4 Research Plan

4.1 Introduction

The principal objective of the research plan is to identify proven technologies and their applications for extending the service life of existing and new bridges with span lengths of less than 300 feet. Bridge service life is at the forefront of bridge owners' concerns in the form of the growing fiscal needs of bridge maintenance. As bridge maintenance costs increase, available funds for new facilities and for the upgrading of existing facilities become more difficult to budget.

There is renewed general public interest in addressing the condition of the nation's bridges that reflects the opinion voiced by the civil engineering community over the last few decades. In an ideal world, the replacement of all bridge structures in question (those that are structurally deficient and/or functionally obsolete and that have low sufficiency ratings) would be a preferred alternative. In reality, budgetary restrictions will not allow replacement of every bridge at issue. Some existing bridges can realize longer service lives using various levels of rehabilitation or modification while others may have to be replaced.

Providing a longer service life for rehabilitated, replacement and new structures through the use of state-of-the-art materials and construction techniques combined with emerging technologies is a financially feasible solution. The research and development of methodologies and details provided in this study are intended to form the basis for providing longer service life by design. This research will examine current practices that have worked well and those that have not in order to develop systems, sub-systems, components and details that overcome past performance issues.

Service life must be addressed on both a structural level and a functional level. It could be argued that the technology needed to achieve 100 plus years of structural service life from our bridges is in existence. The question to be asked then is this: "What general philosophy should we be using to achieve longer service life from our existing and new bridges?" Some nations have taken the route of dividing their bridges into various levels of importance and of using different service life design provisions and details for these various levels.

There must be a balance between first cost, maintenance cost and the planned functional life of a structure in order to provide optimal usage of financial resources. In the design arena, first cost and projected maintenance cost are combined to establish the best value for a facility. Functional life usually is not considered to a great extent. Typically, the operational capacity of roadways is based on traffic projections for a Design Year that is 20 to 25 years beyond the opening of the facility. In rapidly growing urban and suburban areas where development is difficult to predict, these facilities can become functionally obsolete before the designated Design Year. Bridge structures are designed to be widened, but many times the facilities that they span are limited based on bridge pier locations. Bridge structures are, therefore, quite often replaced to allow increased lane capacity underneath the structure. There are many locations where bridges have been widened 3 or 4 times when the capacity needs of the facility they cross have not changed. Bridges over waterways, in rural areas and over minor arterial streets are relevant examples where the need for long service life can be historically documented. These factors need to be carefully weighed when establishing priorities for service life, whether for 50 years, 100 years or more.

Many of the durability issues faced by bridge owners today have design solutions that could have maximized design life and minimized maintenance costs. A thorough research of these maintenance issues and associated bridge failures has historically led to improvements in bridge design codes and construction details. These same issues can be studied to develop similar provisions for a defined service life.

A survey conducted by the proposing research team during the preparation phase of the proposal indicated that many State Bridge Engineers believe the elimination of bridge joints and improvement in durability of concrete decks will have a significant impact on improving the service life of bridges. Other items mentioned by State Bridge Engineers included fatigue and fracture, scour, corrosion of reinforcing bars and exposure of sub-structure elements to corrosive environments.

The following is a list of some of the design, fabrication, construction and maintenance issues that will be considered in the proposed project.

- There is a need to fully comprehend past design practices that led to observed problems in the field with respect to the service life performance of bridges.
- There is a need to fully comprehend past construction and maintenance practices that led to observed bridge service life problems.
- Rehabilitation strategies used in the past, successfully or otherwise, to extend the service life of bridges, need to be identified and studied.
- Existing databases related to past performance of bridges should be studied carefully.
- Experience and solutions gained by designers and researchers in other engineering fields with similar problem types (getting longer service life performance at optimum total cost) should be studied carefully and lessons should be drawn. For instance, the new Mechanistic-Empirical design approach for pavements is driven by long-term service life performance criteria.
- International experience should be studied carefully and lessons drawn, with the understanding that bridge design and construction practices as well as demands in terms of traffic and loads vary within different countries.
- The project should offer a long-term vision for future practices to be used in bridge design, construction and maintenance.
- There is a need to develop a set of criteria to categorize bridges into various levels of importance. This could allow using different sets of details, components or even maintenance provisions for different classes of bridges.
- The design for durability should be accompanied by tools (such as equations or other appropriate analysis tools) capable of predicting expected service life. For instance, we

can predict the fatigue life of various details within a steel bridge in terms of the number of trucks needed to pass over the bridge before the detail in question fails by fatigue. (It should be noted, however, that in most instances, details in steel bridges are designed to have infinite fatigue life.) We need to develop similar capabilities for other observed modes of failure. For example, chloride penetration and carbonation in concrete decks create a corrosive environment for embedded reinforcing bars. The project outcome should go beyond recommending a series of test-verified alternatives to eliminate chloride penetration and carbonation in concrete decks to inclusion of design equations capable of predicting the passage of time required to develop corrosion due to chloride penetration and carbonation.

- The project outcome should include construction practices leading to bridges with higher levels of service life performance.
- Project outcomes should include alternative maintenance options for suggested details. Building a perfect detail that will be very costly to replace could be more expensive than building a detail that is easily replaced but that provides lower service life.
- The outcome of the project should be put into a simple form, adoptable by AASHTO design specifications, and should include sufficient background information, sound reasoning and documentation. For some of the factors affecting the service life of bridges, the known definition of loads and resistance may need to take a different form. However, the final outcome should be put in a form that is familiar to the typical designer. For example, loads that could be related to environmental factors such as de-icing chemicals and resistance take the form of the properties and dimensions of the concrete deck resisting the penetration of these chemicals. Despite this, the end result should be in the form of a table of load and resistance factors applied to environmental loads and the resistance properties of concrete.
- The project outcomes should include new areas, products, systems, etc. that academia, government and industry need to consider for further work. Not every promising idea and concept can be completely resolved within this project.
- The project outcomes should include recommendations for the next generation of inspection reporting complying with the objectives of FHWA's Long Term Bridge Performance Program (LTBPP). Further, for some of the concepts to be identified in this project, LTBPP provides an opportunity for collecting quantifiable data that will assist in developing design criteria for service life.
- The project recommendations should be practical in nature and should use readily available material and equipment.
- The project recommendations should not confine, take away or hinder in any way the imagination and innovative approaches that designers exercise in providing a solution for any given problem.
- Finally, the project outcomes should result in the development of an "owner's manual" to be given to the owner after construction. This owner's manual should include all the information that is needed to make crucial decisions during the service life of the bridge. The "Owner Manual" to be developed will be generic and global. The "Owner Manual"

will be developed by considering span lengths, bridge type, type of systems, components and sub-components used.

One important departure from the past practice in the design and construction of bridges will be the development of the information needed to design bridges for durability and service life. Current codes include very limited information on design for service life. For instance, current design provisions allow designing steel bridge elements for fatigue and fracture to obtain 75 years of service life. The current methods could easily be changed to provide for 100 years of service life by changing only one number (from 75 to 100) in equation 6.6.1.2.5-2 of the AASHTO *LRFD Bridge Design Specifications,* or simply AASHTO *LRFD Specifications* (AASHTO 2007). However, the same is not true for other service life related failure modes or design issues. The service limit state provisions we currently have for steel and concrete bridges are limited in scope and lack the capability to predict service life performance. It would be preferable if the design for service life and durability were in a quantitative form. The proposed methodology should include both existing and new bridges.

The lessons to be drawn from past experience could be obtained by studying all bridge types and span lengths. For instance, the corrosion of steel reinforcement in concrete decks has many similarities for short, medium and long span bridges. There are lessons to be learned from long span bridges with applicability to short span bridges. On the other hand, the major focus of the project needs to be on the type of bridges that dominate our bridge inventory, namely bridges with spans less than 300 feet.

This proposal provides an opportunity to use information gleaned from the experience of others as well as information collected from other engineering disciplines with similar longevity issues to facilitate a new look at the way we design bridges. Current practice focues predominantly on the strength limit state. A similar practice needs to be developed to design for durability. This project will allow, for the first time, the creation of AASHTO LRFD design and construction specifications that addresses design for strength and durability, both in a quantifiable way. This will be unique worldwide and will allow for the development of a set of guidelines that the rest of the world could follow.

4.1.1 Outline of the Efforts Leading to Preparation of the Proposal

A significant amount of effort was devoted to the preparation of this proposal. Every research team member contributed to its writing. Prior to writing the proposal, Dr. Azizinamini visited the group who developed the European approach towards design for service life presented in ***fib*** Bulletin 34, *Model Code for Service Life Design* (CEB 2006). Dr. Azizinamini met with the group for several days in Denmark. During these working meetings, the details of the European approach for the design for service life were thoroughly discussed. Furthermore, the limitations of the European approach were identified.

In addition, a survey was sent out to all states asking them to identify factors and details that limit the service life of bridges and the solutions used by them to address these challenges. Several State Bridge Engineers were also interviewed to seek their opinions about issues related to the project.

The information gained from the Denmark visit, survey results and results of phone interviews, together with the research team's personal experience and knowledge, were among the factors influencing the development of the proposal content and outline. Weekly phone conferences and a day-long meeting in Chicago on September 8, 2007, with almost all research team members present, were also part of the effort leading to the development of this proposal.

4.1.2 Research Team Members

The initial research for this topic along with the interviews of State Bridge Engineers and the team that developed the European approach to designing for service life identified numerous areas of expertise that would be required to meet the objectives of this research project. The research team members were selected based on these project needs and to achieve the goals and objectives of the R19-A project. Two members of the original group responsible for developing the new design provisions for service life in Europe (***fib*** Bulletin 34) are also research team members (Dr. Carola Edvardsen and Ove Sorensen). The research team members are shown in Figures 1 and 2. Research team members have a long history of working together and have carried out joint research and design projects in the past. The greatest feature of the research team is that it has the expertise and knowledge required to incorporate any necessary changes into the project if deemed necessary by SHRP 2 staff after the project is initiated.

Refer to Appendix C, located in Section 12 of this proposal, for complete résumés of the research team.

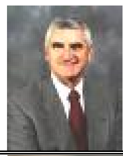
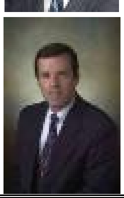
Research Team Members & Affiliations			Expertise
University of Nebraska-Lincoln		Atorod Azizinamini, Ph.D., P.E. Project Director, UNL	Steel Bridges, Seismic Design, High Performance Materials
		Maher K. Tadros, Ph.D., P.E. UNL	Concrete Bridges
		Andrzej S. Nowak, Ph.D. UNL	Risk, Reliability, and Life-Cycle Analysis
PBS&J		Glenn F. Myers, P.E. PBS&J	Bridge Design
		Morad G. Ghali, P.E. PBS&J	Bridge Designer, Concrete Bridges
		Omar Jaradat, Ph.D., P.E. PBS&J	Bridge Design
HDR		Edward H. Power, P.E. HDR	Bridge Design and Management, Steel Bridges
		Brian J. Leshko, P.E. HDR	Bridge Maintenance and Inspection
		Richard T. Horton, P.E. HDR	Bridge Design

Figure 1. Members of the research team

Research Team Members & Affiliations			Expertise
Consultant		H. Celik Ozyildirim, Ph.D., P.E. Consultant	Durability of Concrete
Vector Corrosion Technologies		David W. Whitmore, P.E. Vector Corrosion Technologies	Corrosion
KTA		Eric S. Kline, PCS KTA	Coating
Georgia Institute of Technology		Donald W. White, Ph.D. Georgia Institute of Technology	Analysis and Simulation
University of Delaware		Dennis R. Mertz, Ph.D., P.E. University of Delaware	Fatigue and Fracture, AASHTO Code
William Kenneth Engineers, LLC		Michelle L. Tragesser, P.E. 100% women-owned, Civil and Structural Engineering Consultant Firm and Certified PBE with US DOT	Seismic Design of Bridges
COWI		Carola Katharina Edvardsen, Ph.D. COWI	Design for Service Life- European Approach – Concrete Durability
		Ove Sørensen COWI	Design for Service Life- European Approach – Maintenance and Inspection

Figure 2. Members of the research team (Continued)

The following is a brief description of the expertise of each team member.

Dr. Atorod Azizinamini, P.E., University of Nebraska-Lincoln -- Dr. Azizinamini has a very diverse background in bridge engineering. He has carried out numerous research projects in both steel and concrete bridge areas. This includes the areas of Steel Bridges, Seismic Design, High Performance Steel, High Performance Concrete and non-destructive testing. From 1985 through 1989, he was employed by Construction Technology Laboratories (CTL) in Skokie, Illinois, where he primarily conducted studies related to performance of reinforced and pre-stressed concrete structures. He holds several patents related to innovative bridge systems. He is a member of a number of national bridge committees. From 2000 to 2006, he chaired the ASCE Bridge Technical Administrative Committee and was responsible for overseeing the technical activities of all ASCE bridge committees. On a consulting basis, he has designed bridges in Nebraska, Wyoming and Montana and has served as expert witness in several states related to bridge engineering. He has received several awards and is a registered Professional Engineer in Nebraska and Montana.

Dr. Maher Tadros, P.E., University of Nebraska-Lincoln – Dr. Tadros has four decades of diverse experience associated with unique and conventional bridge system selection and design, construction engineering, alternate and value-engineering design and bridge aesthetics. He holds several patents related to innovative bridge systems. At the 50th Anniversary of the Precast/Prestressed Concrete Institute, in 2004, Dr. Tadros was named "Industry Titan", one of the 50 most influential persons in the 50-year history of the precast/prestressed concrete industry. He is the holder of eleven patents. He has received several awards, including the prestigious T.Y. Lin Award (which he has received an unprecedented four times). Dr. Tadros is the current Chair of the PCI Committee on Bridges.

Dr. Andy Nowak, University of Nebraska-Lincoln – Dr. Nowak's will provide assistance in the statistical analysis of experimental data. More importantly, he will advise the research team on issues that this project should consider in developing guidelines for further research to be carried out in the R19-B project.

Glenn Myers, P.E., Morad Ghali, P.E. and **Dr. Omar Jaradat, P.E.,** PBS&J – Bridge Consultant, Bridge Program Management, Maintenance and Inspection, addressing concrete bridge design. PBS&J provides relevant experience with the inspection, design, rehabilitation and construction of a variety of bridges and has managed transportation programs, including the development of design criteria, for numerous state DOTs and other transportation agencies across the nation. PBS&J's experience with factors affecting bridge longevity for concrete structures includes the evaluation of corrosion issues in extreme environments, seismic retrofitting, jointless bridges and the research and development of criteria for longer recurrence level events based on structure importance.

Ed Power, P.E., **Brian Leshko, P. E.** and **Todd Horton, P.E.**, HDR – Bridge Consultant, Bridge Management, Maintenance and Inspection, mainly steel bridge design. HDR provides relevant practical design, maintenance and construction experience relating to steel bridge systems and details. With offices across the country, HDR has experience with many of the factors involved with increased service life including jointless bridges, deck durability, fatigue and fracture and high performance coatings, and has relationships with nearly all state DOTs, which provides the ability to gather and evaluate various industry experience.

Dr. Celik Ozyildirim, P.E., Consultant – Dr. Celik Ozyildirim, Principal Research Scientist with VTRC, is considered by the profession to be one of the top experts on the durability of concrete. His strength is in understanding the practical problems and challenges facing the industry as related to the durability of concrete and provided rational but practical solutions. He has carried out numerous research projects related to concrete durability. He is a member of a number of key national and international committees. He is past chairman of the TRB section on concrete and member emeritus of TRB Committee AFN10, Basic Research and Emerging Technologies Related to Concrete.

David Whitmore, P.E., Vector Corrosion Technologies -- Mr. Whitmore is president of Vector Corrosion Technologies. From 1989 through 1997, he was involved with the SHRP project Electrochemical Chloride Extraction and the SHRP implementation program. He is considered one of the foremost experts in the area of corrosion. He is a member of several national and international committees related to corrosion. He holds several patents related to corrosion protection of steel in concrete (US patent 6027633 -- Electrolytic Restoration of Concrete, US patent 6183624 -- Restoration of Concrete Decks, US patent 6165346 -- Cathodic Protection of Concrete, US patent 6572640 -- Cathodic Protection, US patent 6793800 -- Cathodic Protection of Steel Within a Covering Material).

Eric S. Kline, PCS, KTA-TATOR, Inc. – Mr. Kline is an expert in the area of coating. He is an SSPC Certified Protective Coating Specialist. He joined KTA-Tator in 1982 as senior coating consultant. He has managed several major bridge repainting projects. Nationally, Mr. Kline’s research has included conceptualization and design of an innovative surface preparation and coating application approach for repainting overpass bridges. Mr. Kline conceptualized the need for and spearheaded the industry effort to develop the now accepted two-coat approach to bridge repainting. He is the co-principal investigator in what has been called the “single coat of the future paint system.”

Dr. Donald White, Ph.D., Georgia Institute of Technology – Analysis and Simulation. Dr. White will address the application of advanced computational analysis and experimental testing methods for the assessment of serviceability performance. Dr. White has extensive prior experience with analysis and testing of steel and steel-concrete structures aimed at the development of structural design standards.

Dr. Dennis Mertz, P.E., University of Delaware -- Dr. Mertz is a Professor of Civil Engineering at the University of Delaware. He was the Co-Principal Investigator of NCHRP Project 12-33, the project that developed the first edition of the AASHTO *LRFD Bridge Design Specification*. He is an expert in Fatigue and Fracture of Steel Bridges and will be one of the team’s contacts with appropriate AASHTO committees, transferring the project findings.

Michelle Tragesser, P.E., William Kenneth Engineering, -- Michelle Tragessor has 13 years of engineering consulting experience in design, analysis and construction for a variety of transportation and marine-related structures. She is well versed in seismic analysis and design. She is an active member of PCI since 1999. She has been involved in all phases of bridge work, from type, size and location studies to construction support of WSDOT "mega" projects. She is president of the William Kenneth Engineering company, a 100% women-owned, civil and structural engineering consultant firm.

Dr. Carola Edvardsen, COWI – Concrete Durability and Design for Service Life Expert in Europe and one of the original members of the European team that developed ***fib*** Bulletin 34. Assisting Dr. Edvardsen will be **Mr. Ove Sørensen**, Chief Bridge Engineer in charge of Operation & Maintenance at COWI.

4.1.3 Management Plan

The SHRP 2 R19-A Project requires management of a large multi-tasked, multi-disciplined team that must be focused on the tasks, deliverables, schedule and budget for each phase. Our Management Approach provides for four elements critical to meeting these goals:

- An Operations Plan in which the management structure for the project is established, the project objectives are fully defined and all team members understand their roles, responsibilities and task requirements.
- A Communication & Coordination Plan outlining the type, function and procedures for all required communication within each of the Phases and Tasks, both within the team and with the SHRP 2 and the Technical Coordinating Committee (TCC).
- A Production Plan detailing requirements for specific task development and execution, as well as standards for document control.
- A Quality Assurance and Quality Control Plan that establishes requirements and procedures to assure the development of quality project deliverables that meet the project objective.

These policies and procedures will be fully developed and included in a single Project Management Plan (PMP) document that will be distributed to all team members, SHRP 2 and the Technical Coordinating Committee (TCC). The PMP will be a living document that is modified as needed to adjust to the refined focus of the study as individual tasks are completed.

An important part of the management plan will be to specify the communication type and level between the experts in the research team and graduate students. The technical leadership will be provided by experts in the research team and students will continuously be supervised.

A) Operations Plan

This plan addresses our procedures for managing and controlling the overall project and individual Phases and Tasks, and provides for individual subcontract management.

Dr. Atorod Azizinamini, P.E., of UNL, will be the Project Director, and will be responsible for overall project activities and delivery, overall schedule, progress reports and coordination with SHRP 2 and TCC staff. Mr. Edward Power, P.E., of HDR, and Glenn F. Myers, P.E., of PBS&J, will serve as Project Coordinators, reporting directly to the Dr. Azizinamini, and will assist with management and coordination of task activities and operations among research team members. Mr. Power regularly visits HDR's Omaha office and has worked with Dr. Azizinamini in similar roles on previous research projects. Dr. Dennis Mertz will serve as the Project Quality Assurance Manager, monitoring conformance with the Quality Control Plan, and will report directly to Dr. Azizinamini. An overall Organization Chart is provided in Figure 3

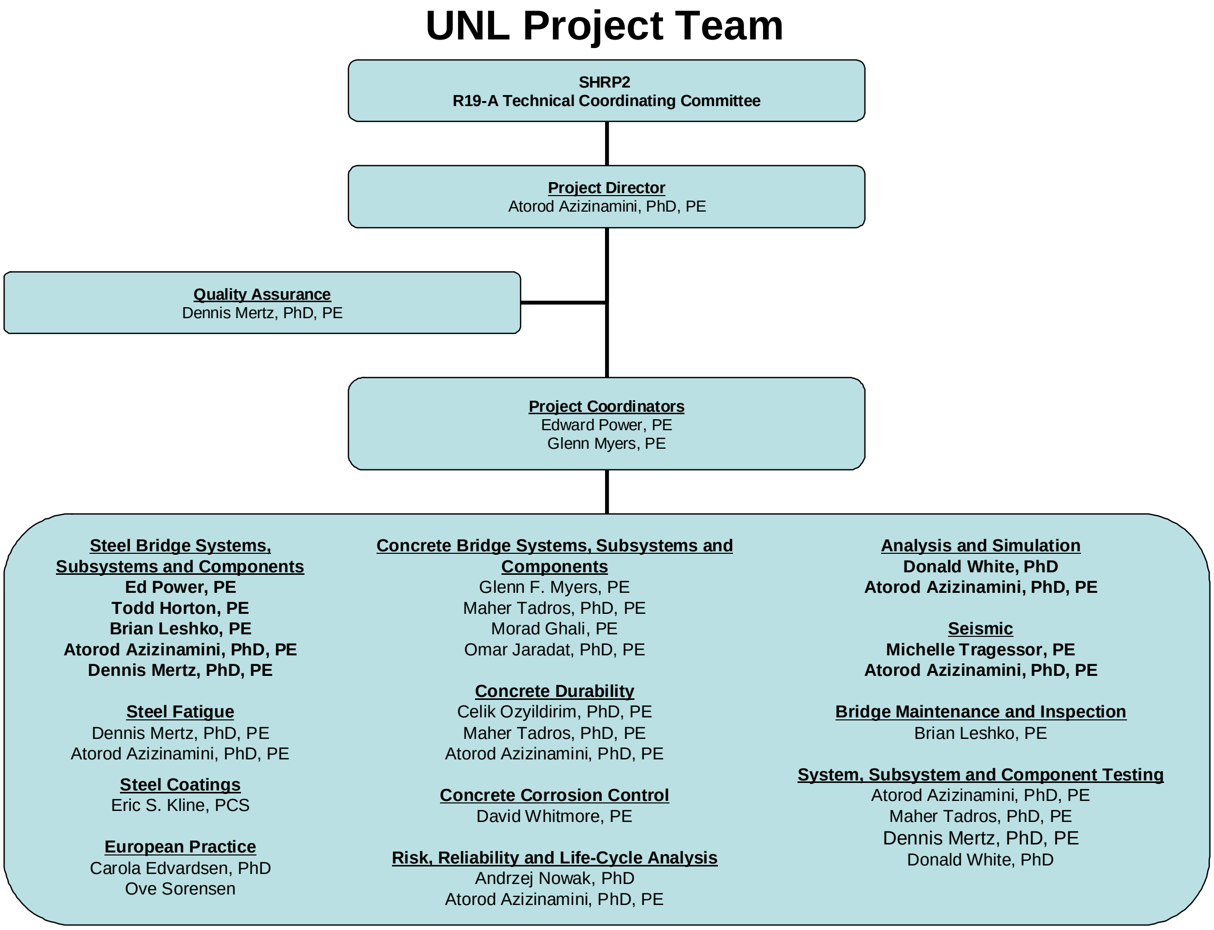


Figure 3. Project Organizational Chart

Dr. Azizinamini and the Project Coordinators will be responsible for developing and compiling the PMP. Dr. Azizinamini and Dr. Dennis Mertz will be responsible for developing the Quality Control Plan in accordance with accepted industry standards.

Our philosophy is to create a seamless project team that works smoothly and efficiently with SHRP 2 and TCC staff. Each research team subcontractor will have a designated single point of contact who will be responsible for managing all contractual and task activities performed by that team member firm, and who will be part of an overall Project Management Team (PMT). Upon execution of a Prime Contract Agreement with UNL, Subcontract Agreements will be executed with each of our team member firms. These Subcontract Agreements will clearly identify the roles, responsibilities and task requirements for each project team member. Each Phase and Task will have a clearly defined Scope, Schedule & Budget (SSB) component with defined objectives and deliverables. The PMP will also address all administrative and contract issues, including billing requirements and progress reporting.

B) Communication & Coordination Plan

This plan addresses our procedures for overall project communication, and outlines required project coordination, document control through the use of web-based systems, scheduled internal and external meetings and communication documentation.

The Project Management Team will meet regularly to address the project status. Overall project progress and individual Task progress will be carefully monitored on a monthly basis for all team members, and the PMT will assess any staffing or level-of-effort adjustments required. Corrective action will be taken as necessary to keep the project on target. In addition to monthly Progress Reports, detailed Progress Reports will be submitted quarterly with curves showing actual progress versus planned progress and budget utilization for each Task.

A major part of this plan will be a dedicated project web site, which will be the repository of all project technical documents, test results, project delivery schedule and status reports, project correspondence including e-mail, meeting notices and minutes. This secured web site will also be used to post internal, bi-weekly progress reports by research team members as well as for communicating ideas and findings. With this site, SHRP 2 and TCC members will have continuous access to view project progress on a daily basis. Another use of the web site will be to web-cast major activities within the project, such as experimental tests. This will allow the entire research team and TCC to observe these activities and to provide input, suggestions, etc.

Several additional items will be included in our communication plan to achieve an excellent level of communication and exchange of information among research team members and the SHRP 2 and TCC staff, including:

- Documentation of the control system, Project Wise, operating on the project web site, which will be used to track and monitor document development and to facilitate management of all documents.
- Communication protocol and uniform distribution lists, which will identify all Project and SHRP 2 and TCC team members, with mail addresses, phone, cell phone, fax numbers and e-mail addresses.
- Procedures for normal communication, which includes mail correspondence, telephone discussion, e-mail and fax communication.
- Scheduling for regular internal team meetings, web meetings and conference calls.
- Scheduling for meetings with SHRP 2 staff, which includes two (2) meetings in Washington, D.C. and one (1) at the University of Nebraska-Lincoln.
- Scheduling for meetings with SHRP 2 staff, which includes one (1) meeting in Washington, D.C. during Task 5.
- Scheduling for one meeting with the SHRP 2 Technical Coordinating Committee for Renewal Research (TCC), in Washington, D.C., Irvine, CA, or Woods Hole, MA.
- Scheduling for major project conference calls between the project team and SHRP 2 and TCC staff.
- Procedures for distribution of meeting and teleconference minutes, which will be prepared within five working days of all meetings.

C) Production Plan

This plan addresses production procedures for required Task and delivery packages, and identifies specific content and format requirements for designs, plans, specifications, testing, reports and other delivery documents. Consistent criteria will be set up and used by all team members. Specific elements in this plan will address requirements for individual Tasks.

D) Quality Assurance & Quality Control Plan

A specific QA/QC plan will be established and implemented at the start of the project, which will provide policies and procedures ensuring quality throughout the entire project duration. The Project Director and Quality Assurance Manager will be responsible for program quality and for the assignment of QC personnel. As part of this plan, all project reports and deliverables will be reviewed for accuracy and for compliance with project objectives and standards. Our approach to quality however, will not be just a review of final documents, but to build in quality throughout the entire process.

A major emphasis will be placed on ensuring that the project addresses the issues that are important to the project and reflects project objectives and TCC comments and recommendations. The Project Director will be responsible for achieving this goal, with assistance by the Project Coordinators.

4.2 Research Approach

This section (4.2) of the proposal provides the background and detailed description of the work to be conducted in this project. Figure 4 provides a brief list of major topics discussed in section 4.2.

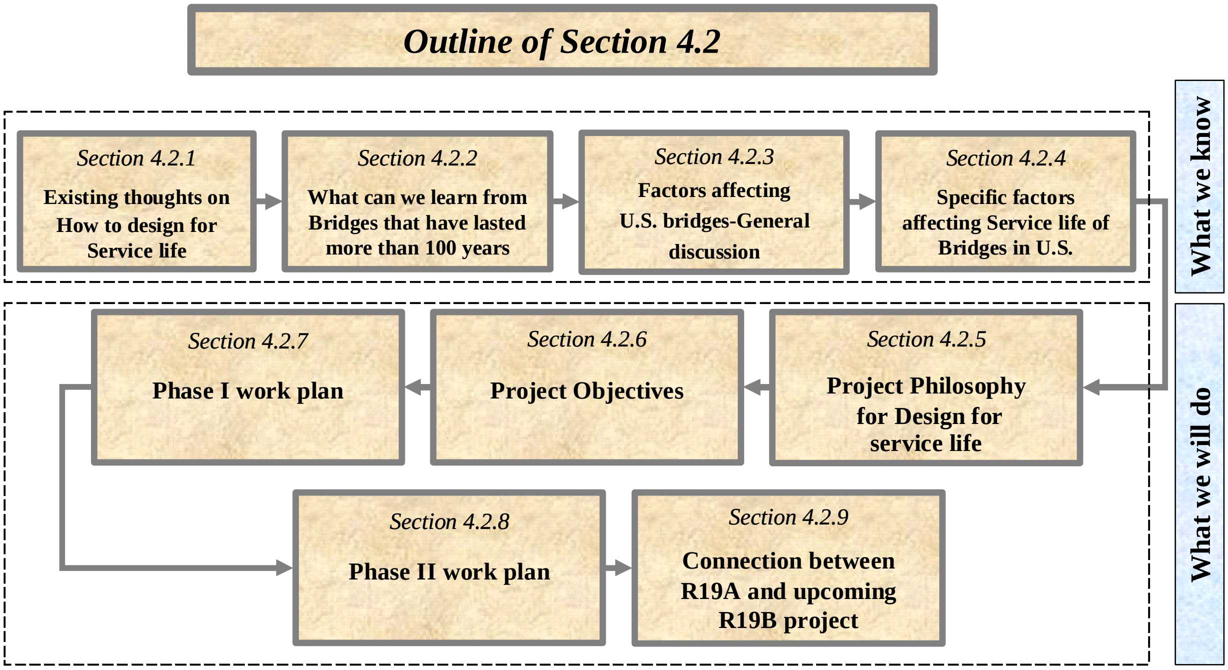


Figure 4. Outline of Section 4.2

Section 4.2.1 provides a brief summary of existing thought and philosophy related to design for service life. A good portion of this section is devoted to emerging philosophy in Europe for design for service life. There are elements of the European approach that could be adopted for U.S. bridges. At the same time, there are aspects that could be further enhanced and used in the U.S. And, finally, there are those aspects that are important to U.S. practice that are not addressed in the European approach.

Section 4.2.2 discusses observations pertaining to existing bridges that have provided service life of more than 100 years and that continue to be in service.

Section 4.2.3 classifies the factors that affect the service life of bridges in two major categories. These are discussed briefly.

Section 4.2.4 is extensive. It discusses factors that are considered important in providing long service life. Research team members are considered top experts in their respective fields. This section, written by a number of team members, represents the latest thinking and state of knowledge in each field discussed. For example, the materials presented under concrete durability, coating, corrosion, etc., are representative of U.S. practice first and secondly of a holistic view of the subject matter.

After presenting the current service life design philosophies and factors affecting service life, section 4.2.5 presents a general philosophy that will be used to achieve the project objectives, which are listed in section 4.2.6.

Sections 4.2.7 and 4.2.8 provide Phase I and Phase II work plans. For the sake of clarity, most of the background materials are presented in section 4.2.4 (factors affecting service life of bridges).

Section 4.2.9 provides an explanation of the linkage between R19-A and R19-B. To be clear and concise, some of the discussion related to connecting R19-A and R19-B is provided in Appendix A, located in Section 12 of this proposal. The materials presented in Appendix A represent significant thinking and new ideas.

4.2.1 Traditional Approaches for Designing Bridges for Service Life

The traditional approaches for service life used in various codes such as AASHTO ***LRFD*** *Specifications*, Eurocode, or British Standards are in an indirect form, specifying use of certain details such as cover thickness, crack width, concrete compressive strength, etc. In short, they are not quantifiable. Eurocode and British Standards assume 50 years for service life, whereas AASHTO ***LRFD*** is based on 75 years of service life. The specified details in these codes are mainly based on experience, field observations and limited research data. The approach taken by current codes does not allow predicting the expected service life of bridges. This philosophy makes the life-cycle cost analysis impossible, hindering the decision making process.

The current approach for durability design has another serious limitation. For instance, consider the concrete piers shown in Figure 5.

Figure 5. Importance of durability design of concrete

Each concrete pier shown could be divided into three different zones. The first zone remains submerged at all times. Another zone, which is located above the submerged zone, is subjected to dry and wet cycles as water elevations change. This is the most critical zone. Finally, there is a zone that always stays above the water level and is subjected to atmospheric conditions only. These three zones behave differently and require different approaches for detailing and design for service life, a requirement that current code approaches are not able to answer.

4.2.1.1 European Approach for Design for Service Life

The emerging European approach for design for service life is mainly concerned with durability of concrete elements within the structure. To date, the procedures are used mainly for bridges and tunnels.

During the preparation phase of the proposal, an extensive dialogue was carried out between the research team members and those responsible for developing the European model code (***fib*** Bulletin 34). The following sections reflect these dialogues.

A) History of the European Approach for Design for Service Life

Between 1996 and 1999, with financial support of the European Commission, a series of research studies were undertaken to develop a scientifically verified method to design and evaluate concrete structures for durability or service life. The project is referred to as the Dura-Crete Project as an abbreviation for "Probabilistic Performance based Durability Design of Concrete Structures." The project was led by COWI and included 12 partners, all from Europe. Dr. Carola Edvarrdsen, one of our research team members, was one of the original Dura-Crete Project team members. She is also the newest representative from Denmark to the ***fib*** Commission 5 -- Design for Durability.

Seven major tasks were undertaken between 1996 and 1999 under the Dura-Crete project. The following lists these tasks and gives a very brief description of their objectives.

- Task 1 -- Design Framework – The aim of this task was to develop a framework for conducting the entire project. It was carried out in two associated smaller projects.
- Task 2 -- Modeling of the Degradations – Under this task, two deterioration models were developed for corrosion of reinforcement in embedded concrete in non-prestressed elements. These models are related to carbonation and chloride induced corrosion.
- Task 3 -- Compliance Tests – The deterioration models developed under Task 2 contain several parameters that need to be determined by testing. Under Task 3, test methods were identified and conducted to obtain these parameters for many different concrete types. It is interesting to note that these studies provide a very critical viewpoint of similar tests we have in our ASTM publications.
- Task 4 – Statistical Quantification – Under this Task, the statistical analysis of data were carried out.
- Task 5 -- Benchmarking of the Conventional Design Methodology – The comparison of the results obtained using existing national European standard provisions and new approaches were the objective of Task 5.
- Task 6 -- Comparative Probabilistic Design Calculations – Calibration to obtain reliability target indexes was the objective of this Task.
- Task 7 -- Documentation of the New Design Methodology – This Task resulted in the development of a final report summarizing the entire work. (Brite Euram 2000)

Task 7 resulted in the publication of a final technical report that formed the basis for producing ***fib*** Bulletin 34, *Model Code for Service Life Design*. The bulletin was published in February, 2006 by Task Group 5.6 of ***fib*** and referenced in the RFP for this proposal.

Eurocode 2 is another document governing the design of concrete structures in Europe. Both the ***fib*** Bulletin 34, *Model Code for Service Life Design* and Eurocode 2 are specifications, or model codes,; it is up to the bridge owner to specify the document to be used in the design process. The major difference between ***fib*** Bulletin 34 and the Eurocode 2 is that the Eurocode approach is based on 50 years service life using approaches similar to AASHTO *LRFD Specifications* (deemed to satisfy design) whereas the ***fib*** Bulletin 34 specifies four different design approaches and is based on 100 years service life.

B) Brief Summary of, *fib* Bulletin 34 *Model Code for Service Life Design,* Approach for Durability Design

Four different approaches for service life design are identified in the ***fib*** Bulletin 34. These are:

- Full Probabilistic Design
- Partial Factor Design
- Deemed to Satisfy Design

- Avoidance of Deterioration

Figure 6 shows a flow chart representing the approach contained in ***fib*** Bulletin 34.

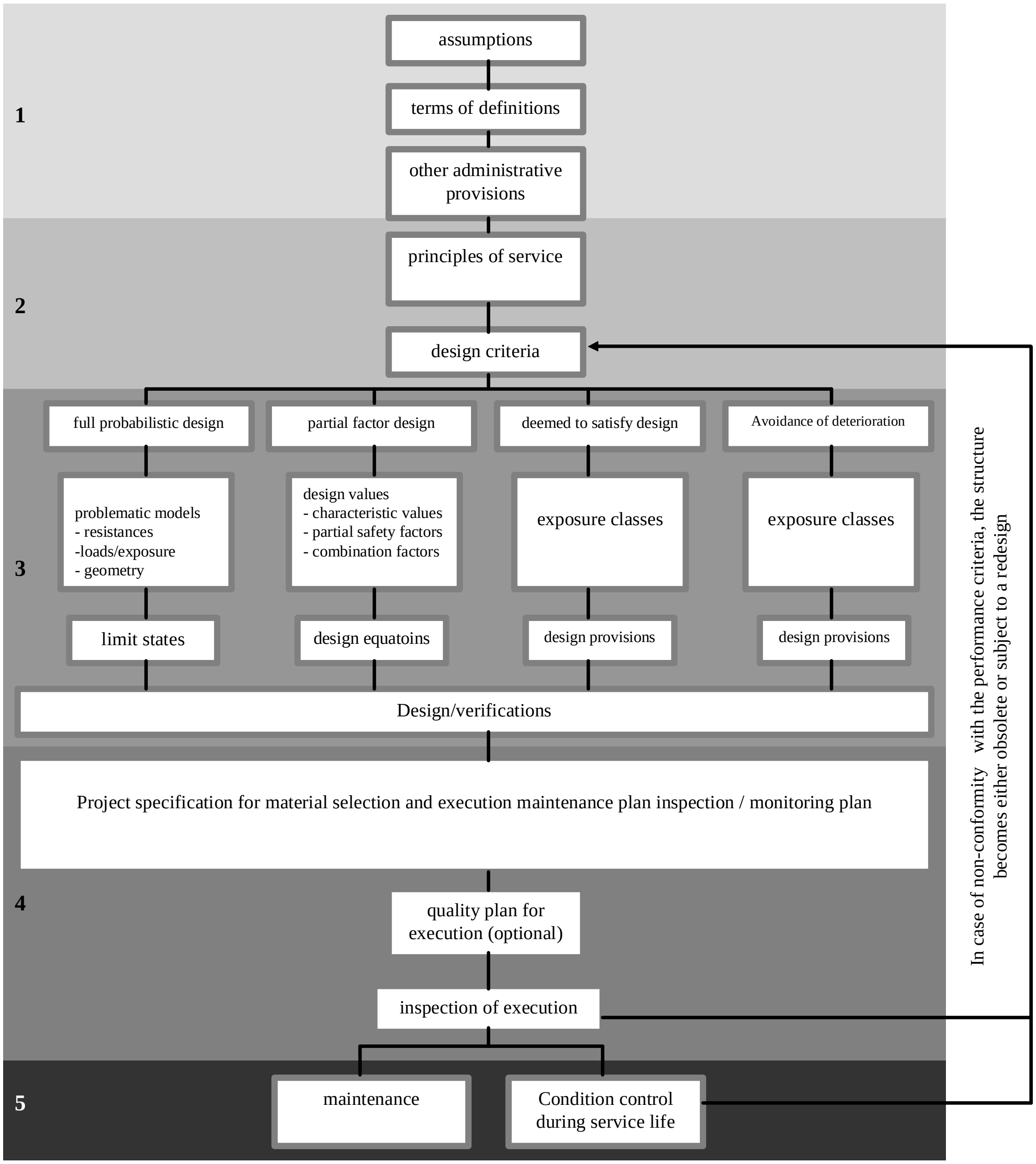


Figure 6. The *fib* Bulletin 34 approach for durability design in a flowchart format (CEB 2006)

The entire document is based on the philosophy that the major sources of induced damage to concrete structures are chloride- and carbonation-induced corrosion of the embedded reinforcement. It is, however, acknowledged that the sulfate attack, Alkali-Silica Reactivity (ASR) and Freeze and Thaw are other sources of deterioration in concrete. Chloride- and carbonation-induced corrosion are addressed through formulas (deterioration models) capable of predicting the time that it takes to start the corrosion process. Sulfate attack, ASR and Freeze and Thaw deterioration are prevented by using quality concrete and are addressed through the "Avoidance of Deterioration" (approach no. 4 listed above) approach.

Design for service life using ***fib*** Bulletin 34 can be carried out using a combination of the four approaches listed. Chloride- and carbonation-induced corrosion can be addressed by all four approaches, while sulfate attack, ASR and Freeze and Thaw are always addressed through the "Avoidance of Deterioration" approach by using quality concrete to prevent the existence of sulfate attack, ASR and Freeze and Thaw problems throughout the specified service life of the structure. The use of the "Avoidance of Deterioration" approach for sulfate attack, ASR and Freeze and Thaw is partly due to the fact that there are no deterioration models for them.

The major difference between the four approaches listed above in ***fib*** Bulletin 34 is the way the chloride- and carbonation-induced corrosion are addressed. The equations given for deterioration of concrete due to chloride- and carbonation-induced corrosion are similar in nature. They are based on diffusion theory and take into account the aging effect. As an example, the equation provided for chloride-induced corrosion takes the following form (CEB 2006):

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

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

The right-hand side of the equation requires several parameters that are related to chloride levels at various points in the cross section of the structure under consideration and the speed at which the chloride is penetrating through concrete (chloride diffusion). By inputting these parameters, equation (1) 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) is then compared to the critical chloride content, C_{crit} which is the value determined to be the point when corrosion starts. When the chloride level at a given depth, x, of the structure is reached, the Critical value, the service life of the structure, is assumed to be exhausted, as depicted in Figure 7.

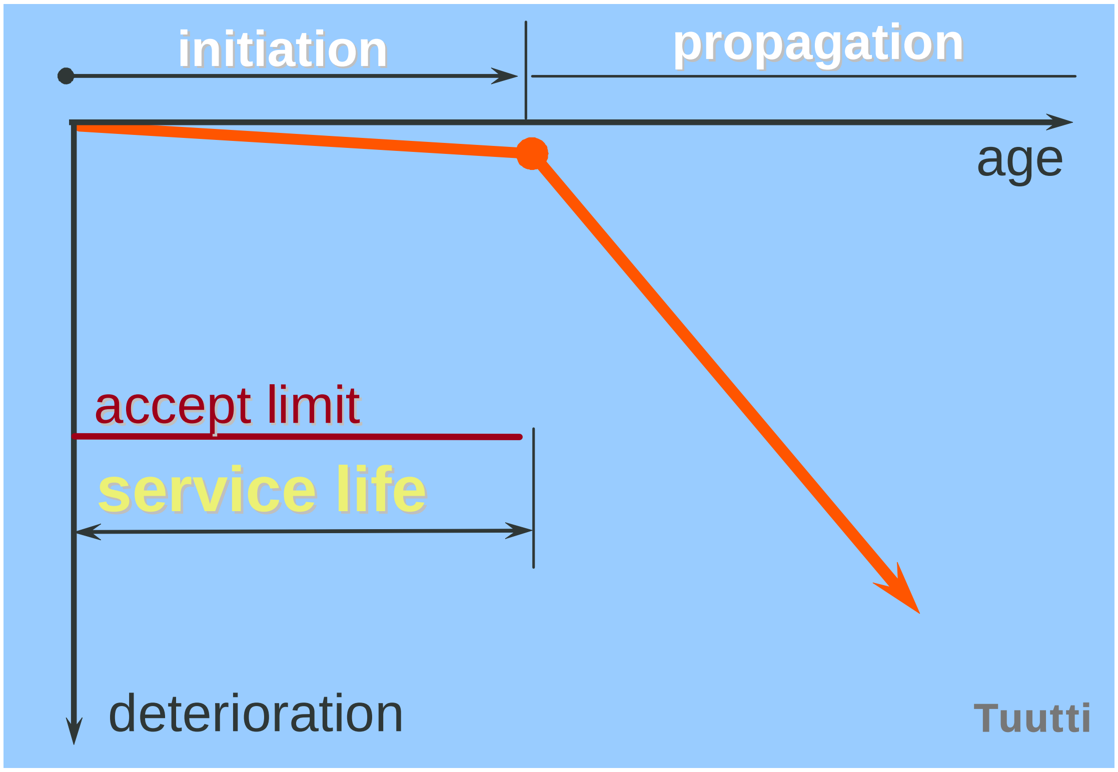


Figure 7. Relationship between damage and service life (Source COWI, Denmark)

The time, t, in equation (1) is the design service life in years. By assuming 100 for t, it is implied that chloride-induced corrosion will not be an issue for 100 years, at which time the process could start.

Use of Equation (1) to address the chloride-induced corrosion would constitute use of approach no. 1 or full probabilistic approach.

If chloride-induced corrosion is addressed through specifying concrete cover thickness, permeability, water cement ratio, etc., in a table format, then it constitutes using approach no. 3 or the “deemed to satisfy deterioration” approach. The traditional code approaches, such as that specified in the AASHTO *LRFD Specifications*, are examples of the “deemed to satisfy design” approach.

In the opinion of those responsible for developing ***fib*** Bulletin 34 (private communication with COWI), approach no. 2 or the partial factor design is similar in nature to approach no. 3 (“deemed to satisfy deterioration” design), if the specified design parameters in the table format (such as cover thickness, concrete permeability, etc.) are obtained based on testing and calibration.

Finally, use of the preventive measures, based on scientific approaches to completely eliminate chloride-induced corrosion would constitute use of approach no. 4 or the “avoidance of deterioration’ approach.

As explained previously, ***fib*** Bulletin 34 addresses chloride- and carbonation-induced corrosion by using approaches no. 1, 2 3 or 4, while deteriorations due to sulfate attack, ASR and Freeze and Thaw are completely avoided by using approach no. 4 or the “avoidance of deterioration” approach (CEB 2006).

C) Limitations of *fib* Bulletin 34, *Model Code for Service Life Design*

fib Bulletin 34 is mainly concerned with service life of bridges as related to concrete durability. Other factors affecting service life of bridges such as expansion joints, bearings, coating, etc., are not addressed to the same level. Furthermore, the deterioration models are only provided for damage to concrete structures due to chloride- and carbonation-induced corrosion. Even for concrete structures, other causes such as sulfate attack, Alkali Silica Reactivity (ASR) and Freeze Thaw are addressed by a requirement that they should be prevented all together. This decision by developers of ***fib*** Bulletin 34 is based on the conclusions that no reliable deterioration models could be developed for the sulfate attack, ASR or Freeze and Thaw.

Another important factor that is not taken into account in developing ***fib*** Bulletin 34 recommendations is that the load effects are not considered. For instance, fatigue caused by dynamic loading leading to time-dependent material degradation, corrosion fatigue developed by simultaneous action of corrosion and environmental factors, etc., are not addressed in ***fib*** Bulletin 34.

4.2.1.2 Other service design approaches

There are lessons to be drawn from other engineering applications, such as durability of pavement design. Following is a brief description of some of the available philosophies that could be examined in developing design criteria for service life (ARA 2004).

A) Mechanistic-Empirical Design Approach

The design of new and rehabilitated pavements during the past 50 years has relied on sound empirical procedures that have been improved incrementally over time. The design methodologies in all versions of the AASHTO Guide for Design of Pavement Structures are based on the empirical performance equations developed using the AASHO [sic] Road Test data from the late 1950’s (AASHTO 1993). The conditions at the road test included one environmental condition, a limited number of axle weights, tire pressures and axle configurations and only 1.1 million axle load repetitions from which empirical design equations were developed. Today, it is common to design pavements for tens of millions of truck passes, including widely varying climatic conditions, highly variable axle weights, tire pressures and different axle configurations. New materials have also been introduced to pavement construction. To overcome these limitations, further improvements depend on a new generation of design tools that combine the knowledge and experience gained from empirical procedures with the real-time effects of traffic loadings, environmental factors and engineering materials.

A.1 - Theoretical background of mechanistic-empirical pavement design guide

The Mechanistic-Empirical Pavement Design Guide (MEPDG) provides a hierarchical methodology with three levels of design ranging from Level 1 (detailed project specific inputs) to Level 3 (default regional inputs).

Mechanistic-empirical (M-E) design combines the elements of mechanical modeling and performance observations in determining required pavement thickness for a given set of design inputs. The mechanical model is based on elementary physics and determines pavement response to wheel loads or environmental condition in terms of stress, strain and displacement. The empirical part of the design uses pavement response to predict the life of the pavement on the basis of actual field performance.

In essence, M-E design has the capability of changing and adapting to new developments in pavement design by relying primarily on the mechanics of materials. For example, M-E design can accurately examine the effect of new load configurations on a particular pavement.

A.2 - Design approach

The design approach followed in MEPDG and a typical flowchart is summarized in Figure 8 and Figure 9. The process takes inputs of structure, materials, traffic data and climate data. Structure data includes information such as layer and slab thicknesses, reinforcement and joint spacing. Material data includes material properties pertaining to all materials present including underlying soils, base courses, slab and any overlay. Traffic data includes truck types, speeds, axle configurations and weights. Climate data can include temperature, precipitation, humidity, and ground water level and fluctuation.

Using the structure, material, and loading information, a structural analysis is performed to determine the stresses and strains induced throughout the structure. Note that the loading is not necessarily due to traffic only. Environmental factors, such as thermal and moisture gradients, can create loads which result in stresses and strains to be combined with those due to traffic. This phase is the mechanistic phase of the design process.

The amount of damage imparted on the structure due to the resulting stresses and strains can now be assessed. Damage is related to stress and strain through the use of transfer functions. These transfer functions are obtained through experimentation and modeling. This is the empirical portion of the process.

Pavement damage is a cumulative process. As damage occurs, the properties of the pavement change as well. Therefore, the previous steps must be performed in an iterative manner. After the damage has been assessed, the properties are updated and the stresses and strains are re-evaluated.

As the iterative procedure converges, the pavement performance is assessed and parameters such as rutting and fatigue are compared with chosen failure criteria to determine if the design is satisfactory. If not, the initial parameters and geometry can be modified and the design repeated until the failure criteria are satisfied.

The values obtained represent the average predicted value. That is to say there is equal probability that the actual observed results will be greater than or less than those predicted. Therefore, a final input parameter is specified, which is the desired reliability.

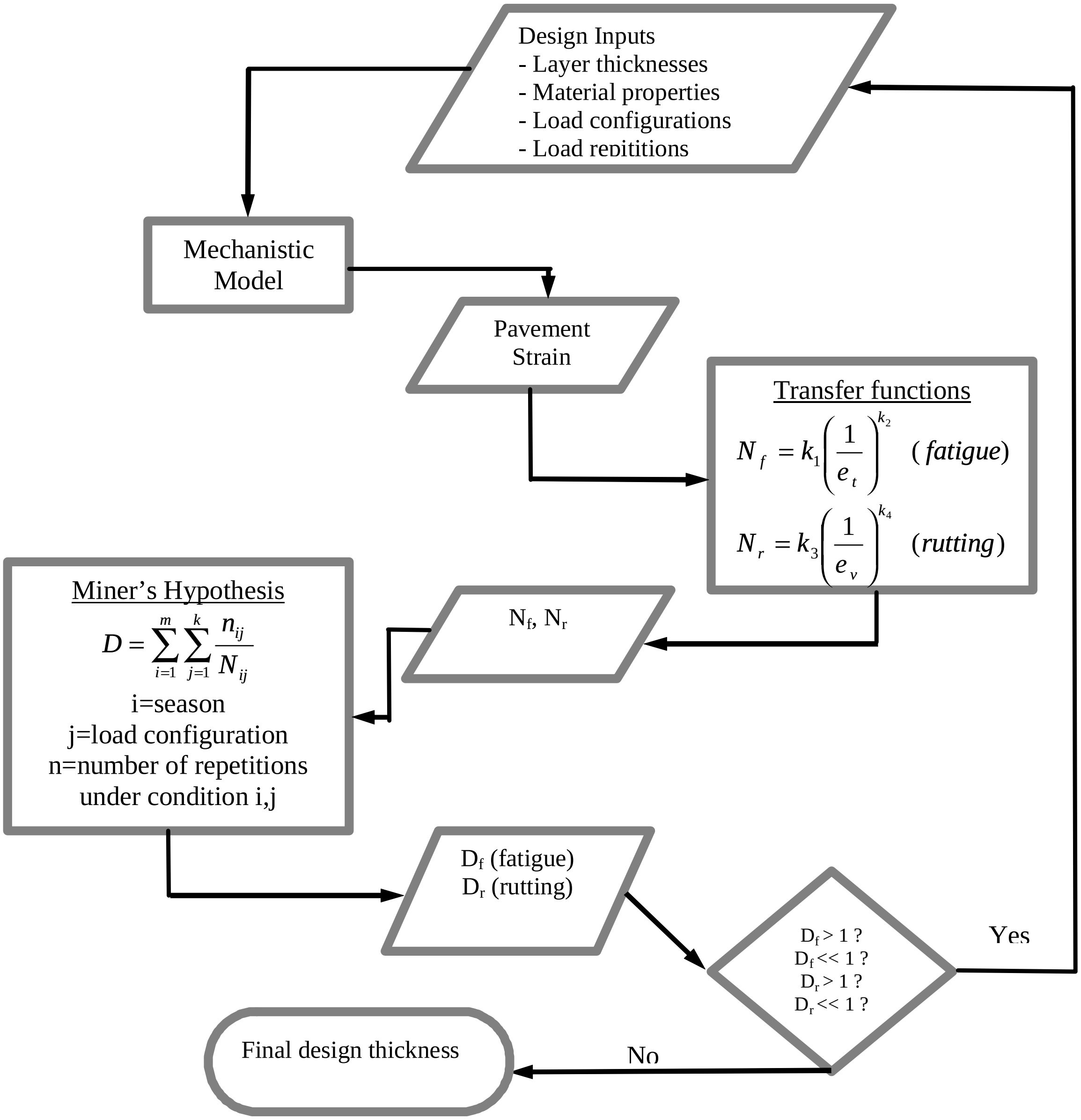


Figure 8. Mechanistic-empirical design framework (Timm 2004)

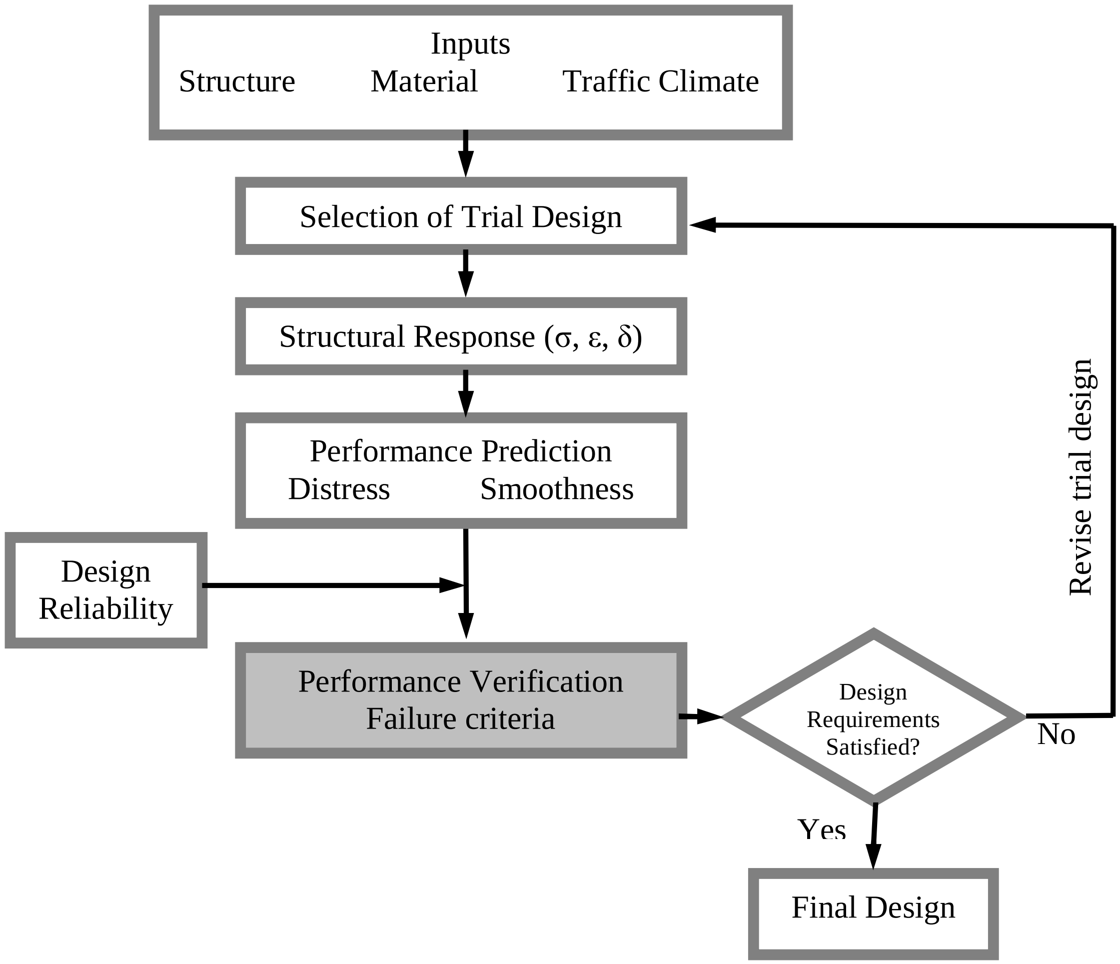


Figure 9. Mechanistic-empirical pavement design framework (Khanum 2005)

4.2.2 Lessons from Bridges that Have Provided Service Life of Beyond 100 Years

Some highway bridges have exhibited over 100 years of service life with no end of service in sight. 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 cost,
- adaptable to changes in functional use as well as service limit state demands and/or
- originally over-designed.

Steel bridges provide some of the best examples of long-lived bridges since there is a longer history of steel-bridge construction in the United States. Reinforced concrete bridges, especially concrete arch bridges, have also demonstrated long service lives. (It is noted that the first prestressed concrete bridge in North America, the Walnut Lane Memorial Bridge in Philadelphia, was completed only in 1951.)

Good examples of steel 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 10 shows the Brooklyn Bridge n 1890.

Figure 10. 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 1988, 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 (NYCDOT) determined that the Williamsburg Bridge should be repaired while it was kept open to traffic. This option was deemed to have the least detrimental impact on motorists and nearby communities. In 1991, the NYCDOT 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. It prepared the bridge for another 100 years of service to the City of New York. Figure 11 shows the Williamsburg Bridge in 1904.

Figure 11. The Williamsburg Bridge, circa 1904

The decision to rehabilitate the Williamsburg Bridge instead of 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 brought about by a poor corrosion-protection choice by the original designer. For the Brooklyn Bridge, its famous designer, John Augustus Roebling, chose a coating of graphite to protect the individual wires of the bridge cable from corrosion. (This choice proved to provide over 100 years of corrosion protection.) 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 relatively poor. However, the cable design utilized a factor of safety of resistance divided by load of about 5. After the significant loss of section observed in 1988 due to corrosion, 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 over-design 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 12 shows the Eads Bridge in 1983. The scale of the bridge was unprecedented: the more than 500-foot span of the center arch exceeded by some 200 feet 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 12. The Eads Bridge looking toward St. Louis and the Gateway Arch, circa 1983

An interesting feature of the history of the Eads Bridge is 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. The last train crossed the bridge in 1974. By the early 1990s, traffic on it had dwindled to about 4,000 cars a day. In 1991, the Eads Bridge was closed. For a while, it was unused altogether. But in 1993, the bridge found new uses. MetroLink, the region's new light rail system, began to use the lower deck, which originally served passenger and freight train traffic. Finally in 2003, the upper deck opened again to buses and automobiles. 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 year old bridges illustrates that for bridges to serve a long life, they must be:

- Maintainable (and subsequently maintained) or relatively maintenance-free
- Adaptable to changes in traveled-way cross section and usage, and
- Designed with growth and change in mind. This seems to be a new bullet or is it relative to being “over-designed.” Otherwise…see next bullet.
- Where did “over-designed” go? See list on p. 23 above.

4.2.3 Assessing Present Conditions of Bridges: Factors Affecting Service Life

Assessing the present condition of bridges and comprehending the factors that influence service life of bridges are essential to designing and building durable bridges. The factors affecting bridge service life typically fall into two categories, functional and structural. The UNL Team, with a balanced make-up of practitioners and researchers, has the background and experience to address the factors affecting bridge service life in both categories.

4.2.3.1 Functional Service Life Factors

The functional issues relate to the bridge's continuing ability to provide safe capacity and are primarily a function of traffic growth and the need for additional lanes or other system improvements to accommodate growth. Many bridges have been modified or replaced well before their expected service life because of required corridor widening or interchange reconfiguration to accommodate increased and unplanned traffic demands. Figure 13 illustrates an example where the interchange accessing Fort Lauderdale/Hollywood International Airport was dismantled and replaced to provide access to a new terminal and rental car facility. Many bridges are also replaced because the facility they cross must be widened or modified. These factors and their probability for occurrence should also be considered when establishing priorities for service life, whether it is 50 years, 100 years or more.

Figure 13. De-construction of Segmental Bridge, in Florida, due to Funtional Obsolescence

Widening of bridges can be easily accommodated and is structurally addressed by the LRFD Bridge Design Specification requirements for exterior beams and substructure design. Interchanges can be designed to accommodate future widening on primary routes by ensuring flyover ramp piers are located to provide required physical space. Similar consideration should be given to the roads and waterways under the bridges as well. These planning modifications, however, result in some additional cost and their implementation may be dictated by the financial decisions relating to the most effective use of available funding.

4.2.3.2 Structural Service Life Factors

Structural issues relate to the condition of individual bridge components and their continuing ability to safely carry loads. They are a function of material longevity and proper detailing and are affected by various loading and environmental factors. Figure 14 is an illustration of most of the factors that affect structural service life.

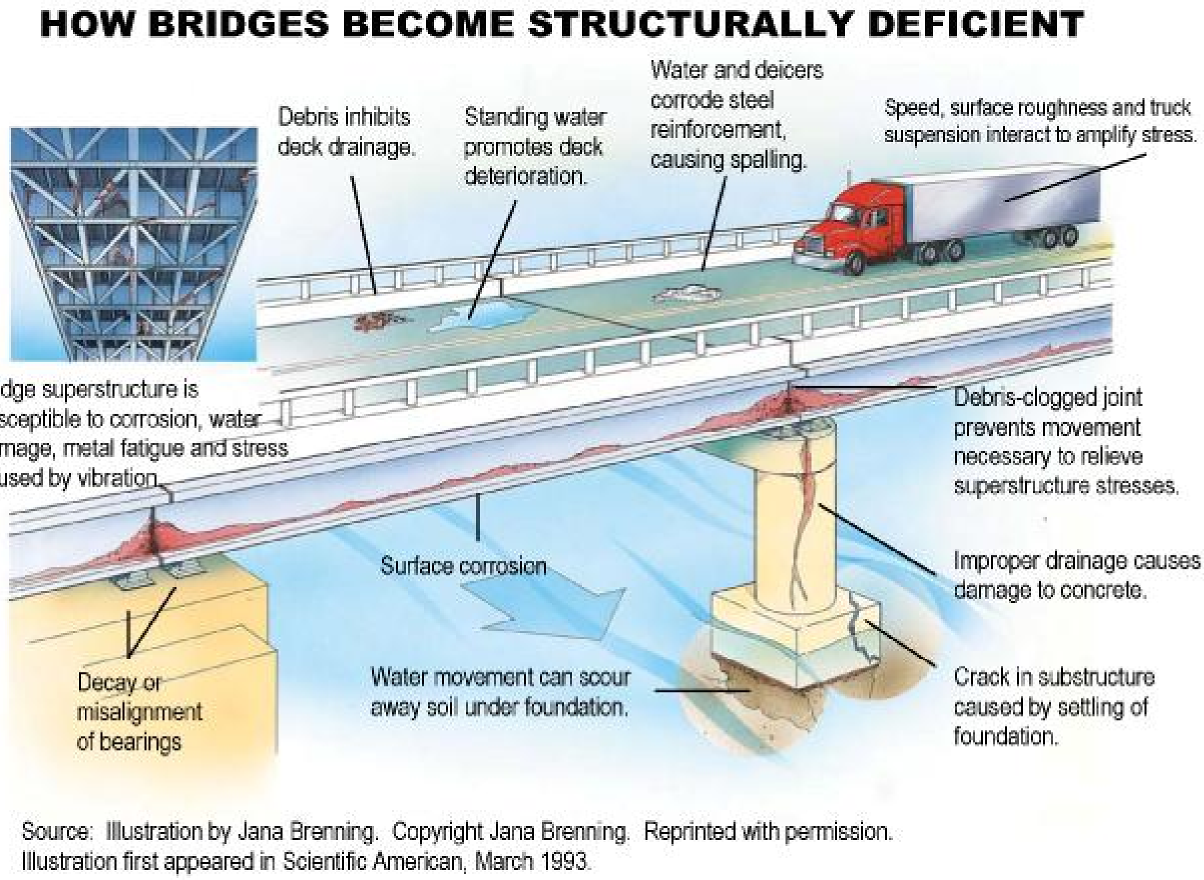


Figure 14. An illustration of the factors that can affect structural service life (From FHWA report to the Committee on Transportation and Infrastructure, United States House of Representatives, Given on September 5, 2007.)

A study of bridge service life, while reviewing bridges that have lasted for 100 years and more, should also be concentrated on elements of bridges that have not lasted their design life and on ways to prolong their functionality through the use of improved materials and/or details. Most of these bridge maintenance issues have solutions that can allow the service life of new and existing structures to be extended. The prioritization of implementing these solutions should be based on the probability that the structure will have a functional life that justifies the added cost for the increased service life.

The UNL Team's combined bridge experience plus preliminary research conducted with various state departments of transportation has established a list of typical bridge elements that consistently require maintenance. The following sections provide discussion on these elements.

4.2.4 Discussions on Important Factors and Issues Affecting Service Life of Bridges

Reviewing the existing status of the bridges in the U.S. with respect to factors and issues affecting service life sets the stage for defining the details of the scope of the work that should be conducted under various tasks.

Figure 15 summarizes typical bridge systems and components that affect overall structural service life and consistently require maintenance. The list provided in Figure 15 represents the UNL Team's experience with bridges combined with preliminary research and contact with various state departments of transportation.

The following sections provide more detailed discussion of each of the bridge systems and elements listed in Figure 15. To aid navigation, Figure 15 is repeated at the beginning of each section with the appropriate box expanded to show additional detail. At the end of each discussion presented below, conclusions are made with regard to subjects needing further research in order to enhance the service life of bridges.

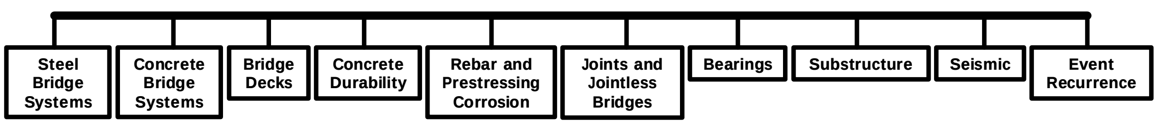


Figure 15. Typical bridge systems and components can affect overall structural service life

4.2.4.1 Discussion on Steel Bridge Systems and Factors Influencing their Service Life

Figure 16 provides a list of factors that will be discussed in this section and that are related to the service life of steel bridges.

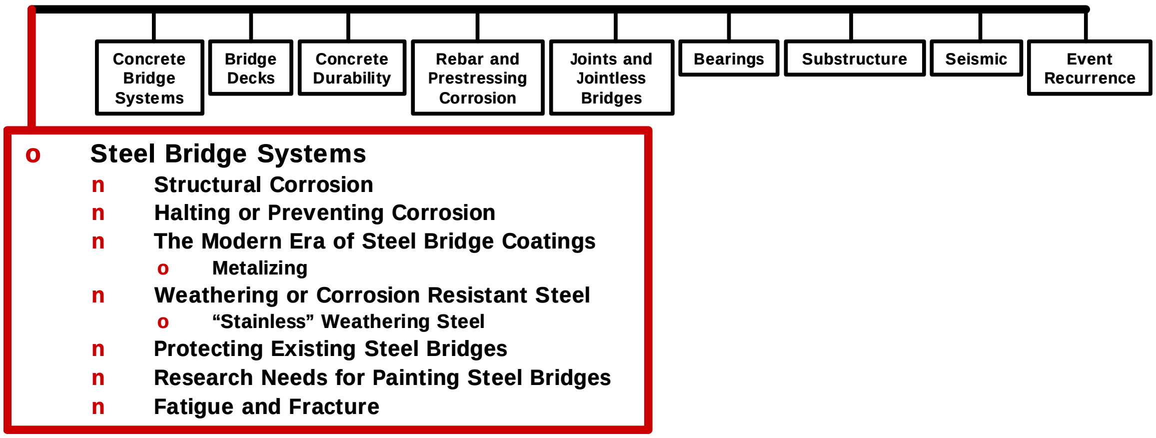


Figure 16. Steel bridge systems

The most common steel bridge systems today are composite multi-girder deck systems using either rolled beams, plate girders or tub girders. These systems can be simple span or multi-span. Multi-span systems are usually fully continuous for dead load (DL) and live load (LL), but new systems, typically with spans up to 150 feet, have been introduced with a "Simple for DL and Continuous for LL" concept. Haunched, steel plate girders have been used for spans up to about 500 feet. Spans up to 400 feet are typically designed with parallel flanges. Trusses, arches, cable-stayed and suspension types are also used for long span applications. Figure 17 shows an I-girder steel bridge. Steel bridges have the potential for service life well over 100 years, but are affected by a number of environmental and loading factors, and are subject to component and detail adverse effects.

Figure 17. Typical I-girder steel bridge

The major causes of reduced service life in steel bridges are corrosion and fatigue and fracture. Each of these topics is discussed in more detail in the following sections. The presence of leaking roadway joints has been the major cause of steel bridge corrosion.

Fatigue and fracture performance is mainly related to tensile stress ranges and types of details. Research studies have resulted in the development of better details and design approaches that have greatly reduced the potential for fatigue and fracture of steel bridges.

The current trends in steel bridge design, especially for short span bridges, are to use less connecting details in the bridges. This approach has several advantages. It reduces the initial cost, results in fewer items to be inspected and is easier and less costly to maintain. An example is the “Simple for DL and Continuous for LL” system. Appendix B, located in Section 12 of this proposal, provides a detailed description of a bridge that uses this system in conjunction with adjacent beam technology. A bridge using the concept presented in Appendix B is designed and will be let in May 2008 in Nebraska.

A) Structural Corrosion

Corrosion is an electro-chemical reaction or oxidation resulting in loss of material. It is a process that occurs in the presence of oxygen and water and is exacerbated in the presence of other chemical elements such as chlorides. Figure 18 shows corrosion at a connection of a steel bridge. The presence of constant wetness allows corrosion to continue unabated.

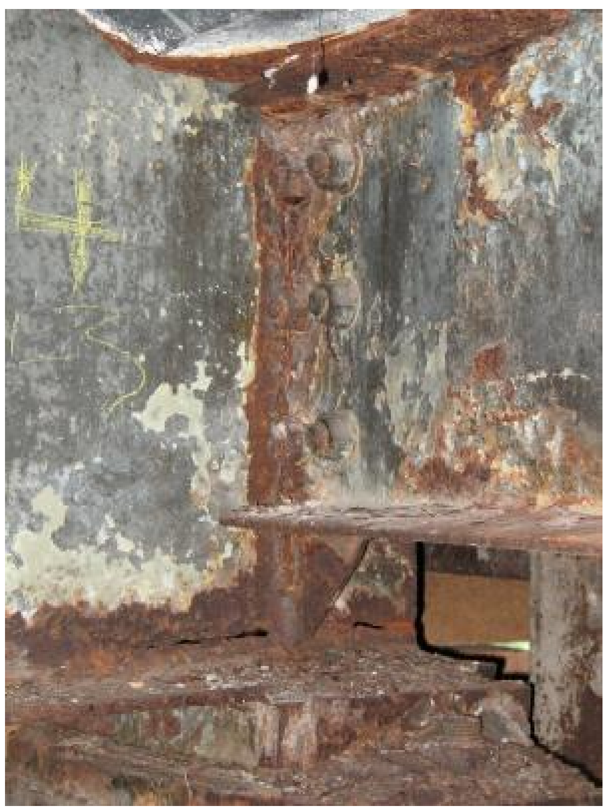


Figure 18. Corrosion at a connection of a steel bridge

The application of chlorides can occur in a number of ways, whether from roadway de-icing materials from above or those which settle on steel surfaces from natural sources such as a seacoast environment. Traffic adjacent to or beneath a structure, especially on grade separations, causes the deposition of de-icing materials from below because of the “rooster-tail” effect behind fast-moving traffic, particularly large trucks.

Other contaminants can increase corrosion rates as well but chlorides are doubtless the most egregious corrosion threat to steel bridges.

A structure can be exposed to a relatively benign macro environment and still have localized micro environments, such as the area beneath scuppers or areas beneath expansion joints where corrosion is rampant. The micro environment in which key elements such as bearings and expansion joints reside, directly affects the overall service life of the structure.

B) Halting or Preventing Corrosion

Knowing the causes of corrosion, the best way to halt or prevent it is to protect the steel from various contaminants such as oxygen, salt and moisture. Deck joints should be minimized or eliminated wherever possible because deck joints eventually leak and create a bath, often laden with salt, for the steel beneath. Coatings are and have been the primary protection system to avoid corrosion in steel bridges, and coating systems have evolved over the years. These systems will be addressed in more detail in the following section.

On each bridge subject to painting and in need of long-term corrosion protection, drainage areas, splash zone areas, fascia beams and other outboard members exposed to windblown debris and sun should be considered worthy of special attention. Likewise, bottom flanges are subject to having debris deposited on their horizontal surfaces and can be exposed to chloride attack because of the deposition of wet contaminants from traffic beneath. Both vertical and lateral clearance is required to provide ways to keep the bridge structure separated from traffic and the related splash and spattering associated with debris propelled by traffic. Finally, gratings, bearings and curbs all represent complex surface areas that are difficult and inefficient to paint in the traditional manner. These detail areas are often galvanized, metalized or fabricated from an inherently corrosion-resistant material such as, stainless steel or plastic.

Finally, corrosion resistant steel can be used but these materials are not effective against corrosion when subjected to chlorides. This will also be discussed in following sections.

C) The Modern Era of Steel Bridge Coatings

Prior to 1970, paint systems consisted of red lead or basic lead silico-chromate pigment in an alkyd or linseed oil binder which could be applied under a wide range of circumstances from very dry to slightly damp. The coating adhered well to mill-scale-covered surfaces and when re-coating was required during periodic repainting cycles, the coating could easily be recoated with an additional layer of the same coating. After decades of exposure to the elements, the alkyd and linseed oil binders would begin to chalk, erode and generally deteriorate. When these “binder” materials deteriorated to the point where the cohesive strength of the coating was exceeded by tensile forces, spontaneous delaminating of the coating was known to occur.

The lead- and chromate-pigmented coatings were inexpensive and were able to be applied to surfaces which had been minimally prepared or cleaned. The required cleaning consisted of only hand or power tool cleaning, which allowed intact mill scale to remain on the steel surface.

Around 1970, after over 100 years of using lead and chromium compounds for carrion prevention/minimization, many state highway departments adopted a completely new approach to protecting steel from corrosion. The approach included much more expensive surface preparation to completely remove mill scale and entailed the application of a multi-coat "high performance" coating system. Shop painting required the use of labor-intensive, dusty open-nozzle blast cleaning or the acquisition of expensive centrifugal blast cleaning equipment. Cleaning costs alone equaled the entire as-applied-coating-cost of the earlier systems.

Except for the fact that the use of lead and chromium pigments has disappeared, current bridge coating technology has actually changed very little between 1970 and today. The introduction of the three-coat system consisting of a zinc-rich primer and two additional coating layers has carried through to today. Protection of the steel substrate is based on the use of a zinc-rich primer and two additional layers of paint aggregating 8 to 12 mils.

The use of tiny metallic zinc particles in intimate, metal-to-metal contact with the steel substrate creates an anode-cathode relationship in which corrosion takes place at the zinc anode. Zinc is sacrificed in a slow reaction which produces harmless by-products and the steel is protected. The zinc corrosion by-products can often impede further corrosion by filling in the scratch. The two coating layers applied over the zinc provide a "blanket" to protect the zinc from the atmosphere, thereby extending the ability of the zinc to provide long term protection to the steel beneath. The layers that are used in the topcoats are chosen for their ability to resist the destructive effects of sunlight, for aesthetic reasons, for the bridge owners' need for a pallet of colors and often for their compliance with air quality regulations, etc. These topcoat materials have evolved and currently (in 2007), the systems commonly in use overlay zinc with an epoxy mid-coat and polyurethane or polyaspartic topcoat; zinc with an epoxy mid-coat and a fluoropolymer topcoat; and zinc with an acrylic mid-coat and topcoats. Some systems consisting of three or more water-borne acrylic coating layers are used in milder service areas. There are even some systems consisting of lead- and chromate-free oil/alkyd coatings. Other systems involve the use of multiple coats of calcium sulfonate-containing materials. Calcium sulfonate/calcium carbonate pigments are alkaline and prevent corrosion by increasing the alkalinity at the steel/coating surface interface.

The service life of high-performance systems using a zinc-rich primer is quite often 30 years before the first touch-up. Many structures coated in the early 1970's are in very good condition after 30+ years and are only now approaching their first touch-up painting. This is true in the northern tier of states that heavily use roadway de-icing materials and in coastal marine areas as well. In the middle latitude and the inland areas of the southern states, the corrosion mitigation afforded by these excellent "high-performance" materials can be considered permanent protection with overcoat application needed occasionally due to color fading, chalking of exposed members or damage from wind or traffic propelled debris.

The metallic zinc particles in the "zinc-rich primer" will protect the steel from corrosion indefinitely. Its service life is limited only by actions which scratch or otherwise pierce or abrade the zinc layer and which cause the zinc to react in order to protect the steel beneath. As long as the layers of coating over the zinc-rich primer are maintained, the true service life of the system based on actual field service life has yet to be reached and is considered permanent.

Usually after 20 to 30 years, it is necessary to apply some touch-up repairs to extend the service life of the high-performance topcoat and to protect the zinc layer beneath. A service life of greater than 100 years should be achievable by using any of the promising topcoat systems currently in use. However, achieving 100 plus years of service life will require some periodic level of field maintenance.

C.1 - Metalizing

Another effective method to prevent corrosion also consists of the application of zinc or a zinc-aluminum alloy to steel surfaces, but the material is deposited by different methods dubbed metalizing or thermal spray. The thermal-spray coatings (TSCs) are used extensively for the corrosion protection of steel in a wide range of environments. Corrosion tests carried out by the American Welding Society and in reports of the LaQue Center for Corrosion Technology confirm the effectiveness of flame-sprayed aluminum and zinc coatings over long periods of time in a wide range of hostile environments. The British Standards Institution code of practice for the corrosion protection of steel specifies that TSCs give protection for greater than 20-years-to-first-maintenance in the 19 industrial and marine environments considered. Only sealed, sprayed aluminum or zinc gave such excellent long-term protection in both sea water immersion and splash zone exposure.

Figure 19. "Metalized" truss bridge over rail yard in South Dakota to minimize maintenance

Finally, in Federal Highway Administration laboratory and field trials of low VOC coatings for the protection of steel bridges, 85/15 Zn/Al, 99.9 Zn, and 99.9 Al TSCs demonstrated the best corrosion performance among 34 coating systems evaluated.

Another advantage is that metalized coatings contain zero volatile organic compounds (VOC).

Thermal spraying is actually a group of processes in which metals of thermal-spray feedstock material are heated to a molten state, atomized and propelled by a compressed gas stream, usually air, to the substrate.

The particles strike the substrate, flatten and form thin platelets (splats) that conform and adhere to the surface. The molten splats produce metallurgical interactions with the substrate and to each other after impact with the substrate. These localized reactions, in effect, form minute weld spots with very good cohesive and adhesive strengths.

TSC's provide protection to steel in the same manner as the zinc-rich paint coatings described above. The zinc or zinc alloy, in intimate metal-to-metal contact with the steel, becomes the anode in the electronic coupling of the two metals. In the presence of moisture, corrosion occurs at the (zinc) anode.

Thermal sprayed coatings provide superior corrosion protection to steel in severe exposure circumstances.

D) Weathering or Corrosion Resistant Steel

Another approach to providing corrosion resistance for steel bridges is the use of weathering steel. Weathering steel has limited corrosion by virtue of the formation of an iron oxide "rust" patina. This stable patina effectively resists further oxidation. Weathering steel also was introduced at about the same time as the change in 1970 away from lead- and chromate-containing paint.

Figure 20. The New River Gorge Bridge (above) in West Virginia is constructed of Weathering Steel

In the late 1970's, however, it was noted that some bridges employing weathering steel, that had areas that were constantly wet and not allowed to dry out, experienced heavy corrosion. Likewise, areas that were subject to salt corroded very heavily and the protective stable "patina" was not able to form. Moratoria on the use of weathering steel were issued and, as a consequence, uncoated weathering steel was not generally used on steel bridges for several years. Figures 20 and 21 show bridges incorporating weathering steel.

Figure 21. These steel girders are fabricated from Weathering Steel

The FHWA Technical Advisory T5140.22 "Uncoated Weathering Steel in Structures" provides guidance to the states for developing their own policies regarding the use of weathering steel (FHWA 1980). This document includes a digest of the primary benefits and concerns regarding weathering steel and provides specific guidance on its appropriate use. The steel industry has made the point that weathering steel, when properly applied, results in a structure that provides both first cost and life-cycle cost savings. A recent FHWA-sponsored study indicated the median cost of shop coating is approximately 7% of the cost of the fabricated girder. With the recent dramatic movement in steel prices, it is difficult to define the premium paid for weathering grade versus carbon steel. Historically, however, the premium has been around 4%.

Assuming that all bridge expansion joints will eventually leak, current guidelines require weathering steel bridge elements to be painted at non-integral beam ends to a length of 1-1/2 times the girder depth. In addition, weathering steel girders are shop blast cleaned to remove mill scale so that the initial protective oxide layer is uniform. These requirements offset some of the potential cost savings associated with weathering steel versus painted steel.

There is a growing body of data taken directly from the performance of bridge structures with weathering steel. The following conclusions are taken directly from the pertinent data:

1. Weathering steel requires some amount of moisture and wet-dry weathering cycle over a period of time to develop a tightly adherent, protective oxide layer. However, excessive moisture and the presence of salt will disrupt this process and result in a structure that corrodes at an unacceptably high rate.
2. Weathering steel will corrode at varying rates from structure to structure and element to element. Corrosion is a complex phenomenon relying on both micro and macro environments, the temperature and specific concentration of contaminants and the specific surface structure of the individual piece of steel. Engineers should expect corrosion rates to occur over a broad range, even in similar situations. For this reason, it is improper to look only at average rates of corrosion. Credence must be given to data from the higher end of the range, for this is where the first potential failure lies.

What is known is that weathering steel provides significantly enhanced resistance to general corrosion rates versus conventional Grade 50 steel in virtually all highway bridge environments.

It is also clear that weathering steel suffers unacceptable rates of corrosion in areas where salt, debris or moisture accumulate on steel surfaces. Many of the failures cited in the literature would seem to be attributable to design details which allowed the accumulation of such salt or debris or the mis-application of weathering steel in locations that simply were not appropriate for its use.

D.1 - "Stainless" Weathering Steel

A new grade of "weathering" steel has emerged in recent years. This "stainless" grade of steel, ASTM A1010, is a 12% chromium stainless steel. The material is described by its supplier as a dual-phase stainless steel product which, when compared to weathering, painted or galvanized steel, has life-cycle cost advantages that permit its effective use. From a corrosion perspective, A1010 is said by its supplier to have roughly seventeen percent less corrosion in sea water compared to regular A588 steel--2.6 mils per year (mpy) vs. 3.1 mpy. Tests at Kure Beach demonstrated that the A1010 steel coupons tested for four years in the 25 meter location had negligible corrosion when compared to A36 mild steel and A588 weathering steel. The use of the A1010 grade of stainless steel appears to address many, if not all, of the problems previously noted when using weathering steel. The development of this new material appears to offer a ray of hope in the battle against corrosion on steel bridges.

E) Protecting Existing Steel Bridges

Problems that are associated with existing bridges are, in a word, related to lead paint management. Since the decision in the early 1970's to discontinue the use of lead-based paint, engineered maintenance painting has become the issue of the day. In the presence of lead-based paint, the logical strategy is to maintain the coating on the structure in a serviceable condition as long as possible to delay the inevitable expensive removal and replacement of the lead-based paint. Lead-based paint has performed very well for over 100 years and the decision to repair or replace it is a crucial one. In and of itself, the removal of lead-based paint is relatively easy. The difficult and expensive part is containing the blast cleaning debris, while protecting both the workers and the environment. Besides removing lead paint, contaminants, etc., removing mill scale is a major cost factor as it sharply limits the productivity of the blast cleaning operator. This makes it a much more problematic and expensive operation both in the fabrication shop and in the field.

The use of an engineered maintenance painting approach means going against-the-grain of traditional thinking. A bridge owner's strategy has often been to wait until the paint on the bridge is beyond repair and the bridge is in poor visual condition prior to repainting. The use of an engineered approach to maintenance painting will often dictate touching-up the paint before it needs to be removed on a wholesale basis. In effect, it means painting the structure before it needs painting so that the areas that are corroded can be managed and minimized. In so doing, the service life of the lead-based paint that is in-place can probably be extended by decades in many cases. Ultimately, lead-based paint does not have to be removed. Its removal can be delayed for years until it is very thick, poorly adherent and perhaps rampant with corrosion. At that point, the paint will possibly be literally falling off and efforts to repair it will be counterproductive.

The development and employment of an engineered approach to the maintenance and repainting of existing steel structures is something that is not routinely undertaken. The tools by which the inventory of bridges can be identified and the condition of each can be measured and assessed, including the identification of "micro-environments", are already in place. Engineered maintenance painting to address smaller areas of the structure while maintaining the condition of the coating on larger areas of the structure is technology that is developed but that has not been applied on a widespread basis.

F) Research Needs for Painting Steel Bridges

Coating durability is always the first issue that comes to mind. One can ask, "If paint is so good and lasts so long, why is it that we have to repaint every 8-10 years?" The reality is and has been quite different for a long time. Since the coating revolution in the 1970's, there have been literally hundreds of thousands of tons of steel bridges painted with the state-of-the-art coating system of the 1970's, consisting of a zinc-rich primer followed by two additional coats of paint. These bridges are now approaching 30-40 years of age. They are, as a rule, in good condition. Most need some touch-up of the coating and perhaps a skim coat topcoat and they will be able to easily last another 40 years before needing attention again. At that time, the zinc-rich primer will be 80 years old and will still be intact and protecting the steel substrate. One hundred years of service will be beckoning.

There are several crucial areas for further research:

1. Removing chloride from steel surfaces. The biggest single reason that coatings fail when highway bridges are re-coated is the inability to remove chlorides from the steel surface prior to re-coating. The removal of the chlorides from the steel is a crucial and major bridge issue.
2. An inexpensive, easy-to-apply, long-lived coating. Lead and chromium were previously the corrosion-inhibitive pigments of choice because they were both effective and inexpensive. Research should be undertaken to find or develop a new material that is both effective and affordable.
3. Implementing a two-coat system. Significant savings could occur with the development of a fast-curing, two-coat paint system that could be applied over blast cleaned steel and which would deliver the same protection to the steel surface as would be obtained using a three-coat system. According to a recent FHWA study, the elimination of one coat of paint will reduce the fabrication cost of a new steel bridge by 3%. In 2004, a two-coat paint system was approved by the North East Protective Coatings Committee (NEPCOAT).
4. Developing a one-coat system. The cost of protecting steel could be halved by developing and using a one-coat system. As yet, however, no suitable one-coat material has been tested or approved. There is a pending pooled fund research study awaiting funding (TPF Study 924 Single Coat).
5. Determining the need to remove mill scale and to what extent removal should be undertaken. While it is acknowledged that in the steel/mill scale electronic couple, steel becomes the anode and should therefore corrode, there is a great deal of "practical" project evidence to suggest that its complete removal is overkill in many mild service

environments. If mill scale were allowed to remain, and a fast curing, effective two-coat (or one coat) paint system was applied, the cost of protecting the steel could be reduced by 75%.

6. Thermal Spray Coatings have shown that they provide excellent long term corrosion protection yet their use is not widespread. This proven technology is available and could be employed readily. Research efforts should be invested to investigate its possible widespread use as a means of protection for steel bridges where superior corrosion protection is critical. While currently costly, it is believed that costs can be reduced by an increase in the volume of use.

G) Fatigue and Fracture

Fatigue of metal structures is the steady-state propagation of pre-existing flaws due to repetitive loads below the critical loads for strength. Steel bridges do not fail in fatigue, but in fracture, which is the unstable propagation of a larger flaw (most likely the result of fatigue).

Fatigue of steel structures is categorized in the AASHTO *LRFD Specifications* as load- (6.6.1.2) or distortion-induced fatigue (6.6.1.3). Bridges designed in accord with the *LRFD Specifications* should not experience fatigue crack propagation during the specified 75-year design life. To extend the 75-year design life to 100 years, the number 100 should be substituted in LRFD Equation 6.6.1.2.5-2 in place of the number 75, as suggested in the commentary to Article 6.6.1.2.5.

Load-induced fatigue cracking is typically cracking normal to the primary stress direction in main members. This cracking can quickly propagate to a flaw size that is susceptible to fracture under heavy loads and during cold periods, jeopardizing the load-carrying capacity of the bridge. Distortion-induced fatigue cracking is typically cracking parallel to the primary stress direction due to secondary stresses which results in serviceability problems. Loss of load-carrying capacity does not occur unless ignored and not retrofitted for a long period of time.

The basic current load-induced fatigue provisions were originally introduced in 1974 in the AASHTO *Standard Specifications for Highway Bridges* with only minor revisions to date (with a significant rewrite in 1994 for insertion in the *LRFD Specifications* but with little change in design results). Thus, steel bridges designed since 1974 should not experience fatigue cracking in the first 75 years of service.

The distortion-induced fatigue provisions, consisting of a catalog of mandated details to minimize distortion-induced fatigue stresses -- mostly connection details between transverse and longitudinal members, were introduced with the first edition of the *LRFD Specifications* in 1994. Thus, steel bridges designed since 1994 in accord with the *LRFD Specifications* should not experience distortion-induced fatigue cracking if common details are used.

Bridges designed prior to 1974 which have experienced load-induced fatigue cracking have required significant emergency repairs, far short of reaching 100 years of service life. The federally-mandated biennial bridge inspection is not an adequate safeguard against fracture from load-induced fatigue cracking, as the chances of discovering load-induced fatigue cracks prior to unstable brittle fracture are slim. Load-induced fatigue cracks propagate relatively slowly; when they are of a size observable by the naked eye, relatively little life remains. In some cases, observed fatigue cracking or an unacceptably low calculation of remaining fatigue life has informed a decision to replace a bridge.

As an example, in the Lafayette Street Bridge over the Mississippi River in St. Paul, Minnesota, the entire bottom flange and the vast majority of the web of one girder of a two-girder system fractured on May 7, 1975. The bridge was not replaced but a repair replacing a significant length of the web and bottom flange was made.

For bridges like the Lafayette Street Bridge, which were designed prior to the establishment of modern fatigue provisions, retrofitting techniques including UIT (ultrasonic impact treatment) can be applied to details when a refined remaining life calculation suggests low remaining fatigue lives. (In many cases, crude remaining fatigue-life calculations yield low or negative lives due to crude estimates of stress along with the high degree of uncertainty associated with fatigue resistance.)

Fracture resistance is specified in the current LRFD *Specifications* by material toughness requirements that vary with certain climatic temperature zones. High Performance Steel has a very high toughness characteristic that provides a safer margin against fracture. It allows more time to detect existing fatigue cracks before they can propagate to a point where brittle fracture can occur.

4.2.4.2 Discussion on Concrete bridge Systems and factors influencing their Service Life

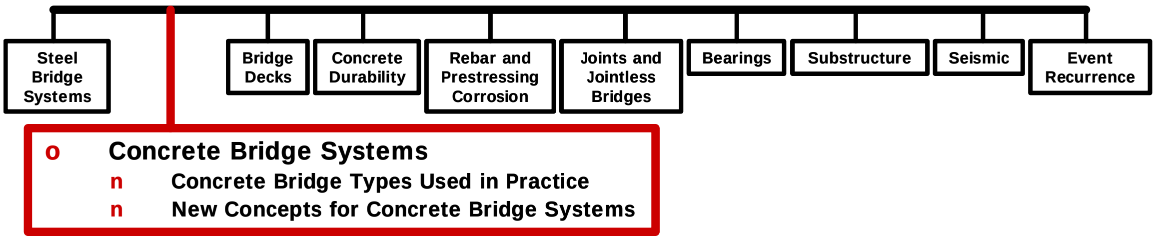


Figure 22. Concrete bridge systems

A) Concrete Bridge Types Used in Practice

The most commonly used concrete bridge superstructures in the U.S. are:

- Cast-in-place slab bridges.
- Precast stringer bridges. Of the stringer bridges, the most common ones are the I-girder and adjacent box-girder systems.
- Segmental post-tensioned concrete box girder bridges, used most commonly in Florida, Texas, Arizona and several other states. They are further divided into cantilever construction and span-by-span construction bridges.
- Cast-in-place post-tensioned box girder bridges, dominant in California and Tennessee.

Other less frequent systems are the arch, cable-stayed and suspension bridge superstructures.

Cast-in-place slab bridges are mostly of the conventional reinforced concrete type. They commonly span less than 50 feet in three or more span arrangements. Another type of cast-in-place concrete slab bridge is the haunched, post-tensioned system used in Kansas. This system can have a span of up to about 100 feet.

Figure 23. Modern concrete segmental bridge (Memorial Causeway Bridge, Clearwater, Florida)

Slab bridges, especially those that are conventionally reinforced, can deteriorate with time. Causes of concrete deterioration include alkali-silica reaction, shrinkage, temperature change and support displacement (due to settlement, scour, etc.). When concrete cracks and the roadway surface is subjected to wet-dry and freeze-thaw conditions, concrete further deteriorates and steel corrodes.

However, full-scale load testing of concrete slab bridges in the field by Azizinamini shows that some older slab bridges are capable of resisting close to 10 times the AASHTO HS-20 truck load, despite the relatively low ratings assigned to them by standard inspection methods (Azizinamini et al., 1994A; Azizinamini et al., 1994B). Slab bridges are not as easily modified for functional obsolescence as other systems. They are normally used for secondary roads on short spans. In most situations, where the bridge width or span lengths are to be modified to remove functional obsolescence, the total superstructure is removed and replaced.

Precast I- and box-girders are the second-most commonly used concrete bridge superstructures. Solid and voided slabs fall into the box-girder category. These girders are made of high performance plant produced materials and are generally very durable. I-girders, with a relatively thick composite deck slab, behave distinctly differently from adjacent box-girders, which are generally not covered with cast-in-place composite slabs. In a bridge system consisting of I-girders with composite cast-in-place slabs, the deck slab generally deteriorates more rapidly than the girders. It is not uncommon to expect removal and replacement of the slab about every 20-25 years while the girders are in near new condition. Many of the girder bridges built in the U.S. end up being replaced every 40-50 years due to functional obsolescence. When this issue is not factored in, it may be reasonable to conclude that the life expectancy of precast concrete girders, as designed and built today, is lower than 100 years. It is the total system that needs to be improved. Moisture and de-icing chemicals damage the deck throughout the bridge length, but seldom penetrate deep enough to create any damage to girder tops. Most of the deterioration takes place at the joints. Although, joints are covered elsewhere in this proposal, it is worth mentioning here that girder ends deteriorate in bridges built as simple spans with details allowing water to leak at girder ends.

Figure 24. Multi-span precast I girder bridge, made continuous for Live Load.

Expansion joints, usually placed over pier supports in long bridges, are also a cause for girder (and pier) deterioration.

Web end cracks are common in pre-tensioned girders. They are caused by the splitting stresses perpendicular to the longitudinal direction of the prestressing. NCHRP Project 18-14, lead by Dr. Tadros, is currently tasked with investigating repair of these cracks to assure preservation of member durability (see Figure 25).

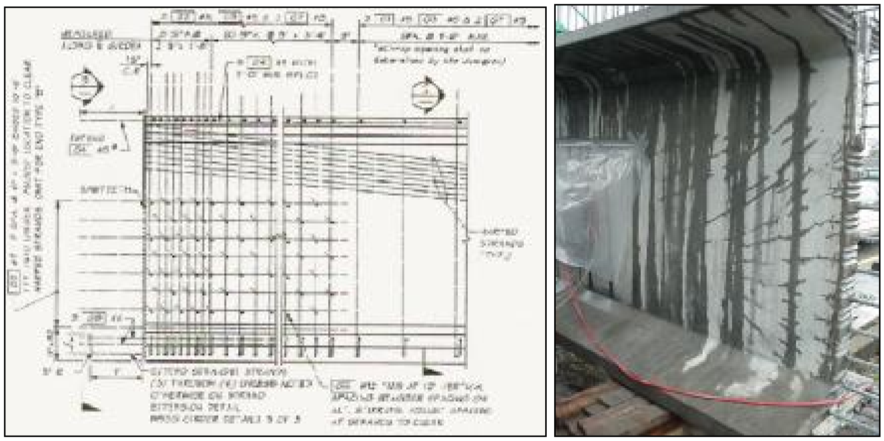


Figure 25. End zone reinforcement detail used by WA State DOT

One of the most commonly used precast, prestressed concrete girder systems is the adjacent box superstructure. This system is expected to gain popularity as pressure mounts to expedite construction and to minimize field forming and placing of concrete. Most of the deterioration of these bridges occurs due to water and chemical leakage into the joints between the girders (see Figure 26). It is possible that water also leaks into the voids inside the boxes. Besides creating a potential for deterioration, corrosion and freeze-thaw damage, accumulation of water could create additional weight unaccounted for in design.

Figure 26. Example of adjacent box girder deterioration due to moisture effects

Figure 27 illustrates a failed box girder in December of 2005. Figure 28 gives another example of an adjacent box-girder failure, one which took place very recently. Both failures were sudden with little deformation prior to collapse. The railroad girder in Figure 28 was seven foot wide, as opposed to the standard four-foot wide highway girder. The adjacent girders in the railroad bridge had no connection. Highway bridges generally have transverse post-tensioning of varying degrees.

Experience with adjacent boxes tends to lead to the thought that transverse design and detailing of adjacent box girders have not been adequately addressed. A possible solution to assure full continuity of boxes in the transverse direction is to follow a system used in Japan and more recently in Korea. Figure 29 shows a conceptual detail developed by the research team. It eliminates the need for transverse post-tensioning, a detail which requires specialty subcontractors in most locations. It also does not require field forming. However, the larger joint and amount of field placed concrete required may reduce its attractiveness. Other alternatives for connecting the boxes should be investigated and developed for skewed bridges to allow for general use.

Figure 27. A collapsed exterior girder in an adjacent box-girder bridge (December, 2005)

Figure 28. Failed interior girder of an adjacent box-girder railroad ridge (August, 2007)

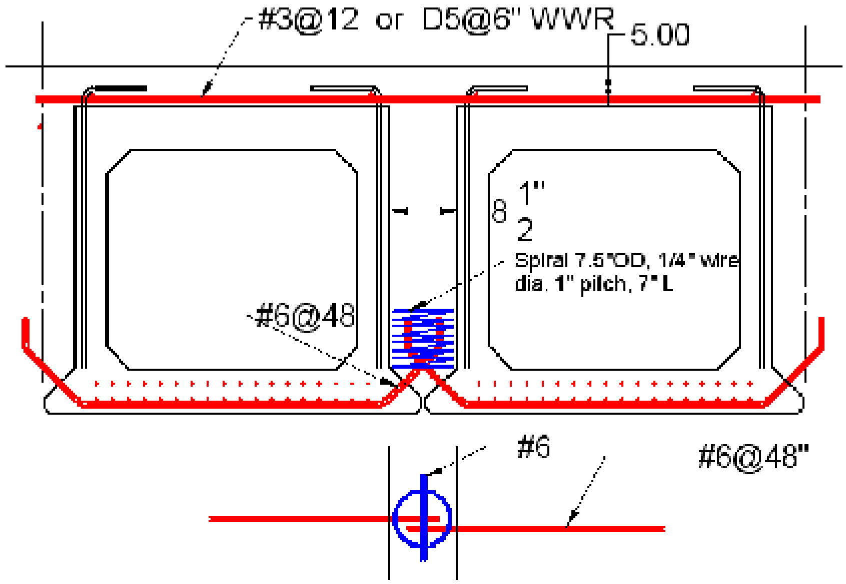


Figure 29. Possible modification in adjacent boxes to eliminate longitudinal joint leakage

B) New Concepts for Concrete Bridge Systems:

To modify current concrete bridge systems for improved service life, the following design elements are recommended.

- Add post-tensioning (PT) to keep the super-structure under compression (spliced girder systems).
- Add PT in adjacent boxes and change the details to assure full transverse direction continuity. (See the PCI Bridge Design Manual, Section 8.9) (PCI 2005).
- Add threaded rod continuity (Nebraska/Alberta/Illinois/Florida) to create full continuity for deck weight and to eliminate potential creep restraint cracking at the piers.
- Create an I-girder with an integral precast deck (the so-called deck bulb tee in the Northwest and Midwest).
- Use new high-performance concrete with corrosion-resistant reinforcement (epoxy, galvanized, stainless steel, carbon fiber, glass fiber or aramid reinforcement).

4.2.4.3 Bridge Deck Systems

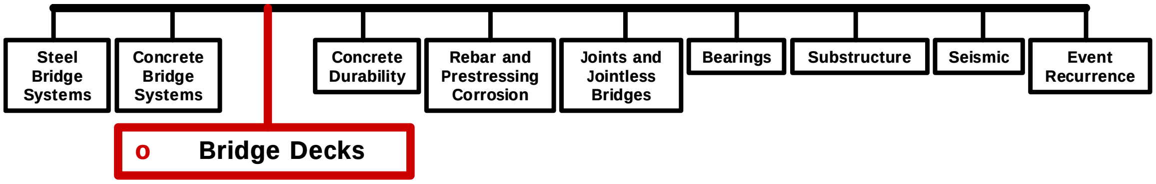


Figure 30. Bridge Deck Systems

Most of the bridge decks in current use are made of cast-in-place reinforced concrete. Other systems include orthotropic steel decks, exodermic decks, fiber-reinforced polymer decks, and precast concrete decks. A major focus of this study will be on conventional concrete decks, either cast-in-place or precast. Additional bridge deck systems will also be considered.

Cast-in-place decks are reinforced with mild steel reinforcement. They are generally 7.5-inch to 9-inch in thickness and are not initially topped with overlays. In most regions of the U.S., deterioration of bridge decks has been a major cause of bridge deficiency. There appears to be no unified method of sizing and detailing the reinforcement. Some owners use the AASHTO *LRFD Specifications'* Empirical Method which generally subscribes #5 bars at 12-inch spacing for the bottom mat and #4 bars at 12-inch spacing for the top mat in each direction. The "Strip Method" of the AASHTO *LRFD Specifications* generally yields larger steel quantities. Previously, the AASHTO *Standard Specifications for Highway Bridges* yielded yet a third set of reinforcement requirements. Despite these variations, there does not seem to be significant effect on the extent of cracking or the degree of deterioration. There seems to be a line of thinking that more steel in the deck provides for more corrosion opportunity and more degradation. Extensive research in Canada has focused in the past 15 years on "steel free" bridge decks. In reality, there are steel straps connected to the top flanges of the stringers, but below the deck. There are more recent recommendations in which the bottom straps are supplemented with fibers in the deck or fiberglass bars as the top mat, to "control temperature and shrinkage cracking."

A number of measures have been taken to control corrosion of the mild steel bars in the deck. A minimum top concrete cover of two inches is required by AASHTO *LRFD Specifications*. Generally, 2.5 inches of cover is used to allow for ½ inch of wearing over the life of the deck. Concrete cover is recognized as an effective method of protecting the steel from corrosion. However, if the deck has extensive cracking, the chlorides might penetrate to the steel though cracks.

Another method of extending deck life is to use epoxy-coated bars. Most highway agencies have now converted to decks with epoxy coated bars. Florida is one exception; it has been found in that state that damage in the epoxy surface creates a higher concentration of corrosion. South Carolina has recently adopted, and Virginia is contemplating, a similar policy.

There is no agreement relative to the effectiveness of galvanized bars in concrete. Early work has indicated that the high PH of fresh concrete before it hardens consumes some of the coating. However, design could allow for enough coating thickness for the expected life of the deck beyond this hydration process. It is possible, therefore, for galvanized reinforcement to be an effective method of corrosion protection.

It appears that there is no definitive direction for cast-in-place mild steel corrosion protection. Obviously, removing all corroding materials from the deck concrete by placing steel straps under the deck or by using fiber reinforced plastic bars, stainless steel bars, etc. would eliminate the problem. This needs to be further synthesized and developed to provide workable design and detailing recommendations, including secondary direction (traffic direction) design and detailing and rail-to-deck connection detailing.

The primary cause of the cracking of cast-in-place decks almost immediately after setting of the concrete is the fact that a composite deck is locked in with the supporting girders as soon as the deck concrete sets. Yet, the hydration cycle dictates that the concrete temperature continues to drop until equilibrium with the ambient air is reached. The shortening due to the temperature drop, combined with early concrete shrinkage (especially if the deck is not cured properly), is resisted by a much stiffer underlying deck system. The result is a set of equal and opposite forces: tension in the deck and compression in the girders. The deck concrete is still relatively young and unable to resist a significant amount of tension. It cracks to relieve itself. It has been observed that a reduced bond of epoxy coated bars, as compared to uncoated bars, contributes to larger cracks at wider spacing. This is perhaps one of the reasons decks with epoxy bars are perceived to have more cracking than older decks with uncoated bars. Regardless, one should focus on the causes of cracking, which are primarily differential temperature and shrinkage effects after the deck has become composite with the girders.

Phase-constructed bridges usually experience longitudinal and transverse cracking in the closure pour (Azizinamini 2002). Public pressure and demand for uninterrupted traffic flow requires that some bridges be constructed using the phase-construction approach. In this method, half of the bridge is constructed (Phase I), while the old half carries the traffic. Next, the old half is demolished and replaced with a new bridge (Phase II) while the newer half (Phase I) carries the traffic. Finally, the two segments are joined together using the closure pour. In some instances, the second segment (Phase I and II) is directly attached to the first segment without a closure pour.

One of the characteristics of phase-constructed bridges is that transverse and longitudinal cracks form near where Phase I and II are joined together. Formation of these cracks in most cases has to be anticipated. It has been shown that water can penetrate through even very small crack widths. Because of this fact, contractors should not use devices that are prone to formation of cracks near construction joints. Using such a practice would drastically increase the chances of having a severe corrosion problem in the mechanical splices. Figure 31 shows the condition of a phase-constructed bridge using a mechanical coupling to join the reinforcing bars of Phases I and II.

Figure 31. The condition of a phase-constructed bridge. The photo shows water seeping through the longitudinal cracks located at the interface of Phase I and Phase II. Additionally, closely spaced transverse cracks are also formed in the Phase II portion (Photo by Azizinamini).

A well constructed bridge of the above type can perform very satisfactorily if detailed correctly. In phase-constructed bridges, the transverse cracks are expected to form mainly on the Phase II side. This is because of the fact that concrete in the Phase I portion is already set and hardened when the Phase II portion of the slab is cast. The fresh concrete in Phase II has an inclination to shrink; the hardened concrete in Phase I, however, restrains this movement. As a result, the concrete in Phase II near the construction joint experiences tensile stresses that over time lead to formation of transverse cracks (normal to traffic) in the Phase II portion of the slab. The formation of longitudinal cracks along the construction joint is also due to the shrinkage of the Phase II concrete and the restraint provided by the Phase I concrete.

The following concepts can lead to higher performing concrete bridge decks:

- The deck could be kept non-composite until it cures. This may be done by creating pockets around relatively large, widely spaced clusters of studs. After the temperature and shrinkage effects dissipate, the deck can then be connected to the girders with high strength non-shrinkage grouts or some form of mechanical connection.
- It is also possible to try to post-tension the deck in the longitudinal direction before the connection is made between the deck and girders.
- A more fundamental concrete materials research discussed in more detail in a separate section will focus on developing a modified high-performance, cast–in-place concrete specifically designed for use in bridge decks. Attributes of such a material would include: low shrinkage (or an expansion to offset shrinkage), low heat of hydration, low modulus of elasticity, high alkali/silica reactivity, high abrasion resistance, high tensile resistance and high freeze-thaw resistance, where needed.
- Precast concrete full-depth, full-width deck panels address the issue of deck concrete shrinkage and temperature drop. They have other advantages compared to cast-in-place systems. These include high speed of construction and utilization of high-performance concrete. Also, they may be prestressed in two directions, rendering the deck crack free. The figures below show application of the full-depth precast deck system for the Skyline Bridge in Omaha, Nebraska. The PCI is in the process of developing recommended practices for full-depth deck panel systems. Dr. Tadros, of the research team, is on the task force developing the report. Its contents should be of significant value to this project.

Figure 32. Deck panel being placed for transport

Figure 33. Completed Skyline Bridge

4.2.4.4 Discussion on Durability of Concrete in General and Factors influencing Service Life of Concrete Deck in Particular

Concrete durability is perhaps one of the most pervasive topics to be investigated in this project. This section provides practical issues related to service life of bridges incorporating concrete as a material. This could be the concrete used in the deck, columns or other elements of the bridge. Figure 34 provides a list of issues that will be elaborated on in this section.

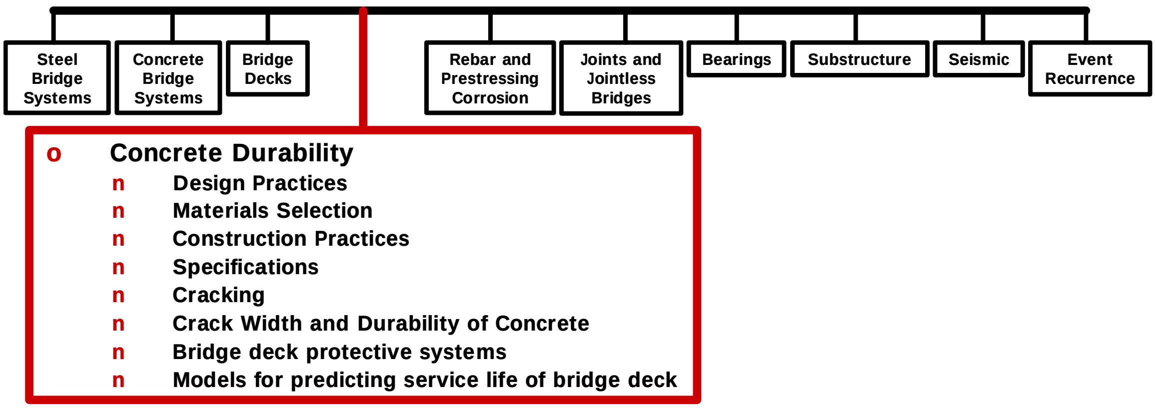


Figure 34. List of factors related to concrete durability discussed in this section

Concrete is a durable material widely used in bridge structures and pavements that can last a long time with minimal maintenance if designed and applied correctly for a particular application. Concerns for safety, limited resources and the desire to reduce lane closures or delays have prompted attention to the durability of concrete bridge structures and pavements worldwide (Ozyildirim, 2007). With new concrete technologies, structures can be built to last a long time. Today, it is possible to design and build concrete structures that will last at least 100 years with minimal maintenance. In such an endeavor, proper design, innovative materials, good construction practices and proper specifications are required. It is also essential to retrofit and upgrade existing structures to extend their lives to provide at least 100 years of service.

In bridge structures, the repeated traffic load and the environment provide a very severe environment for decks and raises issues with durability (PCA 1970; NCHRP 1970). The types of deterioration experienced in bridge decks are mainly scaling, mortar flaking, spalling, delaminations, abrasion damage, alkali-aggregate reactivity and cracking (NCHRP 2004; Ozyildirim, 1993). The most costly and extensive damage is due to the corrosion of the reinforcing steel that results in cracking, spalling and delaminations (NCHRP 1979). It is generally believed that the corrosion of the reinforcement begins when the chloride content at the level of the reinforcing steel reaches a threshold value of 1.0 to 1.5 lb/yd^3 (NCHRP 2004). Chlorides that reach the steel reinforcement accelerate the corrosion process. In the alkaline environment of the concrete, there is a protective layer over the steel; however, chlorides above the threshold values destroy this layer. Another factor that promotes corrosion is the carbonation which results in reduction of the pH level; thus, the reduced alkalinity at the level of steel adversely affects the protective layer over the steel, making it prone to corrosion (Neville, 1994).

To protect against the four major types of environmental distress affecting concrete bridges -- corrosion of the reinforcement, alkali-aggregate reactions, freeze-thaw deterioration and attack by sulfates -- water or solutions should be prevented from penetrating the concrete (Ozyildirim, 1993). For resistance to freezing and thawing and salt scaling, concretes must also be air-entrained (Klieger, 1994). Specifications generally require a certain volume of total air content depending on the maximum size aggregate and exposure conditions. However, for adequate protection from freeze-thaw damage, air voids must be closely spaced in the paste. The average spacing factor (the distance of any point in a cement paste from the periphery of an air void) for satisfactory resistance is accepted as 0.008 in. or less (Mather, 1990). However, low permeability and a proper air-void system do not always ensure durability if the concrete contains excessive cracks that facilitate the intrusion of aggressive solutions. This cracking can be due to many factors related to both environmental effects and structural loads (TR Circular E-C107 2006).

Figures 33 and 34 provide a brief summary of physical and chemical related factors affecting durability of concrete.

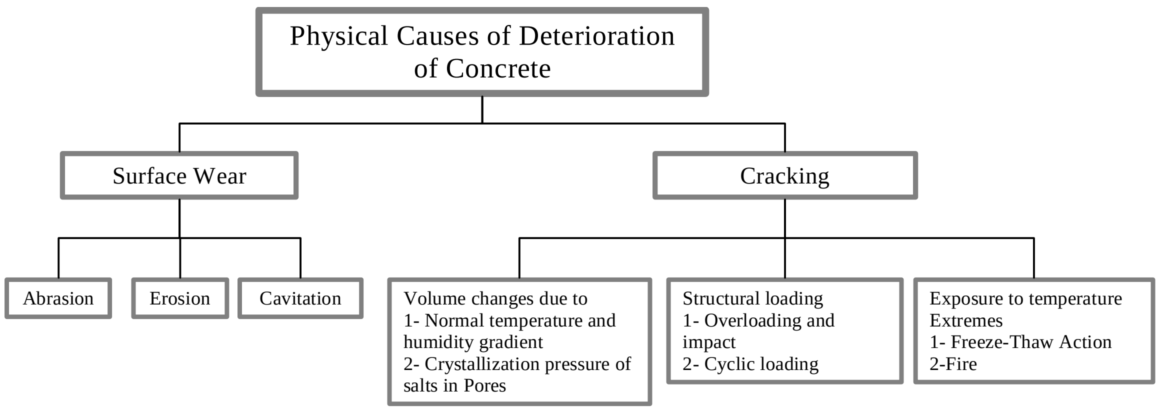


Figure 35. Physical causes of concrete deterioration, "Concrete Structures, Properties, and Materials" by Mehta, P.K., and Monteiro, P.J.M.

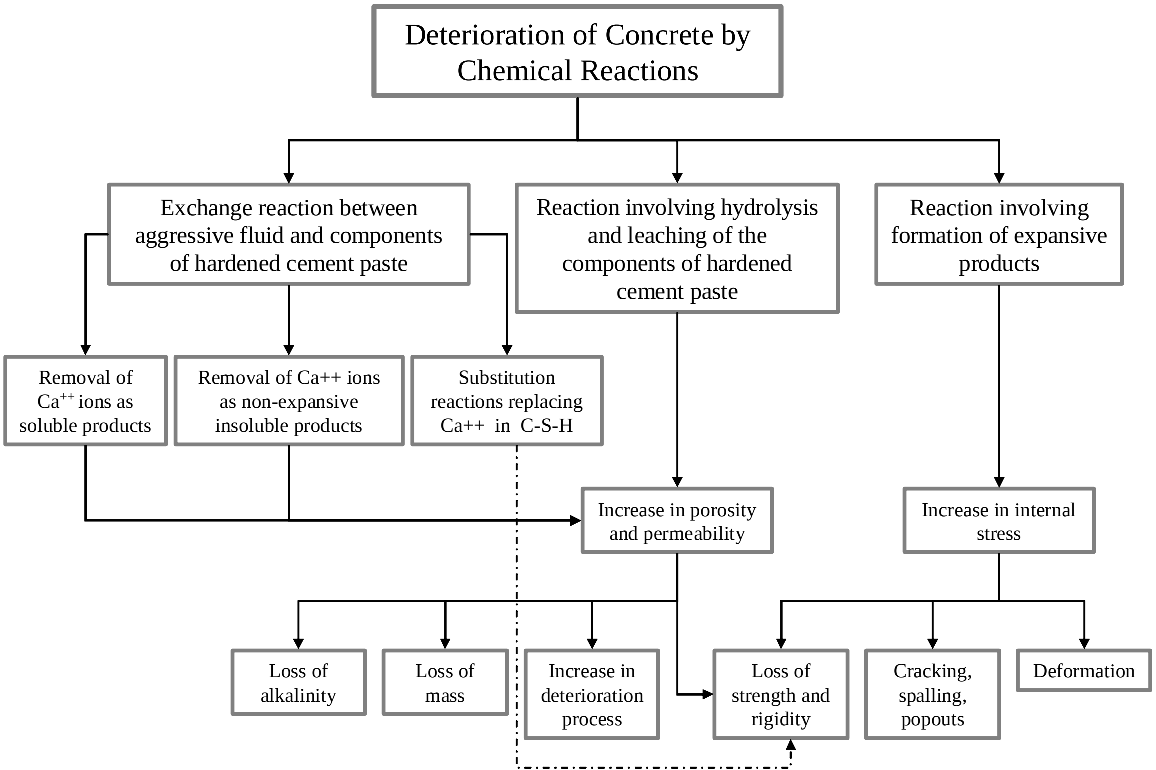


Figure 36. Chemical causes of concrete deterioration, "Concrete Structures, Properties, and Materials" by Mehta, P.K., and Monteiro, P.J.M.

Building durable bridge structures requires innovation and the use of available resources in an efficient manner. An ideal durable structure needs to have a low permeability concrete with a proper air-void system and no cracks and must not be subject to deleterious chemical reactions. These characteristics are discussed below under design, materials, construction, specification and cracking. Any distress that occurs in decks should be repaired; otherwise, interaction of different distress mechanisms will accelerate deterioration. This fact emphasizes the need for regular maintenance activities.

A) Design Practices

Certain design parameters can lead to distress and cracking. Good drainage detail can minimize ponding and prolonged exposure of bridge components to solutions, minimizing corrosion and other environmental distress. Jointless bridges eliminate the possibility of seepage of solutions through the joints which is a common problem. Bridge designs with long span length, large angle of skew, more flexible steel beams and the use of continuous structures are more prone to cracking (NCHRP 2004). More rigid concrete beams minimize flexing of the deck and the growth of cracks. Thicker concrete cover provides more resistance to the penetration of solutions to the level of reinforcement. Avoidance of skews on structures can aid in durability as this design feature introduces torsional stresses that lead to diagonal cracking at the transverse joints. Smaller size and closer spacing of reinforcing bars result in less cracking (NCHRP 2004).

The selection of reinforcement is important as well; bars made of tough and intrinsically corrosion-resistant materials can minimize corrosion potential (Clemena and Virmani, 2004). Bars clad with stainless steel and solid stainless steel bars appear to last at least 15 times longer than carbon steel bars. Low-carbon, chromium steel bars appear to be a cost-effective option for extending the service life of future concrete bridges. They cost about the same as the commonly used epoxy-coated reinforcement and are 4.5 times more corrosion resistant than carbon steel bars. Epoxy coated reinforcement has raised concerns in moist environments subjected to high chloride levels (Weyers et al., 1997; Pyc et al., 2000). Another noncorrosive reinforcement used on a limited basis has been fiber reinforced polymer (FRP) bars (Erki and Rizkalla, 1993; NCHRP 2003). However, FRP may be susceptible to other forms of deterioration (ACI 1996).

B) Materials Selection

A very efficient way of resisting the intrusion of chlorides is to use pozzolans and slag to reduce concrete permeability (Lane and Ozyildirim, 2000). Besides the use of supplementary cementitious materials (SCM's), a proper water-cementitious materials ratio (w/cm) is effective in achieving low permeability. A lower w/cm leads to lower permeability; however, low w/cm concretes usually have higher autogeneous shrinkage, stiffer consistency, higher cement content, less bleed water, high strength and are more prone to cracking, which negates the concrete impermeability. Low shrinkage, low modulus of elasticity, high creep and low heat of hydration are desirable in reducing cracking (NCHRP 2004; Schmidt and Darwin, 1999; NCHRP 1996). The w/cm should not be too low; a range between 0.40 and 0.45 is commonly used for bridge decks. This, in combination with the use of pozzolans and slag, provides for low permeability (Ozyildirim, 1998; 1999).

Today new technologies such as nanotechnology are introduced into concrete making to enhance the properties. Nanoclays appear to be very effective in reducing the permeability of concrete. These supplementary cementitious materials (pozzolans including nanoclay or slag) resist chemical distress. SCM in large replacement rates (75% slag replacement was used in Virginia) can control heat rise. Heat generated especially in mass concrete can lead to high disruptive stresses. Europeans have been using membranes on decks with asphalt overlays to prevent the intrusion of water and salts into concrete. This system should be compared to using low permeability concrete. Low permeability concrete that has extensive cracking does not provide desired longevity. As part of a research project to design and construct high performance concrete bridges in Nebraska, Azizinamini conducted an extensive amount of material tests (M. Kebraei et al., 1997).

One of the major conclusions from these tests is summarized in Figure 37.

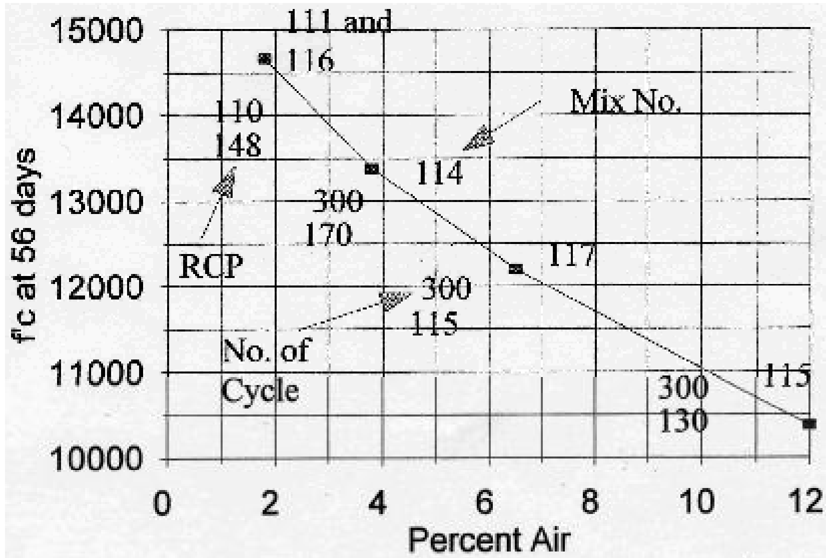


Figure 37. Relationship between chloride permeability and freeze/thaw

The vertical axis in this figure is the concrete compressive strength at 56 days. The horizontal axis is the percent air content. Four data points are shown in this figure, connected by straight lines between them. Above the curve and beside each point there is a number that indicates mix number. Below the curve and beside each data point, there are two numbers. The top numbers are number of freeze thaw cycles before failure and producing a relative dynamic modulus of zero as obtained from ASTM C 666 procedures. The bottom numbers are the results of rapid chloride permeability (RCP) tests conducted based on ASTM C 1202 procedures. Lower RCP numbers are usually taken as an indication of concrete durability. The number of freeze thaw cycles that the concrete can survive is also an indication of concrete durability. Closer inspection of the data presented in Figure 37 indicates the following important conclusions:

1. Even for very high strength concrete, some level of air content is needed to produce durable concrete.
2. The criteria for measuring long term durability of concrete should be based on freeze/thaw test results. There is no relationship between RCP and freeze/thaw test results. For instance, the mix numbers 111 and 116 (upper most right hand data) had a lower RCP number (148) than for mix 114 (170). However, mix 111 and 116 survived only 110 cycles of freeze and thaw, compared to 300 for mix 114.

The discussion above highlights the importance of controlling the cracking when low permeability concrete use is desired. Ways of reducing the severity of cracks and controlling the width of the cracks through material selection and proportioning and proper curing procedures are needed. The addition of fibers to concrete or post-tensioning the elements should also be considered in controlling cracking.

Paste of concrete is where most of the volumetric changes and distress take place. Therefore, to improve durability, the amount of paste should be reduced. This can be accomplished using the largest size aggregate possible and through better grading of the aggregates. This will lead to reduced water, cementitious materials and paste contents.

Water freezing during cycles of freezing and thawing can generate very high stresses in critically saturated concrete. For protection, concrete must be properly air entrained, must have sound aggregates and must have the maturity to develop sufficient strength for long-lasting service (Mather, 1990). A minimum compressive strength of 4000 psi is commonly specified in bridge decks which are recommended for resistance to freezing and thawing and salt scaling (Mather, 1990). Air entraining admixtures provide small, closely spaced and uniformly distributed air voids by stabilizing voids with a diameter less than 1 mm. The size of the air voids is indicated by the specific surface, which is equal to the average surface area divided by the volume of the voids. The average distance water must travel to reach a protective air void in concrete undergoing freezing is indicated by the spacing factor. It is generally accepted that a specific surface greater than 600 in2/in3 and a spacing factor less than 0.008 inch is needed for adequate protection during freezing and thawing (Whiting and Nagi, 1998).

C) Construction Practices

Good construction practices, mainly proper consolidation and curing, are essential for longevity. Proper consolidation will minimize entrapped air voids that can reduce strength and durability. For areas that are hard to reach and areas that are congested with reinforcement and when stiff concretes are used, self-consolidating concrete (SCC) may be the solution, as shown in Figure 38. SCC can flow into the formwork and encapsulate the reinforcement without any mechanical consolidation (Okamura and Ouchi, 1999). SCC can be very useful in constructing prestressed bulb-T beams with many strands in the bottom flange and in other structural elements with thin walls and congested reinforcement.

Figure 38. Using self-consolidating concrete

Proper curing for formation of the binder and control of volumetric changes are needed. In curing, both moisture and temperature control must be addressed. In bridge structures, the deck surfaces require special attention since they have large surface areas where loss of moisture is a concern. For bridge decks, many use seven days of moist curing with wet burlap covered with plastic, as illustrated in Figure 39. After seven days, some states spray curing compound as soon as the plastic and wet burlap are removed. The wet burlap should be placed immediately after the screeding is completed. If there is a delay, the surface should be fog misted. Concerns that prompt placement of the burlap causes marring of the surface are unfounded. There is no problem with the surface marks from the burlap itself, especially since the transverse grooves cut on hardened concrete or a tined surface make surface marks unnoticeable. However, deep indentations from a footprint or heavy object placed on the burlap may be an issue. In some states, the wet cure is extended to 10 days and even to 14 in New York. Immediate attention to moisture loss after screeding and limiting the temperature differences between the core and the surface in mass concrete and between the deck and the beams are important to avoid large temperature differentials that can induce cracking.

Figure 39. Curing concrete deck

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. This may cause tearing of the deck surface during finishing, producing a poor surface finish. Delivery of the concrete to the forms through pumping can result in loss of slump and air content. Loss of air occurs because bubbles shrink due to pressure in the pump line, crush from the impact of falling concrete and expand and dissipate due to the vacuum created when concrete slides in a vertical pipe (Yingling et al., 1992). A steady flow of concrete during pumping should be provided and a large free drop in the pump line eliminated. This generally results in satisfactory freeze-thaw resistance even though the total air content that is adequate before pumping may be lower than specified after pumping (Ozyildirim, 2004).

Additional protection is needed in cold weather conditions. Thermal blankets to retain the heat may not be enough; external heat may be needed. For example, the deck of the first high-performance concrete structure in Virginia was placed during the month of December and the cold weather necessitated the use of heaters and a plastic enclosure (Ozyildirim, 2002).

Traffic induced vibrations during rehabilitation where part of the deck is open to traffic has caused concerns (NCHRP Synthesis 86). Laboratory tests have shown no detrimental effect (NCHRP Report 297). Field evaluations have also confirmed that deflections associated with vibrations are small and do not cause cracking during the hardening process (NCHRP Report 380).

D) Specifications

Current specifications are generally of a prescriptive type requiring a recipe of ingredients. They restrict innovation by limiting the use of many possible combinations of materials. In performance-type specifications, the characteristics of the mixture are specified rather than the mixture itself, allowing the producer to be innovative in the selection of materials and in the proportioning of the mixture. The contractor and the user share responsibility; the contractor is responsible for the development of the product and the user for its acceptance. Compensation can be adjusted depending on the quality of the product. For example, VDOT has developed an end-result specification, where strength and permeability are specified without any limits on the minimum cementitious materials or maximum w/cms and has been evaluating it on pilot projects (Hughes and Ozyildirim, 2005).

E) Cracking

Cracking can be due to loads or environmental effects (ACI 2007; TR Circular E-C107 2006). In bridge structures, cracking is mainly attributed to moisture loss and temperature change. Transverse cracking is common especially in continuous structures. Selection of materials also affects the extent of cracking. Mixtures with high water and paste content are prone to shrinkage cracks that occur over a period of time. 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. The rate of evaporation can be determined using a chart. Means of reducing the evaporation rate can be established. Concrete with low bleed water, stiff consistency and low w/cm are prone to plastic shrinkage cracking. Prevention of plastic shrinkage cracking depends on prompt and effective curing.

To reduce cracking, shrinkage should be reduced; however, cracking also depends on other factors such as restraint, elastic modulus and creep. Low elastic modulus and high creep help to minimize cracking. All these factors should be considered in predicting cracking potential (TR Circular E-C107 2006).

Cracking can also occur in bridge decks where temperature management is not used. High temperatures and high temperature differentials can cause cracking. In bridge structures, a maximum temperature differential of 22°F between the beams and the deck is recommended for at least 24 hours after concrete is placed (Babaei and Fouladgar, 1997). To minimize the potential for temperature-related cracking, the amount of Portland cement should be minimized, concrete delivery temperature reduced and pozzolans or slag included.

The desire to reduce cracking due to shrinkage has led to studies with fibers and shrinkage reducing admixtures. Use of structural fibers in large amounts has been shown to reduce the severity of cracks in terms of length and width (Ozyildirim, 2005). In one study, an overlay with the shrinkage-reducing admixture demonstrated that the least shrinkage occurred when this admixture was used (Sprinkel and Ozyildirim, 1998). However, due to prompt and proper curing, none of the overlays in that study exhibited shrinkage cracking, making this the more important factor.

High performance fiber reinforced cementitious composites (HPFRCC) provide more load carrying capacity after the first crack and large strain capacity due to the special fibers and use of small particles (no coarse aggregate). A new engineered cementitious mortar composite (ECC) developed by Dr. Victor Li of the University of Michigan is one of those HPFRCC that has high strain capacity and is expected to replace the joints in decks (Li 2002, 2003). In HPFRCC including ECC when cracking occurs, multiple tight cracks are formed which prevent the intrusion of harmful chemicals and water. These cracks are less than 0.1 mm in width. Such tight cracks are not expected to facilitate the intrusion of harmful solutions (Wang et al., 1997, Lawler et al., 2002).

F) Crack Width and Durability of Concrete

A sufficiently thick and impermeable concrete cover is recognized as providing excellent durable protection against corrosion due to the alkalinity of the pore water resulting in a so-called passive layer on the steel surface. The reinforcement may lose its protection against corrosion if the concrete is carbonated.

Traditionally, cracks in concrete have been considered an essential source of premature deterioration of concrete structures due to the opening up of the concrete to ingress of deleterious substances. This has led to strict limitations on crack widths as part of the design basis. For example, AASHTO ***LRFD Specifications*** require crack width of 0.1 mm for marine structures. Similar requirements are noted in British code (BS 5400-4, the Code of practice for design of concrete bridges). This limitation could lead to reinforcement quantities in excess of what is required for ultimate limit state design. This strict crack width limitation approach is not based on a scientific rationale or on a general experimental verification. It is rather based on subjective considerations and engineering judgments..

Some of the available research data point to the fact that controlling the crack width to a limited value and using a good quality concrete between these small cracks could still result in a durable concrete.

According to (Schiessl; German Institute for Reinforced Concrete 1989) crack widths smaller than 0.4 mm should not be a concern in relation to corrosion as the oxygen diffusion (cathode process of the corrosion mechanism) of the uncracked concrete between the cracks is the decisive parameter determining the reinforcement corrosion. In addition, no crack influence could be demonstrated for permanently depassivated steel surfaces in the crack zone at crack widths up to approximately 0.4 mm.

The explanation for this observation is illustrated in Figure 40. If carbonation or chloride reaches the reinforcement, depassivation of the reinforcement may occur. Current corrosion measurements (Makita and Mori 1980; Murray Vennesland and Gjørv 1981; Okada and Miyagawa 1980) show that the steel in the cracked region is acting anodically while the cathodic process takes place in uncracked areas beside the crack. In this process, the crack width does not play a major role because the cathodic process is the main rate-determining factor. This process depends on the diffusion rate of oxygen through the concrete cover and, thus, on the quality (density) and the thickness of the concrete cover. Results of long-term exposure tests on uncracked and cracked concrete members and site inspections support these findings (Rehm et al.; Marin and Schiessl, 1970; Bertandy, 1978; Darwin et al., 1985).

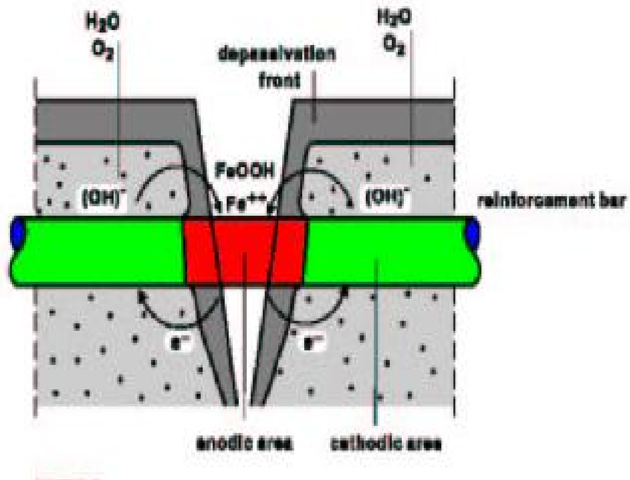


Figure 40. Corrosion of reinforcement in the region of cracks

The argument presented above may not be true in cases where chlorides accumulate due to ponding salt-water and cause a local build-up in the cracks. This could be a case for bridge decks that are frequently salted. In these cases, the salt water may settle or pond and subsequent evaporation results in an increased concentration at and in the crack, starting localized corrosion in the vicinities of cracks.

The discussion provided above is reflected in several available specifications. For instance, Section C5.7.3.4 of the AASHTO *LRFD Specification* states *"Previous research indicates that there appears to be little or no connection between crack width and corrosion."*

Other Specifications reflecting these same thought processes are Eurocode 2 and the ***fib*** Model Code. The newest versions of BS EN 1992-1-1 and BS EN 1992-1-2 (superseding BS 8110 and BS 5400) and the ***fib*** Model Code allow a crack width of 0.3 mm for all exposure classes.

There is a need to conduct additional research and to study the crack width effect in a simulation of a bridge deck subjected to salting.

G) Bridge deck protective systems

There are certain systems that are designed to protect concrete and reinforcement from distress. The primary systems are overlays, membranes, sealers and cathodic protection (NCHRP 2004).

Overlays provide a protective barrier over a concrete deck. They are mainly used in rehabilitation projects. Overlays can be of asphalt, latex-modified concrete (LMC), low-slump dense concrete, concretes containing pozzolans such as silica fume or slag or polymer concrete. LMC and silica fume overlays are used widely (Ozyildirim, 1992; Sprinkel, 1992; Ozyildirim, 1994; Sprinkel and Ozyildirim, 1998; NCHRP 2004).

Membranes are placed on top of the concrete and are protected by asphalt layer(s) that also function as a riding surface (NCHRP Synthesis 220). The asphalt top layer typically has to be replaced after about 10 to 15 years (NCHRP 1987). Water accumulating above the membrane can weaken the bottom layer of the asphalt and cause debonding from the membrane. The use of membranes has been on the decline in the states. Membranes are widely used in Europe and Ontario, generally exhibiting good performance (NCHRP 1996). However, the top asphalt layer must be replaced periodically, necessitating traffic interruptions.

Sealers are expected to minimize the intrusion of aggressive solutions into concrete. Years ago, linseed oil was used; it was determined, however, that the linseed oil retarded but did not slow loride penetration and was ineffective in resisting moisture penetration (Clear, 1976). Today, silanes are popular sealers followed by siloxanes, silicates, epoxies, gums, acrylics, urethanes, chlorinated rubber, silicones and vinyls (NCHRP 1994). Sealers have been difficult to assess and have been reported to have a wide range of performance (NCHRP 2004).

Cathodic protection systems are designed to protect the reinforcement. The potential of the reinforcement is shifted in the negative direction either by impressed current or sacrificial anodes. If the steel is made cathodic, corrosion would stop (Virmani and Clemena, 1998). Impressed current is most common and uses titanium mesh anode in a concrete overlay (Kepler et al. 2000). Some states have reported successful use of the cathodic protection system but others have cited difficulties with reliability and maintenance of the system. Cathodic protection systems have not been proven to be maintenance free or cost effective (NCHRP 2004).

Another system employed when high level of chlorides are present at the level of steel is the electrochemical chloride extraction (Virmani and Clemena, 1998). This procedure enables the extraction of chlorides from concrete and leads to an increase in the pH of concrete that would repassivate the steel. The life expectancy of a treated reinforced concrete is not known.

The prudent approach used in new construction can be used to update existing structures. For example, the porous and contaminated concrete around the steel can be replaced with low permeability concrete. Overlays can increase the cover and minimize the infiltration of solutions and joints can be replaced with continuous slabs without joints. If found satisfactory, membranes with asphalt overlays can be placed. Sealers and coatings are other possibilities that need to be investigated.

H) Models for predicting service life of bridge deck

Models can be used to predict the service life of structures. Service life is the period of time the bridge is expected to be in service (AASHTO 2007). Such models, when calibrated for the environment and the materials particular to a region, can assist in the selection and design of the mixtures and design parameters such as cover thickness. Models could include the rate of chloride penetration, degree of saturation, presence of cracks, design parameters (cover, membrane, skew) and different reinforcement, and could relate them to distress, which would help determine expected service life. Models can be used for new as well as reconstruction projects.

Some of the service life prediction models available are: Bhide, 2002; NCHRP Synthesis 333: Life-365, developed by Thomas and Bentz, addressing time-dependent diffusion of chlorides and predicting service life and life-cycle costs for various protection strategies; CIKS, Computer-Integrated Knowledge System, developed by Bentz, predicting chloride ion diffusivity coefficients and time to initiation of corrosion; Duramodel, developed by W.R. Grace, using effective diffusion coefficients to account for mechanisms other than pure diffusion; ConFlux, developed by Boddy, Bentz, Thomas and Hooton, accounting for diffusion, permeability, chloride binding and wicking; ClinConc, a chloride penetration model developed by Tang, based on mass balance and genuine flux equations to predict chloride profiles in submerged parts of structures; HETEK Model, developed by AEC Laboratory, Denmark, and applicable to marine structures and salt water splash zones.

4.2.4.5 Discussion on Corrosion of Reinforcing Bars and Prestressing Strands Embedded in Concrete

Corrosion of steel elements embedded in concrete is a major maintenance problem and is one of the major sources for reduction in service life of bridges. The FHWA Cost of Corrosion Study (2002) estimates the annual direct cost of corrosion for highway bridges alone to be $8.3 billion per year. This section provides a discussion on the topic of corrosion. Figure 41 provides a list of issues discussed in this section.

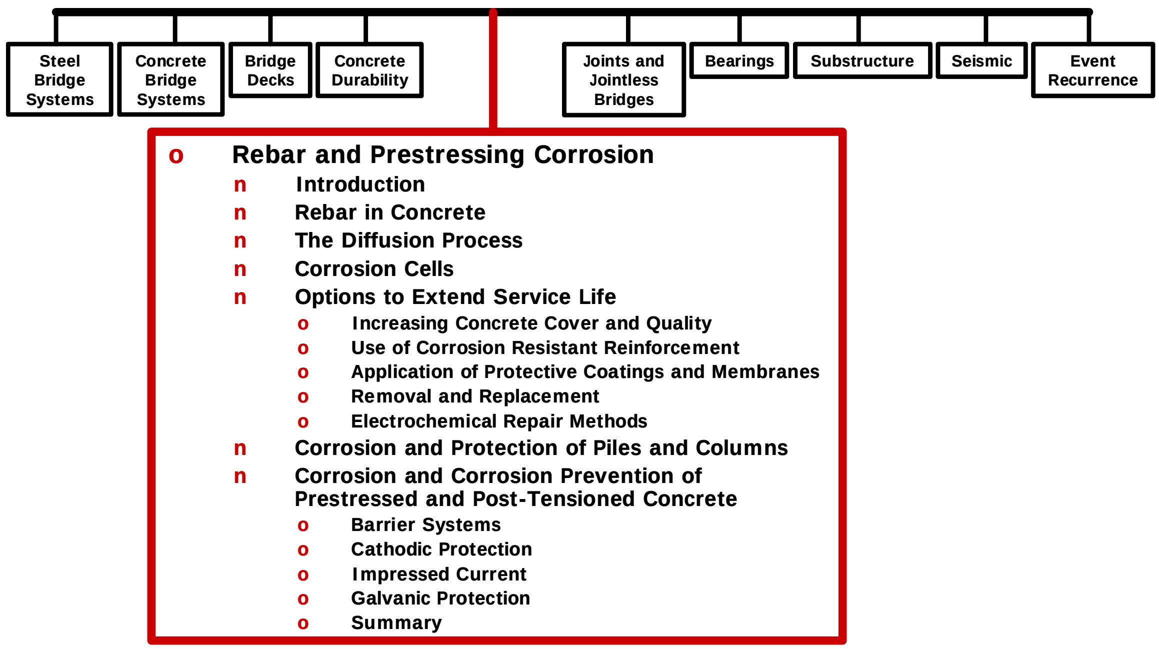


Figure 41. List of factors related to rebar and prestressing corrosion

A) Introduction

Many structures are exposed to salt solutions in their day to day existence. The degree of exposure for concrete bridges can vary from limited exposure to de-icing salts to direct exposure to a marine environment. Initially, this exposure does not appear to have any detrimental effect on the concrete structure, but over time, signs of distress such as cracking and spalling will become evident. The service life of the structure and any repairs which are completed depend on whether the underlying cause of corrosion is prevented or addressed.

B) Rebar in Concrete

Within good quality concrete, rebar is protected against corrosion by the alkalinity of the cement paste, which is typically between pH 12 and 14. In a highly alkaline environment, rebar is passivated and protected from further corrosion by the formation of a surface oxide film.

The passive oxide film is stable at pH values greater than approximately 9.5 in a chloride free environment. The pH value necessary to maintain the passive oxide film increases with increasing chloride content.

For corrosion to be initiated on the rebar, it is necessary for this protective film to break down. This may occur as a result of ingress of sufficient chlorides into the concrete matrix. The chlorides may come from the use of de-icing salts, exposure to a marine environment or through the use of a concrete admixture which contains chlorides. Alternatively, corrosion can be initiated by carbonation, the reaction of CO and CO_2 in the air with available alkali in the concrete which cause the pH of the concrete to drop over time. Once the pH drops below 9.5, the passivating oxide film will start to break down. Although carbonation induced corrosion is a problem with building structures, it is rarely a problem with reinforced concrete bridges due to the concrete cover and concrete quality used.

As a result, more attention will be given to the effects of chloride induced corrosion.

C) The Diffusion Process

To the casual observer, 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 are able to 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 in Figure 42 as follows;

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

Where,

$C_{(x,t)}$ = chloride concentration at depth x and time t

C_0 = surface chloride concentration (kg/m^3)

D_c = chloride diffusion constant (cm^2/yr)

erf = error function

From this expression, it can be seen that chloride concentration within the concrete will tend to equalise with the surface chloride concentration over time. As expected, the chloride concentration within the concrete is greater near the exposed surface and increases with time at any point x 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.

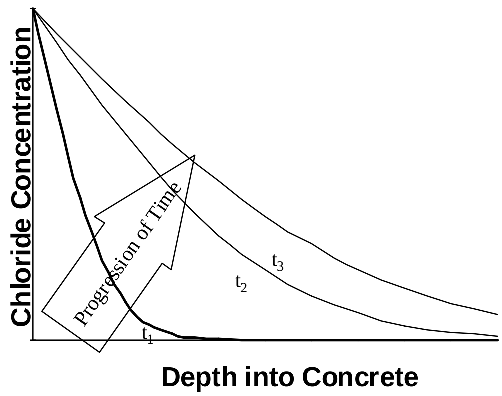


Figure 42. Fick's Law of Diffusion

D) 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 similar to a catalyst in the corrosion process. Chlorides are involved in the corrosion reaction, but are not consumed by the reaction itself. They are re-released such that a single chloride ion can be responsible for the corrosion of many atoms of iron.

On a localised basis, corrosion cells can be formed due to differences in the chloride concentration at various locations along a single bar. These variations can result in localised 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 43 on the following page. Macro-cell corrosion can be very aggressive and severe and is responsible for much of the severe structural damage seen on bridges and other structures. Both pitting type corrosion and general corrosion may be seen as a result of macro-cell corrosion.

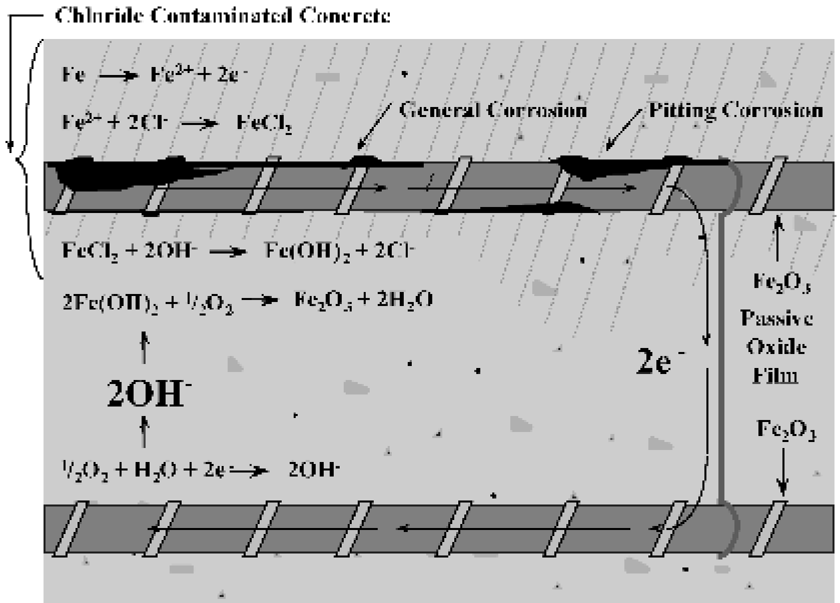


Figure 43. Macro-cell corrosion

E) Options to Extend Service Life

As with most problems, there is more than one possible solution depending on the situation and desired result. Many "solutions" have been proposed or used to address chloride induced corrosion including increasing concrete cover, improving concrete quality, application of coatings and membranes, complete removal and replacement of contaminated concrete, electrochemical remediation and prevention techniques. The cost and performance of the various options may vary greatly (FHWA 2002). Following are some of the more common repair strategies used in the field.

E.1 - Increasing Concrete Cover and Quality

Increasing concrete cover and improving the quality (reducing the permeability) of the concrete are two of the most effective methods to increase the service life of new reinforced concrete structures. Doubling the concrete cover alone will increase the service life by a factor of four (4). Likewise, reducing concrete permeability will significantly increase the service life. These factors are generally considered and must be incorporated into any Design for Service Life.

E.2 - Use of Corrosion Resistant Reinforcement

The degree of corrosion damage will depend on the type of reinforcement and its resistance to corrosion. Consideration should be given to the various types of corrosion resistant alloys -- stainless steel, stainless clad, FRP, epoxy coated and conventional black reinforcing steel -- available. Each of these materials has different corrosion resistant properties. They also have different physical and design properties which need to be considered.

E.3 - Application of Protective Coatings and Membranes

The application of a protective coating or membrane is a potential solution which can be considered. This option works very well for a new structure or one which has not been contaminated sufficiently to initiate corrosion. For these structures, a significant increase in life expectancy can be achieved by the selection and application of an appropriate coating.

If the existing structure is already contaminated with chlorides and is showing signs of distress, the application of a coating or membrane is generally of limited value. The structure is already contaminated with sufficient chlorides to induce corrosion; this corrosion will continue to the extent that moisture and oxygen are available. For these structures, corrosion has already been initiated and it will continue regardless of whether a coating is applied or not. If the coating can significantly reduce the moisture availability or if it can reduce the availability of oxygen, the coating may slow down the overall corrosion rate and buy some additional life from the structure. The corrosion will not stop, however.

E.4 - Removal and Replacement

Removal and replacement of the chloride contaminated portion of a structure is a long-term rehabilitation solution since this method addresses the underlying presence of chloride ions by physically removing contaminated concrete and replacing it with fresh concrete. If all of the chloride contaminated material is removed, the electrochemical incompatibility is removed and corrosion is no longer of concern.

The problem with removal and replacement is that generally the cost of this option is high and may be cost prohibitive or impractical.

E.5 - Electrochemical Repair Methods

Chloride induced corrosion can be addressed by electrochemical repair methods. The two most common electrochemical methods available are Cathodic Protection and Electrochemical Chloride Extraction.

Cathodic Protection is a method whereby a system is either applied to the surface of the concrete or embedded within the concrete itself. The system provides a low-level direct current sufficient to provide corrosion protection and overcome the natural tendency for the reinforcing steel to corrode. Cathodic protection systems can effectively stop on-going corrosion even if sufficient chlorides, water and oxygen are present. (Pedeferri, 1996; Bertolini et al., 1996; NRC 1993)

There are two basic types of cathodic protection systems available (BSI 2000; NACE 2000). Impressed Current Cathodic Protection (ICCP) systems utilize a low voltage DC rectifier or power supply to provide current to the steel. Galvanic Cathodic Protection (GCP) systems utilize galvanic (sacrificial) anodes which are designed to corrode in place of the reinforcing steel.

Impressed current systems have the advantage that the current output from the rectifier can be adjusted if additional protection is required. However, one of the disadvantages of this type of system is that the system needs to be maintained and needs to remain energised for the life of the structure in order to maintain protection. A schematic diagram showing a typical Impressed Current Cathodic Protection (ICCP) System is given in Figure 44 .

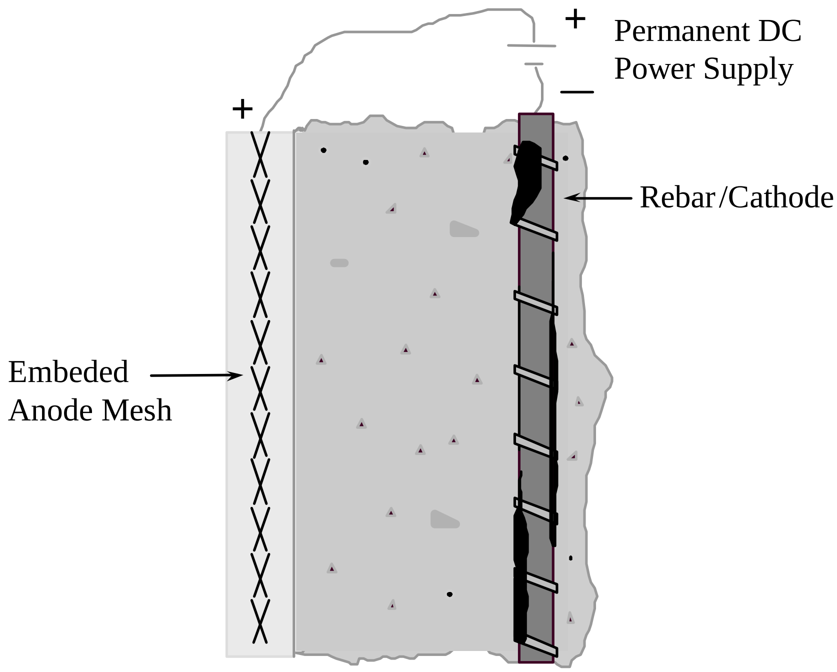


Figure 44. Typical impressed current cathodic protection system

Galvanic systems have the advantage that they require no electrical equipment or systems to operate. As a result these systems are generally attractive to DOT and government agencies as they require little or no maintenance. Disadvantages include the limited driving voltage of galvanic systems. A schematic diagram showing a typical Galvanic Cathodic Protection (GCP) System is given in Figure 45 .

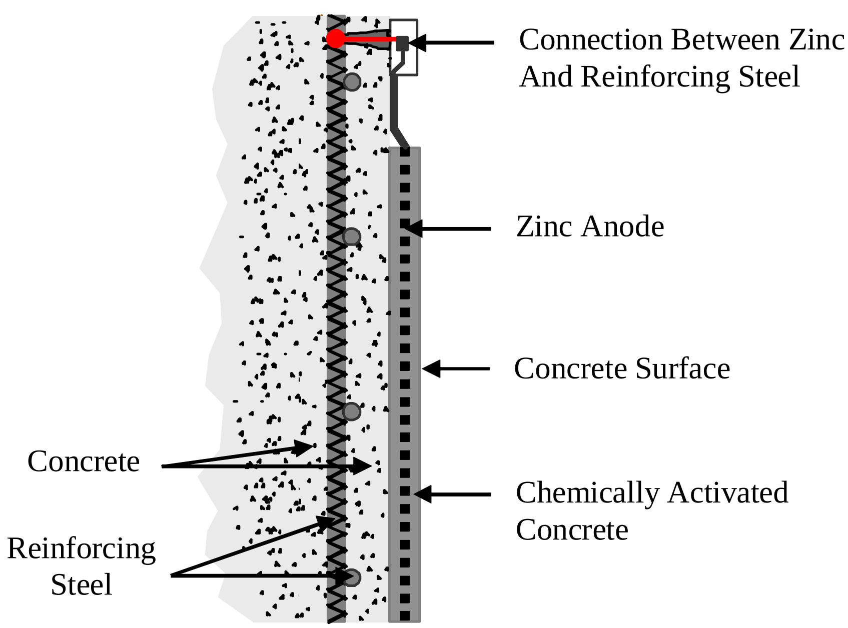


Figure 45. Typical galvanic cathodic protection system

Electrochemical Chloride Extraction (ECE) is similar to cathodic protection with a couple of significant differences. An ECE system is not permanently installed but is rather installed and operated as a treatment process until the chloride content within the concrete is reduced and the passive oxide film on the rebar is re-established (Figure 46) (ACI 1994; VTRC 1996). When the chloride concentration and corrosion potentials are sufficiently reduced, on-going corrosion activity will stop and the ECE system can be removed from the structure. If desired, the structure can be further protected after treatment by the application of a protective coating to prevent it from becoming re-contaminated with chlorides.

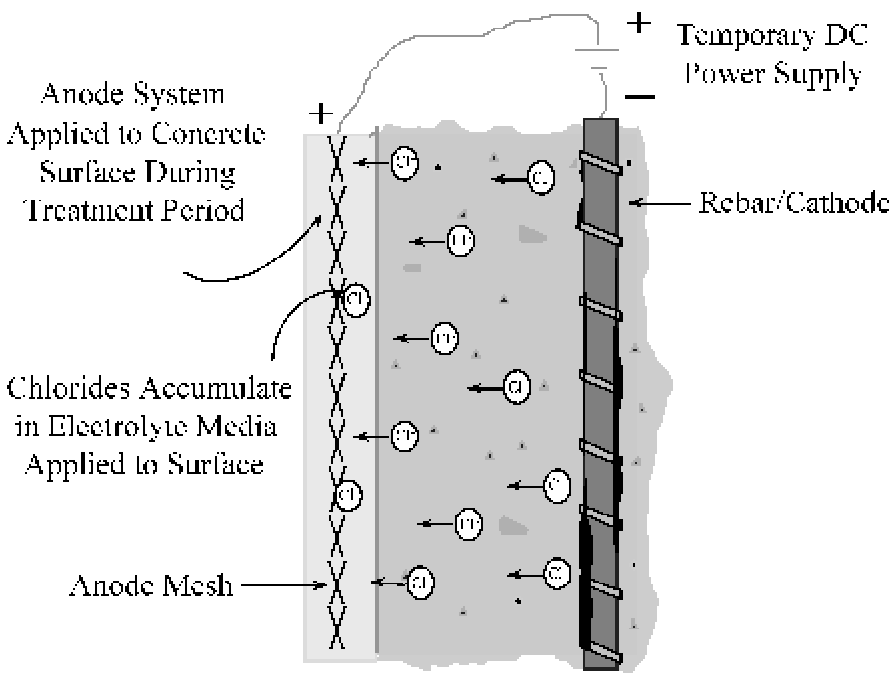


Figure 46. Electrochemical chloride extraction schematic

Research is currently under way on the benefit and use of Electrochemical Pretreatment of Concrete to change the chemistry of the concrete surrounding the reinforcing steel to increase the corrosion threshold. Initial results are promising (Glass 2003). If successful, this approach would allow a structure constructed using normal concrete and black reinforcing steel to be electrochemically treated and the corrosion threshold increased sufficiently to provide over 100 years of service life.

F) Corrosion and Protection of Piles and Columns

Steel and concrete piles are often subjected to severe and corrosive conditions. Examples include piles in marine environments, columns exposed to damage due to ice and steel piles under concrete abutments which may corrode and lose cross section due to soil chemistry or the presence of voids which result from settlement. Structures must be designed to address these issues if 100 year service lives are to be achieved.

G) Corrosion and Corrosion Prevention of Prestressed and Post-Tensioned Concrete

The corrosion of high tensile strength steel under load in prestressed and post-tensioned concrete can be particularly damaging and presents special considerations for corrosion protection, rehabilitation and the selection of corrosion protection systems.

Similar to the corrosion of conventional reinforcing steel in concrete, corrosion of prestressed and post-tensioned steel is an electrochemical reaction that is influenced by various factors including chloride-ion content, pH level, concrete permeability and availability of oxygen and moisture.

G.1 -Special Considerations for Prestressed and Post-Tensioned Concrete

Corrosion activity in prestressed and grouted post-tensioned concrete is generally localized in nature and can usually be attributed to design, construction and maintenance issues. Examples of these deficiencies may be improper drainage, leaking cold joints, failed concrete overlays and membranes or leaking expansion joints allowing chloride containing water to saturate prestressed concrete or post-tensioning ducts not designed for severe exposure conditions. For new structures, these deficiencies must be addressed and contamination of the tensioned elements must be prevented in order to achieve a 100 year service life.

Corrosion in concrete elements with tensioned reinforcing potentially has greater consequences than conventionally reinforced concrete. While the corrosion of non-prestressed reinforcing is generally considered to be more of an issue of concrete durability and aesthetics than imminent structural failure, the corrosion of prestressed steel and the subsequent loss of section have greater ramifications. Since the applied stress level is typically between 55-65% of ultimate, loss of cross section and an increase in stress on the remaining cross section can lead to a point of yielding or fracture. Secondly, when compared to reinforcing bar, prestressed tendons are made up of smaller diameter wires that lose cross sectional area more rapidly at the same corrosion rate (ACI 2001).

Because of unique issues surrounding the corrosion of prestressing steel, the American Concrete Institute specifies lower acceptable chloride thresholds when using prestressed reinforcing in new construction.

As noted earlier, industry recognized corrosion-damage repair procedures for conventionally reinforced structures include removal of concrete and the complete exposure of embedded metals. For prestressed concrete, this procedure would release the stored energy of the prestressing strand and significantly reduce the applied prestressing force. This is of particular concern if the corrosion damage is located in load bearing areas, as is typically the case.

High tensile strength prestressing steel also has a greater susceptibility to stress corrosion cracking and hydrogen embrittlement. Stress corrosion cracking is a special type of corrosion that occurs when steel is under tensile stress and is subject to a corrosive environment. Hydrogen embrittlement is caused by the adsorption of atomic hydrogen into the metal. These issues can be of particular concern to structural elements since they cause a loss of ductility of the reinforcement and increase the potential for brittle failure.

G.2 -Corrosion Protection Systems for Prestressed and Post-Tensioned Structures

Barrier Systems

Reducing exposure to the concrete surface and the use of barrier systems such as increased cover, sealers, coatings and lower permeability concrete are often considered as methods to provide a long service life. Barrier systems primarily function by preventing the ingress of chloride ions, moisture and/or carbon dioxide into the structure. Depending upon the environment and the material selected, an increase in service life can be achieved when the proper system is applied to a new structure or to a structure without significant chloride contamination at the level of the reinforcing steel. If the structure is exhibiting corrosion distress, the benefit of barrier systems will be limited.

Cathodic Protection

One corrosion mitigation technique which is sometimes used is cathodic protection. Cathodic protection mitigates corrosion activity by supplying sufficient electrical current from an external source to overcome the on-going corrosion current in the structure.

Impressed Current

Impressed current systems are widely recognized as an effective method of corrosion control but may create risks for prestressed and post-tensioned structures. Impressed current systems are a potential source of hydrogen generation if the polarization is sufficiently high. If this occurs, some of the hydrogen generated may enter the steel network. Some research has indicated that potentials more negative than -977 mV_{cse} (copper sulfate electrode) can cause hydrogen embrittlement (Hartt et al., 1993). Due to this concern, the use of impressed current cathodic protection on prestressed concrete elements has been limited, especially in North America.

Galvanic Protection

An alternate approach to achieve long-term corrosion protection is to utilize galvanic protection systems. Galvanic protection can be used alone or in combination with barrier systems for multilevel protection.

Unlike impressed current cathodic protection systems, the galvanic system voltage is fixed and is unlikely to cause hydrogen embrittlement. The current generated by galvanic systems will vary with the surrounding environment. Galvanic systems generate higher current output when the environment is more corrosive or conductive. For example, current output will likely exhibit a daily and seasonal variation based on moisture and temperature changes. Some have referred to this type of sacrificial system as a “limited form of intermittent cathodic protection” (Glass et al. 2003).

As noted above, when placed on prestressed steel, the polarized potentials of cathodic protection systems should remain less negative than -977 mVcse to prevent the generation of hydrogen. Since the driving voltage of zinc is relatively low, zinc based galvanic systems are generally suitable for use on prestressed and post-tensioned structures.

Summary

Recent advances in concrete and admixture technology have allowed concrete structures to be more durable than ever. Use of appropriate materials, consideration of exposure conditions, care in specifying construction details and use of appropriate protection systems will greatly increase the service life of new structures. The service life of existing structures which are corroding can be extended through the use of conventional and electrochemical corrosion mitigation methods. With regard to the selection of corrosion mitigation systems, prestressed and post-tensioned concrete structures present special challenges.

4.2.4.6 Discussion on Joints and Jointless Bridges

It is well known that elimination of leaky joints could have a significant affect in enhancing service life of bridges. This section provides a discussion about joints. Figure 47 provides a list of issues discussed in this section

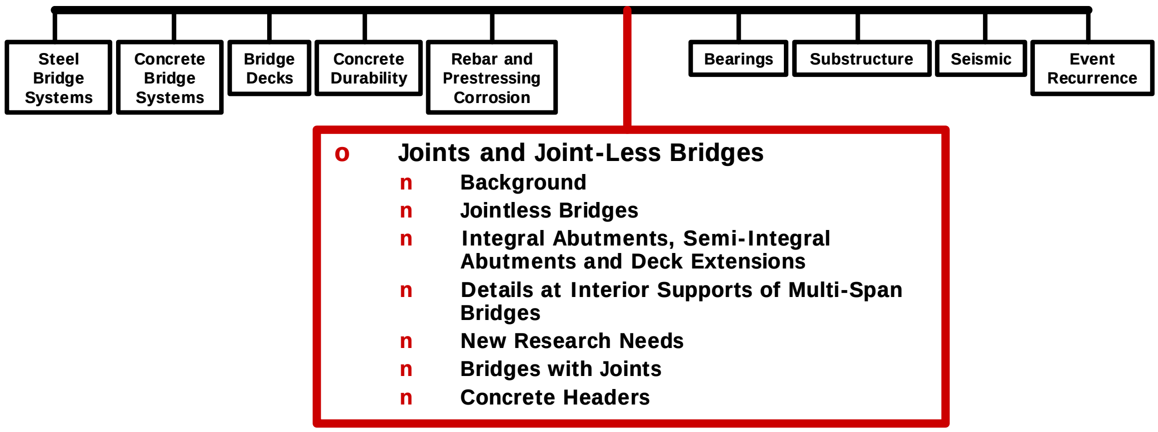


Figure 47. List of factors related joints and jointless bridges

A) Background

Deck joints have been a leading cause of bridge deterioration over the years because they ultimately leak and permit salt-laden run-off water from the roadway surface to attack girder ends, bearings and supporting reinforced concrete substructures. Figure 48 shows deterioration of a bridge due to leakage from a joint. Maintenance of bridge deck joints is without a doubt a persistent and costly problem. Many states have moved towards eliminating deck joints where possible as a means of preventing deterioration. Jointless structures indeed promote reduced maintenance costs, improved riding quality, improved seismic resistance and longer bridge life (Mistry 2005).

Figure 48. Deterioration of bridge due to leakage from a joint

Long bridges will still need joints. Joint types and details will need to be developed that will function structurally and prevent drainage from leaking through.

B) Jointless Bridges

Henry Derthick, former Engineer of Structures at the Tennessee Department of Transportation, once stated, "The only good joint is no joint." Most states have adopted design procedures that reduce or eliminate the use of joints for short to moderate span lengths. However, design methods and construction details vary significantly from state-to-state.

C) Integral Abutments, Semi-Integral Abutments and Deck Extensions

Experimental studies of integral abutment bridges began in the 1930's (McCullough 1930). Early bridges of this type were relatively short, ranging in length from 50 ft. to 100 ft. Subsequent increases in allowable length were based empirically on reports of successful performance on prototype bridges. Over time, various highway agencies have developed their own design criteria along with concomitant limits on length, skew and horizontal curvature. In January 1980, the Federal Highway Administration released Technical Advisory T 5140.13, "Integral, No-Joint Structures and Required Provisions for Movement." This advisory was aimed at providing state and local highway agencies with data and state-of-the-art information pertaining to integral abutments, continuous bridge lengths and specification oriented movement requirements. Since this time, significant progress has occurred in the development of various jointless bridge concepts. These have included:

- Full integral abutments, where the bridge girders are cast into a concrete end diaphragm that is connected to a concrete pile cap typically supported by a single row of piles (Figure 49),
- Semi-integral abutments, where the concrete end diaphragm is not rigidly connected to the substructure (Figure 50),
- Deck extensions, where the end of the deck slab is simply extended over a traditional backwall and into the adjoining approach pavement (Figure 51) and
- Shifted joint details, where the joint is shifted between a semi-integral diaphragm and a fixed backwall (Figure 52).

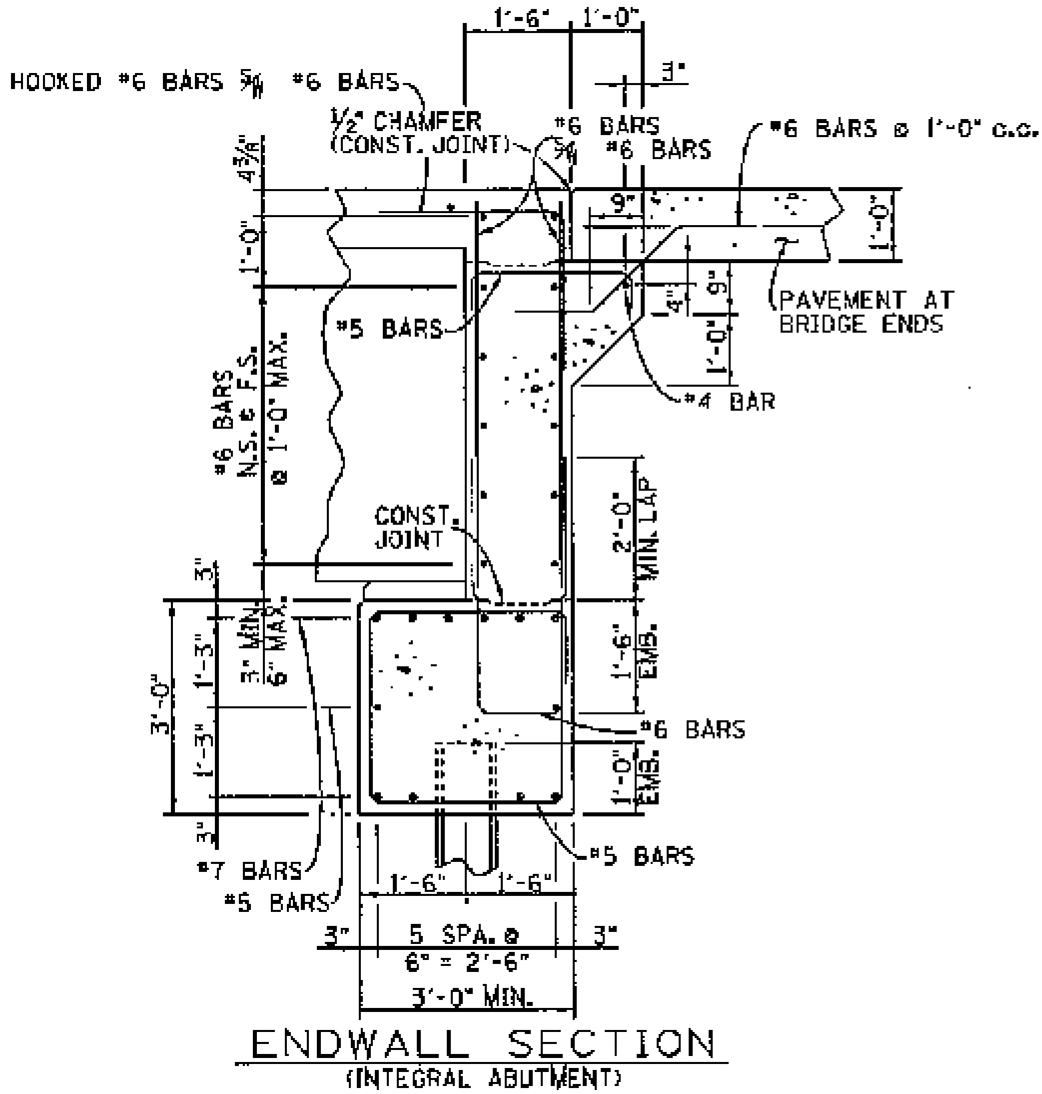


Figure 49. Example detail of a full integral abutment (courtesy of Tennessee DOT)

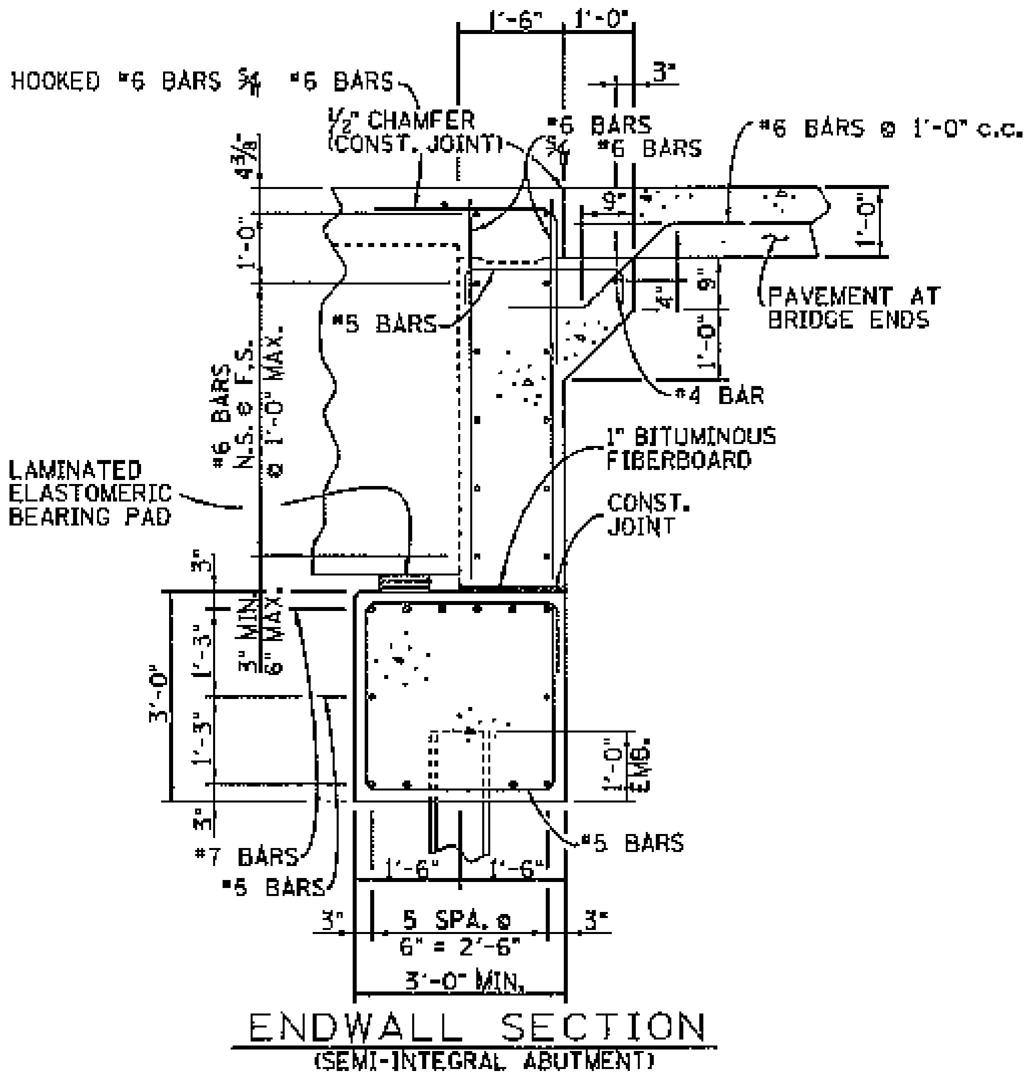


Figure 50. Example detail of a semi-integral abutment (courtesy of Tennessee DOT)

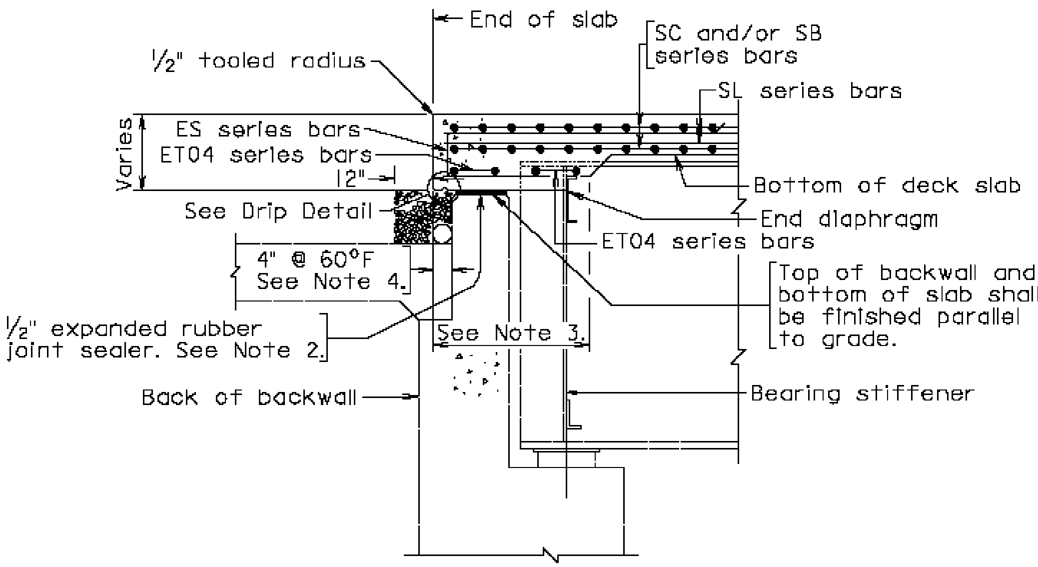


Figure 51. Example deck extension detail (courtesy of Virginia DOT)

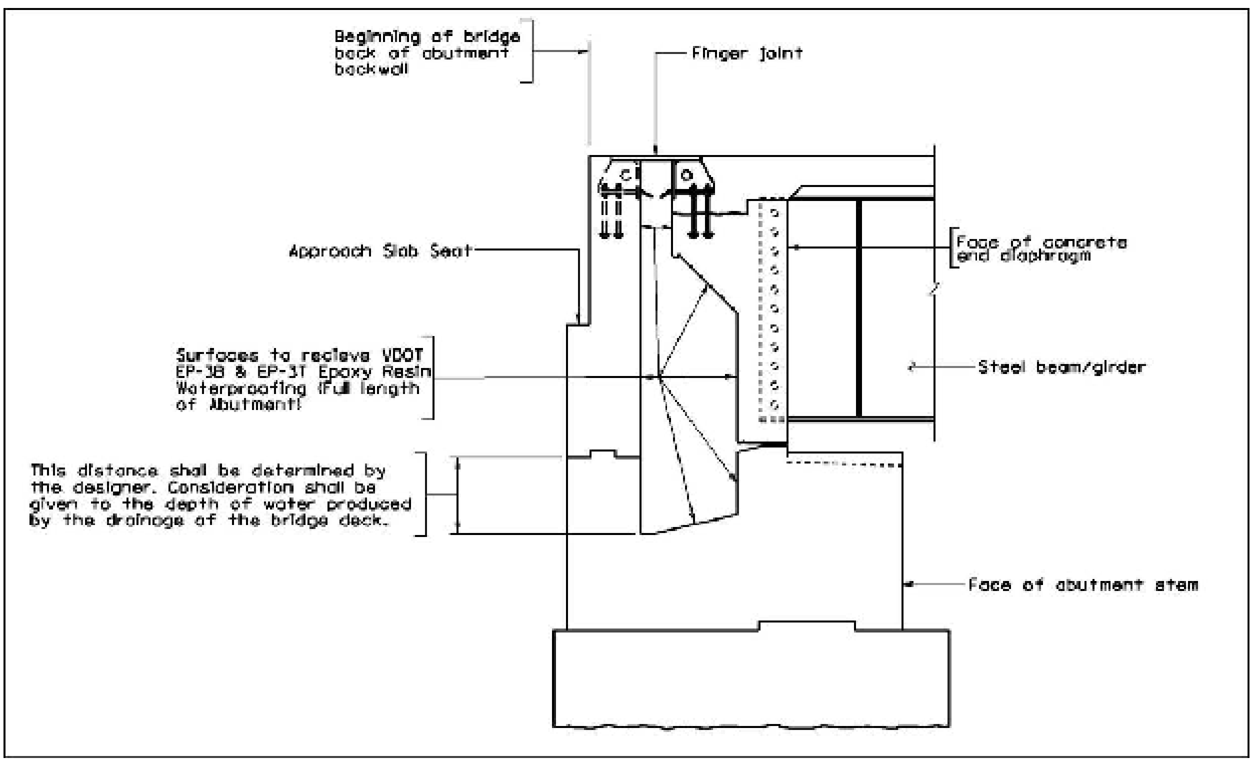


Figure 52. Example shifted joint detail (courtesy of Virginia DOT)

Full integral abutments are used commonly in many U.S. states for bridges having maximum total lengths from 250 ft. up to 400 ft. Thermal movements are accommodated within the foundation and are typically assumed unrestrained in the design of the superstructure. Various devices are utilized by different organizations to accommodate the thermal movements without causing damage to the substructure or superstructure. These include:

- limiting the bridge length, skew and/or horizontal curvature,
- use of select backfill materials and/or uncompacted backfill,
- spanning the area disturbed by the foundation movements immediately behind the abutments with the approach slab, thus avoiding settlement of the slab and the associated surcharge loads,
- limiting the foundations to a single row of vertical piles,
- limiting the pile type and requiring a minimum pile length,
- orienting vertical H-piles such that they are subjected to weak-axis bending due to the longitudinal movements,
- providing a hinge detail within the abutment to limit the moment developed at the tops of the piles,
- anchoring the approach slab to the superstructure with a detail that allows rotation of the approach slab at the abutment to accommodate settlement of the approach fill and
- provision of an expansion joint at the roadway end of the approach pavement.

In cases involving longer bridges, larger skew angles, abutments resting on rock, massive cantilever abutments, etc., where the foundation is less likely to accommodate the required movements, semi-integral abutments are a second option to eliminate the deck joints. In this case, the girders typically are integral with the backwall, but the required movements are accommodated by separating the backwall from the abutment stem as shown (Figure 50). In semi-integral abutments, the girders are seated on expansion bearings.

Deck extensions can be a good option for situations similar to those requiring a semi-integral abutment. Also, they can be useful for retrofitting existing structures (Weakley, 2005).

Finally, other shifted joint details have been used in longer bridges to move the joint away from the bridge structural elements. The particular detail shown in Figure 52 uses a concrete flume connected to a drainage system located beyond the end of the bridge.

D) Details at Interior Supports of Multi-Span Bridges

In addition to the above details at the abutments, states and owner agencies have adopted various other jointless concepts at interior piers of multi-span bridges. One of the concepts implemented by a number of owner agencies has been the use of simple-spans for dead load made continuous for live load (Freyermuth 1969; Oesterle et al., 1989). A common implementation of this type of construction involves the use of precast, prestressed girders connected with a continuous cast-in-place deck slab as illustrated in Figure 53. 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 interior pier diaphragms.

Badie et al. (2001) discuss the alternate use of an interior steel pier diaphragm with prestressed girders to speed the construction and achieve better overall design economy. The concept of simple-span made continuous has also been applied to eliminate interior joints and to improve the construction speed and design economy for short and medium span steel girder bridges (Talbot, 2005).

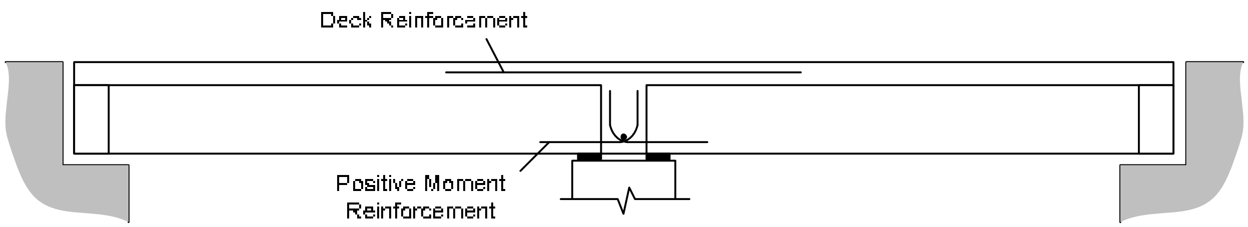


Figure 53. Precast, prestressed girders connected with a continuous cast-in-place deck slab

Unfortunately, some simple-made-continuous prestressed concrete girder bridges have experienced severe cracking in the girders near interior diaphragms. One example of this 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 creep of the prestressed girders under the influence of the prestressing. According to Telang and Mehrabi, "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 discussions of this problem and solutions to avoid it have been published by Oesterle et al. (2004) and Arockiasamy and Sivakumar (2005).

Interestingly, Oesterle et al. (2004) pointed out that the earlier studies by Oesterle et al. (1989) on simple-made-continuous bridges indicated that the positive moment connection in the diaphragm does not provide any structural advantage under service load conditions and is not required.

A "jointless" deck concept implemented by a number of states is commonly referred to as link slab construction. In this type of deck detail, the slab spans continuously over the length between the adjacent girders while the adjacent girders are kept as simple-spans (see Figure 54). The length of the deck connecting the two adjacent simple-span girders is called the link slab (Caner and Zia 1998). Link slabs generally require less deck reinforcement, but place more positive moment demands in the girders than "simple-made-continuous" designs. Caner and Zia (1998) provide a complete design procedure validated by analysis and laboratory experimental tests. The stiffness of the link slab is much smaller than that of the girders. Therefore, the girders are designed as simply-supported and the link slab is designed to accommodate the simply-supported girder end rotations and to limit crack sizes. Caner and Zia recommend the use of this system for eliminating all the interior joints in bridges for up to four spans, thus eliminating 60 percent of the expansion joints. Systems of this type potentially can be used for even longer spans as long as thermal movements are not restrained significantly by the link slab regions.

Link slabs are not common in the "snow belt" states. A crack is invariably formed due to deck slab rotation as the bridge is loaded with live loads. Some states in the South and Southeast use the system due to its simplicity and because de-icing chemicals are not needed.

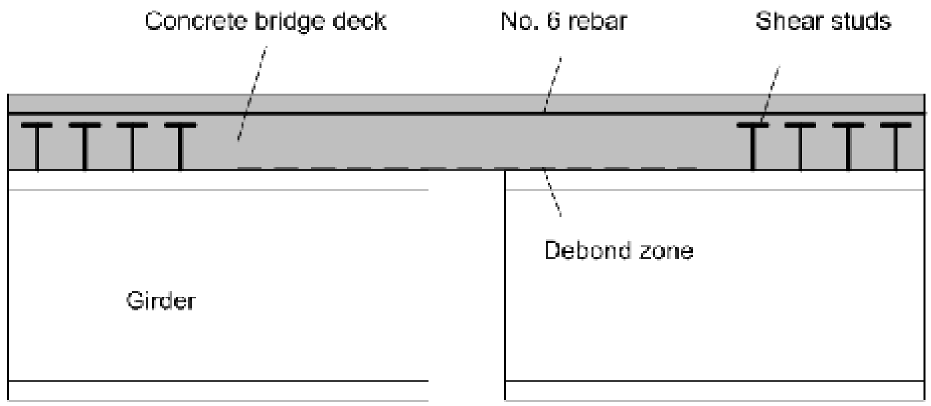


Figure 54. Link Slab concept for jointless deck

More recently, Wing and Kowalsky (2005) describe the monitoring and assessment of a pilot study link slab bridge in North Carolina. Furthermore, Kim et al. (2004) have researched the application of Engineered Cementitious Composites (ECC) to enhance the crack-width control, deformation capacity, fatigue performance and construction/placement of link slabs. These researchers have employed polyvinyl alcohol (PVA) fibers, approximately 2 % by volume, to achieve these improved material characteristics. Their design has been implemented in a prototype bridge by the Michigan DOT. A number of expansion joints are being replaced by link slabs using the Kim et al. ECC design.

E) New Research Needs

Tremendous advances have been achieved in understanding the behavior of jointless bridge systems over the last three decades, particularly in the last several years. This is evidenced by a 1994 British colloquium (Pritchard 1994), a 1996 FHWA workshop (Burke 1996), and a 2005 FHWA Conference on the topic (FHWA 2005) in addition to various research studies funded by different states (e.g., Arochiasamy et al. (2004), Fennema et al . (2004), Brena et al. (2007) and Civjan et al. (2007)) and other guideline papers and documents on the topic (e.g., Wasserman and Walker (1996), Wasserman (1997) and Yannotti et al. (2005)). Extensive experimental, analytical and field observation studies have been conducted in recent FHWA supported research. A number of conference papers (e.g., Oesterle et al. (2004), Oesterle and Lotfi (2005), and Oesterle and Volz 2005) present some of the results from this research. Nevertheless, there are still a wide range of concepts and details being used in design practice. The most promising details need to be compared and contrasted such that the best practices can be better understood.

At the present time, none of the above studies provide any definitive assessment of the service life associated with different details. Wing and Kowalsky (2005) state, based on a one-year monitoring of their prototype bridge, "While the size of the crack in the link slab exceeded the design criteria, it thus far has not impaired the service of the bridge. Furthermore, the large size of the crack (1.6 mm) is due to the saw cut which forces all the deformation into one crack. Revised crack control criteria should be investigated for link slabs that contain saw cuts to control crack location." The introduction of advanced materials, such as ECC for link slabs, is certain to improve the service life performance of jointless bridge details. However, the more traditional detail developed by Caner and Zia (1996) may provide sufficient service life performance in certain applications. The service life performance of the various jointless bridge components and details needs to be assessed. These estimates can be accomplished by a combination of field observations, testing and analytical modeling. For instance, various advanced link slab systems can be subjected to physical (load-deformation) and accelerated environmental testing to determine their deterioration response.

The potential application of advanced materials beyond the enhancement of link slab systems should be studied. For example, it is unlikely that the link slab concept can be combined with integral abutments to achieve a completely jointless bridge design. The girder continuity may be needed to accommodate the forces from the integral abutments. The usage of link slab or other simplified continuous details at interior piers with some type of jointless or shifted-joint detail at the ends of the bridge should be studied. If girder continuity is necessary, advanced materials such as ECC might be applied to improve the crack control in interior diaphragms and in prestressed girders adjacent to the interior diaphragms.

The influence of skew and/or horizontal curvature on the overall response of jointless bridges is still not well understood. Large skew generally leads to lateral movement at integral abutments and overall torsional rotation of the bridge in plan as the soil friction is exceeded against the abutment wall. Depending on the transverse stiffness of the abutment, significant transverse forces can be generated. For skewed continuous bridges, this twisting induces additional forces in intermediate pier bents. Significant progress in the analysis of this behavior is described by Seinberg et al. (2004) and by Oesterle and Lotfi (2005). However, further analysis and assessment of the behavior is needed for various span lengths and for various details, including semi-integral abutments and slab extensions.

The 4th Edition AASHTO *LRFD Specifications* provides only limited guidelines for design of integral abutments. No unified procedure has been established for the analysis, design and construction of these types of bridges. The level of analysis used in designing integral abutment bridges varies significantly. Designs of the superstructure often are not based on any consideration of soil-structure interaction from the integral abutments. Some fixity at the abutments is considered by others in the girder design. The assumption of simply-supported end conditions for the girder design is a reasonable one for longer spans; however, economies can be gained by consideration of the continuity with the foundation and the resulting frame action in shorter spans. Nevertheless, the influence of various potential loadings on the resistance provided by the foundation system must be accounted for in some simplified manner. Generally, some analysis of the loads on the piles is addressed. Researchers have developed sophisticated capabilities for analysis of soil-structure interaction considering nonlinear effects. Refined capabilities are also available for analysis of continuity effects in simple-made-continuous prestressed concrete girder bridges. All of these procedures are useful for analysis of tests and for quantifying and bounding the overall behavior of various systems. Simplified models are necessary for design. The various analysis approaches that have evolved in recent research and practice should be synthesized and appropriate simplified analysis models recommended for different details and conditions. Details that accommodate larger movements while eliminating joints from the bridge system should be investigated. Advanced analyses methods can be coupled with experimental testing and field observations to quantify the performance of new details. Completed studies must demonstrate the behavior clearly and provide a clear and complete step-by-step illustration of the calculations recommended for design.

F) Bridges with Joints

When bridge lengths get beyond what can be tolerated with integral or semi-integral abutments, joints must still be considered. Typically, bridges up to about 1,500 feet have been accommodated with joints only at the abutments. Figure 55 shows a long continuous bridge without joints in the middle. Some concrete bridges have gone as far as 2,500 feet with joints only at the abutments. Longer viaduct bridges have typically been split up into segments, with intermediate joints over piers at about 1,000 feet maximum spacing. When joints are needed, today's practice is still to minimize the number of joints as much as possible.

Figure 55. A long continuous steel bridge with joints only at the abutments

Bridge deck joints are subjected to continual wear and heavy impact from repeated live loads and continual cycles of movement from expansion and contraction caused by temperature changes, creep and shrinkage and/or long-term movement effects such as settlement and soil pressure. In some cases, joints are subjected to impact loadings that exceed their design capacity. In addition, retaining hardware for joints is often damaged and loosened by snowplows and the relentless pounding of heavy traffic and can become a hazard to motorists and a liability to owners.

A wide variety of joints have been developed over the years to accommodate a range of movements, and promises of long lasting, durable, effective joints have led states to try many of them. Some joint types perform better than others, but all joints can cause maintenance problems.

Joints typically fall into two categories: open joints and sealed joints.

Open joints, such as sliding plate and tooth or finger joints, allow drainage to pass through and to be collected in a trough below. Without constant maintenance, these troughs fill with debris, corrode or break, and then allow drainage to spill out below. Neoprene troughs have been used in some cases to try to provide a non-corroding element, but these can fill up with dirt and debris and ultimately tear.

Sealed joints, such as neoprene compression seals, strip seals or modular joints, are intended to prevent drainage from passing through the joint. Failures have occurred, however, resulting in early and costly joint replacements. Strip seal joints are now used in favor of compression seal joints, offering better performance due to the positive attachment of the end of the strip to an embedded extrusion. Compression seals tend to pull out leaving the joint open. Typically, strip seals can accommodate movement up to about five inches, although some manufacturers have products that claim to accommodate movement up to seven inches.

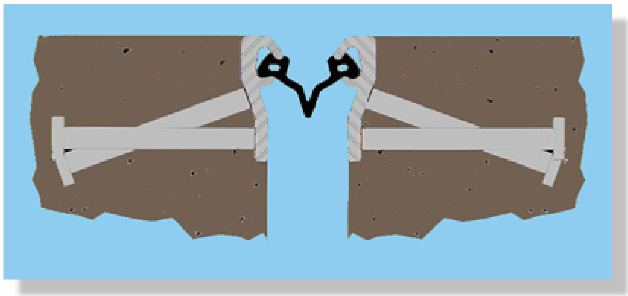
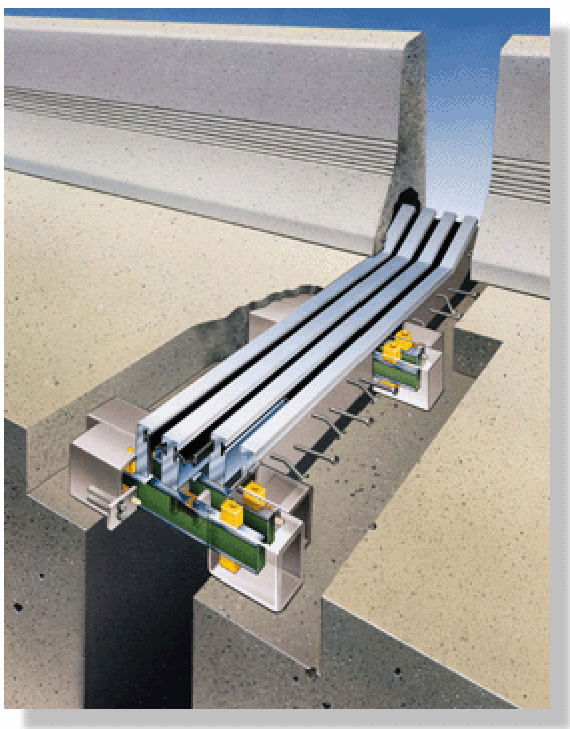


Figure 56. A neoprene strip seal joint



D.S. Brown Steelflex® Modular Expansion Joint System

Figure 57. A modular expansion joint

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 inches. These types of deck joints have evolved over the years leading to improved performance and reliability. Many premature failures of earlier modular joints were attributed to fatigue problems. Expansion joints are subjected to more load cycles than other superstructure elements, but the load types and magnitudes and fatigue-stress ranges that are applied to these joints were originally not well defined. Additionally, sufficient data was not available on field measurements and laboratory testing of fatigue-critical joint details. Neither the AASHTO *Standard Specifications for Highway Bridges* nor the AASHTO *LRFD Bridge Design Specifications* included fatigue design criteria for these joints (AASHTO 1996, 1994). As a result, research was undertaken to develop performance-based specifications and supporting commentary for the fatigue resistant design of modular bridge expansion joints. The researchers identified critical parameters that influence fatigue performance of modular bridge expansion joints and the predominant causes for fatigue failure of in-service joints. Using field tests of modular bridge expansion joints, they were able to assess behavior under static and dynamic truck loading and to define critical stress locations. Fatigue criteria were developed through laboratory testing. These activities culminated in specific proposals to ensure the design of durable modular bridge expansion joints. The findings of this study are documented in NCHRP Report 402, Fatigue Design of Modular Bridge Expansion Joints.

G) Concrete Headers

Strip seals and modular joints are typically set in a deck block-out, which is then filled with concrete after the joint is properly adjusted to grade. This concrete fill has a higher probability of spalling and cracking due to wheel loads and tensile forces.

Elastomeric concretes were developed to prevent the spalling of the Portland cement concrete adjacent to bridge deck expansion joints. Field tests were conducted in Wichita, starting in 1991, to survey both elastomeric and conventional concrete header materials over a ten-year period. Spalling at each joint, rutting of the elastomeric materials and overall condition of the materials were measured and recorded. Laboratory tests of field-cast specimens were performed to determine the mechanical properties of the materials. The results of the tests and surveys show that the elastomeric concretes reduced spalling at bridge expansion joints.

4.2.4.7 Discussion on Bearings

This section provides a general discussion on bearings. Figure 58 provides a list of issues discussed in this section.

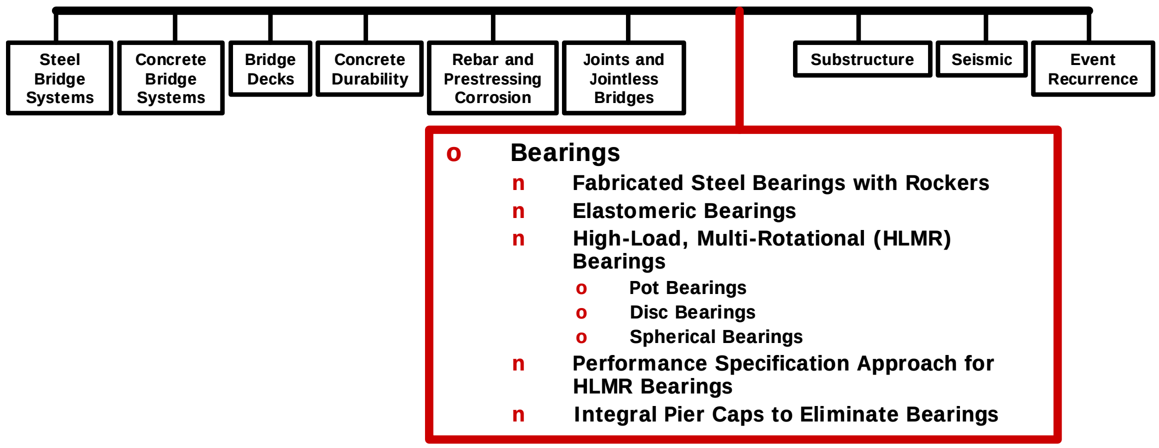


Figure 58. List of factors related to bearings discussed in this section

Bearings are an important bridge component that must be considered when evaluating potential ways to extend overall bridge service life. Bridges experience translational movements and rotations caused by traffic loading, thermal effects, creep and shrinkage, initial construction tolerances and other sources. Bridge bearings are designed and built to accommodate these movements and rotations while supporting required gravity loads and providing the necessary restraint to the structure. There are many types of bridge bearings that have been used over the years and many are in current practice today. Lack of proper maintenance, particularly at expansion bearings and at bearings below deck joints, have caused many bearings to undergo major rehabilitation and replacement before their expected service life and well before the expected bridge service life. Newer types of bridge bearings using elastomeric materials currently have expected service lives well below 100 years. Therefore, it is important to provide bearing installations that are properly detailed to allow for easy inspection, maintenance and potential replacement.

In today's practice, bearing type selection is typically based on function, i.e., load and movement requirements and cost. Various types of bearings have been used in the industry to meet these specific needs. Each has distinct advantages, application limitations and potential shortfalls in performance. The following compares the more common types, discusses issues that affect service life and describes the potential for extending service life.

A) Fabricated Steel Bearings with Rockers

Through the 1970's, steel bridges were normally supported by fabricated steel bearings with rockers, which were expensive to fabricate and install and typically provided uni-directional movement. Figure 59 illustrates a fabricated steel bearing with rockers. These bearings were assumed to have the same service life as supported girders, but steel bearings, particularly those located below deck joints, are susceptible to corrosion due to roadway drainage with de-icing salts and other dirt accumulation. Corroded expansion bearings can "freeze", subjecting beams and substructure elements below to additional load. Thus, steel bearings require continual cleaning and painting to insure proper structural performance. The initial cost of these types of bearings along with the associated maintenance requirements has led designers to investigate other options for new steel bridges.

Many existing steel bridges, however, still have fabricated steel bearings with rockers that need to be addressed as part of extending service life. Proper field cleaning and coating with high-performance paint systems will provide extended life. (This is similar to the previous discussion on coatings for existing steel bridges.) The main issue for extending service life for existing steel bearings will be elimination of deck joints above that can ultimately leak and allow drainage and salt contamination, leading to corrosion. Another option is the replacement of the steel bearings with more serviceable bearings.

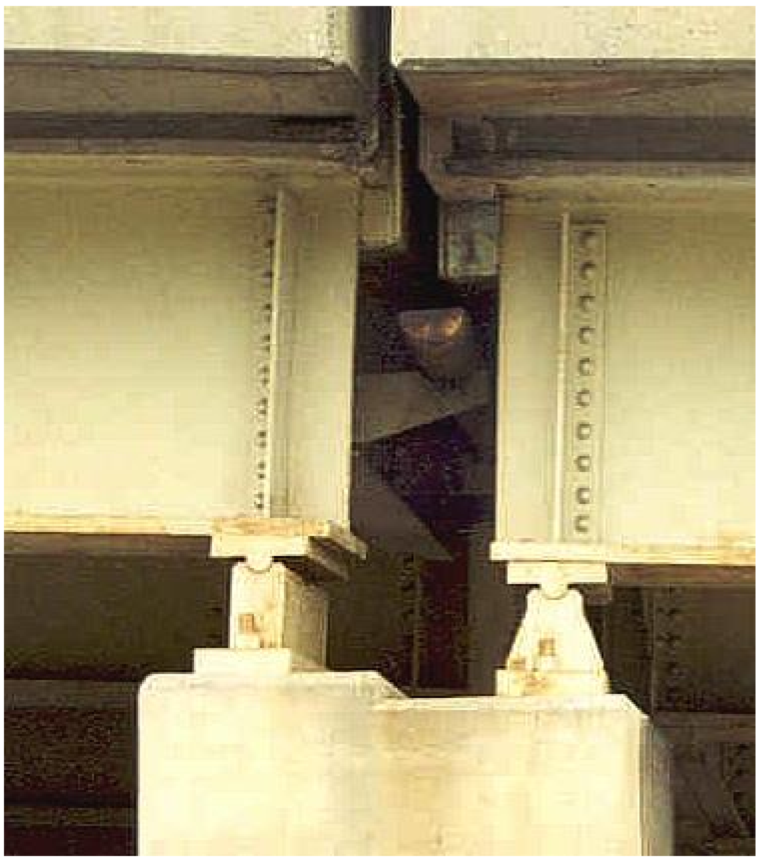


Figure 59. Fabricated steel bearing with rockers

B) Elastomeric Bearings

Elastomeric bearings using neoprene pads have become very common in short and medium span bridges. Neoprene bearing pads were first developed and used for precast, prestressed concrete beams. They were later considered and adapted for steel beams and girders as a simpler, more cost-effective bearing type than fabricated steel bearings with rockers. Elastomeric bearings have no moveable parts and accommodate movement and rotation by deformation of the elastomer. Lateral and longitudinal movement is accommodated by the pad's ability to deform in shear, but the bearing must be carefully designed to control the strains and deformations in the elastomer to assure a long service life and good bearing performance. As requirements increase for vertical load and movement, thin steel plates can be combined with multiple layers of elastomer to form a laminated steel-reinforced elastomeric bearing assembly, illustrated in Figure 60. Correctly designed and installed, elastomeric bridge bearings can be confidently expected to function efficiently for 20 to 40 years. However, some states have indicated problems using elastomeric bearings, resulting in much shorter service life.

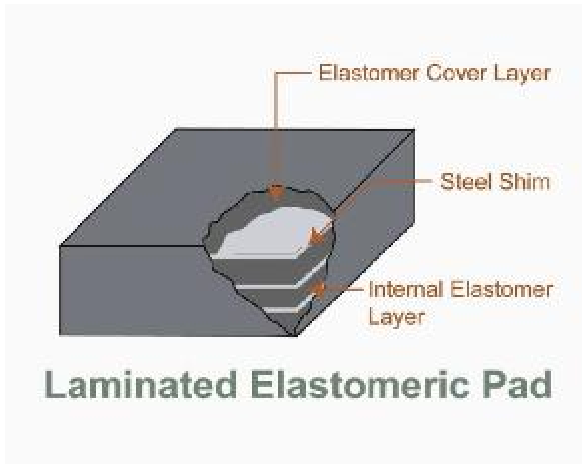


Figure 60. Laminated elastomeric pad

Current problems experienced with elastomeric bearings need to be researched to determine causes and potential solutions. As discussed previously, new structures using elastomeric bearings need to be detailed to accommodate easy bearing replacement.

C) High-Load, Multi-Rotational (HLMR) Bearings:

When elastomeric bearing assemblies will not accommodate anticipated loads and displacements, high-load, multi-rotational (HLMR) bearings are considered. HLMR bearings include pot, disc and spherical bearings. The following describes and compares each.

C.1 - Pot Bearings

Pot bearings, which are traditionally the most economical and common HLMR bearing, accommodate rotation by deformation of a confined elastomeric pad. They have been noted to have problems such as leakage of the elastomer, broken seals, abraded elastomeric pads and metal-to-metal contact. However, they are readily available from numerous sources and have been implemented on bridges throughout the country. Figure 61 shows a typical pot bearing.

Experience to date indicates that many state DOTs recommend avoiding pot bearings because of performance problems and quality control issues related to fabrication and installation. One item to note is that fabrication processes have improved substantially since the development of pot bearings in the early 1960's. 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 in order to achieve satisfactory performance.

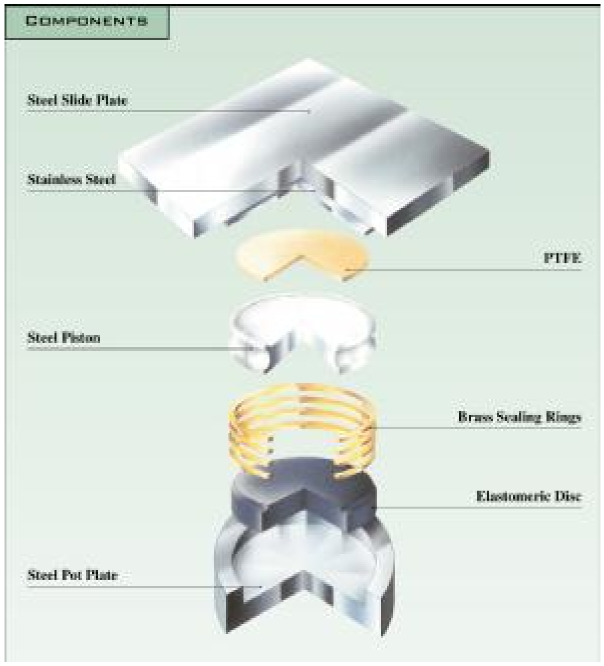


Figure 61. Typical Pot Bearing Components

C.2 - Disc Bearings

Disc bearings consist of a hard polyether urethane disc between upper and lower steel plates with a ball-and-socket like device to resist horizontal load. The discs are stiff enough to accommodate vertical deflection, yet flexible enough to permit rotation. Figure 62 illustrates a typical disk bearing. The failures that have been noted are generally not in the bearing, but are associated with the disc and are linked to the compressive and rotational stiffness of the material. In addition, there have been failures of the PTFE surfaces on the guide bars, which are considered to be a manufacturing issue rather than a design issue. Disc bearings are reasonably economical, but there is generally a concern in a bidding environment due to limited sources. Many states have had little or no experience with disc bearings. Widespread use has been limited because it is patented, and until recently, was only available from a single source. Now there are several bearing manufacturers that can supply disc bearings.

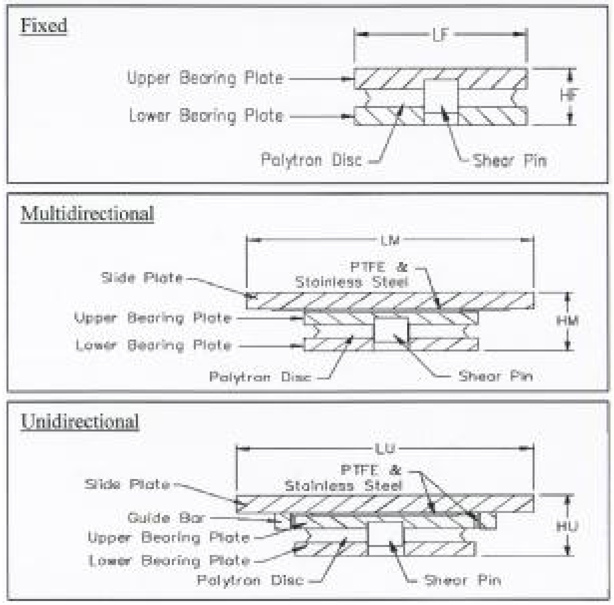


Figure 62. Typical Disc Bearing

C.3 - Spherical Bearings

Spherical bearings are traditionally considered to be the most expensive HLMR bearing type. They are also typically considered to be the most reliable. As a result, a number of states (California and Washington in particular) have adopted policies to restrict the use of HLMR bearings and only allow spherical type bearings. These two states are of particular interest because of the large facilities (with large bearing demands – movements and loads) in their programs. Some states, however, have had problems with spherical bearings. It appeared that the difficulties were attributable to the fabrication of the bearings by less experienced manufacturers.

Spherical bearings accommodate rotation by sliding a convex metal surface against a concave spherical PTFE surface. Translational movement is sustained by incorporating flat PTFE sliding surfaces, as shown in Figure 63. 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. For any condition other than the typical sliding case, they generally require more space. These are more highly machined fabrications and are more sensitive to initial manufacture, quality of installation and maintenance than disc bearings. As a result, they are generally the most expensive type of HLMR bearing.

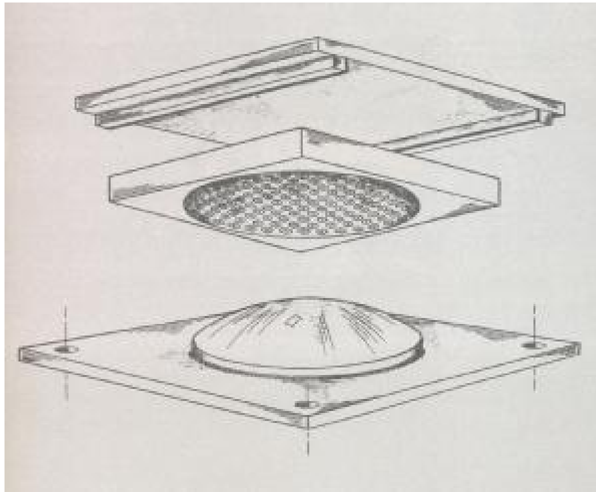


Figure 63. Typical Spherical Bearing

D) Performance Specification Approach for HLMR Bearings:

There are several traditional approaches to the specification of HLMR bearings on projects. The most common is the use of a performance specification with a generic design that allows the contractor to submit an HLMR bearing of their choice, subject to the review and approval of the owner. In a competitive bid scenario, this approach usually results in the most economical bearing being provided. As noted above, that often results in the implementation of a pot bearing, unless the specifications are quite restrictive. The argument against this approach is that while all the bearing types may satisfy initial design criteria, you may not get a bearing that best suits both the design and long-term maintenance goals for the bridge.

Another approach is to specify the type of bearing, as determined by the owner, to best suit the needs of the project. This approach is taken when project demands are considered unique enough to predicate exclusion of alternative HLMR bearings (i.e., when the alternatives are determined/perceived to be less desirable for design reasons or performance concerns). This approach has been adopted by several states on a program-wide basis (as noted above). The argument against this approach is that by excluding competition you may artificially drive the cost of the bearing higher than could be obtained in an open bidding environment.

As noted above, the quality design, manufacture and installation of HLMR bearings are paramount to achieving satisfactory service. Stringent Special Provisions should be developed to insure a reputable manufacturer and a high-quality bearing.

Similar to bridges using plain or reinforced elastomeric bearings, bridges using HLMR bearings should also be detailed to accommodate easy inspection, maintenance and probable bearing replacement during the bridge life span.

E) Integral Pier Caps to Eliminate Bearings

Integral pier caps have been used primarily to accommodate vertical clearance issues in interchange ramp configurations, but they can also serve to eliminate bearings and associated future maintenance. Cast-in-place, post tensioned bent caps have been used for many years now but are not widely employed. The concept allows steel girders to pass directly through the pier's cap, rather than over the top of the cap in the traditional manner. This can overcome clearance restrictions and avoid extreme skews. Integral caps can also maximize column efficiency by halving the column design moments and shears compared to conventional cantilever columns. Use of integral pier caps also enhance seismic performance of bridges. Figure 64 shows a steel bridge with integral pier cap.

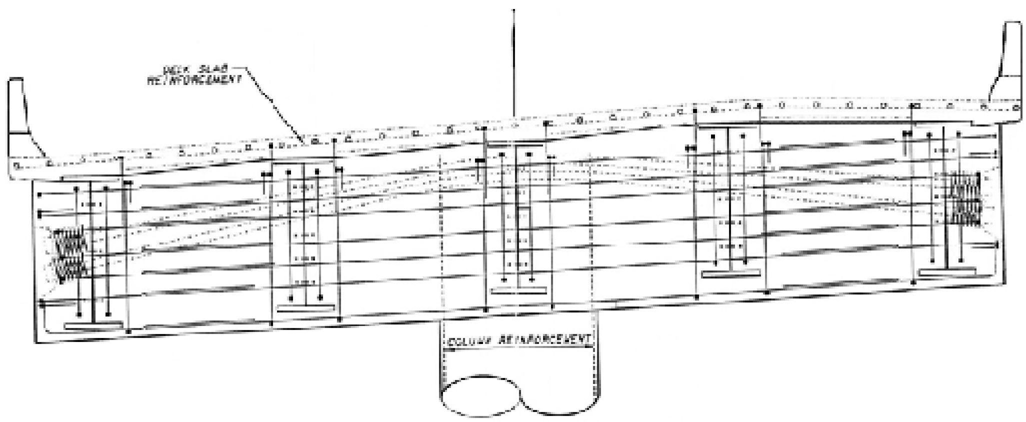


Figure 64. Integral pier detail used in Ohio

The use of integral piers using a single column pier is shown in Figure 65 or using a straddle bent is shown in Figure 65. Continuity of the cap with the pier as shown in Figure 65 maximizes the pier column efficiency as stated previously. Furthermore, integral pier caps help satisfy vertical clearance requirements and improve the design aesthetics. Abu-Hawash et al. satisfy redundancy requirements at integral steel pier caps by using twin HPS I-girders (2005).

Figure 65. Typical post-tensioned concrete integral bent cap with a single column pier (Wasserman 1997) (courtesy of Tennessee DOT and NSBA).

Figure 66. Straddle bents with integral steel pier caps (Abu-Hawash et al. 2005) (courtesy of Iowa DOT, HDR Engineering and NSBA)

Performance of integral pier detail for steel bridges in seismic regions was researched by Patty, Seible and Uang (2002).. They concluded that in seismic regions, post-tensioned concrete diaphragm provided the best behavior. Figure 67 shows the type of specimens investigated.

Figure 67. Type of specimen investigated -- note the detail over the pier. Steel bridge system with integral pier

According to the same study, post tensioning the concrete diaphragm is an effective way of controlling torsional type cracks that can form in the diaphragm when the bridge is subjected to ground motion parallel to span length (longitudinal).

4.2.4.8 Discussion of Substructure Elements

A general discussion of substructure elements as related to service life of bridges is provided in this section with Figure 68 providing a list of issues discussed.

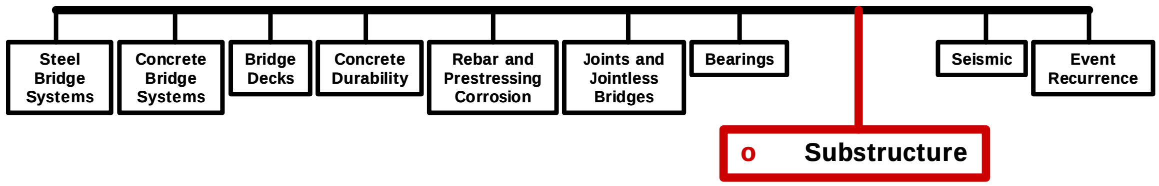


Figure 68. Substructure

Substructure degradation leading to structural deficiencies is an important aspect of bridge life that will be researched thoroughly in this study. Figure 69 (Chase et. al., 1997) shows a comparison of ratings for key items, and indicates the high number of bridges in the national bridge inventory with a substructure that is rated poor or worse.

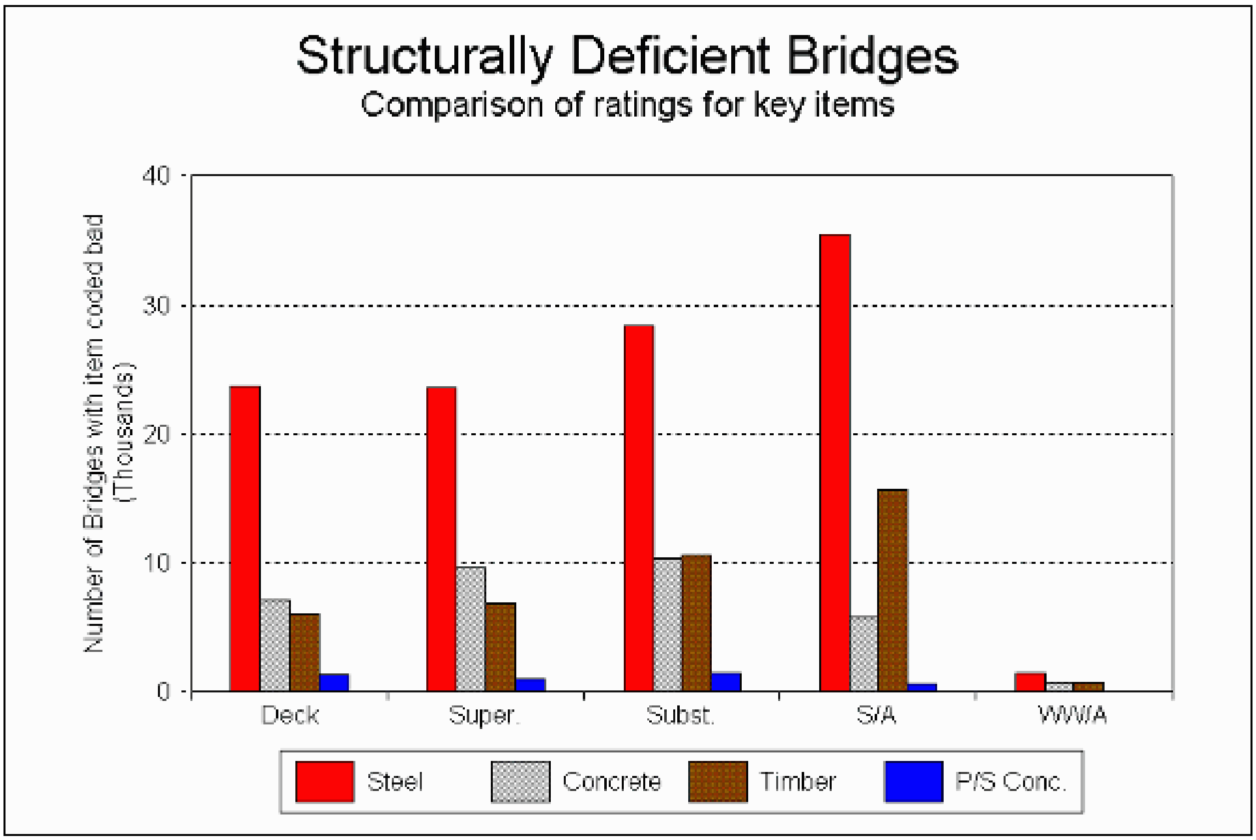


Figure 69. Comparison of rating for key items in the National Bridge Inventory

Various reasons for substructure degradation include:

- Corrosion due to chloride intrusion. These chlorides are found in marine and brackish water environments affecting piles and other substructure units at the waterline, or from leakage of expansion joints where salt spray and/or de-icing salts are found on bridge decks. The leakage of these joints concentrates the chlorides at the top of the pier cap, affecting not only superstructure elements but the top and sides of substructure elements. Similar concentrations can also occur due to breaches in bridge drainage systems. Chloride intrusion and corrosion of reinforcement have been addressed at length in the previous sections.

- Undermining of bridge embankments and foundations. The 1987 collapse of the I-90 Bridge over Schoharie Creek has emphasized the importance of providing adequate hydraulic openings and flow characteristics under bridges. Potential undermining and structural stability under check flood conditions has resulted in the implementation of scour countermeasures on numerous bridges.
- Seismic susceptibility. Recent advances in the understanding of structure stability and ductility requirements for bridges have resulted in the implementation of seismic retrofits on numerous bridges.
- Insufficient pier protection. Impact forces from automotive vehicles and marine vessels have resulted in structural damage to substructure units. Susceptibility to damage from these aberrant vehicle/vessel impacts underscores the importance of pier protection. Susceptibility to fire damage will also be reviewed.
- Underwater bridge inspection techniques and other means for detecting substructure degradation will be investigated since there are very few visual observation and non-destructive testing options available. Many of the techniques for monitoring and testing performance of structures available for above-ground bridge elements do not apply well to below-ground elements.

4.2.4.9 Discussion on Seismic Issues

This section provides a general discussion on seismic design-related issues and their relation to service life.

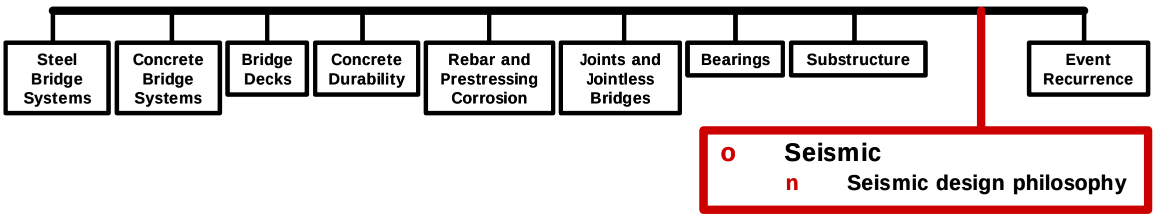


Figure 70. Seismic

A) Seismic Design Philosophy

The present AASHTO design guidelines for bridges in high seismic zones are based on a 50-year service life. Design ground motions are specified with a 10% probability of exceedance during the life of the bridge. These design ground motions relate to criteria based on the preservation of life-safety through prevention of collapse for the structure.

As mentioned in the preceding section, if the expected service life of a bridge is 75 years, the probability of exceedance will increase. In terms of serviceability criterion, there is a greater chance the bridge will be subjected to an event that will cause significant structural damage – which may or may not be repairable.

More probable "moderate" earthquakes require that the structure not experience significant structural damage, implying that the response remains within the elastic range. A longer service life requires that the design ground motions for a "moderate" earthquake be increased from present levels.

The current seismic design criteria published in the Bridge Design Manual by the Washington State Department of Transportation (WSDOT) states that "WSDOT has implemented the new MCEER seismic design criteria for only the design of specific large projects that involve substantial financial investment, such as the Alaskan Way Viaduct Replacement, using the Life Safety performance level only. It is expected that AASHTO will eventually adopt the new criteria. At that time, WSDOT will then use it for the design of new bridges." (WSDOT Bridge Design Manual M23-50 2006)

As mentioned above, the AASHTO Subcommittee on Bridges and Structures has very recently voted to adopt the MCEER seismic design criteria, which specifies a longer return period of the design earthquake (1000 years) and also incorporates displacement-based principles in design. A description of the research underlying these developments may be found at MCEER/ATC (2003).

The overall impact of the recent AASHTO seismic design changes on designing bridges for longer service life needs to be assessed as part of the proposed research.

Caltran presently requires designers to look at two levels of ground motion and structural response:

1. Functional level
2. Safety level

Figure 71 from Caltran's Seismic Design Criteria Memo to Designers 20-1 summarizes these requirements. Bridges are also divided into "ordinary" and "important" categories. The safety-evaluation earthquake has a long return period (1000+ years) and is represented as the maximum credible earthquake. The function-evaluation earthquake has a 40% exceedance probability during the service life of the bridge. Both earthquakes are determined probabilistically.

Table 1 - Seismic Performance Criteria

Ground Motion at Site	Level of Damage and Post Earthquake Service	
	Ordinary Bridge	Important Bridge
Functional - Evaluation Ground Motion	Service: Immediate Damage: Repairable	Service: Immediate Damage: Minimal
Safety - Evaluation Ground Motion	Service: Limited Damage: Significant	Service: Immediate Damage: Repairable

Definitions:

Functional - Evaluation Ground Motion: This ground motion may be assessed either deterministically or probabilistically. The determination of this event is to be reviewed by a Caltrans-approved consensus group.

Safety - Evaluation Ground Motion: This ground motion may be assessed either deterministically or probabilistically. The deterministic assessment corresponds to the Maximum Credible Earthquake (MCE). The probabilistic ground motion for the safety evaluation typically has a long return period (approximately 1000-2000 years).

MCE: The largest earthquake, that is capable of occurring along an earthquake fault, based on current geologic information as defined by the 1996 Caltrans Seismic Hazard Map.

Service Levels:

- *Immediate:* Full access to normal traffic is available almost immediately following the earthquake.
- *Limited:* Limited access (e.g. reduced lanes, light emergency traffic) is possible within days of the earthquake. Full service is restorable within months.

Damage Levels:

- *Minimal:* Essentially elastic performance.
- *Repairable:* Damage that can be repaired with a minimum risk of losing functionality.
- *Significant:* A minimum risk of collapse, but damage that would require closure to repair.

Figure 71. Seismic Performance Criteria (Caltrans 1999)

Design for longer service life would impact the choice of ground motions – especially for the functional level. Also, much of the recent research on seismic design has concentrated on the determination of structural performance for the life-safety and prevention of collapse criteria for major seismic events. Moderate-level earthquakes that may occur cumulatively during a longer service life have not been examined in such detail. A background study of the seismic design criteria would be conducted to assess the considerations.

4.2.4.10 Recurrence Level of Events

This section provides a general discussion on recurrence level of events and their relation to service life.

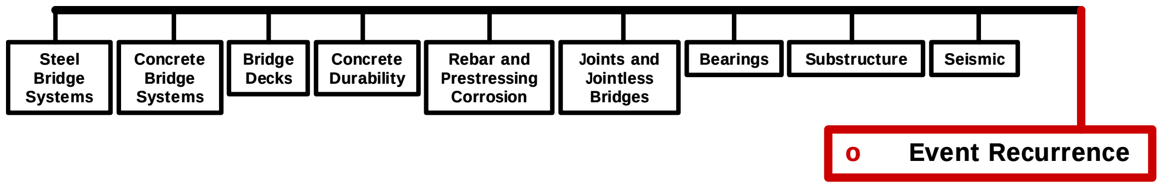


Figure 72. Event Recurrence

As bridge service lives increase, bridges are subjected to environmental conditions for a longer period of time. Many of these conditions are accounted for in design by use of traditional recurrence values of extreme environmental conditions such as for hydraulic stages, wind loads and seismic events. Design for ship impact is treated similarly. The assumed recurrence intervals used for the design of these events needs to be reexamined. The increased service life from 50-year (pre-LRFD) to 75-year (LRFD) to 100-year or 200-year design life increases the statistical probability of exceeding the "design" recurrence level event.

As an example, consider wind loading. Hurricane wind recurrence levels are developed from a statistical 20,000 year simulation using a database of historical hurricane strengths and storm paths (Vickery et. al., 2000). For most coastal geographic areas where the wind load is strongest, the 1.4 load factor for Strength Limit State III of the *AASHTO LRFD Bridge Design Specifications* defines an ultimate loading condition approximately equivalent to a 500-year recurrence storm (AASHTO 2007). The statistical probability of exceeding that criterion in 50, 75, 100 and 200 years is 10, 14, 18 and 33 percent respectively.

As bridge life increases, the recurrence levels used in design must address an acceptable statistical reliability. The recurrence levels currently shown in the AASHTO *LRFD Specifications* are:

- Seismic: 475-year essential bridges, 2500-year critical bridges
- Ship Impact: 1000-year regular bridges, 10,000-year critical bridges
- Scour Check Flood: 500-year with no reserve, no differentiation based on structure importance
- Hurricane Wind: 500-year (+/-), no differentiation based on structure importance

Importance characteristics for all recurrence type design events will also be investigated in this study to ensure adequate reliability.

4.2.5 Overall Philosophy for Developing a New Service Life Design Approach

Previous sections provided some of the available philosophies related to design for service life as well as a complete discussion of factors that affect service life of bridges in the United States. The material presented in previous sections is in part responsible for shaping the general approaches that will be taken in this project to achieve project goals. This section presents the general philosophy of the research team.

This project provides an opportunity to develop guidelines that could assist the design profession in developing bridges with higher levels of service life performance for both existing and new bridges. It offers an opportunity to take a fresh look at the entire spectrum of activities needed to develop long lasting bridges. Comprehending the causes of observed problems and understanding the methods used by bridge owners to resolve these problems is important. Comprehending available methods related to design for service life should be a major task within the project. The methodologies to be developed may be fundamentally different than our past practice; however, the form of presentation should not differ significantly. Designing for loads using load and resistance factors is an unambiguous and well-accepted principle. In new thinking, our loads and resistances will take new and different forms. This is a challenge that may be overcome by presenting the results in familiar forms and formats.

The new approach for design for service life has to be quantifiable. This, in itself, is a major departure from the past. Currently, the AASHTO ***LRFD Specifications*** does contain provisions leading to better service life performance. However, application of these provisions in general does not assure a quantifiable level of service life. This does not mean that current recommendations are not able to provide 100-plus-years of service life. We are simply unable to assign a number, in terms of years of service, when we use these recommendations. For instance, a recommendation that crack width should be limited for different exposure classes (as specified in LRFD Article 5.7.3.4) is not based on a rational service-life calculation ensuring, say, 100-plus-years of service life. In that sense, the new approach for service life design has to be quantifiable. It has to be able to predict whether the structure will last for 50, 75 or 100-plus-years without major maintenance. The methodology should be applicable to both new and existing bridges, where the existing level of deterioration can be assessed. Further, the procedure should be able to predict the service life of the bridge, following a retrofit/rehabilitation process.

In order to achieve long service life, the design process should include steps that traditionally have been left to others. The introduction of the LRFD Specifications tasked the designer to consider durability, maintenance and inspection with little in the way of quantifiable requirements. The ability to perform inspection is a factor in selecting the details that go into a bridge. Details that are very expensive to replace, repair or recoat should not be used. The bridge owner should have all information at their disposal, allowing them to make crucial decisions during the service life of the bridge. The contractor, when handing the bridge to the owner, should also provide the owner with a document containing all essential information about the bridge. This document could be referred to as an “owner’s manual.” The “owner’s manual” should include basic information necessary to make maintenance, rehabilitation and other future decisions, and should assist the inspector in his or her focus with critical parts of the bridge. The “owner’s manual” should reflect the verifiable as-built condition and not the values assumed during the design phase by the designer. For example, permeability of the concrete should be obtained from core samples obtained from the structure and not the specified values contained in the job specification. This will allow the bridge owner to predict with confidence the remaining life of the bridge and to make crucial maintenance decisions using tools that will be developed by the R19-A and R19-B projects. Last but not least, the "owner's manual" should be incorporated into the next generation of NBIS data base, providing a reference point. Recommendations will also be made on how to incorporate the data in the Owner Manual into AASHTO Manual for Condition Evaluation of Bridges in the Form of an Appendix.

4.2.6 Objectives

The objectives of this project are to develop methodologies, concepts and ideas that can improve existing systems, subsystems and components that have limited the service life of bridges, and to introduce promising concepts for alternative systems, subsystems and components capable of achieving longer service life. More specific objectives are:

- To identify the bridge systems, subsystems, components and details that have provided long service life.
- To identify the bridge systems, subsystems, components and details that, with modification, can provide longer service life.
- To identify promising alternative bridge systems, subsystems, components and details that could result in longer service life at optimum total cost.
- To identify causes limiting service life of bridges and current procedures used for eliminating these problems.
- To evaluate promising systems, subsystems, components and details capable of prolonging the service life of existing and new bridges, by conducting proof-of-concept investigation (a combination of experimental, numerical and analytical work).
- For the deterioration modes identified as significant, to develop models capable of predicting rates of damage. For each deterioration mode, the team will establish performance and acceptance criteria. These criteria can then be used to define service limit states.
- To develop general framework and methodologies for predicting service life of bridges in a quantifiable way. The team will use this methodology to establish a framework for the type of information that should be included in the AASHTO *LRFD Specification* and AASHTO *Construction Specifications* to achieve certain levels of service life, such as 100-plus-years of service life with minimal maintenance. In this methodology, the team will develop a scheme that could be used to categorize the bridges into several importance classes and will use this classification scheme as part of the criteria for selecting appropriate systems, subsystems, components and details. This classification will also allow the establishment of different levels of risk that could be accommodated during service life of the bridge.
- To develop a concept of an "owner's manual" for bridges that could be given to the bridge owner after construction, containing all the information needed by the owner to

make crucial decisions during the life of the structure. The type of data to be included in the “owner’s manual” should be information assisting in inspection, maintenance and rehabilitation of the bridge. The “owner’s manual” should include sufficient information to allow the owner to conduct life-cycle cost analysis for the purpose of selecting various activities during the service life of the bridge. One of the most important pieces of information to be included in the “owner’s manual” should be “hot spots” or regions of the bridge that inspectors should pay more attention to during the routine inspection of a bridge. These “hot spots” should be defined during the design process by the designer, and could require more active inspection and maintenance during the service life of the bridge.

- To develop information that could be used by FHWA to incorporate the findings of this project into future versions of the National Bridge Inspection Standards (NBIS) as charged through the Long Term Bridge Performance Program (LTBPP).
- To provide flexibility in the reporting of project findings to allow its usage as new information becomes available.
- To provide training material with detailed examples demonstrating the use of new technologies.

4.2.7 Phase I Work Plan

The objectives outlined in the previous section will be obtained by conducting Tasks 1 through 8 as later described. Figure 73 shows a brief description of the tasks to be carried out and the interrelation among various tasks.

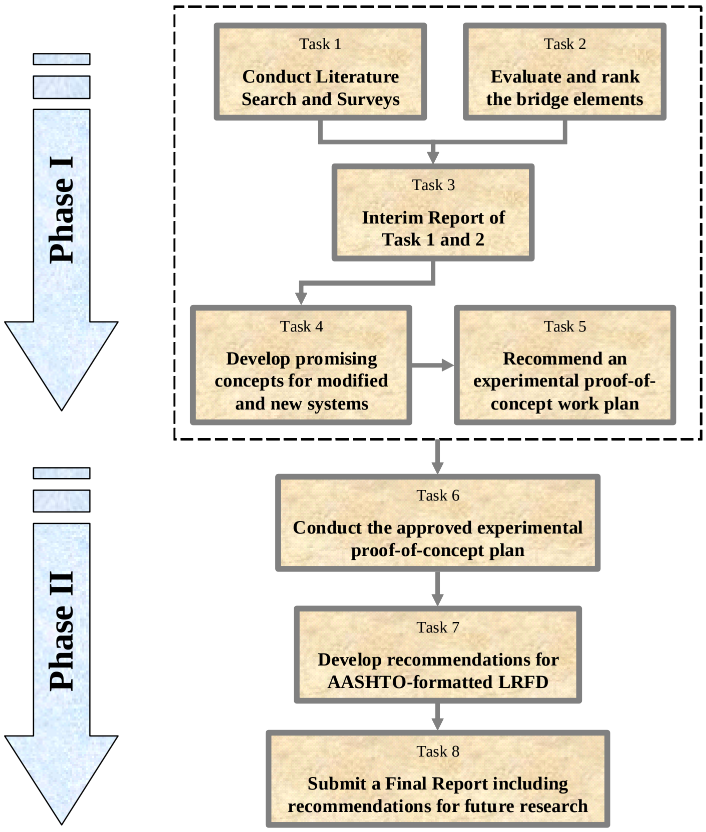


Figure 73. Phase I work Plan

As shown in Figure 73, the project will be initiated in Phase I of the study by conducting a literature search. Evaluating the data obtained in this search and classifying and ranking the results will be the Task 2 objectives. Results of Tasks 1 and 2 will be put in an interim report and will be submitted to TCC under Task 3. Task 4 is one of the most important activities within this project. Under Task 4, promising concepts and ideas for new and existing bridges will be developed and will be put into a recommendation form (Task 5) for TCC evaluation and approval.

Task 6 is where most of the proof-of-concept tests will be carried out on concepts approved by TCC. The findings will be put in an AASHTO-formatted language and will be recommended to AASHTO under Task 7. Finally, Task 8 will summarize the entire body of work and will publish it in a final report.

Task 1: Literature Search

An important aspect of the proposed project is to collect and comprehend information available worldwide. The design for service life is a topic that has gained a great level of interest internationally. This issue is not limited to bridge structures. Pavement engineers in the United States have developed an extensive amount of information leading to design of pavement mainly for service performance. There are a vast number of publications related to Mechanistic-Empirical Pavement Design; the research team is fully aware of these efforts. Dr. Azizinamini, for example, is assisting the Nebraska Department of Roads at the present time with adopting this new approach for rigid pavement design.

Since 1996, the European Union has undertaken several closely-related research projects that have resulted in comprehensive methodologies for the design of concrete structures for longer service life. The methodology developed is mainly used for concrete bridges and tunnels. The approach is quantifiable and can be used to design concrete bridges for service life of more than 100 years. The major components of these European initiatives consist of the following elements:

- DuraCrete, "Probabilistic Performance Based Durability Design of Concrete Structures" (1996-1999). This project, which included 12 European partners, provided the basis for developing ***fib*** Bulletin 34, referenced in the RFP for this project.
- DuraNet, "Network for Supporting the Development and Application of DuraCrete" (1998-2001). This initiative was aimed at promoting the use of ***fib*** Bulletin 34.
- DARTS (2001-2004), "Durable and Reliable Tunnel Structures" (2001-2004). This effort, which is aimed at tunnels, is similar in nature to that developed for bridges in ***fib*** Bulletin 34. Two related programs developed in Europe for tunnels include FIT (Fire in Tunnels) (2001-2005) and UPTIN (Cost Effective, Sustainable and Innovative Upgrading Methods for Fire Safety in Existing Tunnels" (2002-2006).

Results obtained from DuraCrete or ***fib*** Bulletin 34 could be used by this project. However, there are lessons to be learned from other European efforts for tunnels (DARTS, FIT and UPTIN). Dr. Carola Edvardsen and Mr. Ove Sorensen, of COWI Denmark, are members of this proposing project. Dr. Carola Edvardsen is one of the original members of the research team developing ***fib*** Bulletin 34 and has worked closely with Dr. Steen Rostam, who unfortunately is extremely ill. The material presented in ***fib*** Bulletin 34 represents a small portion of research projects carried out between 1996 and 1999. For instance, a series of unpublished research was carried out attempting to develop deterioration models for freeze and thaw. However, the models were judged to be unreliable. For these modes of deterioration (Freeze/Thaw, Sulfate attack and ASR), the design recommendations listed in ***fib*** Bulletin 34 recommend use of "Avoidance of Deterioration" or approach number 4 listed in the RFP. Through Dr. Edvardsen, the research team will have access to important publications and thought that were incorporated in developing the European approach for design for service life.

The European experience is a good starting point. However, some of the recommendations may not be directly applicable to the objectives of this project. For example, one of the main assumptions that the entire ***fib*** Bulletin 34 provisions are based on is that the concrete is "un-cracked". In ***fib*** Bulletin 34 terminology, concrete elements with small (not through crack) crack width (smaller than 0.03 mm in width) are assumed to behave as an un-cracked concrete element. Further, these small cracks should not be through the element thickness. In the case of a concrete bridge deck, in most cases, we do have through cracks. In these cases, a good portion of ***fib*** Bulletin 34 recommendations are no longer applicable and new approaches will have to be developed to estimate the service life of new and existing elements. This necessitates further work in this area. The research team will be reviewing available information and will critically evaluate it to identify what is applicable and not applicable to U.S. practice.

The research team will carefully examine and review existing U.S. bridge databases. The existing U.S. database includes the New York state database on bridge failures and NBIS. In addition, several states have their own version of bridge databases that could be utilized by the project. These databases will be identified through a survey. In addition, a detailed survey will be conducted to seek state Departments of Transportation input into the process. It is very important to develop and construct recommendations that suit AASHTO needs.

It is also important to have personal contacts in conducting surveys of state Departments of Transportation. Two of the consultants on the team, PBS&J and HDR, have offices in almost every state across the U.S. and have worked closely with various bridge owners. Figure 74 shows the offices that HDR and PBS&J have in various locations. PBS&J and HDR will be responsible for reviewing the results of state DOT surveys and will be complementing them by making personal contacts with state DOTs if needed.

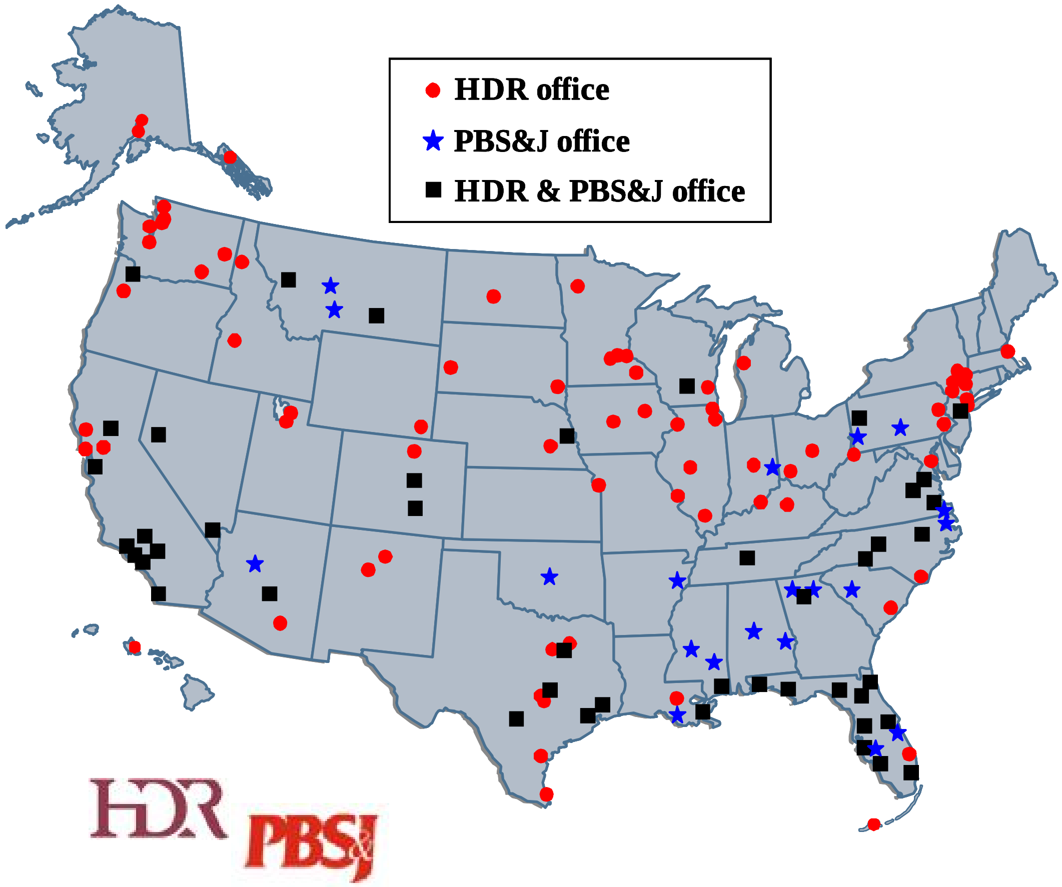


Figure 74. U.S. map with locations of branch offices for HDR and PBS&J

Research team members are very active in various disciplines related to this project and are fully aware of current knowledge in their field of expertise. Research team members are also members of key organizations and committees working closely with bridge owners, designers, fabricators and contractors. This will allow an additional means of collecting information on past, current and future projects.

More importantly, the research team has expertise that will allow the SHRP 2 program to make necessary adjustments during the course of the project as more is learned through literature research.

Results of the literature research should result in a) identifying available methodologies for design for service life, b) identifying bridge systems, subsystems and components that have provided long service life or have potential to provide long service life if modified and c) identifying potential solutions for currently observed service life related problems.

To capture the latest published work, literature searches will be conducted continuously throughout the project. With respect to innovations and new ideas, we will keep the scope of the project very flexible. Changes will be incorporated only after obtaining the SHRP II approvals. We will communicate necessary changes that could be incorporated into R19A project with TRB staff and if agreed, incorporate the necessary changes. Re-visiting the scope of work to be conducted will be true for every task, especially while conducting tasks 6 through 7.

Task 2: Ranking of the Identified Bridge System, Subsystems and Components

One of the activities of Task 1 will be identifying the bridge systems, subsystems and components that can minimize or completely eliminate the causes of observed problems related to service performance of bridges. Under this task, the concepts identified will be placed in different categories. The criteria for ranking the systems, subsystems and components will include such factors as design, required maintenance, construction practices, observed performances and traffic levels. For instance, reinforcing bars used in concrete elements could be classified into several categories. Stainless steel can eliminate corrosion of reinforcing bars in the bridge deck altogether regardless of traffic and construction practices. Figure 75 shows two identical bridges in Mexico, one about 35 years older than the other. The older bridge utilized stainless steel and is still in service, while the younger bridge, utilizing black reinforcement, has been completely destroyed due to the corrosion of rebars (the slide was provided by research team member, Dr. Edvardsen of COWI).

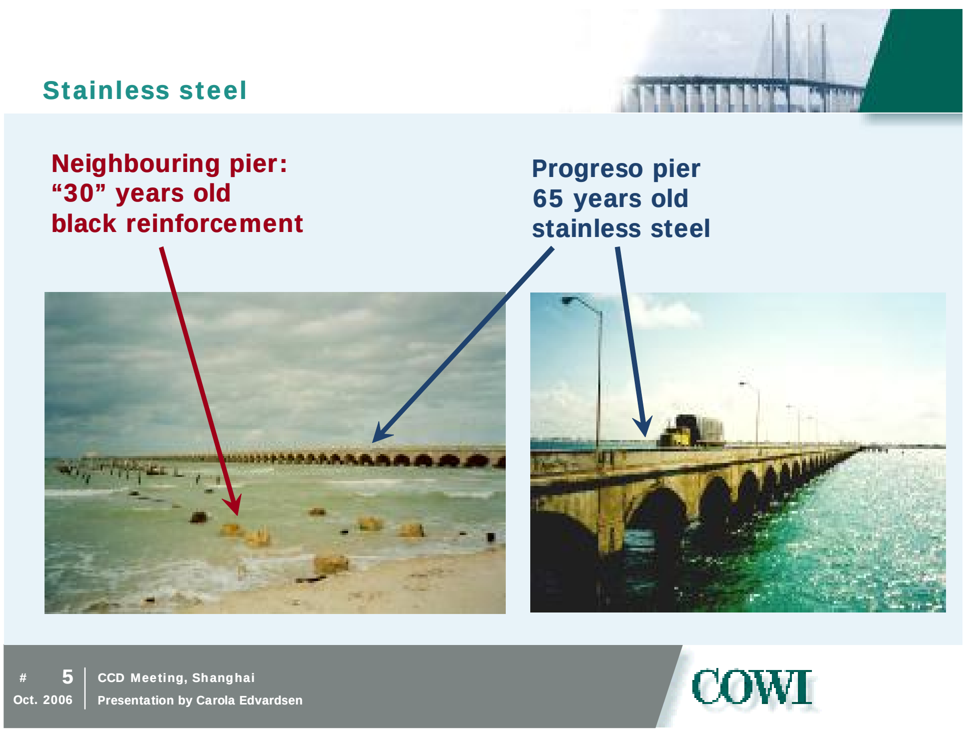


Figure 75. Two identical bridges in Mexico

On the other hand, epoxy-coated bars are sensitive to construction practices, to the presence of moisture at the reinforcement level, etc. The use of conventional reinforcing bars in a bridge deck, to a large extent, will depend on the quality of the concrete, cover thickness and level of traffic. For the example given (stainless bar, Epoxy-Coated bar and Conventional bar), the observed performance of different reinforcement types will be ranked based on the data available. This ranking may also be altered at the end of the project.

For some systems, subsystems or components identified, there may be a need to develop additional data to better quantify their performances for developing deterioration models. For example, the different expansion joint systems that are available provide a case worthy of consideration. Beyond certain bridge length, most designers feel that there is a need for expansion joints. Ranking the available expansion joints based on their performances and developing models capable of predicting their service life could be of interest. This is where the Federal Highway Administration Long Term Bridge Performance Program (LTBPP) will be helpful. The LTBPP is a 20-year program aimed at developing quantifiable data that could be used in a variety of ways to improve design, construction, maintenance and inspection of bridges and that could result in higher performing bridges at optimum total cost. Recommendations will be made to LTBPP to instrument and monitor certain systems, subsystems or bridge components that have shown good field performances but that lack quantifiable data for assessing their field performances. The data to be collected through LTBPP will be helpful in better assessing the performance of some promising systems to be identified in this project. Further, such activities should ultimately lead to development of better ways of assessing and managing the nation's bridges (the next generation of NBIS). The University of Nebraska-Lincoln is a major sub-contractor (led by Dr. Azizinamini) for a proposal that was submitted on September 11, 2007 to FHWA. The prime organization submitting the proposal is the University of Missouri-Rolla.

Task 3: Interim Report

The results of Tasks 1 and 2 will be used as a basis to prepare an interim report for SHRP 2.

Task 4: Development of Concepts for 100+ Years of Service Life

A) Introduction

In Task 4, the UNL Team will continue its research, drawing on what was learned and documented in Tasks 1 through 3, to develop concepts for new systems and components that will provide a reliable long service life, ideally 100+ years. We believe that this task will be the most important project phase in that concepts identified and developed here will become the basis for achieving the overall project objectives. Our team consists of the appropriate combination of experts to address all the material performance issues, and can also address all bridge systems, including steel and concrete, as well as specific components such as decks, joints and bearings. Levels of required maintenance will also be considered. The concept development will focus on systems and components that are rapidly and economically maintained, and will also consider whether certain components have to be replaced at various intervals during the extended life of the overall bridge system. As part of the concept development, the team will also evaluate related construction practices, where applicable, that could lead to longer and more predictable service life. The team will further identify promising new concepts that will need more extensive investigation and development by others. Project will also address issues such as bridge widening, replacement and accelerated construction. These issues will be part of all tasks listed in this proposal.

As part of the project the research team will develop an evaluation matrix to provide a basis for decision making processes when selecting various systems and subsystems for life cycle cost analysis. It will include the different bridge parameters, such as span length, location, function, traffic volume, and importance level.

The purpose of these efforts will be to facilitate the development of tools for assessing the costs and benefits associated with various systems and subsystems. The aspect of the work will continue into tasks 6 and 7.

In Sections 4.2.2 through 4.2.4 of this proposal, we identified a list of bridge systems and components and how they are affected by various external factors resulting in shortened service life. As part of that discussion, we also identified issues within each major bridge system and component that could extend or enhance service life. In describing our approach to Task 4, we will follow up on our discussion in Sections 4.2.2 through 4.2.4 to identify potential material improvements or structural concepts that our team would consider as part of this project. The concepts discussed are not intended to be a final or complete list of ideas to consider, but rather are intended to illustrate our team's understanding of the broad range of potential topics involved, and to represent the current thinking of our team in approaching this task.

B) Bridge Systems and Components

Many of the bridge systems, component design methods and materials used today are based on achieving certain levels of acceptable service while minimizing initial cost. These methods do not always result in certain designs that achieve extended service life, particularly with many bridge components, such as decks, steel coatings, joints and bearings. Our approach to developing new overall systems and components will be to:

- Minimize or eliminate, where possible, all problematic bridge components or conditions within bridge systems that cause deterioration.
- Evaluate design of various problematic components that can result in extended system service life. Enhance the performance, protection and accessibility of these vulnerable areas to allow easier inspection, maintenance and potential replacement.
- Consider upgraded materials that will not deteriorate for problematic components that cannot be eliminated. This will be more expensive but will permit longer system service life.

B.1 - New Steel Concepts and Processes

The following is a partial list of factors that could be considered to enhance service life of steel bridges:

- Simple for DL, Continuous for LL – Steel stringers erected as simple-spans but made continuous to carry live loads similar to the well-established concept used for precast, prestressed concrete girders are being introduced by several states with various continuity details. The concept should be studied and various details evaluated. (Appendix B, located in Section 12, provides a more detailed description of this system, which was researched at the University of Nebraska-Lincoln).

- Innovative HPS girder concepts for increased fracture resistance – The introduction of high-performance steel (HPS) with higher levels of fracture toughness (high-performance steels are tough enough to arrest a fracture from another less tough monolithically connected component.) provides an opportunity to rethink steel bridge design utilizing enhanced toughness. (States are constructing bridges with fracture-critical members (FCM's) of HPS to mitigate FCM considerations.)
- Elimination of fatigue-sensitive details in design – Rather than proportioning a steel stringer and adding welded details to connect them transversely into a system, a holistic design approach should be taken to proportion and detail steel-bridge systems to minimize, if not eliminate, fatigue-sensitive details.
- Use of ultrasonic impact treatments (UIT) to improve fatigue resistance – When fatigue-sensitive details cannot be eliminated through better detailing or repositioning of details, the fatigue resistance of the remaining critical details may be enhanced through the introduction of beneficial compressive residual stresses from UIT.
- Integral pier caps to eliminate deck joints and bearings – Where possible, pier caps may be made integral with the superstructure to eliminate bearings and/or joints. Distortion of flexible tall piers can serve to replace expansion bearings.
- Integral abutments to eliminate deck joints and bearings – Within certain geometric limitations (for example, expansion length, angle of skew, etc.), integral or semi-integral abutments can eliminate problematic joints and bearings at an abutment assuming the foundation accommodates thermal movements.
- Further use of weathering steel and structural stainless steel to resist corrosion.

B.2 - Steel Coatings Concepts

Coating steel bridges will most likely be a practice that is employed within the bridge industry for years to come. Therefore, investigating more economical approaches for coating is an important issue to be considered. The following is a partial list of issues related to coating that could be considered.

- Criteria for painting, metalizing or use of corrosion resistant steel to achieve specific service life goals, using regional and a level of importance matrix approach.
- Guidelines for coating maintenance that consider specific local area requirements as well as the overall bridge.
- Coatings research needs:
    - Removing chloride from steel surfaces. The biggest single reason that coatings fail on highway bridges is the inability to remove chlorides from the steel surface prior to re-coating.
    - Developing a new, inexpensive and easy to apply, long-lived coating. Lead and chromium were the corrosion-inhibitive pigments of choice because they were both effective and inexpensive.

- Implementing a two-coat system. Significant savings could occur if a fast curing, two-coat paint system could be applied over blast cleaned steel which would deliver the same protection to the steel surface as obtained using a three-coat system.
- Developing a one-coat system. The cost of protecting steel could be halved by developing and using a one-coat system. As of yet, however, no suitable one-coat material has been tested or approved. There is a pending research study awaiting funding (TPF Study 924 Single Coat).
- Determining the need to remove mill scale and to what extent such removal should be undertaken.

B.3 - New Concrete Concepts

The following are a list of issues and ideas that could result in enhancing the service life of concrete bridges.

- Adding post-tensioning to keep the super-structure under compression (spliced girder systems).
- Adding PT in adjacent boxes and changing the details to assure full transverse direction continuity.
- Adding threaded rod continuity (Nebraska/Alberta/Illinois/Florida) to create full continuity for deck weight and to eliminate potential creep restraint cracking at the piers.
- Creating an I-girder with integral precast deck (the so called deck bulb tee in the Northwest) and discussing camber and continuity issues.
- Developing new high performance concrete with corrosion resistant reinforcement (epoxy, galvanized, stainless steel, carbon fiber, glass fiber, aramid fiber or electrochemically treated steel reinforcement).
- Using Reactive Powder Concrete (a LaFarge product called Ductal) for entire concrete bridge systems or incorporating it in some elements of current bridge systems.

B.4 - Bridge Deck Concepts

Envisioning bridge deck systems that could provide longer service life at optimum total cost is an important issue. Sections 4.2.4.3 and 4.2.3.4 provided a detail discussion of the factors influencing the service life of bridge decks. The major concentration within this project will be concrete decks. Bridge deck systems such as Fiber Reinforced Polymer (FRP) decks will also be evaluated.

Some of the issues that are crucial to bridge deck systems that the project may consider for developing promising concepts include a) material durability, b) construction, c) maintenance and d) innovative deck systems.

B.5 - Reinforcing Concepts

Numerous reinforcing concepts are capable of achieving 100+ years of service life if researched or implemented appropriately. These reinforcing concepts can be considered independently or can be used in conjunction with other systems which provide increased protection, such as increased concrete cover, improved concrete quality and proven coatings, sealers and membranes. Reinforcing concepts with merit and deserving of further consideration include:

- Epoxy coated reinforcing with sufficient concrete quality and cover,
- Stainless and stainless clad,
- Pseudo stainless and other lower corrosion alloys and coated bars,
- FRP rebar,
- Black bar with electrochemical treatment to increase corrosion threshold and
- Steel free decks.

One of the practices to be considered is to use a combination of different reinforcing bars within a given bridge deck or other bridge elements. For instance, it is beneficial to use stainless steel bars only in those areas where historically bridges have exhibited deterioration problems, such as in splash areas, and to use other reinforcing bars in the rest of the element. Traditionally, designers have had a tendency of specifying the same reinforcement types throughout the concrete element.

Selection of the appropriate system will depend on the specific service and on environmental exposure conditions, installed cost and construction requirements.

B.6 - Jointless Bridges

For most bridges, joints could be eliminated. Some existing bridges incorporating joints could be made jointless. Section 4.2.4.6 provided a detailed discussion of joints and some of the issues involved. The following is a brief list of factors that should be considered in developing promising concepts related to jointless bridges.

Consistent integral abutment details that reflect current research for performance.

A wide range of integral abutment details is used by different owner agencies at the present time. This project will compare and contrast various promising details so that a range of best practices can be identified. Advanced analysis will be coupled with the results from prior and new experimental testing and field observations to investigate the potential of promising integral abutment details that can potentially accommodate larger movements, thus increasing jointless bridge length limits.

Up-to-date consistent criteria for length, skew and horizontal curvature.

Recommended maximum lengths and skew angles for integral abutment bridges vary significantly from state-to-state at the present time. The project will aim to provide a clear and fundamental understanding of the length, skew and horizontal curvature limits associated with different abutment details and bridge geometries based on the results from the above experimental testing, field observations and advanced analysis studies. Simplified methods of analysis will be recommended for evaluation of bridges where jointless concepts may be extended to systems with skew and/or horizontal curvature.

Shifted joint details at abutments for long bridges that need some type of joint.

As shown previously by the example of Figure 52, innovation is possible so that joints can be moved away from the superstructure in cases where joints are necessary. Details of this type will be studied and recommended for broader use where merited.

Quantification of the service life performance of various simple-span-made-continuous and link-slab details at interior piers of multiple-span bridges.

The service life performance of various jointless bridge components and details needs to be assessed. These estimates can be accomplished by a combination of field observations, experimental testing and analytical modeling. For instance, various link slab systems can be subjected to physical (load-deformation) and accelerated environmental testing to determine their deterioration response. Details using both traditional as well as new high-performance materials will be considered.

B.7 - Bridges with Joints

Detailed discussion of factors related to bridges with joints and bearings are provided in section 4.2.4.6(F). Beyond certain total span lengths, designers like to incorporate joints. Where possible, more durable armored joints should be used. Criteria for the maintenance and replacement of seals and whole joints should be specified in advance. The replacement of joints should not be expensive or time consuming.

More durable armored joint concepts.

More research is needed to quantify the expected service life of existing joint types that are used in the industry. Depending on the level of importance for a particular bridge, joints can be planned to be replaced at certain intervals throughout the overall bridge life. Bridges with high importance levels that also require joints will need to have joint options with improved long term durability. These types of joints may be more expensive initially, but will be expected to perform for much longer durations.

Criteria for maintenance and replacement of seals and whole joints.

More information needs to be developed as part of an overall maintenance guide for various joint types, which will identify maintenance types and frequency, service life expectancy of individual joint components, such as neoprene seals, and the entire joint system.

B.8 - Bearings

Current Specification versus field Observation

Figure 76 is taken from the AASHTO *LRFD Specifications*. This table "was based on general judgment and observation," according to AASHTO *LRFD Specifications* Commentary. It is interesting to note that the cotton duck reinforced pad is shown as "Unsuitable" ("U") for any movement and is only "Suitable" ("S") for vertical load resistance. At the same time, some states has used cotton duck reinforced pads with success. (Typical NDOR "fixed bearing" and "expansion bearing" details using cotton duck pads are shown in Figure 77.)

Evaluating the performance of various bearing types used and recommending use of a simple detail will be a major objective. Promising bearing detail to be recommended for investigation should be durable and easy to replace if needed. Further, inspection of the bearings should not be difficult to accomplish.

More durable bearing concepts.

Similar to joint systems, there is a need to quantify the expected service life of existing bearing types that are used in the industry and to develop a line of bearing types that can be expected to perform for extended durations.

Inspection, maintenance and replacement criteria.

Similar to joint systems, an overall maintenance guide for bearing types needs to be developed that includes criteria for inspection access and frequency, types of required maintenance and criteria for when various bearing types may need to be replaced based on service life expectations.

Table 14.6.2-1 Bearing Suitability.

Type of Bearing	Movement		Rotation about Bridge Axis Indicated			Resistance to Loads		
	Long.	Trans.	Long	Trans.	Vert.	Long.	Trans.	Vert.
Plain Elastomeric Pad	S	S	S	S	L	L	L	L
Fiberglass-Reinforced Pad	S	S	S	S	L	L	L	L
Cotton-Duck-Reinforced Pad	U	U	U	U	U	L	L	S
Steel-Reinforced Elastomeric Bearing	S	S	S	S	L	L	L	S
Plane Sliding Bearing	S	S	U	U	S	R	R	S
Curved Sliding Spherical Bearing	R	R	S	S	S	R	R	S
Curved Sliding Cylindrical Bearing	R	R	U	S	U	R	R	S
Disc Bearing	R	R	S	S	L	S	S	S
Double Cylindrical Bearing	R	R	S	S	U	R	R	S
Pot Bearing	R	R	S	S	L	S	S	S
Rocker Bearing	S	U	U	S	U	R	R	S
Knuckle Pinned Bearing	U	U	U	S	U	S	R	S
Single Roller Bearing	S	U	U	S	U	U	R	S
Multiple Roller Bearing	S	U	U	U	U	U	U	S

S = Suitable

U Unsuitable

L = Suitable for limited applications

R – May be suitable, but requires special considerations or additional elements such as sliders or guideways

Figure 76. AASHTO *LRFD Specifications* Bearing Types and Suitability Table (appears to not favor cotton duck reinforced pads.

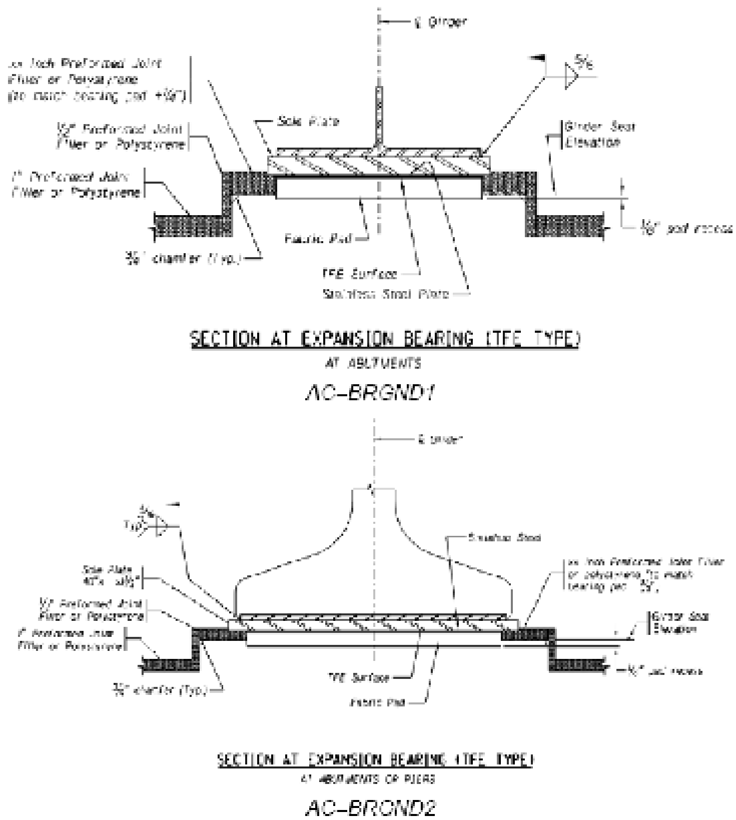


Figure 77. Nebraska Cotton Duck Reinforced Bearing detail for Expansion Bearings

<u>Task 5:</u> Phase I Report

Based on the results of Tasks 1, 2, 3 and 4, bridge systems, subsystems, components and details will be identified to address the needs of existing and new bridges. The solutions that are judged to be most promising will be identified and will be recommended for further investigation under Task 6. It is expected that part of the recommendation will be related to developing deterioration models that could ultimately lead to development of predictive models capable of estimating the service life of various components of the bridge with a given reliability index. Experiences by others, especially those of Europeans, will be considered in order to reduce the level of efforts involved in establishing deterioration models.

It is expected that some promising concepts identified and capable of enhancing service life of existing and new bridges will need further research by others. For example, development of advanced one-layer coating systems for steel bridges would fall in that category. Toward that end, a list of problems that should be considered by others, including NCHRP, will be identified.

A summary of Tasks 1 through 5 will be put in an interim report format and will be submitted to SHRP 2; the research team will await SHRP 2 approval before proceeding with Phase II of the investigation.

4.2.8 Phase II Work Plan

Task 6: Conduct an experimental proof-of-concept work plan

Under Task 4, promising concepts to enhance the service life of bridges will be identified. Under Task 5, TCC will finalize the concepts to be further researched. The purpose of this task will be to continue the project based on the decision made by TCC, to conduct proof-of-concept tests and to conduct tests to develop deterioration models, if needed. The type of investigation to be conducted will depend on the type of problems to be addressed. It is important to note that this section of the proposal includes examples which may or may not be investigated. The examples given (use of Membrane or Chloride induced corrosion) are mainly presented to explain the team's general approach and thoughts on the type of work that could be conducted under Task 6 and are not necessarily the reflection of the exact concept (Membrane or Chloride) to be investigated.

The type of investigation to be carried out to conduct proof-of-concept study for promising concepts will most likely include a combination of experimental, numerical and analytical work. The following are potential main categories of work that could be conducted within a framework of proof-of-concept investigation.

- An investigation to develop deterioration processes and models allowing the assessment of the service life of promising concepts subjected to environmental type loading or chemical loadings. The end results could be the development of procedures and methodologies to establish the expected service life of the bridge or to estimate the remaining service life of existing bridges in a quantifiable way. The investigation would make it possible to say, for instance, that an existing bridge has 30, 40 or 50 years of remaining life or to estimate the service life of new bridges using the developed concept.
- An investigation to comprehend the structural response of promising concepts to environmental type loading as well as traffic type loads. The end result will be development of design, construction and maintenance recommendations.

A critical aspect of the work to be conducted will be the establishment of performance criteria and acceptance or rejection criteria for promising concepts.

The following sections expand on the type of investigations listed above.

A) Investigation to develop deterioration processes and models allowing the assessment of the service life of promising concepts subjected to environmental or chemical type loadings.

One of the main causes of observed damages to bridge components are environmental type loads such as temperature fluctuation, freeze/thaw or application of chemicals such as salt. There will be a need to develop tools that could be used to assess in a quantifiable way the service life of promising concepts. This will demand development of deterioration models for some of the main factors causing damage. Examples may include a concrete bridge deck using membrane and asphalt overlay or a concrete deck with steel fiber concrete overlay.

Figure 78 shows a concrete surface covered by an asphalt membrane. Note the trapped air beneath the membrane. Such a situation could result in deterioration of the concrete behind the membrane as shown in the middle photo in Figure 78.

Figure 78. A concrete surface covered by an asphalt membrane

The membrane system has received mixed evaluations to date. Some states in the U.S., such as Colorado, have used the membrane system and have had good experience while others have different opinions. According to the Colorado State Bridge Engineer, Colorado has used it with success.

Questions that have to be resolved before a quantifiable approach is developed to predict the service life of these systems include:

- What is the damage mechanism?
- How does the system respond to such loadings as cyclic salting of the deck, cyclic freeze/thaw or chloride penetration?
- Is the damage to the concrete deck almost zero until the membrane is damaged, or does damage start even before the membrane is impaired?
- Once the membrane is damaged, what is the remaining service life of the deck?
- What are the permeability models that could be used for asphalt or fiber overlay and what is the level of damage when moisture is trapped under the membrane or overlay?

These questions will require that we conduct cyclic durability tests such as Freeze/Thaw tests on scaled models or on the systems. The first task will be to develop a model capable of explaining the damage mechanism. Europeans are using the following equation, for example, to explain damage caused by diffusion of any chemical (CEB 2006).

$$C_{(x,t)} = C_o\left(1 - erf\frac{x}{2\sqrt{D_c t}}\right) \quad \textbf{(3)}$$

Where,

$C_{(x,t)}$ =Chemical concentration at depth x and time t

C_0 =surface chemical concentration (kg/m^3)

D_c =Chemical diffusion constant (cm^2/yr)

erf =error function

The equation shown is developed based on experimental work and through the use of Fick's second law. This equation can be used to predict the level of various chemicals at any point within the concrete element. For instance, equation (3) could be used to predict the chloride content at the level of reinforcement within a concrete column, at any given time t, in a marine environment. Corrosion is believed to start when chloride content reaches a critical value. Comparison of the chloride content at the point of interest obtained from equation (3) and a comparison to the critical value will establish the time that it will take to start the corrosion process. If we define the failure to be the time that it will take to start the corrosion process, we can then estimate the service life of the concrete column under consideration.

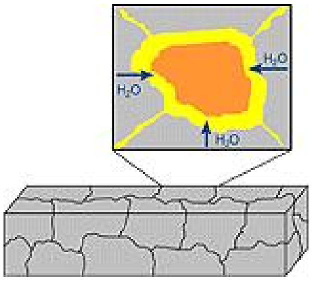
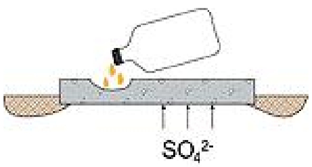
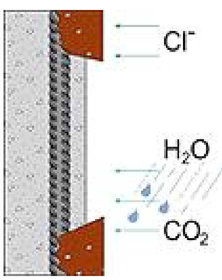
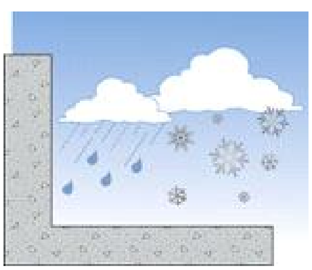
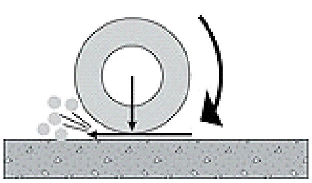
Equation (3) could also be used to estimate the remaining life of existing concrete elements. As an example, let us assume that it is desirable to estimate the remaining life of an existing concrete deck consisting of steel fiber overlay. Let's further assume that the equation shown above (equation (3)) is what will be used to estimate the chloride diffusion within different layers of concrete. Let us also assume that we know what level of chloride content at the reinforcement level can start the deck corrosion and that we are defining failure as the start of corrosion. Estimating the remaining life of the concrete element will then be a relatively easy task and will involve the following steps.

- Taking a core sample from the existing structure and estimating chloride content based on available ASTM tests.
- Using equation (3) to estimate the time that it will take for the chloride content at the reinforcement level to reach the critical level. This will be the remaining service life of the system if the failure is defined as initiation of corrosion. (The commencement of failure is the definition used by some Europeans in some countries; this definition needs to be re-visited).

Equation (3) could also be used to determine the thickness of the concrete cover that is needed to result in a specified service design life, t.

We may also use available ASTM tests to evaluate durability characteristics of selected concrete mixtures. Table 1 taken from the PCA web site, shows available ASTM tests related to durability of concrete. It is also possible that revised ASTM methods may be used in some of the tests.

Table 1: Available ASTM tests related to durability of concrete

Durability Issue	Mechanism	Test Methods
Alkali-Aggregate Reaction 	-Alkali-Silica Reaction -Alkali-Carbonate Reaction	ASTM C 227 ASTM C 289 ASTM C 441 ASTM C 586 ASTM C 856 ASTM C 1260 ASTM C 1105 ASTM C 1293 ASTM C 1567 Los Alamos Method
Chemical Resistance 	-Sulfates -DEF -Seawater -Acids	ASTM C 1012 ASTM D 516 ASTM C 1582
Corrosion of Reinforcement 	-Chloride Resistance -Chloride Content -Carbonation -Corrosion -Corrosion Potential -Resistivity Testing -Depth of Concrete Cover -Delamitation Survey	ASTM C 1202 ASTM C 876 AASHTO T 259 ASTM C 1556 AASHTO T 260 ASTM C 1152 ASTM C 1218 ASTM C 1524 AASHTO TP 11 AASHTO TP 22 AASHTO TP 26 AASHTO TP 55 ASTM D 4580
Freeze-Thaw 	-Freezing and Thawing -Deicer Scaling -D-Cracking	ASTM C 666 AASHTO T 161 AASHTO TP 18 ASTM C 457 ASTM C 672
Miscellaneous 	-Abrasion -Erosion -Fire Resistance -Efflorescence	ASTM C 131 ASTM C 535 ASTM C 3744 ASTM C 1137 AASHTO TP 58

The outcome of the efforts described above should result in:

- The development of a damage mechanism,
- The definition of "failure",
- The ability to predict the service life of existing and new bridges using promising concepts.

B) Methodology to assess structural responses and development of design models for promising systems

In addition to comprehending the behavior of promising systems to environmental type loading, structural response of same system to traffic type load may also have to be investigated. For example, assume that one of the promising concepts will be the use of a concrete bridge deck in conjunction with a steel fiber reinforced concrete overlay. The bonding between the concrete overly and the concrete deck needs to be investigated under traffic type loads. This will be in addition to conducting durability tests as described above.

The test specimen could consist of a full-scale model or scaled portion of the system. The load and/or displacement boundary conditions applied in the tests will simulate the condition where the element is part of the entire bridge system. This objective would be achieved by comprehending the forces and/or displacements that the element will be subjected to as part of the entire structural system. During experimental testing, the structural element will be subjected to these pre-defined load or displacement types at appropriately defined boundaries. To eliminate the scale factor, the components to be tested will be full-scale. The expertise of Dr. Azizinamini and Dr. Tadros in experimental testing and the convenience of having two fully-equipped structural laboratories -- one in Lincoln, Nebraska and one in Omaha -- will allow the simultaneous testing of the performances of more than one component and will achieve project goals in a shorter time period.

From past experience, the best approach is deemed to be a combined numerical, analytical and experimental method when investigating structural components. Numerical work is usually performed prior to conducting experimental work. This allows comprehension of the effect of different variables and identification of critical variables to be investigated through experimental testing. A reduction in the number of experimental tests that need to be carried out will, thus, be accomplished.

The general philosophy of investigation of the response of structural elements to applied loads as described above is shown schematically in Figure 79. Information shown in the figure is a summary of work conducted. Figure 79, has been used by research team members to develop innovative bridge systems. Each step taken in the process is depicted in the Figure 79 flowchart and is explained below.

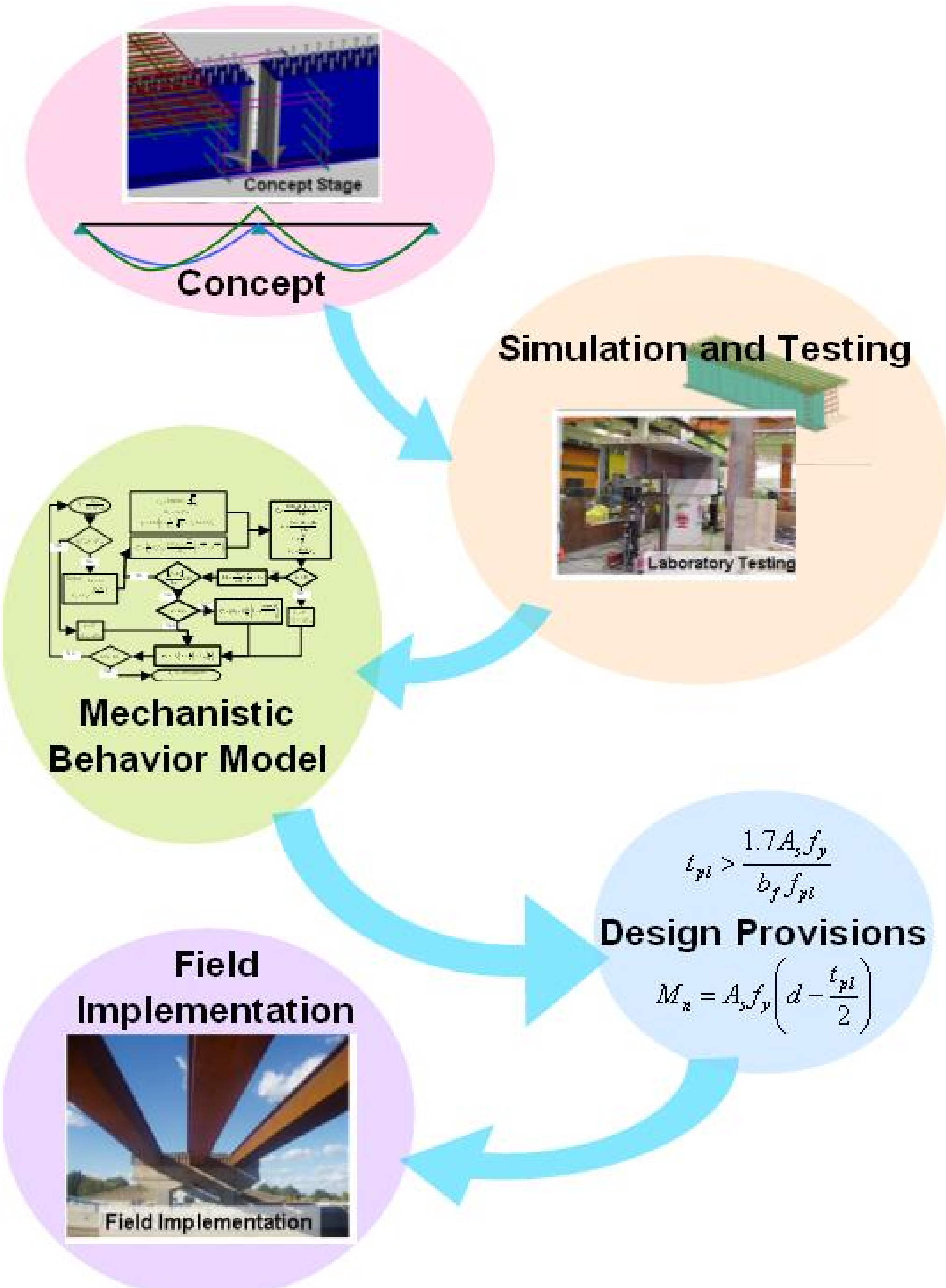


Figure 79. Flowchart displaying the envisioned development process

The first step is concept development, in which a connection detail to join the girders over the support is envisioned. Next, a critical region of the system is identified as needing investigation – in this case, it is the region over the pier where steel girders are joined together by a concrete diaphragm. A prototype bridge is then designed and forces that the girders are subjected to at inflection points are estimated. A test setup is then created representing a portion of the bridge over the pier.

In past uses of this combined numerical, analytical and experimental method of investigating structural components, several test specimens were subjected to various loading types to comprehend short- and long-term performance of the system. For example, 75 year traffic loads were simulated by subjecting the specimen to cyclic loads. Because experimental testing is expensive, the first few tests were used to develop a detailed computer model. The computer model then allowed researchers to investigate the effect of various parameters on system behavior in an economical manner.

Using the information obtained from experimental testing and numerical models under this system, a detailed mechanistic model was then developed to describe the behavior of the system in a mechanistic manner. While this model was not totally suitable for use in a design office atmosphere, the ultimate objective of developing a simple design provision suitable for use by hand calculator was achieved by eliminating parameters that were judged to be insignificant from the detailed mechanistic model. The end result, as depicted in Figure 79, is a simple design provision.

Research team members should be able to use similar philosophies to investigate structural responses of promising systems to traffic type loading.

Task 7: Developing Recommendations for AASHTO-Formatted LRFD Design and Construction Specification

Recommendations will be developed for the AASHTO *LRFD Bridge Design Specifications* and the *LRFD Bridge Construction Specifications* in the form of draft agenda items to be considered by the AASHTO Subcommittee on Bridges and Structures (SCOBS) through the appropriate technical committees. Based upon the scope of this project, the recommended provisions for enhanced service-life expectancy will be more qualitative with various alternatives for designs or details which provide increasing, yet unquantified, levels of service life. The findings of Project 19B should ultimately add quantitative provisions to complement or replace these.

The traditional service limit states and newer limit states relating to 100-year durability will be considered in this project. The traditional service limit states such as the Service III limit state of the *LRFD Specifications* (given in LRFD Table 3.4.1-1) typically consider stress, wherein an applied stress (load) is compared to a stress limit (resistance) in the basic LRFD equation (LRFD Equation 1.3.2.1-1). This currently uncalibrated limit state must be reviewed and if the limit-state function is considered appropriate ultimately calibrated to achieve the desired service life.

In addition to reviewing these limit-state functions, the research team will consider additional limit-state functions appropriate to achieve a quantifiable service life. An example of a new limit-state function may be chloride penetration into bridge decks from de-icing agents. In this case, the amount of de-icing agent applied to the deck may be considered the load and the depth to the reinforcing bars may be considered the resistance. In the final product (in conjunction with the project R19-B calibration), the depth to the rebar may be determined for different probabilities of achieving a 100-year service life as a function of the amount of de-icing agents applied. For example, the following table template could be filled in:

Table 2:Depth to Reinforcement to Achieve Various Reliabilities of 100 Years of Life

Depth to Reinforcement to Achieve Various Reliabilities of 100 Years of Life				
tonnage of de-icing agent applied annually	probability of 100 years of service			
	β = 1.0	β = 1.5	β = 2.0	β = 1.5
xxxx	a”	d”	g”	j”
yyyy	b”	e”	h”	k”
zzzz	c”	f”	Ii”	l”

This project will develop the limit-state functions that require calibration. Project 19B will consider appropriate reliability, will determine the uncertainties involved and will perform a calibration.

The research team is aware of the importance of the incorporation of the results into various AASHTO documents and the time it takes to get them into the documents.

The research team members include individuals who hold key positions within steel and concrete technical committees, and as a result, the research team has excellent communication channels with various AASHTO committees. Research team members will take full advantage of these communication levels and expertise. Research team members also have excellent expertise in the area of developing AASHTO formatted design provisions. The draft of the recommended languages to be presented to AASHTO committees will be developed by expert individuals who will be conducting the work and developing the new knowledge. However, the final recommendations to be shared with the profession, which includes AASHTO committees, will be discussed and agreed by the entire research team. To help the process, part of the management plan to be developed will include the establishment of a small panel responsible for ensuring the appropriateness of the provisions to be recommended to various AASHTO committees. This includes both the formatting and the language that should be used. The members of this panel will include Dr. Dennis Mertz, Dr. Tadros and Dr. Azizinamini at a minimum.

The recommendations that will be made by the R19A project will be in a form ready for implementation without waiting for other pending projects. It is, however, possible that some of the recommended work would need to be revised over time, for example to include the results of the calibration work carried out by R19B.

Task 8: Final Report

At the conclusion of the project, a final report summarizing all activities completed within the scope of the project will be prepared and submitted to SHRP 2.

4.2.9 Connecting R19-A to R19-B

When combined, R19-A and R19-B should result in the development of AASHTO formatted provisions for the design and construction of bridges capable of providing more than 100 years of service life. The provisions should address both existing and new bridges. The procedures will have to be quantifiable for both existing and new bridges.

The R19-B project is scheduled to start approximately six months after R19-A. R19-A will develop promising systems, subsystems, components, details and retrofit concepts capable of prolonging the service life of bridges at optimal total cost. One of the major activities within R19-A will be the development of deterioration models for some of the promising ideas and concepts. While developing higher performing concrete decks, for example, there will probably be a need to develop deterioration models capable of predicting the time that it will take for corrosion to be initiated due to chloride penetration, considering type and quality of concrete used. The focus of the R19-A project is concentrated on development of Level 3 (the Satisfy Design Approach) and Level 4 (the Avoidance of Deterioration Approach) design aids. Development of a Level 4 approach would be simple in some cases such as when requiring the use of stainless steel. Development of the Level 3 approach, however, would require providing a table in the AASHTO specifications specifying such details as cover thickness, permeability of concrete, etc.

For the example discussed above, determining the numbers that will go into these tables will require two main pieces of information: 1) the deterioration model and 2) performance acceptance criteria. These two pieces of information will be developed in the R19-A project. To carry the task to the final step and develop the final numbers such as concrete cover thicknesses, a calibration process will have to be carried out, which will be left to R19-B.

Appendix A, located in Section 12 of this proposal, provides an example of a calibration process that could be conducted to take results from R19-A to develop the final version of the tables to be included in the AASHTO specifications. The example shown in Appendix A was developed by Dr. Azizinamini in collaboration with Professor Fred Choobineh, an Industrial Engineering faculty member at the University of Nebraska-Lincoln (not a member of the R19-A team), Dr. Andy Nowak of UNL and Dr. Carola Edvardsen of COWI. Members of the research team have the background and expertise required to anticipate information that will be needed for the R19-B team to accomplish their work.

In summary there will need to be close coordination of activities between R19-A and R19-B. It will be crucial to have joint meetings, etc. between the members of both research teams in order to achieve the goals of R19-A and R19-B.

(It should be noted that the example given above (Chloride induced corrosion) represents a small part of the activities to be undertaken in this project. As outlined in previous sections of this proposal, the diverse and variable nature of U.S. practice will require additional issues to be addressed beyond those discussed in ***fib*** Bulletin 34.)

4.3 Anticipated Research Results

The main product of this research will be the development of AASHTO formatted design and construction provisions capable of providing 100 or more years of service life at optimal maintenance cost, both for existing and new bridges, in a quantifiable manner. Specific components of the work product will include:

- At the end of Task 2, an interim report will be submitted which could help those who will be carrying out the R19-B project.
- At the conclusion of Task 5, promising concepts to be researched by R19-A and those to be considered by other agencies and entities will be identified. For the promising concepts to be addressed by R19-A, a plan of action in the form of a Proof-of-Concept work plan will be developed and will be submitted for approval before proceeding with Phase II activities.
- A final report will summarize the entire project activities, including recommended language that could be incorporated into AASHTO LRFD Design and Construction Specifications to reflect project findings.
- An “Owners Manual” concept will be developed along with detailed description of its content. This document will contain the information needed by the bridge owner to make crucial decisions to ensure that the bridge provides long service life at optimal maintenance cost.
- Design and analysis tools for promising concepts will be developed. For instance, available information for jointless bridges will be collected and further developed to create specific AASHTO formatted design and analysis recommendations.
- A complete methodology for design for service life in a quantifiable form for new and existing bridges will be developed.
- Specific recommendations will be made that could be used by the FHWA Long Term Bridge Performance Program (LTBPP) to further validate the findings related to promising concepts identified in this project.
- Specific recommendations will be provided for modifying the designer’s role in addressing various aspects of bridge performance during the entire life of the bridge. For example, the concept of an “Owners Manual” will require designer involvement in more than just providing design plans.
- Specific recommendations will be made that could be used by various agencies for construction, procurement, contracting, maintenance and inspection related activities.
- Specific recommendations will be provided to FHWA to evaluate the inspection cycles for bridges using the technologies developed by R19-A.
- Training and educational materials for the technology transfer aspect of the project will be developed.

4.3.1 Project Deliverables

The project deliverables will include:

- An Interim Report outlining Tasks 1 and 2.
- Draft and Final Phase I Reports outlining the outcomes of Tasks 1 through 5.
- A Final Report summarizing the entire project activities.
- Interim meetings with SHRP 2 staff, two in Washington, D.C. and one at the University of Nebraska-Lincoln.
- One interim meeting with SHRP 2 staff and review panel in Washington, D.C. during Task 5.
- One interim meeting with the Renewal Technical Coordinating Committee (TCC) at a place designated by SHRP 2 staff.
- Other means of communication such as e-mails, telephone conferences and video conferences, as needed.

4.4 Applicability of Results to SHRP 2 Objectives

The SHRP 2 goals and objectives are best described by the following statements:

"…America's highway system includes more than 3.9 million miles of highways, arterials, and local roads and streets. These roads, which carry more than 90% of passenger trips and account for some 84% of freight value, are critical to meeting the mobility and economic needs of local communities, regions, and the nation. In addition to commercial and private vehicles, the roadways accommodate buses, bicycles, and pedestrians and provide vital links to all other modes of transportation. To address these challenges, Congress established the second Strategic Highway Research Program." (SHRP 2 web site)

SHRP 2 focuses on the following four areas.

Safety	Preventing or reducing the severity of highway crashes by understanding driver behavior
Renewal	Addressing the aging infrastructure through rapid design and construction methods that cause minimal disruption and produce long-lived facilities
Reliability	Reducing congestion through incident reduction, management, response and mitigation
Capacity	Integrating mobility, economic, environmental and community needs in the planning and designing of new transportation capacity

The proposed project is in the Renewal area. The overall goal of the SHRP 2 renewal program is to "…develop a consistent, systematic approach to performing highway renewal that is rapid, causes minimum disruption, and produces long-lived facilities…" (SHRP 2 web site)

The proposed project meets the objectives of SHRP 2 goals by developing the technologies and the knowledge needed to design, construct and maintain bridges to last more than 100 years at optimum maintenance cost. Longer lasting and higher performing bridges will eliminate major interruption and minimize the overall costs of having bridges in service.