = 2R'(\beta - \gamma) \quad \textbf{EQ F.15}$$

In which,

$$\beta = \frac{L}{2R} \quad \textbf{EQ F.16}$$

12. Using the new coordinates of the bridge corner $x_{A'}$ and $y_{A'}$, the components of bridge corner displacement are found as follows:

$$\Delta_x = x_{A'} - R_A \quad \textbf{EQ F.17}$$

$$\Delta_y = y_{A'} \quad \textbf{EQ F.18}$$

F.3. OPTIMUM PILE ORIENTATION

In curved bridges, the optimum orientation of the piles depends mainly on the bridge geometry. Therefore, in contrast to straight bridges, the optimum direction is not the same for all curved bridges. In this section, a method is presented to find the optimum pile orientation in a curved bridge. This method is based on finite element simulation of several curved integral steel I-girder bridges (Doust 2011). The same concept employed for straight bridges is also

used for curved girder bridges; namely, the piles should be oriented so that the strong axis of their sections is perpendicular to the direction of bridge maximum displacement.

The following steps should be used to obtain the optimal abutment pile orientation:

1. The critical load combination for design of the piles should be determined to be either expansion based or contraction based. Figure F.3 may help for determining the controlling load combination.

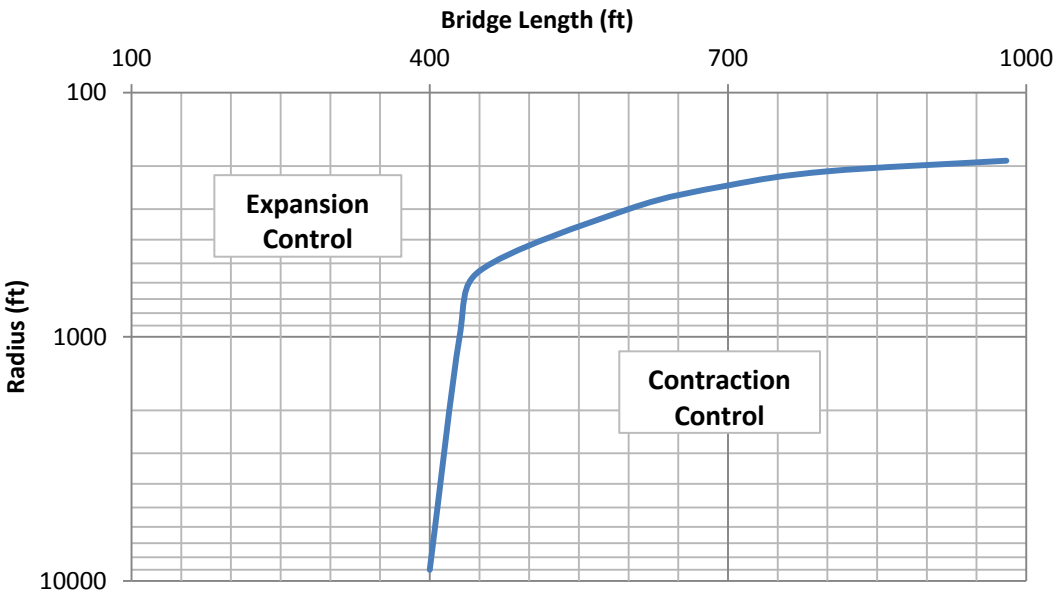


Figure F.3. Controlling type of load combination.

2. The direction of bridge maximum end displacement, as defined in Figure F.4, should be determined using the curves presented in Figure F.5.

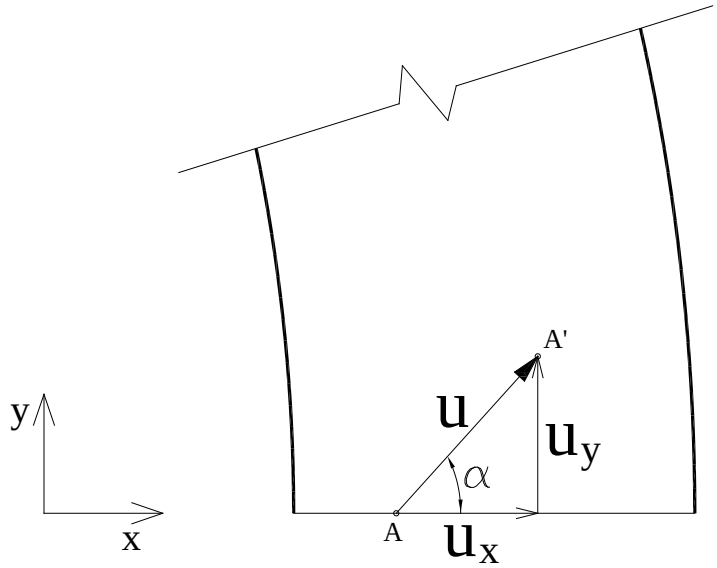


Figure F.4. Direction of bridge end displacement.

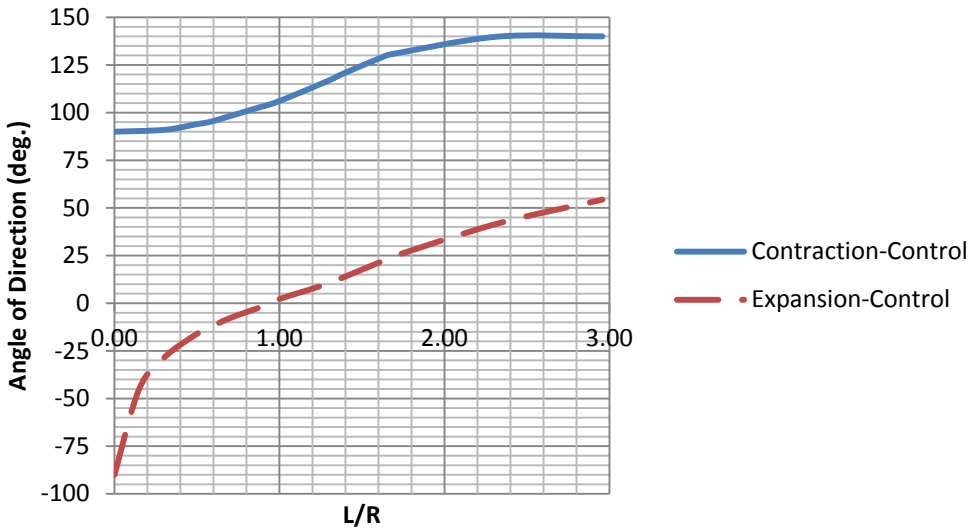


Figure F.5. Angle of direction of bridge end displacement.

3. The strong axis of the abutment piles perpendicular to the displacement direction found in Step 2 should be oriented. If the type of critical load combination cannot be distinguished for a specific bridge, the bridge should be analyzed for both expansion-control and contraction-control pile orientations from Step 2 and then the optimum orientation should be chosen.

APPENDIX G

DESIGN PROVISION FOR SLIDING SURFACES USED IN BEARING DEVICES FOR SERVICE LIFE

G.1. INTRODUCTION

This appendix provides a procedure for designing sliding surfaces for service life that is applicable to various bearing devices that allow rotation and use sliding surfaces to allow horizontal movements. A key factor in the process is being able to predict the service life of sliding surfaces, which is based on the following essential parameters:

1. Wear rate of the sliding material, which can be obtained through experimental work;
2. Total accumulated movements, which can be approximated from loading demand (traffic and thermal loads) and analysis; and
3. The speed or velocity of movement, which can be determined from analysis depending on movement due to truck load or temperature change.

The target service life of the bridge system is established by the owner. The designer must ensure that the bearing device incorporating a sliding surface can provide a service life exceeding the bridge system. If the service life of the sliding surface is less than the service life of the bridge system, steps must be taken to accommodate replacement of the sliding surface or the entire bearing.

The following sections provide detailed descriptions of the parameters listed and the design steps.

G.2. ELEMENTS OF DESIGN PROVISIONS

G.2.1. Wear Rate

Tests have shown that plain PTFE will wear over time causing reduction in thickness, which ultimately affects service life. If the right type of sliding material is selected along with the right thickness, there is greater probability of achieving the desired service life.

The rate of wear, which can be identified as the anticipated thickness reduction per length traveled, can be used to approximately predict service life. The rate of wear is affected by contact pressure, travel speed, temperature, and

lubrication. Considering these factors, the following equation can be used to estimate the wear rate for a sliding surface:

$$wear\ rate = base\ wear\ rate\ (material, P, V) \times CT \times CL \qquad \textbf{EQ G.19}$$

Where:

wear Rate	=	defined in terms of mil thickness per mile of travel distance
base wear rate(material, P, V)	=	defined as a function of material type, contact pressure, and velocity, based on experimental tests
CT	=	modification factor for the effects of low temperature (function of material type)
CL	=	modification factors for the effects of lubrication (function of material type)
P	=	contact pressure acting normal to the sliding surface
V	=	travel speed of the sliding bearing (Section G.2.3)

The base wear rate defined in this procedure is the wear rate determined from tests conducted at various combinations of speed and contact pressure at room temperature, without lubrication. Research (Stanton et al. 1999) showed that low temperature and lubrication also contributed to wear rate. Low temperatures increased wear, while lubrication significantly reduced wear. The effects of these parameters can be seen in Table G.1 below. To account for these effects, the factors CT and CL are added to Equation G.19. These factors are a function of material type and must be determined from tests. At this time, there is insufficient data to develop these factors accurately for final service life design, but estimates can be drawn from Table G.1

Research performed by Campbell and Kong (1987) on wear of PTFE sliding surfaces, indicated that the value of pressure times velocity, referred to as the PV factor, could be used as a base parameter to predict the corresponding rate of wear. Their research indicated that there was a PV threshold below which there would be a low wear regime, and above which, there would be a high wear regime.

In a study conducted by *SHRP 2 Project R19A*, limited proof of concept testing resulted in preliminary development of PV curves for two types of PTFE sliding materials, and an alternate non-PTFE sliding material. These studies confirmed the concept of PV factor affecting wear rate for PTFE based materials, and further confirmed the concept of PV threshold. However, because of the limited amount of data, additional tests need to be carried out to develop final PV vs. wear rate curves that can be reliably used for actual service life design.

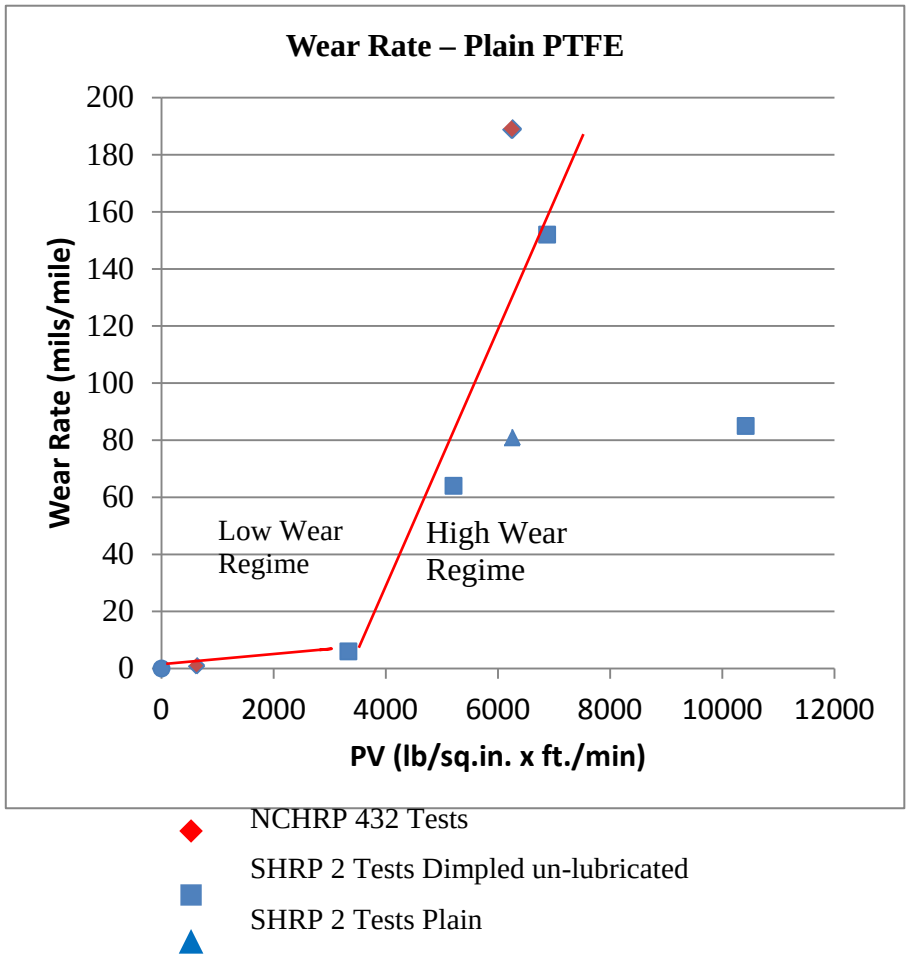


Figure G.1. Wear rate vs. PV factor for plain PTFE.

Figure G.1 shows wear rate vs. PV data for plain PTFE sliding surfaces. It combines data from the *SHRP 2 R19A* study with data from *NCHRP Report 432* (Stanton et al. 1999) and shows the relative low wear and high wear regimes. Data shown in red is from *NCHRP 432*. As stated previously, further testing is required to develop more accurate curves within each of these regions. Figure G.2 shows a similar curve for glass reinforced PTFE (Fluorogold®) from *SHRP 2 R19A* rests. Rates of wear for reinforced PTFE are significantly reduced from plain PTFE and could be considered as an alternative for plain PTFE in conditions of high PV.

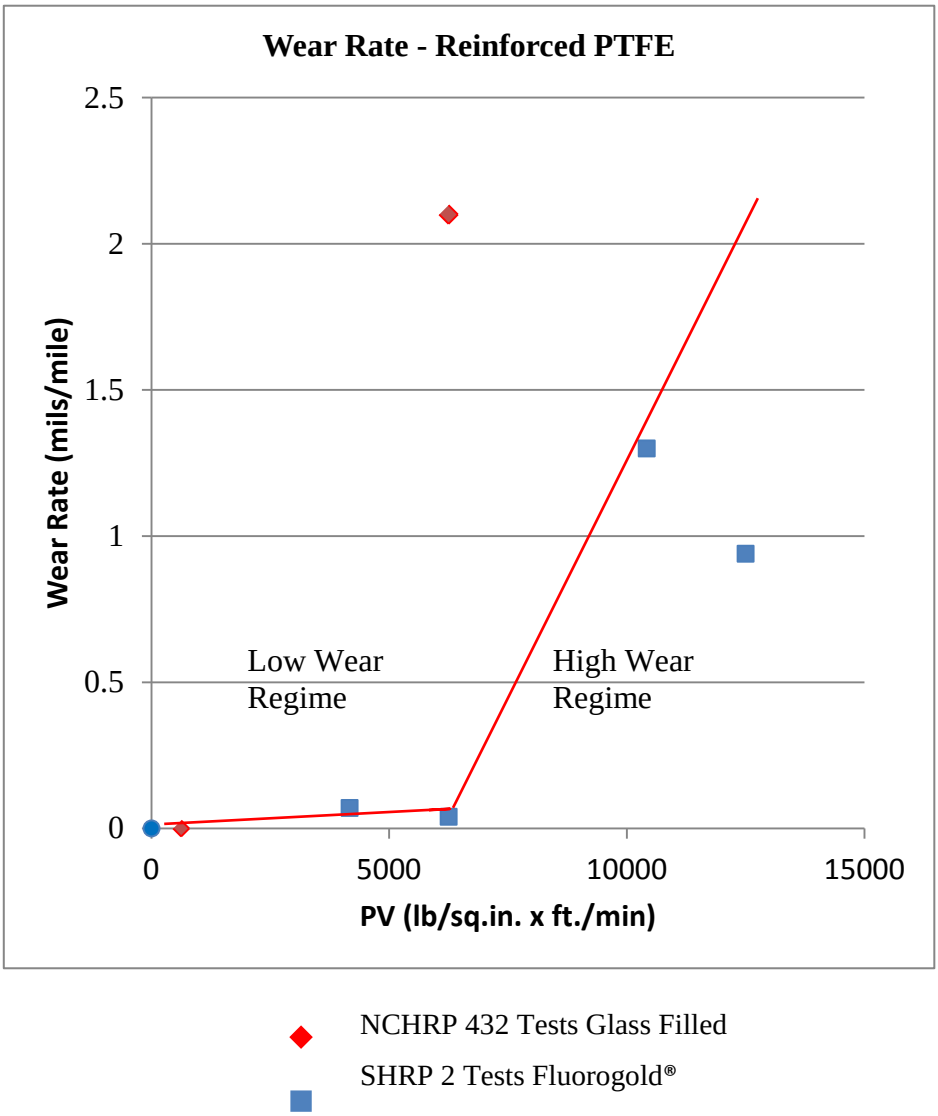


Figure G.2. Wear rate vs. PV factor for glass reinforced PTFE.

Table G.1 presents wear data from *NCHRP Report 432* (Stanton et al. 1999) and shows wear rates for various PTFE based materials at constant pressure, but with variations in sliding speed, temperature and lubrication. This data can be helpful in providing input for parameters in Equation G.19, but as mentioned, further testing is required to establish final values.

Table G.1. PTFE Wear Rates. (Stanton et al., 1999)

Material	Lubrication	V (in./min)	T (°F)	PV (lb/sq.in. ft/min)	Wear Rate (mil/mile)
Unfilled PTFE	Dimpled-lubricated	2.5	68	625	0.3
		25	68	6250	0.5
	Flat-unlubricated	2.5	68	625	0.7
		25	68	6250	189
		2.5	-13	625	10
		25	-13	6250	259
Woven PTFE	Flat-unlubricated	2.5	68	625	0.3
		25	68	6250	17
		2.5	-13	625	27
		25	-13	6250	24
15% Glass Filled	Flat-unlubricated	2.5	68	625	-1
		25	68	6250	-0.5
		2.5	-13	625	No Result
		25	-13	6250	6
25% Glass Filled	Flat-unlubricated	2.5	68	625	-0.3
		25	68	6250	2
		2.5	-13	625	4
		25	-13	6250	46

Note: Pressure 3000 psi

G.2.2. Estimating Total Accumulated Movements

The total travel demand is the total accumulated distance that the sliding surface will be traveling throughout the service life of the bridge system. This total travel demand can be estimated using a) specified bridge system service life, b) traffic and thermal loading demands, and c) calculating horizontal movements, related to applied traffic and thermal loadings.

Sliding surfaces are means to accommodate horizontal movements associated with traffic load and daily and seasonal bridge superstructure expansion and contraction. The total bridge movement at the bearing, $(TD)_{Demand}$, in miles, is produced by the following three mechanisms.

a. Traffic-induced horizontal movement, $(TD)_{Tr}$ The total accumulated travel with this type of movement can be considerably greater than that associated with b and c.

b. Daily temperature-induced horizontal movement, $(TD)_{DT.}$

c. Seasonal temperature-induced horizontal movement, $(TD)_{ST.}$

The total movement due to temperature is the combination of daily and seasonal movements.

G.2.2.1. Traffic-Induced Horizontal Movement, $(TD)_{Tr}$

Equation G.20 estimates total horizontal movement of the sliding surface, in miles due to traffic movement, for the designed service life, $(SL)_B$ in years.

$$(TD)_{Tr} = 2 \times A \times \theta \times D_1 \times n \times 1.33 \times (ADTT)_{SL} \times (SL)_B \times \frac{365}{63360} \qquad \textbf{EQ G.20}$$

Where:

$(TD)_{Tr}$	=	traffic-induced horizontal movement (miles)
A	=	1, if each end of the girder is free to move in horizontal direction 2, if all horizontal movements are accommodated at one end, with the other end pinned against horizontal movement.
$(ADTT)_{SL}$	=	single lane ADTT (average daily truck traffic)
θ	=	rotation of girder end with sliding bearing (rad)
D_1	=	depth of neutral axis measured from the bottom flange (in.)
$(SL)_B$	=	design service life (years)
1.33	=	impact factor for truck load
n	=	number of equivalent full amplitude horizontal movement cycles per truck passage (due to free vibration) initially taken equal to 1.0 for this procedure

In EQ G.20, A is a parameter that accounts for boundary conditions at both ends of the span. The term $\theta \times D_1$ is the horizontal movement due to girder end rotation. The factor 2 accounts for the full cycle of movement, which includes deflection and rebound.

When a truck passes over a span, the girders deflect to a maximum amount as the truck approaches mid span, and then recover as the truck moves toward the end of the span. However, because of dynamic behavior, the girders may continue to vibrate until the girder deflection is damped out. The cycles produced after truck passage have successively smaller amplitudes and the decay is dependent on the damping ratios. This characteristic is represented by the term n, which is the equivalent number of cycles with full amplitude that corresponds to the total number of cycles with decreasingly smaller amplitude. For the purposes of this procedure, however, this term can be neglected (using $n = 1$). Although it is recognized that this behavior occurs, its true magnitude as it applies to bearing movement requires further study along with field verification.

In the equation, ADTT is the average daily tuck traffic and $(SL)_B$ is the owner specified service life of the bridge system. The constant terms in EQ G.20 are conversion factors.

G.2.2.2. Daily Temperature-Induced Horizontal Movement, $(TD)_{DT}$

Equation G.21 estimates total horizontal movement of the sliding surface, in miles, due to daily temperature fluctuation over the designed service life of the bridge, $(SL)_B$ in years.

$$(TD)_{DT} = \Delta L_{Daily} \times (SL)_B \times 365/5280 \qquad \textbf{EQ G.21}$$

Where:

ΔL_{Daily} = $2\alpha L \Delta T_{Daily}$
α = coefficient of thermal expansion
L = maximum span length, ft., or length contributing to expansion in the case of multiple spans
ΔT_{Daily} = maximum daily temperature fluctuation

G.2.2.3. Seasonal Temperature-Induced Horizontal Movement, $(TD)_{ST}$

Equation G.22 estimates total horizontal movement of the sliding surface, in miles, due to yearly temperature fluctuation over the designed service life, $(SL)_B$ in years.

$$(TD)_{ST} = \Delta L_{Annual} \times (SL)_B \times \frac{1}{5280} \qquad \textbf{EQ G.22}$$

Where:

ΔL_{Annual} = $2\alpha L \Delta T_{Annual}$
ΔT_{Annual} = maximum annual temperature fluctuation

G.2.2.4. Total Induced Horizontal Movement, $(TD)_{Demand}$

$$(TD)_{Demand} = (TD)_{Tr} + (TD)_{DT} + (TD)_{ST} \qquad \textbf{EQ G.23}$$

G.2.3. Estimating Speed of Sliding Surface Movement

The service life calculation, as described previously, involves the use of PV curves that are specific for the material used for the sliding surface. The term V is the speed at which the sliding surface moves, which depends on whether the movement is caused by truck load or temperature load. This section provides a procedure for calculating the term V in the PV expression.

G.2.3.1. Speed of Movement per Truck Passage

The speed of travel, V, for the sliding bearing for movement caused by truck passage, can be determined from the following general equation;

$$average\ travel\ speed = \frac{total\ horizontal\ movement\ per\ truck\ passage}{travel\ time} \qquad \textbf{EQ G.24}$$

The total horizontal movement of the sliding surface per truck passage can be determined from the following equation:

$$\text{total horizontal movement per truck passage} = 2 \times 1.33 \times A \times \theta \times D_1 \times n \qquad \textbf{EQ G.25}$$

Where:

A	=	1, if each end of the girder is free to move in horizontal direction 2, if all horizontal movements are accommodated at one end with the other end pinned against horizontal movement.
θ	=	Rotation of the girder end with sliding surface (Rad)
D_1	=	Depth of neutral axis measured from the bottom flange (in.)
1.33	=	Impact factor for truck load
n	=	Number of equivalent full amplitude horizontal movement cycles per truck passage (due to free vibration) initially taken equal to 1.0 for this procedure

The total travel time is the time that it will take for the accumulated horizontal movement due to passage of one truck to occur. It is the time for the first cycle and for all succeeding dynamic vibration cycles to take place. If the component of the time due to dynamic vibration cycles is neglected as described in Section G.2.2, The resulting time for the movement can be determined from EQ G.26 below:

$$t = \frac{bridge\ span\ length}{truck\ speed} \qquad \textbf{EQ G.26}$$

G.2.3.2. Speed of Movement per Temperature Variation

The speed of travel, V, for the sliding bearing for movement caused by daily and seasonal temperature change is a much slower velocity. It can be estimated from EQ G.27 below:

$$average\ travel\ speed = \frac{total\ horizontal\ movement\ per\ temperature\ change}{travel\ time} \qquad \textbf{EQ G.27}$$

The total horizontal movement of the sliding surface due to temperature movement is estimated by determining the total yearly temperature movement due to a) daily temperature change and b) seasonal temperature change.

$$\text{total movement due to daily temperature change} = \Delta L_{Daily} \times 365 \qquad \textbf{EQ G.28}$$

$$\text{total movement due to seasonal temperature change} = \Delta L_{Annual} \qquad \textbf{EQ G.29}$$

$$\text{total temperature movement} = (\Delta L_{Daily} \times 365) + \Delta L_{Annual} \qquad \textbf{EQ G.30}$$

The total travel time for the total temperature movement as defined in EQ G.30 is 365 days, which can be converted into consistent units.

G.3. DESIGN PROCESS FOR SLIDING SURFACES

G.3.1. Steps in Design Process

The following steps could be used to select the type of sliding material and its required thickness to meet service life requirements.

Step 1. Calculate the total travel distance demand, $(TD)_{Demand}$, in miles, using Section G.2.2.

Step 2. Determine the velocity of movement based on traffic load or temperature movement, using Section G.2.3.

Step 3. Select a trial sliding surface type and determine the corresponding wear rate, based on PV curves for the type of material, in in./mile, using Section G.2.1.

Step 4. Calculate the thickness demand, which is the total predicted wear or reduction in thickness for the sliding surface using the following equation:

$$(\text{thickness})_{\text{Demand}} = (\text{TD})_{\text{Demand}} \times \text{wear rate} \times \alpha \qquad \textbf{EQ G.31}$$

$$\text{gross thickness} = (\text{thickness})_{\text{Demand}} + \text{thickness of recess} \qquad \textbf{EQ G.32}$$

Where:

$(\text{TD})_{\text{Demand}}$ = total induced horizontal movement (see Equation G.23)

α = factor to assure that the thickness will not be zero at the end of the service life to prevent undesirable metal to metal contact (>1.0).

It should be noted that the $(\text{Thickness})_{\text{Demand}}$ is the thickness that is subject to wear. Accordingly the gross thickness is the thickness subject to wear plus the recessed thickness that is used to positively connect the sliding surface to backing plate.

Step 5. Establish the gross thickness of the material to be specified in the design plan. The thickness of commercially available sliding surfaces must be larger than the gross thickness calculated in Step 4.

Step 6. If the commercially available thicknesses are less than the required gross thickness, then there are two available approaches: 1) select another material, such as a reinforced or braided PTFE or other sliding material type, that could meet the demand by repeating Steps 3 through 5, or 2) calculate the service life of the commercially available thicknesses and develop a replacement strategy accordingly.

G.3.2. Design Process Application

The process has application to all bearing types that use sliding surfaces to permit horizontal movement, and where horizontal movement is caused by truck load or temperature load. It can also be applied to evaluate the service life of sliding surfaces that are used in combination with elastomeric pads. In these cases, the elastomeric pad is designed to accommodate the high-cycle, low-amplitude horizontal movement due to truck load, and the sliding surface is designed to accommodate the larger-amplitude, low-cycle movement due to temperature. This approach has advantages for design of expansion bearings at the end of a series of continuous spans where the temperature movement is large and the superstructure reactions are low. Combining elastomeric pads with sliding surfaces reduces the required thickness of the elastomeric pads and permits the use of more durable elastomeric bearings in cases in which HLMR types would have been required because of the excessive height required for the elastomeric pads. Further advantages of the reduced elastomeric pad thickness include better stability during construction and operation, and reduced instantaneous and long-term compressive deflection.

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