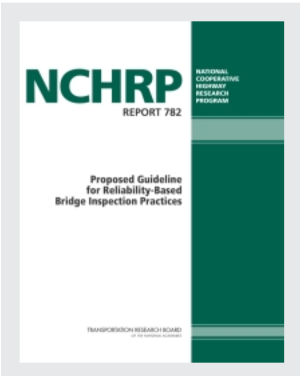

Proposed Guideline for Reliability-Based Bridge Inspection Practices

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AUTHORS

Washer, Glenn; Nasrollahi, Massoud; Applebury, Christopher; Connor, Robert; Ciolko, Adrian; Kogler, Robert; Fish, Philip; and David Forsyth

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

NCHRP REPORT 782

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Glenn Washer
Massoud Nasrollahi
Christopher Applebury
UNIVERSITY OF MISSOURI
Columbia, MO

Robert Connor
PURDUE UNIVERSITY
West Lafayette, IN

Adrian Ciolko
KPFF CONSULTING ENGINEERS
Evanston, IL

Robert Kogler
RAMPART, LLC
Arlington, VA

Philip Fish
FISH AND ASSOCIATES, INC.
Middleton, WI

David Forsyth
TRI/AUSTIN
Austin, TX

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway and Transportation Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Federal Highway Administration, United States Department of Transportation.

The Transportation Research Board of the National Academies was requested by the Association to administer the research program because of the Board's recognized objectivity and understanding of modern research practices. The Board is uniquely suited for this purpose as it maintains an extensive committee structure from which authorities on any highway transportation subject may be drawn; it possesses avenues of communications and cooperation with federal, state and local governmental agencies, universities, and industry; its relationship to the National Research Council is an insurance of objectivity; it maintains a full-time research correlation staff of specialists in highway transportation matters to bring the findings of research directly to those who are in a position to use them.

The program is developed on the basis of research needs identified by chief administrators of the highway and transportation departments and by committees of AASHTO. Each year, specific areas of research needs to be included in the program are proposed to the National Research Council and the Board by the American Association of State Highway and Transportation Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are the responsibilities of the National Research Council and the Transportation Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

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CRP STAFF FOR NCHRP REPORT 782

Christopher W. Jenks, *Director, Cooperative Research Programs*
Christopher Hedges, *Manager, National Cooperative Highway Research Program*
Waseem Dekelbab, *Senior Program Officer*
Danna Powell, *Senior Program Assistant*
Eileen P. Delaney, *Director of Publications*
Scott E. Hitchcock, *Editor*

NCHRP PROJECT 12-82(01) PANEL
Field of Design—Area of Bridges

Matthew Farrar, *Idaho Transportation Department, Boise, ID* (Chair)
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William R. "Randy" Cox, *American Segmental Bridge Institute, Driftwood, TX*
Arthur W. D'Andrea, *Louisiana DOTD, Baton Rouge, LA*
David A. Juntunen, *Michigan DOT, Lansing, MI*
Peter C. McCowan, *New York State DOT, Albany, NY*
Barton J. Newton, *California DOT, Sacramento, CA*
Andre V. Pavlov, *Florida DOT, Tallahassee, FL*
Lance D. Savant, *AECOM, Mechanicsburg, PA*
Thomas D. Everett, *FHWA Liaison*
James W. Bryant, Jr., *TRB Liaison*

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FOREWORD

By Waseem Dekelbab
Staff Officer
Transportation Research Board

This report presents a proposed Guideline for reliability-based bridge inspection practices and provides two case studies of the application of the proposed Guideline. The Guideline describes a methodology to develop a risk-based approach for determining the bridge inspection interval according to the requirements in the "Moving Ahead for Progress in the 21st Century Act (MAP-21)" legislation. The goal of the methodology is to improve the safety and reliability of bridges by focusing inspection efforts where most needed and optimizing the use of resources. The material in this report will be of immediate interest to bridge engineers.

The National Bridge Inspection Standards (NBIS) mandate the frequency and methods used for the safety inspection of highway bridges. The inspection intervals specified in the NBIS require routine inspections to be conducted every 24 months, and that interval may be extended to 4 years for bridges that meet certain criteria and are approved by FHWA. For bridges with fracture-critical elements, hands-on inspections are required every 2 years. The specified intervals are generally not based on performance of bridge materials or designs, but rather on experience from managing almost 600,000 bridges in the National Bridge Inventory.

These inspection intervals are applied to the entire bridge inventory, but they may not be appropriate for all bridges. For example, recently constructed bridges typically experience few problems during their first decade of service and those problems are typically minor. Under the present requirements, these bridges must have the same inspection frequency and intensity as a 50-year-old bridge that is reaching the end of its service life. In the case of bridges with fracture-critical elements, newer bridges with improved fabrication processes and designs that mitigate the effects of fatigue are inspected on the same interval and to the same intensity as older bridges that do not share these characteristics.

A more rational approach to determining appropriate inspection practices for bridges would consider the structure type, age, condition, importance, environment, loading, prior problems, and other characteristics of the bridge. There is a growing consensus that these inspection practices should meet two goals: (1) improving the safety and reliability of bridges and (2) optimizing resources for bridge inspection. These goals can be accomplished through the application of reliability theory.

Research was performed under NCHRP Projects 12-82 and 12-82(01) by the University of Missouri to develop a proposed bridge inspection practice for consideration for adoption by AASHTO. The methodology developed is based on rational methods to ensure bridge safety, serviceability, and effective use of resources.

The report includes two parts: Part I—Proposed Guideline for Reliability-Based Bridge Inspection Practices and Part II—Final Research Report: Developing Reliability-Based Inspection Practices that documents the entire research effort.

CONTENTS

PART I Proposed Guideline for Reliability-Based Bridge Inspection Practices

PART II Final Research Report: Developing Reliability-Based Inspection Practices

Note: Many of the photographs, figures, and tables in this report have been converted from color to grayscale for printing. The electronic version of the report (posted on the Web at www.trb.org) retains the color versions.

PART I

Proposed Guideline for Reliability-Based Bridge Inspection Practices

SUMMARY

Proposed Guideline for Reliability-Based Bridge Inspection Practices

This guideline describes a methodology for developing Risk-Based Inspection (RBI) practices for highway bridges. The goal of the methodology is to improve the safety and reliability of bridges by focusing inspection efforts where most needed and optimizing the use of resources. The guideline provides the framework and procedures for conducting reliability assessments to develop suitable inspection strategies for bridges based on an engineering assessment of inspection needs. The methodology considers the structure type, age, condition, importance, environment, loading, prior problems, and other characteristics that contribute to the reliability and durability of highway bridges.

RBI practices differ from traditional approaches that are generally calendar based, because the setting of inspection frequencies (or intervals) and scope are not fixed or uniform. Rather, reliability-based engineering analysis is conducted to assess the inspection *needs* for a particular bridge or family of bridges, and inspection requirements, i.e., frequency and scope, are aligned with those needs. This is achieved by analyzing the likelihood of anticipated damage modes and the associated outcomes or consequences. As a result, RBI practices can focus attention specifically on the damage and deterioration mechanisms that are most important for ensuring bridge safety. As such, they provide a better linkage between damage modes that affect bridges and the inspection approaches that will best reduce the associated risks, leading to improved bridge safety. This approach has been widely accepted in many industries with facilities that can be considered analogous to highway bridges: very large, expensive, and complex structural systems that are exposed to rugged environmental conditions and mechanical loading.

The purpose of this document is to provide guidance for bridge owners for conducting reliability-based assessments to determine inspection needs. The methodology requires bridge owners to perform a reliability assessment of bridges within their bridge inventory to identify those bridges that are most in need of inspection to ensure bridge safety, and those where inspection needs are less. An expert panel assembled at the owner level performs this assessment. The assessment considers the reliability and safety attributes of the bridges to assess the likelihood of damage and evaluate the potential outcomes or consequences in terms of safety and serviceability. Through this process, inspection needs are prioritized to improve the safety and reliability of the bridge inventory overall.

Definitions

Attributes: Characteristics that affect the reliability of a bridge or bridge element.

Condition Attributes: Characteristics that relate to the current condition of a bridge or bridge element. These may include element ratings, component ratings, and specific damage modes or mechanisms that have a significant effect on the reliability of an element.

Consequence Factor: Factor describing the expected outcome or result of a failure.

Damage Mode: Typical damage affecting the condition of a bridge element (e.g., spalling of concrete, cracking, etc.).

Design Attributes: Characteristics of bridge or bridge element that are part of the element's design. These attributes typically do not change over time except when renovation, rehabilitation, or preservation activities occur.

Deterioration Mechanism: Process or phenomena resulting in damage to a bridge element (e.g., corrosion, fatigue, etc).

Element: Identifiable portions of a bridge made of the same material, having similar role in the performance of the bridge, and expected to deteriorate in a similar fashion.

Failure: Termination of the ability of a system, structure, or component to perform its intended function (*1*). For bridges, the condition at which a given bridge element is no longer performing its intended function to safely and reliably carry normal loads and maintain serviceability.

Loading Attributes: Loading characteristics that affect the reliability of a bridge or bridge element, such as traffic or environment.

Occurrence Factor: Factor describing the likelihood that an element will fail during a specified time period.

Operational Environment: The operational environment is a combination of the circumstances surrounding and potentially affecting the in-service performance of bridges and bridge elements. These include typical loading patterns, ambient environmental conditions, construction quality and practices, maintenance and management practices, and other factors that may vary between different geographic regions and/or organizational boundaries.

Probability: Extent to which an event is likely to occur during a given time interval (*1*). This may be based on the frequency of events, or on degree of belief or expectation. Degrees of belief about probability can be chosen using qualitative scales, ranks, or categories such as "remote/low/moderate/high."

Reliability: Ability of an element or component to operate safely under designated operating conditions for a designated period of time.

Risk: Combination of the probability of an event and its consequence.

Risk Analysis: Systematic use of information to estimate the risk. Sources of information may include historical data, theoretical analysis and engineering judgment.

Screening Attribute: Characteristics of a bridge or bridge element that:

- Make the likelihood of serious damage unusually high,
- Make the likelihood of serious damage unusually uncertain, and
- Identify a bridge with different anticipated deterioration patterns than other bridges in a group or family.

CHAPTER 1

Introduction

This guideline describes a methodology for developing Risk-Based Inspection (RBI) practices for highway bridges. The goal of the methodology is to improve the safety and reliability of bridges by focusing inspection efforts where most needed and optimizing the use of resources. The guideline provides a framework and procedures for developing suitable inspection strategies based on a rational engineering assessment of inspection needs. The methodology considers the structure type, age, condition, importance, environment, loading, prior problems, and other characteristics that contribute to the reliability and durability of highway bridges.

The methodology requires bridge owners to perform a reliability assessment of bridges within their bridge inventory to identify those bridges that are most in need of inspection to ensure bridge safety, and those where inspection needs are less. This assessment is conducted by considering the reliability and safety attributes of bridges, assessing the likelihood of damage and associated deterioration mechanisms, and evaluating the potential outcomes or consequences in terms of safety and serviceability. Through this process, inspection needs are prioritized to improve the safety and reliability of the bridge inventory overall.

This chapter of the document provides an introduction and overview of the process, as well as background information on the underlying theories and common practices for RBI and reliability assessments. Chapter 2 of the document describes the methodology for conducting a reliability assessment for bridges. This includes providing a definition of element failure suitable as an analysis tool, and a description of the key factors to be assessed in the typical reliability assessment conducted for inspection planning purposes. This chapter also describes the composition of the Reliability Assessment Panel (RAP) that will conduct the assessments.

Chapter 3 describes the process for determining the appropriate maximum inspection interval and scope of inspection, based on analysis as described in Chapter 2. The underlying approaches for identifying inspection intervals and the techniques or methods to be used for the inspections are discussed. Finally, Chapter 4 provides an overview of the overall process, guidance for bridge owners on beginning an RBI program, transitioning from traditional, calendar-based approaches, and general guidance on the training that may be required.

There are six appendices in the document that describe in more detail the process and mechanics of the analysis. Guidance for determining the factors necessary to perform a reliability assessment are included in Appendices A, B, and C. Guidance on inspection methods and nondestructive evaluation (NDE) technologies that can be used for conducting RBIs is described in Appendix D. Appendix E contains commentary regarding specific, common attributes of bridges that influence damage modes and deterioration mechanisms, and relate to bridge reliability. Finally, Appendix F includes three example implementations of the methodology applied to bridges of common design: a multi-girder concrete bridge with prestressed superstructure elements constructed in the past 5 years, a multi-girder steel bridge constructed more than 50 years ago, and a multi-girder reinforced concrete bridge constructed in 1963.

1.1 Process

The process involves an owner (e.g., state) establishing an expert panel to define and assess the durability and reliability characteristics of bridges within the state. The expert group analyzes portions of the bridge inventory to assess inspection needs by using engineering rationale, experience, and typical deterioration patterns to evaluate the reliability characteristics of bridges and the potential outcomes of damage. This is done through a relatively simple process that consists of three primary steps:

Step 1: What can go wrong, and how likely is it? Identify possible damage modes for the elements of a selected bridge type. Considering design, loading, and condition characteristics (attributes), categorize the likelihood of serious damage occurring into one of four *occurrence factors* (OFs) ranging from remote (very unlikely) to high (very likely).

Step 2: What are the consequences? Assess the consequences, in terms of safety and serviceability, assuming the given damage modes occur. Categorize the potential consequences into one of four *consequence factors* (CFs) ranging from low (minor effect on serviceability) through severe (i.e., bridge collapse, loss of life).

Step 3: Determine the inspection interval and scope. Use a simple 4 × 4 matrix to prioritize inspection needs and assign an inspection interval for the bridge based on the results of Steps 1 and 2. Damage modes that are likely to occur and have high consequences are prioritized over damage modes that are unlikely to occur or are of little consequence in terms of safety. An RBI procedure is developed based on the assessment of typical damage modes for the bridges being assessed.

Inspections are conducted according to the RBI procedure developed through this process. Results of the inspection are assessed to determine if the existing RBI procedure needs to be modified or updated as a result of findings from the inspection.

Through this process, individual bridges, or groups of bridges of similar design characteristics, can be assessed to evaluate the inspection needs based on an engineering analysis of the likelihood of serious damage occurring and the effect of that damage on the safety and serviceability of the bridge. This approach considers the structure type, age, condition, and operational environment in a systematic manner to provide a rational assessment process for inspection planning. A documented rationale for the inspection strategy utilized for a given bridge is developed. The damage modes most important to ensuring the safety and serviceability of the bridge are identified such that inspection efforts can be focused to improve the reliability of the inspection results.

1.1.1 Scope

This guide is focused on the inspection of typical highway bridges of common design characteristics. Atypical structures, such as long-span truss bridges, cable-stayed bridges, suspension bridges, and other unique or unusual bridge designs may require certain considerations not presently captured in this guideline; this guideline provides for inspection planning for the superstructure, substructure, and deck for typical highway bridges. Scour and underwater inspections have existing methodologies for evaluation, and, as such, are not included herein. Bridges assessed using this methodology are assumed to have a current load rating that indicates that the structural capacity is sufficient to carry allowable loads.

1.1.2 Purpose

The purpose of this document is to provide guidance for bridge owners for conducting reliability-based assessments for determining the frequency and scope of inspections for typical highway bridges. This document is intended to be used by bridge owners for assessing their bridge inventories in order to prioritize inspection needs based on an engineering analysis that considers the bridge type, age, loading, condition, and other characteristics of a bridge. This guideline is intended for application to typical bridges with common and ordinary forms of deterioration and damage. Advanced deterioration and/or specific defects such as fatigue cracks due to primary stresses or severe corrosion damage in concrete typically require more detailed engineering analysis than provided herein.

1.2 Background

The periodic inspection of highway bridges in the United States plays a critical role in ensuring the safety, serviceability, and reliability of bridges. Inspection processes have developed over time to meet the requirements of the National Bridge Inspections Standards (NBIS)(*2*) and to meet the needs of individual bridge owners in terms of managing and maintaining their bridge inventory. The inspection frequency mandated by the NBIS requires the inspection interval (maximum time period between inspections) not to exceed 24 months. Based on certain criteria, that interval may be extended up to 48 months with approval from the Federal Highway Administration (FHWA) (*3*). Maximum inspection intervals of less than 24 months are utilized for certain bridges according to criteria developed by the bridge owner, typically based on age and known deficiencies. Most bridge owners utilize the uniform maximum inspection interval of 24 months, as mandated by the NBIS, for the majority of the bridges in their inventory, and the reduced intervals for bridges with known deficiencies. Only 15 states utilize the 48 month policy, often only for culverts. The uniform inspection interval of 24 months was specified at the origination of the National Bridge Inspection Program in 1971 based on experience, engineering judgment, and the best information available at the time. The uniform approach provides a single maximum inspection interval for most bridges, regardless of the bridge age, design, or environment. To date, this mandated inspection interval has provided an adequate level of safety and reliability for the bridge inventory nationwide. However, such a uniform inspection interval does not consider explicitly the likelihood of failure based on bridge condition, design, or operating environment, or the potential consequences of a failure. A uniform inspection interval does not recognize that a newly constructed bridge with improved durability characteristics and a few years of exposure to the service environment may be much less likely to develop serious damage over a given time interval than an older bridge that has been exposed to the service environment for many years. Bridges that are in benign, arid operating environments are inspected at the same interval as bridges in aggressive marine environments, where significant damage from corrosion may develop much more rapidly. Current practices make it difficult to distinguish if the same or improved safety and reliability could be achieved by varying inspection methods or frequencies to meet the needs of a specific bridge based on its design and operational environment. The current approach also makes it difficult to analyze if a given inspection activity is excessive, or if it provides little or no measure of increased assurance of the safety and reliability of bridges. Given that any inspection activity carries with it a certain amount of risk to both the inspector and to the traveling public, inspections that are excessive or that provide little benefit may present added, unnecessary risks. Otherwise, inspections that are inadequate or fail to distinguish the importance of critical damage modes may also present certain added risks that require analysis.

Recognizing the variability in the design, condition, and operating environments of bridges would provide for inspection requirements that better meet the needs of individual bridges to improve both bridge and inspection reliability. Other industries are increasingly recognizing the limitations of prescribed inspection frequencies and are developing methodologies for efficiently assessing inspection needs, ensuring the safety and reliability of systems, and focusing inspection resources most effectively (*1, 4–6*). Methodologies for assessing inspection needs based on the likelihood of a service failure, combined with the consequences of such a failure, is a common approach to inspection planning and to developing effective inspection strategies. These approaches are typically described as *risk-based*, where inspection planning is conducted considering the reliability of a component, i.e., how likely is it that the component or machine will fail during a certain time period, and the consequences of such an event. Damage modes and deterioration mechanisms are typically assessed explicitly to determine the likelihood of failure during a given time period, and to identify the appropriate inspection methods to detect critical damage prior to failure.

A risk-based approach has been adopted in many industries as a tool for inspection planning, to focus attention on the component or machine that represents the greatest "risk." Risk is defined as the product of the probability of an event and the associated consequences:

$$Risk = Probability \times Consequence$$

Probability in this equation is the likelihood of an adverse event or failure occurring during a given time period. This is sometimes expressed quantitatively as a probability of failure (POF) estimate for a given time interval, or as a qualitative assessment of the likelihood of an adverse event based on experience and engineering judgment. *Consequence* is a measure of the impact of the event occurring, which may be measured in terms of economic, social, safety, or environmental impacts.

Risk can be expressed quantitatively using POF estimates or models and quantitative measures of consequences, such as the cost of a certain event or the loss of service of a component. Risk can also be expressed qualitatively by estimating whether the likelihood of a certain event is high, medium, or low, and determining a qualitative estimate of the consequences. Presenting risk qualitatively is a common and effective method for evaluating risk and for assessing relative risk efficiently. Figure 1 shows a qualitative risk matrix (*1*, *5*). This matrix shows a good representation of the overall concept and basic principles of risk. A high likelihood (probability) of occurrence combined with a high consequence results in a high risk, located in the upper right corner of the figure. Low likelihood combined with a low consequence results in low risk, located in the lower left-hand corner of the figure. High risk and low risk elements typically do not create challenges in decision making; items that are "high risk" may not be acceptable and actions are required to lower the risk, either by reducing the likelihood of an event, or by reducing the consequences, or both. Items that are "low risk" are typically acceptable and may require little or no action. In the "medium risk" area, questions may arise about how much risk is acceptable, and what the appropriate decision-making strategies are for mitigating that risk. In terms of inspection strategies, items that are "high risk" are prioritized for more frequent and possibly more intense inspections to reduce uncertainty and to monitor the development of damage to ensure that safety is maintained. Items that are "low risk" may have longer inspection intervals and have less intense inspection protocols.

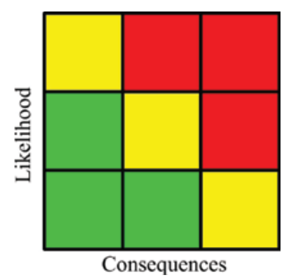

Figure 1. Risk matrix showing high-, medium-, and low-risk values.

An important concept in risk analysis is to understand that high likelihood does not necessarily mean high risk, if the consequences are small. Similarly, high consequence does not necessarily mean high risk, if the likelihood is small. The level of risk can only be determined once both of these variables are assessed.

A risk-based planning approach focuses attention not on the items that are most likely to fail, but rather those items whose failure is most important, by considering both the likelihood of failure and the associated consequences. The setting of inspection frequencies or intervals is not a rigid process, such as is the case for uniform or calendar-based inspection frequencies. Rather, it is a process that evolves and changes over the life of a component such that inspection frequencies change as risk increases (or decreases). Therefore, the frequency of inspection is aligned with the needs and the associated risks, focusing attention on the most at-risk items. This approach has been widely accepted in many industries with facilities that can be considered analogous to highway bridges: very large, expensive, and complex structural systems that are exposed to rugged environmental conditions and mechanical loading *(1, 4, 6)*.

1.2.1 Reliability and Probability

Reliability is defined as the ability of an item to operate safely under designated operating conditions for a designated period of time or number of cycles. For bridges and bridge elements, reliability typically decreases as a function of time due to deterioration and the damage accumulated during the service life of a bridge. That is, the likelihood of failure typically increases with time as the element ages, due to deterioration mechanisms such as corrosion or fatigue. The reliability of a bridge or bridge element can be expressed as:

$$R(t) = \Pr(T \geq t)$$

Where $R(t)$ is the reliability, T is the time to failure for the item, and t is the designated period of time for the item's operation. In other words, the reliability is the *probability* (Pr) or *likelihood* that the failure time exceeds the operation time. Sometimes, the probability is expressed as a probability density function (pdf) that expresses the time to failure of an item (T) as some generic distribution, such as normal, log normal, etc. This distribution can be used to calculate a POF function, $F(t)$, to express the probability that the item will fail sometime up to time t. This time-varying function describes likelihood of failure up to some given time, or the *unreliability* of the item, and the reliability is then:

$$R(t) = 1 - F(t)$$

In other words, the reliability is the probability that the item *will not* fail during the time period of interest. When a large population of test data of identical or near identical components exposed to the same operational environment are available, a probability function describing the failure characteristics of the component may be determined and verified based on the results. If test data are not available, a suitable distribution must be assumed based on the general characteristics of the population, typical failure behavior, and known deterioration mechanisms. These distributions are typically based on experience and assumptions regarding the anticipated performance of the system or component. This is challenging and can lead to unsubstantiated confidence in the model when the design characteristics, construction quality, condition, and operational environment of the components vary. Even if substantial data were readily available, design and construction practices are constantly evolving such that past performance may not indicate future performance. Critical damage modes may have yet to manifest in observable damage, and as such may not be included in the data. Given the large variation in the design, construction, construction quality, and operational environments, the utility of probabilistic models to effectively predict the future performance of a specific bridge or bridge element is problematic.

Under these circumstances, engineering judgment and experience is needed to estimate the expected reliability of a specific component, or set of components, of similar design and construction quality operating within a specific operational environment. Engineering judgment is required to estimate the reliability of bridge elements based on past experience, engineering knowledge, and a rational process to systematically assess bridges of common design and construction characteristics. The process involves engineers with experience and expertise in the performance of bridges within a particular operational environment using engineering judgment to assess the probability (likelihood) of failure during some future time period. When combined with an assessment of the consequences, an effective analysis can be conducted to identify inspection needs efficiently.

1.2.2 Consequences

The primary purpose of bridge inspection is to ensure the safety and serviceability of highway bridges. As a result, the consequences to be assessed in prioritizing the importance of different damage modes are assessed in terms of bridge safety and serviceability. The consequence of failure, or of serious damage developing in a bridge element, typically depends on the role of that element in the structural system of the bridge, and on the operating environment surrounding the bridge. For example, the consequence of an abutment having severe corrosion damage might be *low*, while the same damage in a main superstructure member may be *high.* The consequence of damage developing at the soffit of a bridge deck, such as concrete spalling, might be *low* if the bridge is over a flood plain, but *high* if the bridge is over an interstate highway. The consequence associated with a given damage mode can be assessed through engineering judgment, through common or related experience, or through theoretical analysis.

The process developed and described herein requires the determination of two key parameters: an estimate of the reliability of given bridge elements, based on the likelihood (probability) that the element would fail during a given time interval, and an assessment of the consequences of that failure. These data are then used to determine an appropriate inspection interval and scope (procedures and methods) for a bridge. As such, the methodology described is a reliability-based bridge inspection planning process for ensuring the safety and serviceability (i.e., reliability) of highway bridges.

CHAPTER 2

Reliability Assessment of Bridge Elements

This section describes the methodology for reliability assessment of the bridge elements. Section 2.1 describes and defines *failure* as applied to typical bridge elements for the reliability assessment. Section 2.2 describes the methodology for evaluating the probability or likelihood that failure will occur (OF). Section 2.3 describes the methodology for evaluating the consequences of that occurrence (CFs). Finally, Section 2.4 discusses the panel that conducts the assessment, the RAP.

2.1 Definition of Failure

It is critical that the conditions that constitute a *failure* be defined before beginning a reliability assessment. For bridges, catastrophic collapse would be one obvious condition that could be used to define failure. For most bridges, the probability of catastrophic failure is very remote. For bridge inspections, important concerns extend well beyond simply avoiding catastrophic failure. Ensuring the safety of the bridge, the safety of those traveling on or below the bridge, and the serviceability of the bridge are each critical. Maintenance and repair activities are needed to support the serviceability of the bridge and ensure the safety of motorists, even while the likelihood of a catastrophic failure remains remote.

Therefore, *failure* requires a suitable definition that captures the need to ensure the structural safety of the bridge, the safety of travelers on or below the bridge, and the serviceability of the bridge. *Failure*, utilized in this context, is defined as when an element is *no longer performing its intended function to safely and reliably carry normal loads and maintain serviceability*. For example, a bridge deck with severe spalling may represent a "failed" condition for the bridge deck even though the deck may have adequate load-carrying capacity, because the ability of the deck to reliably carry traffic is compromised. The condition rating of 3, "serious condition" according to the NBIS rating system, is used in the analysis described herein as a general description of a "failed" condition. It is not envisioned that any bridges or bridge elements assessed using a risk-based approach are allowed to deteriorate to this condition. *Rather, inspection intervals are adjusted to ensure that the likelihood of* failure *in the time intervals between inspections always remains* low. Bridge components that have deteriorated to this extent may no longer be performing their intended function, and remedial actions are typically planned to address such conditions. The subjective condition rating of 3 is defined within the *Recording and Coding Guide* (*7*) as follows:

> *NBI Condition Rating 3: SERIOUS CONDITION: Loss of section, deterioration, spalling or scour have seriously affected primary structure components. Local Failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.*

This condition description is widely understood and there is significant past experience in the conditions warranting a rating of 3 throughout the bridge inventory. This condition description is not absolute, but provides a frame of reference for the analyst considering the likelihood of damage occurring to a serious extent. In terms of the AASHTO Bridge Element Inspection Guide, this condition generally aligns with elements in condition state (CS) 4, "severe. " (*8*)

2.2 Occurrence Factors

What can go wrong, and how likely is it to occur?

The first step in the reliability assessment is to address the question "*What can go wrong, and how likely is it to occur?*" The first part of this question, "what can go wrong" addresses the *damage modes* that affect typical bridge elements. In other words, what damage is likely to develop over the service life of the bridge, which may result in the *failure* of a given element? "*Failure*" used in this context is serious damage to the element such that its performance as intended cannot be assured, as described in Section 2.1 (e.g., condition rating = 3 or CS = 4). For concrete elements, spalling and cracking of the concrete is a typical *damage mode.* For steel elements, section loss or cracking are typical *damage modes.* The second part of this question, "how likely is it to occur," describes the likelihood, or probability, of failure due to that damage mode occurring, given the design, materials, and current condition of a bridge element. The OF categorizes this likelihood on a qualitative scale that provides an assessment of the likelihood of serious damage, i.e., failure, occurring.

For the assessment of bridge inspection needs, the OF is usually an assessment of the likelihood that a given damage mode will result in failure (i.e., serious condition), over a time period of 72 months (6 years). The deterioration mechanism resulting in the damage is considered in the assessment. In some cases the OF may be an estimate of the likelihood of a certain adverse event occurring, such as impact from an over-height vehicle or an overload. Each damage mode or adverse event must have a separate OF, based on the likelihood of the damage mode or the event resulting in failure of an element during the specified time interval.

The OF describes the likelihood of failure of an element in one of four categories. The scale ranges from remote, when the likelihood is extremely small such that it would be unreasonable to expect failure, to high, where the likelihood of the event is increased, as shown in Table 1.

To assess the appropriate OF for a given bridge element, key characteristics, or *attributes*, are considered. "Attributes" are characteristics of a bridge element that contribute to the element's reliability, durability, or performance. These attributes are typically well-known parameters affecting the performance of a bridge element during its service life. This includes relevant design, loading, and condition characteristics that are known or expected to affect the durability and reliability of the element. For example, consider the damage mode of spalling due to corrosion damage in a concrete bridge deck. A bridge deck may have "good" attributes, such as being in very good condition, having adequate concrete cover, epoxy-coated steel reinforcing, and minimal application of de-icing chemicals. Given these attributes of the deck, it may be very

Table 1. OF rating scale for RBI.

Level	Category	Description
1	Remote	Remote likelihood of occurrence, unreasonable to expect failure to occur
2	Low	Low likelihood of occurrence
3	Moderate	Moderate likelihood of occurrence
4	High	High likelihood of occurrence

unlikely that severe damage (i.e., failure) would occur in the next 72 months. This is based on the rationale that the deck is presently in good condition, and has attributes that are well-known to provide resistance to corrosion damage. As such, an OF of "Low" or "Remote" might be used to describe the likelihood of failure due to this damage mode. Alternatively, suppose the deck is in an environment where de-icing chemicals are frequently used, the reinforcement is uncoated, and the current rating for the deck is a 5, Fair Condition, indicating that there are signs of distress in the deck. Based on this rationale, the likelihood of serious damage developing would be much greater, resulting in an OF rating of "Moderate" or "High." Past experience with decks of a similar design, characteristics of the specific operating environment, and attributes of the deck are combined with engineering judgment and used to support the assessment of the specific OF for a given deck. Methodologies for determining credible damage modes and their associated attributes are included in Appendix A.

Certain key attributes will ideally be identified as part of criteria for reassessment of bridge inspection requirements, following subsequent RBIs. These key attributes are typically associated with condition, which may change over the service life of the bridge as deterioration occurs. When changes in these condition attributes cause a change in the likelihood of a given damage mode resulting in failure (i.e., the OF), reassessment of the inspection requirements is necessary.

Deterioration rate data, trends, and theoretical models can be used to support the categorization of the OFs by providing insight regarding the average, typical, or expected behavior of elements of a similar design. Transition probabilities, Weibull statistics, or regression trends, developed based on past inspection results, can provide insight into the anticipated behavior of a group of similar bridge elements. Care should be taken to ensure that the bridge elements being assessed have similar or the same attributes as those represented by the data. Theoretical models may also be used to support these assessments. However, the complexity and variations in the operational environment, construction variability, and current condition can be difficult to capture in these models. Results need to be verified using engineering judgment.

2.3 Assessment of Consequences

What are the consequences?

The second factor to be assessed within an RBI process is the Consequence Factor, CF, a categorization of the likely outcome presuming a given damage mode were to result in failure of the element being considered. The assessment of consequence is geared toward assessing and differentiating elements in terms of the consequences of the assumed element failure. It should be noted that *failure of an element is not an anticipated event when using an RBI approach*, rather the process of assessing the consequences of a failure is merely a tool to rank the importance of a given element relative to other elements for the purpose of prioritizing inspection needs.

The CF is used to categorize the consequences of the failure of an element into one of four categories, based on the anticipated or the expected outcome. Failure scenarios are considered based on the physical environment of the bridge, typical or expected traffic patterns and loading, the structural characteristics of the bridge, and the materials involved. These scenarios are assessed either qualitatively, through necessary analysis and testing, or based on past experience with similar failure scenarios. The four-level scale used to assign the CF is shown in Table 2. The CF ranges from low, used to describe failure scenarios that are benign and very unlikely to have a significant effect on safety and serviceability, through catastrophic scenarios, where the threat to safety and life is significant. Thus, both short-term (generally safety related) and long-term (generally serviceability related) consequences can be considered.

Table 2. CFs for RBI.

Level	Category	Consequence on Safety	Consequence on Serviceability	Summary Description
1	**Low**	None	Minor	Minor effect on serviceability, no effect on safety
2	**Moderate**	Minor	Moderate	Moderate effect on serviceability, minor effect on safety
3	**High**	Moderate	Major	Major effect on serviceability, moderate effect on safety
4	**Severe**	Major	Major	Structural collapse/loss of life

In assessing the consequences of a given damage mode for a given element, the RAP must establish which outcome characterized by the CFs in Table 2 is the most likely. In other words, which scenario does it have the most confidence will result if the damage were to occur. Using the illustration of brittle fracture in a girder, it is obvious that the most likely consequence scenario would (*and should*) be different for a 150-foot span two-girder bridge than for a 50-foot span multi-girder bridge. For the short-span, multi-girder bridge, an engineer may state with confidence that the most likely consequence scenario is "high" or "moderate" and that the likelihood of "severe" consequences is very remote for a multi-girder bridge, based on his or her experience and the observed behavior of multi-girder bridges. For the two-girder bridge, the consequence scenario is likely to be "Severe." As this example illustrates, the CF simply ranks the importance of the damage mode as being higher for a two-girder bridge than for a multi-girder bridge. For many scenarios, qualitative assessments based on engineering judgment and documented experience are sufficient to assess the appropriate CF for a given scenario; for others, analysis may be necessary using suitable analytical models or other methods. A series of more detailed criteria for specific elements [i.e., decks, steel girders, prestressed (P/S) girders, etc.] are provided in the Appendix B that can be utilized during the assessment to determine the appropriate CF for a given element failure scenario. These criteria, combined with owner-specific requirements developed in the RAP or from other rational sources for assessing bridges and bridge redundancy, are then used to determine the appropriate CF for a given scenario.

2.4 The Reliability Assessment Panel

An important component of the analysis process is the elicitation of expert judgment regarding the likelihood of damage and the level of associated consequences. Because design features, construction specifications and practices, materials, environment, and bridge management strategies differ from state to state, or even within a particular state, the expert panel should be selected keeping in mind the need to have membership which is familiar with the operational environment of the inventory of bridges being evaluated.

The RAP typically will consist of four to six experts from the bridge-owning agency. This panel should include an inspection team leader or program manager that is familiar with the inspection procedures and practices, as they are implemented for the inventory of bridges being analyzed. The team should include a structural engineer who is familiar with the common load paths and the overall structural behavior of bridges, and a materials engineer who is familiar with the behavior of materials in the particular environment of the state and has past experience with materials quality issues. A facilitator may be used to assist in the analysis process. The general characteristics of members of a RAP include the following:

1. **Bridge Inspection Expert:** Inspection team leader or program manager that oversaw the specific inspection process and the reports for the bridges being evaluated. This individual should be able to represent the inspection results reported in the bridge file, understand the

notes and sketches included in the file, and have an understanding of the scope and the methods of the inspections used for the bridges under consideration.

2. **State Program Manager or Bridge Management Engineer:** Individual familiar with the characteristics and the behavior of the bridge inventory throughout the state.
3. **Bridge Maintenance Engineer:** An individual familiar with the standard methods and techniques used for bridge maintenance, the level of maintenance typical for the bridges under consideration, and the outcomes of bridge maintenance.
4. **Materials Engineer:** A materials engineers who is familiar with the history of materials performance within the state. This individual should be experienced with the materials historically used within the state, be knowledgeable of any prior problems with the quality or with the performance of the materials used, and be knowledgeable of typical deterioration patterns.
5. **Structural Engineer:** An engineer with sufficient training and experience to understand the consequences, in a structural sense, of bridge element failures. For example, the structural engineer should be able to recognize the load paths in a structure and to understand the importance of elements in the overall structural system of the bridge.
6. **Independent Experts:** The RAP may include independent experts, academics, or consultants to address specific or complex damage modes, provide independent review, and/or supplement the knowledge of the panel as needed.
7. **Facilitator:** A RAP facilitator may be used to assist in the RAP analysis, to lead expert elicitations, and help build consensus during the analysis process.

The expert panel may also include representatives from the FHWA to monitor the process, to fulfill oversight responsibilities, and to assist with the implementation of the methodology used for inspection planning.

CHAPTER 3

Determination of Inspection Interval and Scope

This section describes the process of determining the inspection interval and scope based on the assessment completed as described in Chapter 2. This process leads to a prioritization of inspection needs, highlights critical damage modes for bridges, and results in an RBI practice.

3.1 Inspection Interval

The inspection interval is selected based on the RAP assessment of the OFs and CFs. Once these factors have been determined, their numerical values are used to place a given damage mode in the appropriate location on a reliability matrix. A typical reliability matrix is shown schematically in Figure 2. In this figure, the horizontal axis represents the CF as determined for a particular damage mode for a given bridge element. The vertical axis represents the outcome of the OF assessment for a given damage mode for a given element. Elements that tend toward the upper right corner of the reliability matrix require shorter inspection intervals, and possibly more intense inspections, than elements that fall in the lower left corner.

The matrix is utilized to determine the appropriate maximum inspection interval for a given bridge or bridge type. These inspection intervals are determined to ensure that the probability, or likelihood, of failure remains low during the inspection interval. The maximum inspection interval is established in order to be consistent with the assessment of the OF, as determined over the predefined assessment interval of 72 months, as described in Section 2.2. Keeping this in mind, the *actual* maximum inspection interval is determined such that the likelihood of occurrence within the time between inspections (i.e., the inspection interval) always remains low. For example, if the OF is "low" over a 72-month period, than it may be reasonable to assign the inspection interval of 72 months (ignoring the influence of consequence for the time being). However, if it were found that the OF were high, the analysis is really indicating a failure is relatively likely to occur before the end of the 72-month interval. Since the goal is to ensure that the possibility of failure occurring before the end of the interval is always low, one would shorten the inspection interval, for example to 24 months. In other words, by inspecting every 24 months, the possibility of failure occurring before the end of the interval (now reduced to 24 months) remains low.

Obviously, the OF is not the only parameter that should be evaluated when setting the interval. The consequence of the failure must also be incorporated into the process of selecting the appropriate interval. Using the example above, where the OF were high and the interval was reduced to 24 months; if the consequence of that same damage was determined to be severe, it would be appropriate to assign a shorter interval of, for example, 12 months. This provides an extra measure of confidence and safety (i.e., a reduction from 24 months to 12 months due to the severe nature of the consequences). Although there are many permutations of the OF and the CF, the above illustrates the concept.

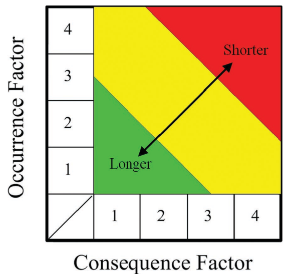

Figure 2. Risk matrix for determining maximum inspection intervals for bridges.

This is a relatively easy task for elements where the OF is high and the CF is severe, and hence an interval of 12 months or less is needed. However, if the OF is remote and the CF is low, then it would also seem reasonable and justifiable that the inspection interval should be greater than the longest interval assumed in the OF assessment (72 months). (If the OF is remote, this indicates the members of the RAP concluded that there is a "remote likelihood of occurrence, unreasonable to expect failure to occur" in the next 72 months for this element and damage mode.) This information, coupled with the observation that failure, should it occur, is a low consequence, may justify the use of an inspection interval longer than 72 months.

The actual inspection interval selected is based on the shortest inspection interval determined from the analysis. In other words, whichever element has the shortest maximum inspection interval, based on the likelihood of failure and associated consequence. In certain circumstances, there may be one element of the bridge that results in a much shorter inspection interval than the other elements of the bridge. In such a case, a different inspection interval may be identified for that particular element, based on engineering judgment and the discretion of the bridge owner. For most cases, multiple elements would be expected to have the same or very similar intervals, with the shortest interval being selected for practical reasons.

3.1.1 Inspection Scope

Under an RBI practice, the inspection scope is determined from the damage modes identified through the reliability analysis. In other words, the inspection methods used are selected based on their effectiveness and reliability for detecting the specific damage mode(s) that are most important. Guidelines for the selection of inspection methods to be used are included in Appendix D. In many cases, visual inspections supplemented with sounding are well-proven approaches to detecting typical damage in highway bridges. However, in a risk-based process, these inspections would include hands-on access to key portions of a bridge, such that damage is effectively identified to support the RBI assessment. For example, when assessing the likelihood of fatigue cracking in a bridge, it would be necessary to know if there were *currently* fatigue cracks in the bridge. Therefore, the inspection scope used to support the assessment must utilize an approach that is capable of making that determination. This would require hands-on access to certain locations where fatigue cracking is likely to occur. In some cases, NDE techniques are required, often based on a limited access for visual inspection (e.g., for detecting a crack in a bridge pin).

Based on the assessment of the OF and the CF, damage modes for a bridge are prioritized based on the product:

$$IPN = OF \times CF$$

Where IPN = Inspection Priority Number. For example, if the fatigue cracking has a moderate likelihood of occurring and the consequence is severe, then the IPN would be $3 \times 4 = 12$. If fatigue cracking were moderately likely, but the consequence were only moderate (minor service disruption), for example, if the bridge in question is a short-span, multi-girder bridge with known redundancy, the IPN for that damage mode would only be $3 \times 2 = 6$. This process highlights the damage modes that are most important, that is, most likely to occur, and have the greater associated consequences, if they did occur.

It should be noted that the calculation of the IPN for each damage mode identified in the process does not limit the scope of the inspection to only *those damage modes. However, it does provide a simple method to prioritize damage modes that are* most important, *based on a rational engineering assessment that incorporates bridge type, age, design details, condition, etc., as well as the consequences of failure.*

3.1.2 Sampling

When using the RBI approach, it may be appropriate to inspect a representative sample of a bridge element, using the inspection method identified. This can be used to reduce or limit inspection activities that provide little or no measure of increased benefit or that introduce risks that are unjustified. The sampling population size (number of locations or area, for example) should reflect the nature and type of damage to be assessed through the inspection. When damage modes are expected to be widespread and relatively uniform, such as spalling in a bridge deck, an appropriate sampling based on area may be justified. For example, inspecting 25% of the bridge deck to assess if delaminations are present. When damage modes are isolated or non-uniform, such as fatigue cracks, sufficient sampling must be based on analysis to identify the location and number of inspections. Criteria and analysis supporting the sampling should be documented.

3.1.3 Maintenance Inspections

RBIs are typically more focused and intense than calendar-based, general-condition inspections, and the maximum interval between inspections may be increased. For bridges with extended inspection intervals, *maintenance inspections* may be specified periodically to ensure the maintenance of traffic safety and to address general maintenance needs. These inspections are typically conducted by maintenance personnel with responsibility for the maintenance of the roadways and the bridges in the district or region where the bridge is located. The purpose of a maintenance inspection is to:

- Identify bridge maintenance needs (minor patching, clearance of debris, vegetation control, etc.).
- Confirm general conditions have not significantly changed.
- Monitor unreported vehicular damage to a structure.
- Evaluate traffic safety issues (maintaining signage, roadway delineations, etc.).

Intervals for maintenance inspection would typically not exceed 2 years. Such maintenance inspections may be integrated into the business practice of a district or region.

3.1.4 Initial Inspections

Initial inspections, the first inspection of a bridge following construction or reconfiguration of a structure (e.g., widening, lengthening, supplemental bents, etc.) are required according to AASHTO's *The Manual for Bridge Evaluation* (*9*). In addition to this initial inspection, at least one RBI should be conducted at the interval of 24 months prior to initiating an RBI practice utilizing an interval greater than 24 months. Newly rehabilitated bridges should also have at least one RBI at the interval of 24 months following rehabilitation. The purpose of these inspections is to ensure that construction errors or deficiencies have not significantly altered the anticipated performance, and that a thorough inspection based on the RAP analysis has been conducted.

3.1.5 Start-Up Inspections

When initiating an RBI practice for a bridge, the first RBI should be conducted at the regular interval for the bridge, typically 24 months under the current NBIS. This start-up RBI will implement the practice as determined through the RAP analysis. Following the start-up inspection, the inspection results should be assessed for conformance with criteria and attributes identified by the RAP to determine if reassessment is necessary before implementing any modifications to the inspection interval.

3.1.6 Quality Control/Quality Assurance

Quality control (QC) and quality assurance (QA) processes should be employed to ensure quality in the implementation of RBI practices. Procedures for QC could include data model reviews, scoring and reliability factor reviews, RAP procedures, and application of inspection intervals based on the RBI analysis. Procedures for QA could include analysis of historical bridge performance, consistency in data models developed from the RAP analysis, and field reviews of bridge performance under the RBI process. Additional methods for QC and QA for bridge inspection programs are available in the literature (*10*).

CHAPTER 4

Establishing an RBI Program

4.1 Overview of Process

The overall process for implementing an RBI is shown schematically in Figure 3. The process begins with the selection of a bridge or family of similar bridges to be analyzed. For the selected bridge or bridges, the RAP identifies credible damage modes for elements of the bridge, given the design, materials, and operational environment. Key attributes are identified and ranked to determine the OFs, and the appropriate CFs associated with the damage modes are analyzed. Based on the assessment of the OFs and CFs for the bridge, the inspection practice is established including the interval and scope (procedures) for the inspection, and criteria for reassessment of the inspection practice. The criteria for reassessment are typically based on conditions that may change as a result of deterioration or damage, and may affect the OFs for the bridge. The RBI practice is then implemented in the subsequent inspection of the bridge. Following the inspection, inspection results are assessed to determine if any established criteria have been violated, or if conditions have changed that may require a reassessment of the OF. If such changes exist, a reassessment of the OF is completed and the inspection practice modified accordingly. If no such changes or conditions exist, the inspection practice can remain unchanged for the subsequent inspection interval.

Using the overall process described above, bridge owners can initiate an RBI practice for bridges in their inventory. However, the process and inspection requirements under an RBI practice may diverge significantly from traditional, calendar-based, and uniform inspection strategies. Therefore, consideration is needed regarding the scope of initial RBI assessments, training for inspectors and RAP members, and integration with existing software and databases. The sections that follow discuss these considerations.

4.2 Setting the Scope of the Analysis

RBI requires increased planning resources relative to calendar-based or uniform inspection processes. An effective strategy for transitioning from a calendar-based inspection practice to RBI is needed to facilitate the process and ensure adequate resources are available to conduct the necessary assessments. A suitable approach for transitioning an inspection program from a calendar-based, uniform inspection strategy to RBI is to identify those bridges where a reliability analysis can most readily be conducted and begin the process by assessing those bridges first. These bridges may be identified by conducting a simple qualitative risk assessment of the overall bridge inventory. This assessment should identify those bridges or family of bridges that are of very common design characteristics, and where significant experience exists regarding the anticipated damage and deterioration patterns. Such an assessment can be rapidly conducted based on general bridge characteristics such as span length, bridge type, number of spans, and

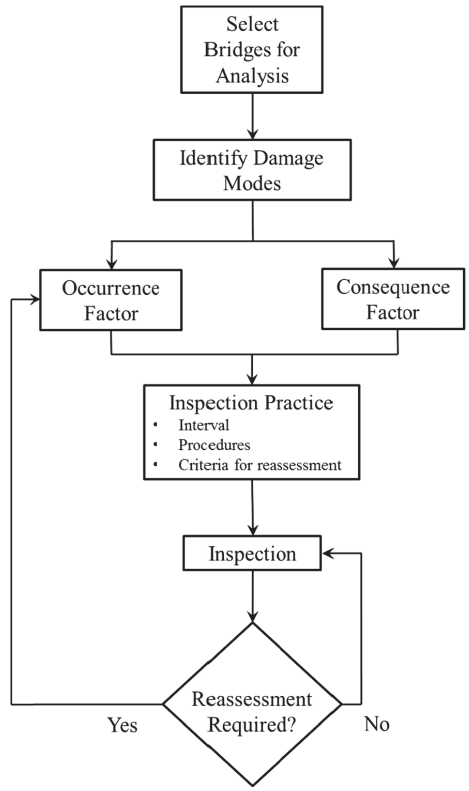

Figure 3. Flow chart showing RBI program activities.

current condition. For those bridges where past experience is greatest, uncertainty regarding both the development of the damage and the associated consequences is reduced. Bridges that are more complex, suffer from advanced forms of deterioration, or have unique design attributes require a higher level of assessment, as shown schematically in Figure 4. More data and a more sophisticated or more specialized assessment may be required. Therefore, to initiate an RBI practice, bridge owners can conduct a general, fully qualitative assessment of their inventory and assign or determine the scope of the initial assessment to be conducted.

Bridges that are of common and simple design, and are in good condition, are identified for analysis first. These bridges can be considered to be in a low risk category because they are of

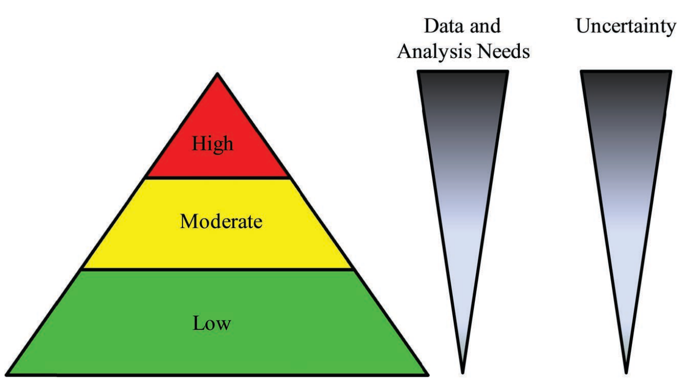

Figure 4. Schematic diagram of qualitative risk assessment for a bridge inventory.

simple design and there is significant experience and confidence in their performance. For example, bridge owners conduct a simple analysis of their inventory to determine bridges that are multi-girder, short span, and in generally good condition for assessment first. Reliability assessment for these bridges may be relatively simple. Conducting the reliability assessment of these bridges first helps develop the RBI practice and develops the knowledge and experience of the RAP members. After this analysis is completed, the assessment moves on to bridges that are more complex, require more data for assessment, or require more sophisticated analysis to determine the factors necessary for a reliability assessment.

4.3 Training Requirements

As noted, the inspection planning process is more involved and complex under an RBI scheme relative to a calendar-based inspection planning process. The approach to inspection planning is more focused on inspection needs for the individual bridge. Further, the assessment of reliability characteristics requires an understanding of the approach and the assessment needs. Therefore, training for both members of the RAP and for inspectors that will implement the results of the RBI planning process will be necessary.

4.3.1 Training for RAP Members

Participants in the RAP process require training to understand the underlying philosophy and processes involved in conducting RBI planning. This training should provide sufficient knowledge in the theory and underlying approach to RBI planning, address methodologies for expert elicitation, and processes for determining the OFs and CFs required for the analysis. A full understanding of the underlying concepts and reliability theories utilized in the process is necessary to conduct effective assessments. Facilitators that may be used to assist in the expert elicitations and overall reliability assessment should be similarly trained.

4.3.2 Training of Inspectors

RBI assessments for inspection planning provide a prioritization of inspection needs for a bridge based on the anticipated or expected damage modes, and the importance of that damage in terms of safety of the bridge. Criteria developed through the RAP process identify key condition attributes used to determine the reliability of individual elements of the bridge. Inspections are necessary that are capable of determining these conditions, and, as such, these inspections are typically more intense than traditional inspections that are intended to report on the general condition of bridge components. Training is therefore required in conducting an element-level inspection to meet the needs of an RBI assessment. Assessments for detecting specific damage modes may be more thorough than under traditional calendar-based practices. For example, training in the detection of fatigue cracks in steel or reliable use of sounding to detect subsurface damage in concrete may be needed. In certain cases, training for inspectors in the application of advanced NDE technologies may be required. Training on the use of NDE technologies is specialized in nature, and certification and training for specific NDE technologies is typically available from commercial sources.

Training for bridge inspectors in the underlying philosophy of the RBI approach is also needed. Appropriate implementation of the inspection prioritization developed through the process, and an understanding of the importance of the quality of bridge inspection outcomes, is needed to implement the process and to transition from traditional inspection approaches.

4.4 Software Development and Integration

The processes for assessing the OFs, such as identifying and scoring key attributes of bridge elements, can be repetitive once established, and therefore lends itself to software implementations. Many of the attributes identified by the RAP may already be stored in existing databases and bridge management systems. Condition attributes and screening criteria for RBI could be implemented through existing software developed for bridge inspection and storing bridge inspection data, or appropriate software may be developed. Therefore, the process of implementing an RBI practice can be simplified by the development of software to more rapidly implement the methodology. Integration with existing software and databases that store relevant information is beneficial.

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APPENDIX A

Guideline for Evaluating the Occurrence Factor

A 1 Introduction

The Occurrence Factor (OF) is used within an RBI to estimate the likelihood of serious damage (i.e., failure) developing in a bridge element during a specified time interval, based on engineering rationale. This rationale is developed through a systematic process that considers and documents the anticipated damage modes for bridge elements. The characteristics, or *attributes*, of bridge elements that contribute to their reliability, considering the expected damage modes, are identified. The damage modes and attributes are identified through an expert panel process described herein, and subsequently used in a rational process that identifies those bridges with elements that are highly reliable and durable, and those bridges with elements that are more likely to suffer from deterioration and damage.

The overall process for estimating the OFs is as follows:

1. Identify the likely damage modes that will affect a bridge element from commonly known damage modes, past experience, and engineering judgment.
2. Identify attributes that contribute to the reliability and the durability of the element considering the damage modes identified.
3. Rank the importance of each attribute's influence on the reliability and the durability of the bridge element.
4. Develop rationale based on the damage modes and attributes of the bridge element to estimate the likelihood of serious damage (i.e., failure) occurring during the specified interval.

An empirical scoring procedure based on the key attributes identified for a given element is used to provide a rational method of estimating the OF. The analysis can be used to construct criteria that can be applied to individual bridges, or groups of very similar bridges, to categorize the likelihood of serious damage (i.e., "failure") occurring in the next 72-month time frame into one of four categories, ranging from "remote" to "high," i.e., the OF.

The OF represents a probability of failure (POF) estimate over a time interval of 72 months. This time period was selected based on engineering factors that included prior research, analysis of data from the National Bridge Inventory (NBI), expert judgment, and data from corrosion and damage models. It was also selected as a time interval for which an engineer could reasonably be expected to estimate future performance within four fairly broad categories, ranging from "remote" to "high," based on key attributes that describe the design, loading, and condition of a bridge or bridge element. In addition, this time interval was selected to provide a suitable balance between shorter intervals, when the POF could be unrealistically low due to the typically slow progression of damage in bridges, or longer intervals, where uncertainty would be increasingly high.

The analysis provides the rationale for categorizing the OF on a rating scale from "remote," when the likelihood is extremely small such that it would be unreasonable to expect failures, to "high," where the likelihood is increased. This rating scale is shown in Table A1. In some cases, the OF may be an estimate of the likelihood of a certain adverse event occurring that results in a failure, such as impact from an over-height vehicle or an overload.

Table A1. Occurrence factor rating scale for RBI.

Level	Category	Description
1	Remote	Remote likelihood of occurrence, unreasonable to expect failure to occur
2	Low	Low likelihood of occurrence
3	Moderate	Moderate likelihood of occurrence
4	High	High likelihood of occurrence

The following sections describe how a Reliability Assessment Panel (RAP) identifies the damage modes to be assessed, determines important attributes for each damage mode, and ranks and scores those attributes to support assessment of an individual bridge or families of bridges of nearly identical attributes, damage modes, and design. The RAP is an expert panel assembled by the bridge owner as described in section 2.4 of the main report.

A 2 Damage Modes

The first step in the process is to answer the question "What can go wrong?" For most common bridges, the damage modes that affect the bridge are well known. Spalling and cracking of the concrete as a result of corrosion, or section loss and fatigue cracking in steel elements, are typical examples. The RAP, through a consensus process, develops a listing of the credible damage modes for the elements of a bridge or a family of bridges being assessed. A *credible* damage mode is one that could reasonably or typically be expected to occur during the service life of the bridge element. Current and past research and experience should be considered in developing the listing. An expert elicitation process described in section A 2.1 may be used to identify the typical damage modes for consideration. This process may also be used to identify unusual or uncommon damage modes that may be relevant for a particular bridge inventory. Table A2 lists damage modes that may be identified by the RAP, as examples to illustrate typical damage modes for several common bridge elements.

A 2.1 Expert Elicitation for Credible Damage Modes

In many cases, the credible damage modes for a given bridge element may be readily identified from past experience and engineering knowledge. In other cases, it may be necessary for the RAP to form a consensus on the credible damage modes for a given element. To identify damage modes that are specific to the type of bridge and elements being considered, the RAP can utilize a process to elicit the expert judgment of the panel based on their experience and knowledge. The process is an expert elicitation of judgments from the panel that consists of the following:

1. Identify the element scenario: The first step in the process is to frame the problem for the panel. This includes describing the element under consideration, including the material and known design parameters. The operational environment for the element should also be described, such as the environment and loading, especially if the operational environment is atypical or unique. For example, if the element under consideration is a concrete beam located in an aggressive coastal environment.

Table A2. Typical damage modes for common bridge elements.

Element	Damage Modes
Steel Girder	Corrosion damage/section loss
	Fatigue cracking
	Fracture
	Impact damage
Prestressed Girder	Corrosion damage (spalling/cracking)
	Strand fracture
	Shear cracking
	Flexural cracking
	Impact damage
Piers and Abutments	Corrosion damage (spalling/cracking)
	Damage to bearing areas
	Unexpected settlement/rotation

2. Identify damage modes: The facilitator poses a question to the RAP such as: "The inspection report indicates that the element is rated in serious condition. In your expert judgment, what is the most likely cause (i.e., damage mode) that has produced/resulted in this condition?" This question is intended to elicit from the panel a listing of damage modes that are likely to occur for the element.

Each expert is asked to independently list the damage modes he/she judges are most likely to have caused the element to be rated in serious condition. The expert records each damage mode he/she identifies, and provides an estimate of the relative likelihood that the damage mode was the cause. This is done by assigning relative probabilities to each damage mode, typically with a minimum precision of 10% (the sum of the ratings should be 100%). The expert notes any supporting rationale for their estimate. The individual results from each member of the RAP are then aggregated to evaluate consensus among the panel on the most likely damage modes for the element. An iterative process may be necessary to develop consensus on the credible damage modes for a given bridge element. However, for most elements, the damage modes are well known and consensus can be reached quickly.

A 2.2 Example of Soliciting Expert Judgment for Damage Modes

This section provides an example of the process for eliciting expert judgment from the RAP for a typical bridge element. In this example, the RAP is provided with the following description for a steel bridge member: *The element under consideration is a painted, rolled steel girder in a simply supported, multi-girder bridge with a typical span length, in a moderate environment. If you were told this girder is rated in serious condition, what would be the most likely cause of this condition?*

Each member of the RAP is then asked to list the damage modes that they identify as the most likely causes (e.g., cracking, section loss) for the member condition, and estimate its relative likelihood of being the cause, relative to other damage modes they identify. The results of this independent exercise are then aggregated as shown in Table A3, showing illustrative results from a six member RAP team assessing the given element scenario.

Following the independent elicitation, the panel discusses the results of the assessments. Any damage mode with an average score of less than 10% may be assessed to determine if that damage mode is credible for the given scenario. Rationale for inclusion or exclusion of the particular damage mode should be recorded. Any damage modes with variance of >20% from the average are also discussed, and RAP members are provided an opportunity to revise their individual ratings based on the discussions.

In this steel girder example, the panel considers the damage mode of corrosion damage/section loss to be most likely to have resulted in severe damage to the steel girder. Less likely damage modes include fatigue, overload, and impact damage. Each credible damage mode identified will be assessed by the RAP to determine its OF.

Table A3. Expert elicitation of damage modes for steel girders.

Damage	Expert 1	Expert 2	Expert 3	Expert 4	Expert 5	Expert 6	Average
Corrosion/ section loss	60%	60%	50%	50%	70%	50%	57%
Fatigue	30%	30%	30%	20%	10%	20%	23%
Overload	10%	10%	10%	20%	20%	20%	15%
Impact	0%	0%	10%	10%	0%	10%	5%
Sum	100%	100%	100%	100%	100%	100%	100%

Table A4. Example damage modes for a steel girder bridge.

Element	Damage Modes
Bare Concrete Bridge Deck	Spalling resulting from steel corrosion
	Widespread cracking
	Rubblization of concrete due to freeze/thaw damage or ASR
Steel Girder	Corrosion damage
	Fatigue cracking
	Fracture
	Impact damage
Piers and Abutments	Spalling resulting from corrosion
	Damage to bearing areas
	Unexpected settlement/rotation

The elicitation process is repeated for each key element of the bridge to develop a listing of damage modes to be considered in the analysis. For example, considering a typical steel girder bridge with a bare concrete deck and concrete piers and abutments, damage modes for each element of the bridge that might be identified by a RAP are shown in Table A4. For the deck in this illustration, the most common damage mode is identified as spalling of the deck due to corrosion damage of the reinforcing steel; widespread cracking, and damage due to alkali-silica reactivity (ASR) and/or freeze-thaw cycles. For the steel girder, corrosion damage (section loss) is identified as the most likely damage mode; fatigue cracking, fracture, and impact are also identified by the RAP. For the piers and abutments, damage modes included corrosion damage that results in spalling, damage to the bearing areas (beams seats, for example), and unexpected settlement or rotation. Such a listing is developed through a consensus process by the RAP for a specific bridge and element types under consideration, as previously discussed.

Once this listing of damage modes has been identified, the next step in the process is to identify key attributes that contribute to the reliability and durability of the element, considering these damage modes.

A 3 Element Attributes

"Attributes" are characteristics of a bridge element that affect is reliability. These attributes are typically well-known parameters affecting the performance of bridge elements during their service lives. For example, bridge elements can have "good attributes" that are known to provide good service-life performance. A bridge deck can have "good" qualities such as having adequate concrete cover and use of epoxy-coated reinforcing steel for corrosion resistance. Alternatively, bridges may have qualities or attributes that contribute to more rapid deterioration or increased likelihood of damage. Using the concrete deck example, heavy use of de-icing chemicals, minimal concrete cover, and unprotected reinforcement would be examples of attributes that contribute to more rapid deterioration. For a steel girder, fatigue-prone details may be an attribute indicating increased likelihood of damage. The identification of key attributes is simply a listing of these attributes and a relative ranking of their importance in terms of the reliability and the durability of the element.

These attributes can be generally grouped into three categories: Design, Loading, and Condition attributes. *Design attributes* are characteristics of a bridge element that are part of the element's design. Design attributes are usually intrinsic characteristics of the element that do not change over time, such as the amount of concrete cover or material of construction [concrete, high performance concrete (HPC), etc.]. In some cases, preservation or maintenance activities that contribute to the durability of the bridge element may be a design attribute, such as the use of penetrating sealers as a preservation strategy.

Loading attributes are characteristics that describe the loads applied to the bridge element that affect its reliability. This may include structural loading, traffic loading, or environmental loading. Environmental loading may be described in macro terms, such as the general environment in which the bridge is located, or on a local basis, such as the rate of de-icing chemical application on a bridge deck. Loading attributes describe key loading characteristics that contribute to the damage modes and deterioration processes under consideration.

Condition attributes are characteristics that relate to the current condition of a bridge or a bridge element. These can include the current element or component level rating, or a specific condition that will affect the reliability of the element. For example, if the damage mode under consideration is concrete damage at the bearing, the condition of the bridge joint may be a key attribute in determining the likelihood that severe corrosion will occur in the bearing area.

Relevant attributes are identified for the damage modes and underlying deterioration mechanisms determined by the RAP. In many cases, attributes are well-known characteristics of bridges and bridge elements that contribute to the reliability and durability of the elements. However, because bridge designs, environments, and management policies differ, attributes and their relative importance may also differ between bridge owners. Therefore, it is necessary that the RAP identify those attributes that contribute most significantly, including any special or unique attributes that might contribute significantly (either positively or negatively) to the likelihood of damage for bridges in their inventory. Attributes that are not relevant or do not have significant impact on durability and reliability should not be included in the analysis.

A 3.1 Screening Attributes

Screening attributes can be used to quickly identify bridges or elements that should not be included in a particular analysis, either because they already have significant damage or they have attributes that are outside the scope of the analysis being developed. Screening attributes are typically attributes that:

- Make the likelihood of serious damage occurring very high.
- Make the likelihood of serious damage occurring unusually uncertain.
- Identify a bridge with different anticipated deterioration patterns than other bridges in a group.

Once the attribute listing has been completed, attributes that match these criteria can be identified. The RAP should identify the appropriate value or condition for the attribute to use as a screening tool. In any scoring scheme there is the possibility, and hence a concern, that the value of key attributes can be diminished when the scoring for all of the relevant attributes are combined. Screening attributes are useful to ensure key conditions are identified, to address this concern.

For example, if considering the likelihood that the steel bridge will suffer corrosion damage that reduces its rating to a 3, and the current rating is 4, the RAP may consider that such condition indicates that there is a high likelihood of further damage developing over the next 72-month period, regardless of other attributes. In such a case, the analysis can move forward to an assessment of the consequences without assessing the specific attributes of the element, since the likelihood has already been assessed to be high.

Design features may be useful as screening criteria, particularly if the features result in the likelihood of serious damage being unusually uncertain. For example, for bridges that possess details susceptible to Constraint-Induced Fracture (CIF), there is a high potential for sudden brittle fracture. For fracture-critical bridges in particular, inspection will provide no protection as the CIF occurs without any warning and before any detectable cracks are observed. Hence, it would be prudent to screen these bridges from the analysis, because the likelihood of serious damage is unusually uncertain. Another strategy, such as retrofitting the critical details, should be performed to ensure safety.

Another example would be to screen steel beam elements in bridges that have open decking. Since the open decking allows drainage directly onto the steel beams, the deterioration of these bridges would not be similar to steel beams with typical concrete decks. Therefore, it would be prudent for these bridges to be screened from the analysis of steel beam bridges, as they may require separate analysis. It may be appropriate to treat these bridges as a separate group, developing the analysis to consider key attributes of those bridges with open decking.

In some cases, it may be more practical to screen bridges from the analysis entirely through a qualitative reliability assessment of the overall inventory, as described in the following section.

A 3.1.1 Qualitative Assessment of Elements and Details

A simple qualitative assessment can also be used early in the RAP process to identify appropriate families or groups of bridges to be analyzed. This tool can be used to separate potentially problematic details or elements that may require more in-depth analysis. These elements may include, for example, rocker bearings in long-span bridges, modular expansion joints, or other details that have the potential to affect the reliability of a bridge uniquely. The qualitative assessment uses a simple three-level scale, as shown in Table A5. This tool can be used to perform an assessment of a bridge inventory and sort bridges that include attributes that are perceived to have low reliability or require special analysis. The assessment is useful for identifying bridges that can be easily assessed from those for which more detailed or individual assessments may be required. For example, assume the RAP is going to assess multi-girder rolled beams, but it considers those beams with rocker bearing to require special analysis and to potentially have low reliability (relative to bridges with other bearing types); these bridges are simply screened from the process using the qualitative assessment, such that the balance of the bridges in that family can be assessed appropriately. A separate analysis that addresses this specific attribute can then be developed, if necessary.

This qualitative screening process would typically be used early in the reliability assessment process to identify an appropriate family or group of bridges and make assessments more efficient.

A 3.2 Identifying Key Attributes

Attributes can be identified generally through a variety of means such as past performance, experience with the given bridge element, previous and contemporary research, analysis of historical performance, etc. While there are potentially many attributes that contribute, in some way, to the durability and reliability of a bridge element, it is necessary to identify those attributes that have the greatest influence on the future performance of an element. Key attributes for a

Table A5. Qualitative reliability scale for screening details.

Relative Reliability
High
Moderate
Low

Table A6. Attributes related to the damage mode of corrosion for a steel girder.

Design Attributes	Loading Attributes	Condition Attributes
Deck Joints/Drainage	Macro-Environment	Existing Condition
Built-Up Members	Micro-Environment	Joint Condition
Deck Type		Maintenance Cycle
Age/Yr of Construction		Condition History/ Trend
		Debris Accum.

given damage mode can be identified through expert elicitation of the RAP. For example, the facilitator could ask the following question pertaining to a particular damage mode, X:

- Consider damage mode X for the subject bridge element. If you were asked to assess the likelihood of serious damage occurring in the next 72 months, what information would you need to know to make that judgment?

The resulting input from the RAP can be categorized appropriately and ranked according to the relative importance of the attribute for predicting future damage for the identified damage mode and element. Rationale for each attribute should be documented. Many of the most common attributes are described in Appendix E, and can be documented by reference. For attributes not included in Appendix E, a brief summary of the rationale for the attribute should be developed and recorded by the RAP.

As an example, Table A6 illustrates typical attributes identified by a RAP for corrosion damage on a steel girder element. Based on an expert elicitation, the primary attributes that contribute to the likelihood of serious corrosion damage developing for a steel girder bridge element include design attributes, loading attributes, and condition attributes, as shown in the table. The rationale for these attributes is relatively simple and straightforward. For example, the presence of deck joints and the quality of the drainage system may indicate whether or not the bridge has deck drainage that is likely to spill de-icing chemicals directly onto the steel girder, thereby resulting in an increased likelihood of corrosion occurring. Built-up members are more likely to suffer crevice corrosion and would therefore be more likely to suffer serious corrosion damage than a rolled or welded section. The attribute of deck type considers if there is open decking that allows de-icing chemicals to drain directly onto the girder, thereby increasing the likelihood of corrosion damage, etc. These attributes are identified by the RAP by applying common engineering knowledge to develop criteria from which a steel bridge element can be assessed to determine if it is likely to suffer serious corrosion damage, or if corrosion damage is unlikely. Elements that have little exposure to de-icing chemicals, are in mild environments, and are currently in good condition may be unlikely to develop serious corrosion in the near future. Conversely, a steel element with active corrosion present, which is in an aggressive environment, and/or is exposed frequently to de-icing chemicals, is more likely to develop serious corrosion damage.

A 3.3 Ranking Attributes

Once the key attributes have been identified, the attributes are ranked on a simple three-level scale according to their importance in assessing the reliability of a bridge element. The ranking is based on the consensus of the RAP. This scale, shown in Table A7, is used to rank a particular attribute's importance as high, moderate, or low. Once ranked, the attributes are assigned a point value corresponding to their importance, to be used in the attribute scoring methodology that

Table A7. Ranking scales for key attributes.

Ranking Descriptor	Total Points
High	20
Moderate	15
Low	10

supports the RAP assessment of the OF. For attributes that are ranked with high importance, a scale of 20 points can be assigned, 15 points for an attribute that has a moderate importance, and 10 points for an attribute that plays a minor role, but is still an important indicator. For example, for the corrosion of a steel beam, a leaking joint which results in drainage of de-icing chemicals directly onto the superstructure is highly important in assessing the likelihood of serious corrosion damage occurring. Therefore, this attribute would be assigned a 20 point scale. Age of the structure contributes to the likelihood of corrosion damage, but to a much lesser extent, relatively, such that it would have 10 points allocated. Maintenance cycle, built-up members, and debris build-up are moderate indicators; these may be assigned a point scale of 15 points.

Once the importance of the attribute is identified, different conditions or situations may be described to distribute points appropriately based on the engineering judgment of the RAP. Again, a simple high-, moderate- and low-ranking model should be used to distribute scores among different conditions or situations that are appropriate for a given attribute. Depending on the number of different conditions or situations, scoring may be distributed over two, three, or four different levels for a given attribute. Using a joint as an illustration, if the joint is leaking or can reasonably be expected to be leaking, it will have the highest effect and might be scored the full 20 points. If the joint is debris-filled or exhibiting moderate leakage, a score of, for example, 15 points may be appropriate; if there is a joint, but it is not leaking, a score of 5 points may be assigned. If the subject bridge is jointless, a score of 0 points may be used. The distribution of scoring for a particular attribute is determined by the RAP. Numerous examples for scoring various attributes are included in the Attribute Index and Commentary located in Appendix E. The RAP should assess if the suggested scores in Appendix E are appropriate, based on the characteristics of the bridges being assessed, and assign appropriate scoring regimes for attributes selected.

A 4 Occurrence Factor Assessment

A 4.1 Estimating the Occurrence Factor

Once the appropriate attributes have been identified and ranked for a given element, the attributes are used to estimate the appropriate OF for a that element. A simple scoring procedure is developed to evaluate the reliability characteristics of a given element based on the attributes and their relative ranking, as described above. The developed scoring procedure provides a data model that is used to assess the OF. Attributes scoring sheets may be used to record the relative scoring of the attributes for a given element, or the data model may be implemented through suitable software. Illustrative examples are included in Appendix F.

A 4.2 Calibration of Scoring Regime

Once the appropriate attributes are selected and ranked, the overall outcome of the scoring procedure (i.e., data model) should be tested to ensure results are adequate for categorizing the subject elements. In some cases, the weighting of particular attributes may need to be increased or decreased to provide suitable results. Since operational environments and design and construction practices vary, rankings for attributes and associated values may need to be adjusted. When a large number of attributes are identified, the relative weight of the most important attributes becomes diminished relative to the overall scoring, and may need to be adjusted to appropriately characterize the anticipated reliability of the element. Screening attributes can also be used for this purpose. Sensitivity studies and Monte Carlo simulation may also be used to assess the relative weights designated for attributes and calibrate the scoring regime developed.

The effectiveness and accuracy of the scoring regime developed can also be evaluated using back-casting, a process for analyzing historical inspection records to verify the effectiveness of the data model (i.e., attributes and scoring) developed. In a back-casting assessment, the attributes and scoring regime are applied to historical inspection records to assess their effectiveness for identifying the likelihood of serious damage occurring.

Regardless of the method(s) used to calibrate the data model, engineering judgment should be used to verify the adequacy of the data model developed.

A 4.3 Occurrence Factor Scale Numerical Estimates

The OF is a qualitative assessment of the likelihood of failure occurring during the next 72 months. Four categories are used to characterize the likelihood considering a particular element and damage mode. Table A8 includes numerical ranges that could be used to describe the OF scale quantitatively. Such numerical values provide ranges or target values for the qualitative rankings that could be used to map quantitative data, if these were available, to the qualitative rating scales. Failure of a bridge element is a relatively rare event, and design and construction details vary widely. As a result, relevant and verifiable frequency-based probability data are scarce. In some cases, modeling may be used to provide an estimate of a particular failure frequency or probability. Probabilistic models or assessments may also be developed for a particular bridge element or elements. The numerical values shown in Table A8 are target values that could be used to map these data or models to the qualitative scales used for analysis. Providing a quantitative estimate of the OFs allows for the data from the probabilistic analysis to be incorporated directly in the reliability-based bridge inspection practice. These numerical categories can also provide a framework for future development of models or data derived from analysis of the deterioration patterns in a particular bridge inventory.

An estimate of a particular damage mode having a “low” likelihood is somewhere between 1/1,000 and 1/10,000. Although the quantitative probability is not necessarily known, engineering judgment supported with an evaluation of the reliability characteristics of the elements is adequate to differentiate between different categories: for example, a likelihood in the “low” category, where the chances are 1/1,000 or less, versus something in the “moderate”

Table A8. Occurrence Factor categories and associated interval estimates.

Level	Category	Description	Likelihood
1	Remote	Remote probability of occurrence, unreasonable to expect failure to occur	≤1/10,000
2	Low	Low likelihood of occurrence	1/1,000-1/10,000
3	Moderate	Moderate likelihood of occurrence	1/100-1/1,000
4	High	High likelihood of occurrence	>1/100

Table A9. Percentage estimates for Occurrence Factor ratings.

Qualitative Description	Expressed as a Percentage
Remote	0.01% or less
Low	0.1% or less
Moderate	1% or less
High	>1%

category, where the likelihood in less than 1/100 but greater than 1/1,000. Estimates from deterioration rate information or from statistical modeling can also be used to support the categorization of the OF.

The quantitative description can be also be used as a vehicle for expert elicitation by using common language equivalents for engineering estimates. For example, if you asked an expert to estimate the probability of serious corrosion damage (widespread spalling, for example) for a particular bridge deck given its current condition, a common engineering response might include a percentage estimate, for example, less than 0.1% chance or less than 1 in a thousand. This estimate can then be mapped to the qualitative scale as being "low." Such estimates are typically very conservative, particularly for lower, less likely events. For engineering estimates of the likelihood of a failure occurring for a given bridge element, the qualitative scale can be interpreted as shown in Table A9.

A 4.4 Use of Deterioration Rate Data

Data on the previous performance of bridge elements can provide some insight into the likelihood of damage occurring for a bridge element. Such data can provide supporting information for decision making regarding the appropriate OFs for a family of similar bridge element types. The user is cautioned that deterioration rate data records only *historical* events that may not reflect the rate or likelihood of *future* events. For example, a state may have never had corrosion damage occur in prestressing tendons; however, this provides little insight into how likely it is that tendon damage will occur in the future. It may be that the population of bridges from which the data is obtained has simply not reached the age where tendon damage would become apparent. Further, deterioration rate data based on condition states or condition ratings may provide little insight into the deterioration mechanisms that caused the condition states or ratings to change. Caution and careful judgment should be used in determining the relevance of the deterioration data to the particular bridge under consideration. Considerations for utilization of deterioration rate data include:

- **Similarity of Operational Environment:** The RAP should consider if the particular bridge under consideration shares the same operational environment as the elements from which data were obtained. Key elements of the operational environment include the average daily traffic (ADT), average daily truck taffic (ADTT), macro-environment of the bridges (severe environment vs. benign environment), micro-environment (salt application, joint and drainage conditions, exposure to overspray), and typical maintenance and management.
- **Similarity of Key Attributes:** Key attributes that affect the damage modes and mechanisms for the bridge element should be similar for the bridge under consideration to those from which deterioration rate data were obtained. This may include materials of construction, design attributes, and condition attributes. Quality of construction and years in service may also be a factor.

Deterioration rate data typically describe the mean or average behavior of the bridge element based on the observed behavior of a population of similar elements. Statistical descriptors of the dispersion of the data, such as the standard deviation, may be provided and then used as indicators of the variability of the data. Applying such data to a specific bridge assumes that the specific bridge has the same design, operational environment, and attributes as those in the larger population from which the statistics were derived. Attributes identified through the RAP process may be used to judge if a particular bridge or family of bridges could be expected to perform above the average or mean, or below.

Statistical data from a bridge management system or other databases can also be used to inform this process if it is available. This data can be useful in determining the damage modes and the overall deterioration behavior of similar bridge elements in the past. However, this data should not be used exclusively because past experience does not necessarily indicate what would occur in the future. Therefore, it is important that the RAP utilize their collective engineering judgment, experience, and rationale for identifying and assessing damage modes that can affect bridge elements.

Lastly, when using such data, one would have to decide which data to use: the mean, or say, two standard deviations below the mean. If the mean is used, there may be a 50-50 chance that the bridge being assessed will deteriorate more quickly than predicted by using the mean deterioration data. However, using some confidence limit, say 2 standard deviations from the mean, may be overly conservative and result in all bridges, good or bad, having unrealistically high estimates of likelihood. Thus, using such data without the ability to also consider or incorporate specific information (condition, design data, details, etc.) from the bridge under consideration must be done with caution, and with a full understanding of the ramifications of such an approach.

A 4.5 Use of Surrogate Data

For many bridges, the use of "surrogates" for the attributes identified in the reliability analysis may be considered to improve the efficiency of the analysis for larger families of bridges. As used herein, "surrogate" refers to specific data that can be used to either infer or determine another piece of information that is required for the reliability assessment. For example, assume a fracture critical bridge was designed and built in the year 2000, which is well after the implementation of the AASHTO/AWS Fracture Control Plan. This information can be used to determine that the steel must at least meet certain minimum toughness requirements, and the bridge meets modern fatigue design requirements. Note that this was determined only from the date of construction and with no detailed review of the design calculations or specifications.

As stated, the use of surrogates is particularly attractive when identifying and assessing a family of bridges. Design and loading attributes identified by the RAP are typically static in nature, that is, they do not change over time. The condition attributes will typically change over time, as damage accumulates and deterioration mechanisms manifest. However, when elements are in generally good condition, specific condition attributes identified by the RAP may not require individual assessment for each bridge or family of bridges; the previous inspection results can simply be used as a surrogate for the individual attributes. This will typically allow for larger groups of bridges of similar design to be grouped into a particular inspection interval, based on the criteria developed by the RAP. For example, again considering steel bridges built to modern design standards, it is known that the design attributes that would increase the likelihood of fatigue cracking and fracture have been mitigated through improvements in the design, fabrication, and construction process. The condition attributes that are required to assess the reliability of the element would include the presence of fatigue cracks due to out-of-plane distortions, fatigue cracking due to primary stresses, and corrosion damage. However, if the component rating is 7, in good condition according the NBIS scale, or CS 1 in an element-level scheme, the existing ratings can be used as a surrogate for the condition attributes. *Note: This assumes the* *inspection result is from an RBI procedure, i.e., the inspection was capable of identifying the necessary condition attributes.* This allows all bridges that are of this same rating (and similar design, loading, and condition attributes) to be treated collectively in a process that is data-driven and does not require much detailed analysis of individual bridges. If the condition rating or condition state changes, then the bridges can be reevaluated, according to the RAP criteria. If the condition does not change between periodic inspections, reassessment may not be necessary.

It is important to note that this process is significantly different than assigning an inspection interval based simply on the current condition of the bridge, for example, deciding to inspect all steel bridges with a rating of 7 on a longer interval than all of those rated a 6. The RAP analysis forms a rationale that identifies not only the current condition attributes that affect the reliability of the element, but also the design and loading attributes of the bridge or bridge element that affect the *potential* for damage to occur. This RAP evaluation forms an engineering rationale for the decision-making process that considers not only the condition of the element, but also the damage modes and the potential for that damage to occur.

For element-level inspection schemes, the attributes identified by the RAP may map directly to an element and element condition state. For example, consider that the RAP identifies leaking joints as an attribute driving the likelihood of section loss in the bearing area of a steel beam. The element condition state (joint leaking) is recorded in the inspection process and can be used as a criterion for that attribute score. In some cases, all of the attributes identified by the RAP as being critical to the likelihood of failure of an element may be included in a comprehensive element-level inspection process, in other cases, they may not.

For NBI-based inspection schemes, attributes identified by the RAP may map to sub-element data collected in addition to the required condition ratings for the primary components of the bridge. These data could be used if it is collected under a standardized scheme for rating and data collection for the sub-elements. For the primary components, the generalized nature of the component rating makes this more difficult for specific attributes.

Mapping of the attributes from the RAP analysis to the elements, sub-elements, or element condition states should not be performed until the RAP analysis has been completed independently. In some cases, the RAP analysis may identify attributes or factors not presently included in the available data, and these data may need to be obtained from other sources. For example, for the case of fatigue cracking in a steel beam, element condition states would indicate fatigue cracking, but not the presence of fatigue sensitive details, i.e., the potential for cracking may be high, even though no cracking is currently present. This is an important consideration in the assessment of appropriate scope and interval of inspection. This data may be readily available in the bridge file, or may need to be ascertained from design plans, records, or other data on the bridge design. In any case, the RAP analysis *shall not* be constrained by the data presently available; the RAP should identify what data is needed *and then* assess if that data is readily available. In some cases, additional data may need to be collected to support the analysis.

A 4.6 Rationale and Criteria Based on RAP Assessments

The RAP assessment for a given bridge or a family of bridges provides an engineering rationale for decision making regarding the appropriate inspection interval and scope. The effects of design, loading, and condition attributes on the potential for failures are considered and documented through the process. For most bridges, the design attributes and loading attributes will not change over time. The RAP assessment should include criteria for modifying the selected inspection interval, and/or for reassessment of a bridge, based on the results of the RBI. These criteria will typically be based on the condition attributes identified during the RAP assessment. If loading conditions change significantly, reassessment may be necessary.

APPENDIX B

Guideline for Evaluating the Consequence Factor

B 1 Introduction

Within an RBI, the Consequence Factor (CF) is used to categorize the outcome or the result of the failure of a bridge element due to a given damage mode. For example, brittle fracture is one of the key damage modes pertaining to steel bridges. Should brittle fracture of a girder occur, the next logical question becomes, "what is the *consequence*?" This would obviously depend on the specific scenario for the fracture. If the member were classified as fracture critical, such an event may be catastrophic, or one that would be considered to be a *severe* consequence. However, if the girder were one member of a multi-girder short-span bridge, the consequence of that fracture would likely to be much less serious, perhaps requiring a lane closure or even temporary closure of the bridge, or a *high* consequence. ("Multi-girder" bridges described herein are bridges with four or more main load bearing members.) In fact, in some cases, such an event may only have moderate consequences.

The CF is used to categorize the consequence of failure of a bridge element into one of four broad categories: Low, Moderate, High, and Severe. Table B1 indicates the general descriptions for each of the CF categories used for the RBI assessment. The general descriptions are indicated in terms of safety and serviceability of the bridge, graduated with qualitative descriptions. Both long- and short-term consequences should/may be considered.

To assess the appropriate category for a particular element and damage mode, typical scenarios or outcomes of a failure must be considered. In some cases, there may be a single scenario that could result from the failure of an element; in other cases, more than one possible scenario needs to be considered. Using the example of brittle fracture of a single beam in a multi-girder, short-span bridge as noted above, it is unlikely that the result from a brittle fracture is a *low* consequence, which has a minor effect on serviceability and no effect on public safety. It is much more likely that such a fracture may have a *moderate* consequence, which has a moderate effect on serviceability and a minor effect on public safety. It is also possible that the fracture will have a *high* consequence, which has a major effect on serviceability and a moderate effect on public safety, and may require urgent repair. There may also be a remote possibility that the fracture causes a catastrophic collapse, or a *severe* consequence. It is necessary to determine which of these consequences is most realistic and establish sufficient rationale based on experience, engineering judgment, and/or theoretical analysis to exclude those consequences that are not credible scenarios.

While the immediate effect on the structure is primarily what is evaluated (e.g., collapse after member failure), it is also appropriate to consider longer term consequences. For example, in the example cited above, if the fracture were to result in a lane closure on a portion of interstate that carries a very high ADTT, the consequence on the traveling public could be high to even severe, though no concerns regarding the *structural* performance of the bridge may actually exist. Rather, the resulting impacts on serviceability could be such that a more frequent inspection interval is justified.

There are many cases in which the critical consequence is obvious. There are also many that require considerable judgment and/or analytical effort to ensure the appropriate CF is selected.

Table B1. General description of the CF categories.

Level	Category	Consequence on Safety	Consequence on Serviceability	Summary Description
1	**Low**	None	Minor	Minor effect on serviceability, no effect on safety
2	**Moderate**	Minor	Moderate	Moderate effect on serviceability, minor effect on safety
3	**High**	Moderate	Major	Major effect on serviceability, moderate effect on safety
4	**Severe**	Major	Major	Structural collapse/loss of life

In these cases, it is important that the rationale used to support the determination is recorded. There are many situations in which analysis and/or experience can be used to justify selecting one scenario over another. However, the level and the type of analysis that is required must be defined, as well as what constitutes sufficient "experience" and when it is appropriate to use experience to justify the categorization of the consequence.

This section describes, through example, situations in which analysis or experience is needed to justify the selection of an appropriate CF. Since not every situation can be included or foreseen, the reader must use the information provided and consider it a road map or framework on how to select the appropriate consequence. The Reliability Assessment Panel (RAP) may use this guidance to develop basic rules or common practices for very common scenarios they anticipate in the analysis. The RAP should consider existing rules, policies, or common practices within its state regarding the considerations for identifying structural redundancy and other factors that may influence the assessment of the consequences. If no rules, policies, or common practices exist, it may be necessary for the RAP to develop its own basic guidelines before performing consequence assessments.

B 1.1 Definitions

This section provides definitions for the terms "analysis" and "experience" as used in the context of this document to support the selection of the most appropriate CF.

Analysis: As used herein, refers to the effort put forth using accepted methods of structural analysis to quantitatively evaluate the outcome of a given event or scenario based on certain initial conditions. Laboratory and field experimental testing are also acceptable methods that can be used to demonstrate, quantitatively, the outcome of a given event or scenario. Analysis requirements may be beyond the scope of most engineering specifications currently used for design and rating, and special assessments may be required in certain conditions. Hence, the owner and the engineer must agree upon the level of analysis, loading, material properties, etc. that will be used for the basis of the analysis. Similarly, any laboratory or field testing must properly simulate or represent in-situ conditions (i.e., scale of the specimen or test, loading, failure mode, etc.) in order to be considered acceptable.

Experience: As used herein, refers to the use of previous knowledge alone to qualitatively evaluate the outcome of a given event based on certain initial conditions. In order to use experience, the user must be able to demonstrate at least the following:

1. The characteristics of the structure being evaluated are identical or sufficiently similar to the structure for which the RAP has previous documented experience.
2. The result of the damage mode is identical for the bridge(s) used as a reference. For example, strand fracture as a result of corrosion or impact may be effectively the same. In both cases, the strand failed.

The information on which the decision is based must be included in the documentation of the RBI assessment. It may consist of the location, structure type, damage type, reason for selection, or other rationale and evidence used to form the decision so that a permanent record is available for future RAPs.

B 2 General Descriptions of Consequence Scenarios

This section provides guidance for the treatment of typical scenarios and situations for each of the four CF categories. A brief description of each CF category is provided, as well as typical examples or scenarios for each category. Methods for selecting the appropriate CF for a given failure scenario are described. This section is intended as guidance for evaluation. Specific situations and scenarios may vary, and the RAP should utilize good engineering judgment supported with analysis or documented experience where necessary. Local rules, policies, and practices of the bridge owner should be considered in the assessments.

As stated, when assessing the CF, the immediate and short-term outcomes, or the results of the failure of an element should be considered. The immediate consequence refers to the structural integrity and safety of the traveling public when the failure occurs. Considerations include whether a bridge will remain standing when the damage mode occurs, and whether the traveling public will remain safe. For example, failure of a load bearing member in a multi-girder redundant bridge is not expected to cause loss of structural integrity, excessive deflections, or collapse. As a result, the traveling public is not immediately affected when the failure occurs. Another scenario would be for a fracture-critical bridge, where the loss of a main member could cause excessive deflection or collapse thereby causing the bridge to be immediately unsafe for the traveling public. The safety of the structure and the public should be considered for determining the immediate consequence.

The short-term consequence refers to serviceability concerns and short-term impacts to the traveling public after a given damage mode occurs. Load posting, repairs, and speed reductions can be considered serviceability concerns. Lane, sidewalk, or shoulder closures as a result of the damage mode impact the traveling public and can cause delays. For example, a multi-girder redundant bridge that experiences the loss of a load bearing member is expected to remain standing; however, once the failure is discovered, a typical response is to close a lane or shoulder until the bridge is repaired. Therefore, the traveling public will be affected. The serviceability of the structure and the impact to the traveling public should be considered when determining the short-term consequence.

For example, the failure of a member in a multi-girder bridge may be a moderate immediate consequence because the bridge is expected to remain standing and no excess deflections are expected to occur. However, if this bridge is located on an interstate located downtown in a major city, the short-term consequence of the member failure may be high or severe because a lane closure may be required, which would cause significant traffic delays. Therefore, the CF for this bridge may be high based upon the short-term consequence.

Tables B2 through B6 provide additional guidance for commonly encountered situations for bridge decks, typical superstructures, and substructures. These tables provide descriptions of typical immediate and short-term effects from common damage modes and sample situations. The tables also include factors the RAP may consider in differentiating CF categories. For example, for the damage mode of spalling in a bridge deck, the CF may be different for a low ADT bridge than for a high ADT bridge, based on serviceability considerations. The CF may be different for a bridge that crosses a roadway than one that crosses a small stream, based on concerns regarding debris falling into traffic, etc. These tables are not intended to be comprehensive, but rather are intended to provide guidance and examples to assist an RAP with developing criteria for determining the CF for typical damage modes for common bridge designs under analysis.

B 2.1 Low Consequence Event

General Description

- Minor effect on serviceability, no effect on safety.

This scenario is the least serious of all the CF categories. The likelihood of structural collapse resulting from the damage mode is not credible and the effect on the serviceability of the bridge is minor.

Table B2. Consequence table for deck elements. Assumed damage mode is spalling.

Consequence for Deck	Description	Sample Situations	Factors to Consider
Low	**Immediate:** Damage to the top of the deck does not present a safety concern for the traveling public. Falling debris from the bottom of deck does not affect the safety of the public. **Short-term:** Minimal serviceability concerns may require maintenance. Little or no impact to traveling public.	• Bridge carrying low- volume and/or low-speed roadway • Bridge with concrete deck over a non-navigable waterway or unused right-of-way land	• ADT/ADTT • Feature under • Feature carried • Stay-in-place forms
Moderate	**Immediate:** Damage to the top of the deck presents a minimal safety concern to the traveling public. Falling debris from the bottom of deck presents a minimal safety concern. **Short-term:** Moderate serviceability concerns. Speed reduction may be needed. Traffic is moderately impacted as a result of lane, shoulder, or sidewalk closure on or under bridge.	• Moderately traveled roadway where damage would cause minimal delays • Bridge with stay-in-place forms over roadway where spalls would not reach roadway or waterway	
High	**Immediate:** Damage to the top of the deck presents a moderate safety concern to the traveling public. Falling debris from the bottom of deck presents a moderate safety concern. **Short-term:** Major serviceability concerns. Repairs or speed reduction may be required. Traffic is greatly impacted as a result of lane, shoulder, or sidewalk closure on or under bridge.	• High-volume roadway where damage would cause reduction in posted speed or potential for loss of vehicular control • Bridge without stay-in-place forms over heavily traveled waterway or high-volume roadway	
Severe	**Immediate:** Damage to the top of the deck presents a major safety concern to the traveling public. Falling debris presents a major safety concern. Possible loss of life. **Short-term:** Potential for significant traffic delays on or under the bridge.	• Bridge over feature where spalling concrete would result in lane closure, loss of life, or major traffic delays	

Requirements for Selection

In order to select the lowest consequence category, the user must be able to clearly demonstrate that the consequence of the damage will be benign. Generally speaking, this decision will most often be based on engineering judgment and experience. Situations in which selection of this consequence scenario may be appropriate are as follows:

- Failure of thin deck overlay.
- Spalling in a concrete deck bridge on a low-volume and/or low-speed roadway.
- Spalling/corrosion damage in an abutment where the bridge is over a non-navigable waterway or unused right-of-way land.

B 2.2 Moderate Consequence Event

General Description

- Moderate effect on serviceability, minor effect on safety.

This scenario can be characterized by consequences that are classified as moderate in terms of their outcome. The likelihood of collapse and loss of life is very remote, and there is a minor effect on the safety of the traveling public.

Requirements for Selection

In order to classify the consequence of a given failure scenario as moderate, the user must demonstrate that the damage mode will typically result in a serviceability issue. The damage mode

Table B3. Consequence table for steel superstructure elements. Assumed damage mode is loss of one primary load carrying member.

<table>
<tr><th>Consequence for Steel Superstructure</th><th>Description</th><th>Sample Situations</th><th>Factors to Consider</th></tr>
<tr><td>Low</td><td>Immediate: Little to no impact on structural capacity is expected based upon structural analysis or documented experience. Public safety is unaffected.

Short-term: Minimal serviceability concerns may require maintenance. Little or no impact to traveling public.</td><td>• Bridge over non-navigable waterway or unused right-of-way land

• Rural bridge with low ADT/ADTT</td><td rowspan="4">• ADT/ADTT
• Feature under
• Feature carried
• Redundancy
• Composite construction
• Load carrying capacity/rating</td></tr>
<tr><td>Moderate</td><td>Immediate: Structural capacity is expected to remain adequate based upon structural analysis or documented experience.

Short-term: Moderate serviceability concerns. Speed reduction or load posting may be needed. Traffic is moderately impacted as a result of lane, shoulder, or sidewalk closure on or under bridge.</td><td>• Bridge over multi-use path, railroad, or lightly traveled waterway

• Bridge on or over moderate-volume urban roadway or high-volume rural roadway that would cause moderate delays for drivers</td></tr>
<tr><td>High</td><td>Immediate: Structural capacity is expected to remain adequate.

Short-term: Major serviceability concerns. Load posting, repairs, or speed reduction may be needed. Traffic is greatly impacted as a result of lane, shoulder, or sidewalk closure on or under bridge.</td><td>• Bridge with alternate load path(s) that has an expectation of adequate remaining structural capacity

• Lane or shoulder closure on or under roadway that would cause major delays for drivers</td></tr>
<tr><td>Severe</td><td>Immediate: Structural collapse. Possible loss of life.
Short-term: Potential for significant traffic delays on or under bridge.</td><td>• Bridge with high ADT/ADTT that requires closure</td></tr>
</table>

poses no serious threat to the structural integrity of the bridge or to the safety of the public. Generally, damage that will require repairs that can be addressed in a programmed fashion (i.e., non-emergency) would be classified as having a moderate consequence. Member or structural redundancy should be a consideration, and, in cases where the member is non-redundant, it may be prudent to classify an event higher in consequence. Situations in which the selection of this CF may be appropriate are as follows:

- Spalling damage in a deck soffit or concrete girder for a bridge over multi-use path, railroad, or low-volume (<10 ADT) roadway.
- Spalling in a concrete deck bridge on a moderate-volume roadway.
- Lane or shoulder closure on a bridge carrying a moderate-volume urban roadway or a high-volume rural roadway that would cause moderate delays for drivers.
- Fatigue cracks that require repair but are not the result of primary member stresses, such as out-of-plane distortion cracks in redundant members

The examples above illustrate some of the element failure scenarios that would typically be categorized as having moderate consequence. In some cases, failure scenarios that could be considered more serious can be categorized as having moderate consequences, if analysis or past experience can be used to better define the outcome of a given scenario. For example, out-of-plane fatigue cracks are not uncommon in some older steel bridges, and are included in

Table B4. Consequence table for reinforced concrete superstructure elements. Assumed damage mode is loss of one primary load carrying member.

<table>
<tr><th>Consequence for Concrete Superstructure</th><th>Description</th><th>Sample Situations</th><th>Factors to Consider</th></tr>
<tr><td>Low</td><td>Immediate: Little to no impact on structural capacity is expected based upon structural analysis or documented experience. Falling debris does not affect the safety of the public.

Short-term: Minimal serviceability concerns may require maintenance. Little or no impact to traveling public.</td><td>• Bridge over non-navigable waterway or unused right-of-way land
• Rural bridge with low ADT/ADTT</td><td rowspan="4">• ADT/ADTT
• Feature under
• Feature carried
• Redundancy
• Composite construction
• Load carrying capacity/rating</td></tr>
<tr><td>Moderate</td><td>Immediate: Structural capacity is expected to remain adequate based upon structural analysis or documented experience. Falling debris presents a minimal safety concern to the public.

Short-term: Moderate serviceability concerns. Speed reduction or load posting may be needed. Traffic is moderately impacted as a result of lane, shoulder, or sidewalk closure on or under bridge.</td><td>• Bridge over multi-use path, railroad, or lightly traveled waterway
• Bridge on or over moderate-volume urban roadway or high-volume rural roadway that would cause moderate delays for drivers</td></tr>
<tr><td>High</td><td>Immediate: Structural capacity is expected to remain adequate. Falling debris presents a moderate safety concern to the public.

Short-term: Major serviceability concerns. Load posting, repairs, or speed reduction may be needed. Traffic is greatly impacted as a result of lane, shoulder, or sidewalk closure on or under bridge.</td><td>• Bridge with alternate load path(s) that has an expectation of adequate remaining structural capacity
• Lane or shoulder closure on or under roadway that would cause major delays for drivers</td></tr>
<tr><td>Severe</td><td>Immediate: Structural collapse. Falling debris presents a major safety concern to the public. Possible loss of life.

Short-term: Potential for significant traffic delays on or under bridge.</td><td>• Bridge over feature where spalling concrete would result in lane closure, loss of life, or significant traffic delays</td></tr>
</table>

the examples above. However, other types of fatigue cracks may be more serious. For example, consider cracking in a single plate of a built-up riveted girder. These types of cracks would normally be expected to be much more serious. They may require categorization as having high or a severe consequence, if it is assumed that the crack propagates such that the load carrying capacity of the girder is lost. However, in many cases, riveted built-up members are composed of two or three cover plates, two angles, and the girder web. If it could be shown by analysis that even after complete cracking of one of these individual components (e.g., complete cracking of one of the cover plates) the member still has plenty of reserve capacity, then it might be reasonable to classify the event as a moderate consequence scenario. The individual making this assessment would also want to consider overall system redundancy and other factors.

Hence, if analysis can be used to show that a condition that is generally perceived to be more serious, but is actually not so, then it may be justified to classify the event as having a moderate consequence. Experience may also be utilized to assess if a given failure scenario is a high consequence event or a moderate consequence event. In cases where a given owner may have had the same or very similar experience with several other identical or sufficiently similar bridges, the owner may

Table B5. Consequence table for prestressed concrete superstructure elements. Assumed damage mode is loss of one primary load carrying member.

<table>
<tr><th>Consequence for Prestressed Superstructure</th><th>Description</th><th>Sample Situations</th><th>Factors to Consider</th></tr>
<tr><td>Low</td><td>Immediate: Little to no impact on structural capacity is expected based upon structural analysis or documented experience. Falling debris does not affect the safety of the public.

Short-term: Minimal serviceability concerns may require maintenance. Little or no impact to traveling public.</td><td>• Bridge over non-navigable waterway or unused right-of-way land
• Rural bridge with low ADT/ADTT</td><td rowspan="4">• ADT/ADTT
• Feature under
• Feature carried
• Redundancy
• Composite construction
• Load carrying capacity/rating</td></tr>
<tr><td>Moderate</td><td>Immediate: Structural capacity is expected to remain adequate based upon structural analysis or documented experience. Falling debris presents a minimal safety concern to the public.

Short-term: Moderate serviceability concerns. Speed reduction or load posting may be needed. Traffic is moderately impacted as a result of lane, shoulder, or sidewalk closure on or under bridge.</td><td>• Bridge over multi-use path, railroad, or lightly traveled waterway
• Bridge on or over moderate-volume urban roadway or high-volume rural roadway that would cause moderate delays for drivers</td></tr>
<tr><td>High</td><td>Immediate: Structural capacity is expected to remain adequate. Falling debris presents a moderate safety concern to the public.

Short-term: Major serviceability concerns. Load posting, repairs, or speed reduction may be needed. Traffic is greatly impacted as a result of lane, shoulder, or sidewalk closure on or under bridge.</td><td>• Bridge with alternate load path(s) that has an expectation of adequate remaining structural capacity
• Lane or shoulder closure on or under roadway that would cause major delays for drivers</td></tr>
<tr><td>Severe</td><td>Immediate: Structural collapse. Falling debris presents a major safety concern to the public. Possible loss of life.

Short-term: Potential for significant traffic delays on or under bridge.</td><td>• Bridge over feature where spalling concrete may result in lane closure, loss of life, or significant traffic delays</td></tr>
</table>

be able demonstrate that a lower CF is justifiable. Very high load ratings (e.g., 150% of the minimum required) and redundancy could also be factors to consider when selecting this CF category. Of course, if experience and judgment are used to determine CF, then sufficient documentation would need to be available to justify the selection of a given CF.

B 2.3 High Consequence Event

General Description

- Major effect on serviceability, moderate effect on safety.

This scenario can be characterized by consequences that are more serious in terms of their outcome. The likelihood of collapse and loss of life may be more measurable, but still relatively remote.

Requirements for Selection

The user must be able to demonstrate that the possibility of collapse and loss of life are still relatively remote when identifying a given failure scenario as having a high consequence. Though the bridge may require repairs, the outcome would not be catastrophic in nature.

Table B6. Consequence table for substructure elements. Assumed damage mode is spalling.

Consequence for Substructure	Description	Sample Situations	Factors to Consider
Low	**Immediate:** Falling debris does not affect the safety of the public. Structural capacity of the bridge remains adequate. **Short-term:** Minimal serviceability concerns may require maintenance. Little or no impact to traveling public.	• Bridge over non-navigable waterway or unused right-of-way land	• ADT/ADTT • Feature under • Load carrying capacity
Moderate	**Immediate:** Falling debris from substructure presents a minimal safety concern to the public. Structural capacity is expected to remain adequate based upon structural analysis or documented experience. **Short-term:** Moderate serviceability concerns. Speed reduction or load posting may be needed. Traffic is moderately impacted as a result of lane, shoulder, or sidewalk closure on or under bridge.	• Bridge over multi-use path, railroad, or lightly traveled waterway	
High	**Immediate:** Falling debris from substructure presents a moderate safety concern to the public. Structural capacity is expected to remain adequate. **Short-term:** Major serviceability concerns. Load posting, repairs, or speed reduction may be needed. Traffic is greatly impacted as a result of lane, shoulder, or sidewalk closure on or under bridge.	• Lane or shoulder closure on roadway that would cause major delays for drivers	
Severe	**Immediate:** Structural collapse, bearing area failure, or loss of load carrying capacity. Falling debris presents a major safety concern to the public. Possible loss of life. **Short-term:** Potential for significant traffic delays on or under bridge.	• Bridge adjacent to high-volume roadway with insufficient horizontal clearance where spalling concrete may result in lane closure, loss of life, or major traffic delays • Bearing area failure resulting in deck misalignment	

Examples of high consequence events would include scenarios that require short-term closures for repairs, lane restrictions that have a major impact on traffic, load postings, or other actions that majorly affect the public. Situations where the selection of this CF may be appropriate are as follows:

- Failure of a main member in a multi-girder bridge with sufficient load path redundancy.
- Spalling damage in a deck soffit or concrete girder for a bridge over a navigable waterway or a moderate-/high-volume roadway.
- Spalling in a concrete deck bridge on a high-volume roadway.
- Lane or shoulder closure on or under a roadway that would cause major delays for drivers.
- Impact damage on a multi-girder bridge.

Again, using brittle fracture of a girder as an example, consider the response to the fracture of an exterior girder in a multi-girder bridge. If the girders are spaced relatively closely, a reasonable strategy would be to place barriers on the bridge to keep traffic off the shoulder and hence, off the faulted girder. Though one girder out of several was compromised, experience indicates the remaining girders have sufficient capacity to carry traffic safely.

In the above example, it is important to note the reaction to the fracture was not based on calculations, but was based entirely upon experience. If the owner performed calculations that quantifiably showed that the bridge had sufficient reserve capacity in the faulted condition, i.e., with one girder fractured, it might be reasonable to identify the event as having a moderate consequence.

Guidance on such analysis exists in the literature and it can be performed for common bridges and common bridge types. However, simplified analytical procedures may also suffice. For example, there is considerable discussion regarding redundancy of multi-girder systems, both concrete and steel, as reported in *NCHRP Report 406: Redundancy in Highway Bridge Superstructures* (*1*). This document provides direction on determining the capacity and the redundancy as a function of span, girder spacing, and the number of loaded lanes using system factors. The research resulted in the development of system factors that quantify redundancy based on an assessment of the reliability of the bridge systems, rather than simply the individual bridge members. Using the recommended system factors may greatly reduce the analytical effort needed in assessing a bridge. The major conclusion from this research was that bridges designed to AASHTO bridge specifications generally possess sufficient reserve capacity. In addition, NCHRP Project 12-87, "Fracture-Critical System Analysis for Steel Bridges" was underway at the time this report was prepared and once complete may be of use in performing system analysis.

If experience is used as the reason to justify a reduction from a high consequence to a moderate consequence, the experience referenced would have to be for a type of structure *and* a damage mode outcome that is nearly identical to the one under consideration, as described in section B 1.1. (For example, corrosion, fatigue, or fracture can all lead to a failed girder. Hence, although the damage modes are different, the outcome is the same.) Therefore, the RAP would have to adequately document and demonstrate that the cited case(s) are of sufficient similarity. Owners may cite examples both in their own state and from other states. Another desirable characteristic would be whether or not the experience with a given response has been observed more than once. For example, an owner may have experience with a certain type of rolled steel beam bridge and truck impact. Experience with truck impacts on several similar steel bridges may demonstrate that for the bridge under consideration, impact to the superstructure would not result in a set of circumstances that justify identifying the event as having a high consequence. Based on this experience, it may be appropriate to identify the event as having only moderate consequences.

Another example would be a case in which there is severe spalling at the bearing of a member in a prestressed, multi-girder bridge that is over a small creek or a flood plain. Hence, there is no concern regarding spalled concrete hitting someone or something below the bridge (minor effect on public safety). If calculations could be made to show that if the bearing were to completely fail, there would only be moderate effects on serviceability, then it would be reasonable to state this is a moderate consequence event. In the absence of detailed calculations and/or substantial experience regarding the specific scenario, it would be required to be identified as having a high consequence, based on the criteria discussed.

B 2.4 Severe Consequence Event

General Description

- Major effect on serviceability and safety.

This is the most critical CF category and can be characterized by events that, should they occur, are anticipated to result in catastrophic outcomes. Structural collapse and loss of life are likely should the failure occur.

Requirements for Selection

Due to the catastrophic nature implied by this consequence scenario, it should not be selected arbitrarily as a catch-all or just "to be conservative." The user must have reasonable justification that shows that the failure scenario being considered is likely to be consistent with a severe consequence event.

Examples of severe consequence events would include failure of the pin or hanger in a bridge with a suspended truss span or a two-girder system, or strand fractures in a pre- or post-tensioned element that results in a non-composite member falling into a roadway below, such as what was observed in Washington Township, PA (*2*). Failure of a pier due to severe corrosion of the reinforcement or to a lack of reinforcement would also be an example of a severe consequence event. Situations in which the selection of this CF may be appropriate are as follows:

- Fracture in a non-redundant steel bridge member.
- Failure of a non-composite girder over traffic.
- Spalling of a concrete soffit, concrete girder, or concrete abutment over a high-volume roadway or pedestrian walkway.
- Lane or shoulder closure on a major roadway that would cause significant delays for the traveling public.
- Bearing area failure resulting in deck misalignment.

Cases for which there is insufficient experience or where reliable calculations cannot be made (*due to lack of analytical models or data for use in the models*) may also be categorized as severe. Examples would be unique, one-of-a-kind bridges or other structural systems for which the result of failure associated with a given damage mode is essentially unknown. In such cases, the only reasonable approach is to conservatively assume and select the worst-case consequence (i.e., a severe consequence), as the actual outcome cannot be well defined.

A common example of a failure that would result in a severe consequence is primary member failure in a fracture-critical bridge. Due to the perceived lack of redundancy, fracture of a main member is assumed to result in a total collapse of a bridge or a portion of a bridge. Though this is a reasonable conclusion in the absence of more rigorous analysis, the bridges can also be good examples of where more rigorous analysis can be used to show redundancy actually exists. For example, a literature review conducted as part of *NCHRP Synthesis 354: Inspection and Management of Bridges with Fracture-Critical Details*, revealed that there were no documented cases of catastrophic failure for any two-girder bridges or cross girders where fractures had occurred (*3*). In some of the failures, an entire girder fractured, but due to inherent redundancy of the unaccounted-for load paths, such as the deck and lateral system, and overall system behavior, the bridges did not collapse. In fact, in some cases, there is little perceived deflection in the faulted state.

In light of the above, owners may wish to perform an after-fracture redundancy analysis to demonstrate that a given bridge possesses sufficient alternate load paths such that the most likely outcome would have only high consequences. Obviously, the owner must select the appropriate live load that must be carried in the faulted state for the analysis. Further, consideration should be given to the fact that the bridge may need to remain in service for some time with the fracture undetected. For example, if the fracture occurred immediately after a scheduled inspection and there was little or no evidence that would alert anyone to the condition and to take action (e.g., no deflection).

Obviously, there are other damage modes that may result in a severe consequence. For those, analysis may also be used to demonstrate that the most likely outcome would have only a high consequence. Downgrading to the less serious consequence scenario is permitted but *only* through the use of analysis. Experience alone may not be used to justify downgrading from a severe consequence to a high consequence, due to the catastrophic outcomes associated with the more severe scenario. While experience may be used in conjunction with analytical studies to make a stronger case for downgrading to a lower consequence scenario, experience alone is not deemed to be sufficient.

B 3 Use of Expert Elicitation for Determining the Consequence Scenario

An expert elicitation of the RAP can be a useful tool for evaluating the appropriate CF for situations that are not well matched to the examples given above, or to establish basic ground rules for the assessment of common situations. The expert elicitation process can be used to establish or to build consensus among the RAP and to assist in identifying the most likely outcomes of damage modes assessed during the reliability analysis. The process should be carefully controlled and systematic to ensure that the judgments of the RAP are effectively ascertained. The process involves a few basic, but critical, steps as follows:

1. *Statement of the Problem*: The RAP should be presented with a clear statement of the problem and supporting information to allow for expert judgment to be made. Care should be taken to ensure the problem statement does not contain information that could lead to a biased decision. The problem statement typically includes data regarding the bridge design, location, typical traffic patterns, and the failure scenario under consideration.
2. *Expert Elicitation*: Independently, each member of the RAP is asked, based on his or her judgment, experience, available data, and given the scenario presented, to determine what the most realistic consequence is resulting from the damage mode under consideration. The expert is asked to express this as a percentage, with the smallest unit of estimate typically being 10%. The expert provides a written statement on what factors they considered in making the estimate.
3. *Comparison of Results*: Once each member of the RAP has rated the situation, the results of the elicitation are aggregated. Generally, there will be consensus regarding the most critical consequence. However, in some cases, the most critical choice will not be clear and there will not be consensus.
4. *Identify Consequence Factor*: If there is consensus among the panel regarding the appropriate CF, then the rationale for making the determination is recorded. This rationale should be consistent with the general guidance herein, or document deviations, changes, and associated rationale.

For cases in which consensus is not reached in the initial elicitation, the experts should discuss their rankings, their assumptions, and rational for their specific judgments. The members of the RAP should then be given the opportunity to discuss the various judgments and to revise their scores based on the discussion. In some cases, additional information may be needed to support developing a consensus regarding the appropriate CF. For example, analysis may need to be conducted or previous experience documented. If consensus cannot be reached, a potential approach would be to adopt the most conservative consequence scenario that was included among the revised scores. Exceptions to the selected CF should also be documented.

When consensus cannot be reached, the RAP may determine that additional analysis is required to determine the appropriate consequence for a given failure scenario. In some cases, additional data collection may be required in order to reach a consensus. Regardless of the approach, the individual RAP should have the flexibility to develop its own methodologies to handle cases for which there is no consensus. However, at the conclusion of the analysis, the method still must result in the selection of the most appropriate consequence scenario, based on the guidelines provided herein and on good engineering judgment.

B 4 References

1. Ghosn, M., Moses, F., *NCHRP Report 406: Redundancy in Highway Bridge Superstructures*. 1998, TRB, National Research Council: Washington, D.C.
2. Clay, N., et al., Forensic Examination of a Noncomposite Adjacent Precast Prestressed Concrete Box Beam Bridge. *Journal of Bridge Engineering*. 15(4): p. 408–418.
3. Connor, R. J., R. Dexter, and H. Mahmoud, *NCHRP Synthesis 354: Inspection and Management of Bridges with Fracture-Critical Details*. 2005, Transportation Research Board of the National Academies: Washington, D.C.

APPENDIX C

Guideline for Determining the Inspection Interval

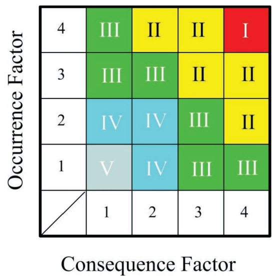

Figure C1. Risk matrix for a typical highway bridge.

C 1 Inspection Intervals

Inspection intervals are determined based on the reliability analysis using a simple four by four matrix as shown in Figure C1, which illustrates a risk matrix for a typical highway bridge. Engineering judgment is required for establishing the specific divisions applied to the risk matrix; the divisions are generally applied to ensure that the likelihood of damage remains *low* during the interval between inspections, such that there are multiple inspections conducted before there is a high likelihood of failure occurring. When consequences are relatively high, should the failure occur, the interval is further reduced to provide an extra margin of safety.

For the risk matrix shown in Figure C1, divisions have been made to separate the bridges requiring more frequent inspections (Category I) from those requiring less frequent inspections (e.g., Categories III, IV, and V). The inspection interval categories are shown in Table C1. Bridges with elements falling in Category II require the typical inspection interval of 24 months, currently used under the NBIS.

The inspection intervals and the divisions on the risk matrix are engineering-based to ensure a high margin of safety and that multiple periodic inspections take place *before* the likelihood of failure becomes high. In other words, the intervals are determined such that the likelihood of failure remains *low*, and the intervals are further reduced as consequences increase to provide additional levels of safety. For example, recall that the RAP assessment of the likelihood of a damage mode resulting in a "failure" (as defined in Section 2.1) is based on a 72-month timeframe. For a given element, if there is low likelihood of a failure (OF = 2), and the consequence of that failure is moderate (CF = 2), the inspection interval of 72 months (Class IV) is identified on the matrix. This is justified because the analysis has indicated that there is a low likelihood of failure, and even if the failure occurs, there will be only a moderate effect on the serviceability of the bridge. However, if the consequence of the failure were high, then the inspection interval is reduced to 48 months (Class III) and 24 months (Class II) if the consequence is severe. Alternatively, if the likelihood of failure is moderate (OF = 3) over 72 months, the maximum inspection interval is *less than* 72 months, regardless of the consequence; 48 months if the consequence were only low (benign) (CF = 1) or moderate (CF = 2) and 24 months if the consequence were high (CF = 3). Similarly, if the likelihood of failure were remote over the 72-month timeframe, it may be justified to have a maximum interval of more the 72 months, particularly if the consequences are assessed to be benign (CF = 1). As the consequences increase, this interval is reduced.

C 1.1 Important or Essential Bridges

As noted, the divisions on the risk matrix require engineering judgment to determine which inspection intervals are acceptable and necessary. For certain bridges, for example, essential bridges along key transportation routes, an owner may wish to provide an additional margin of reliability. Under these circumstances, the divisions on the risk matrix may be adjusted down

Table C1. Maximum inspection interval categories.

Category	Maximum Interval
I	12 months or less
II	24 months
III	48 months
IV	72 months
V	96 months

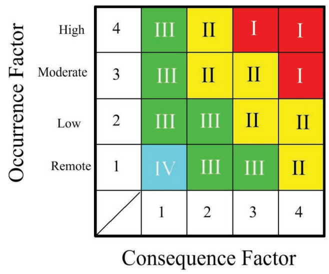

Figure C2. Risk matrix that may be applied to "essential" bridges.

and toward the lower left corner of the matrix. For example, Figure C2 illustrates a risk matrix an owner could apply to bridges for which an additional measure of reliability is desired. This may be due to the importance of the bridge to the effectiveness of the transportation system overall, and/or because the bridge serves essential purposes. Criteria for identifying these essential or important bridges should be developed by the bridge owner, but would typically consider such factors as ADT, functional classification of the route, and importance to local transportation functions. Owners may already have criteria for identifying essential or important bridges for which added measures of reliability are desired.

APPENDIX D

Inspection Technologies

D 1 Introduction

This appendix provides general guidance for the inspection methods to be utilized in a risk-based inspection (RBI) practice. The section includes a description of nondestructive evaluation (NDE) technology's technical readiness and relative costs to assist decision makers in determining appropriate and practical technologies for the detection and evaluation of typical damage modes and deterioration mechanisms in highway bridges. This section also includes tables that indicate the relative reliability of different inspection methods and NDE technologies to assist decision makers regarding the application and effectiveness of the technologies.

D 1.1 NDE Method Technical Readiness Levels and Costs

This section provide general guidance on the technical readiness levels (TRLs) and costs of the most common NDE technologies that may be applied for damage detection in highway bridges. Technologies have been evaluated on relative scales using expert judgment and experience. Table D1 indicates the scale used to assess the TRL of the technologies. This scale is intended to assist engineers in understanding the practicality and the availability of NDE technologies, and to discriminate between those techniques that are readily available and well proven, from those that may be more experimental in nature. The scales provide a five-level discrimination that indicates if an NDE technology is experimental in nature, or if it is a widely available and widely implemented technology.

NDE technologies are rated according to the cost scales shown in Table D2. These scales are intended to provide engineers with general information regarding the relative costs of implementing NDE technologies for bridge inspection. Relative costs are based on a typical, multigirder highway bridge approximately 150 ft in length.

The TRL and costs for NDE technologies are shown in Table D3.

Table D1. TRLs for NDE technologies.

Technical Readiness Level		
Description	TRL No.	**Examples**
Fundamental Research: basic research in the laboratory	1	Fundamental sensor research, nano-sensors, laser-induced breakdown spectroscopy (LIBS)
In Development: laboratory equipment, starting field testing and experimental applications, proof of concept testing	2	In-situ corrosion sensors, positron annihilation, backscatter x-ray, thermal crack detection
Application Development: Applications for the technology are being developed, commercially available research equipment, field testing is experimental/developmental, initial assessments of effectiveness in the field, reliability unknown	3	Electromagnetic-acoustic transducer (EMAT) sensors, ultrasonic stress measurement, magnetic flux leakage for embedded strands thermal crack detection
Controlled Implementation: Commercially available equipment and service, application by specialist/consultant, (certification may be available), assessments of reliability/effectiveness are ongoing	4	Ground penetrating radar (GPR), radiography, impulse response, phased array ultrasonics, infared thermography (IR)
Widespread Implementation: Certification available, widely used, commercially available equipment, commonly available, application by suitably trained technician, generally accepted reliability/effectiveness	5	Ultrasonic pulse velocity (UPV), dye penetrant, eddy current, magnetic particle, covermeters, half cell

Table D2. Relative cost scales for NDE technologies.

Cost Scales		
Description	Symbol	**Examples**
Low cost, state forces, or $100s of dollars to apply/bridge	○	Dye penetrant, magnetic particle, impact echo, ultrasonic thickness, thermography
Moderate Cost, $1,000–$10,000 typical costs/bridge	◉	GPR, ultrasonic crack detection, impact echo
High cost, >$10,000 to apply	●	Health monitoring, x-ray diffraction, radiography

Table D3. TRL and cost for typical NDE technologies.

Code	Name	TRL	Cost	Material	Primary Usage
MP	Magnetic particle testing	5	○	Steel	Surface-breaking cracks in steel
PT	Dye penetrant testing	5	○	Steel	Surface-breaking cracks in steel
UT	Ultrasonic testing	5	◉	Steel	Surface and subsurface cracks in steel, volumetric defects
UT-T	Ultrasonic thickness gage	5	○	Steel	Plate thickness, section loss
ET	Eddy current testing	5	○	Steel	Surface-breaking cracks in steel
AE	Acoustic emission	4	●	Steel	Monitoring growth of fatigue cracks
IR	Infrared thermography	4	○	Concrete	Subsurface delaminations in concrete
GPR	Ground penetrating radar	4	◉	Concrete	Detecting damage in concrete associated with corrosion, rebar depth, locating embedded metal objects
UPV	Ultrasonic pulse velocity	5	○	Concrete	Deterioration of concrete, concrete moduli/strength, subsurface voids, cracks
IE	Impact echo	4/5	◉	Concrete	Delaminations in concrete, deterioration of concrete, subsurface voids
CD	Chain drag	5	○	Concrete	Delaminations in concrete
HC	Half-cell potential	5	◉	Concrete	Corrosion potential
RT	Radiographic testing	4	●	Concrete	Internal voids, loss of section/fracture in embedded steel
S	Sounding	5	○	Concrete	Delaminations, deterioration of concrete
SAW	Surface acoustic wave	4	◉	Concrete	Cracking and deterioration in concrete, delaminations
MFL	Magnetic flux leakage	3	◉	Concrete	Loss of section for embedded steel element (prestressing strand, rebar)

D 2 Inspection Methods and Technologies

The tables included in this section (Tables D4 through D9) qualitatively describe the reliability and effectiveness of NDE technologies and inspection methods including routine inspection and hands-on inspections. In making the assessments of reliability and effectiveness, it was assumed that a routine inspection was conducted without hands-on access to the bridge element. The reliability assessment indicated in Tables D5 through D9 is intended to provide general guidance on effective inspection methods for detecting and evaluating certain damage modes and deterioration mechanisms. Key monitoring or sampling methods that provide tools for assessing the likelihood of corrosion damage developing in concrete have been included. The key to the symbolic guide is shown in Table D4. Methods that are low reliability typical do not provide effective detection or assessment, and are not recommended for the damage mode or deterioration mechanism indicated.

Table D4. Symbolic guide to inspection method reliability and effectiveness.

Key	
Low	○
Moderate - low	◒
Moderate - high	◓
High	●

Table D5. Inspection methods for bare concrete decks.

Damage Mode or Mechanism	Routine Visual	Hands-On Visual	Sounding[1]	IR	GPR	Impact Echo	Chain Drag	Half Cell	Chloride Ion Content
Spalling/patches	◒	●	◓	◓	◓	◓	◒	NA	NA
Delamination (dry)	○	○	◓	◓	◒	◓	◓	NA	NA
Deck cracking (distributed)	◒	●	○	○	◒	○	○	NA	NA
Corrosion damage	○	○	◓	◒	◓	◓	◒	NA	NA
Freeze-thaw/ pulverized/ cracks	◒	●	●	◓	◓	◓	◓	NA	NA
Delamination in soffit[2]	○	○	◓	◓	◒	◓	NA	NA	NA
ASR	◒	●	◒	◒	◒	◒	◒	NA	NA
Active corrosion/ corrosion potential	○	◒	○	○	◒	○	○	◓	●

[1] Based on FHWA visual inspection study results.

[2] NDE technologies applied to the soffit surface.

Table D6. Inspection methods for concrete decks with overlays.

Damage Mode or Mechanism	Routine Visual	Hands-On Visual	Sounding	IR	GPR	Impact Echo	Chain Drag	Half Cell	Chloride Ion Content
Spalling/patches	○	○	◒	◒	◓	◓	○	NA	NA
Delamination	○	○	◒	◒	◒	◒	○	NA	NA
Debonding/ overlay delamination	○	○	●	◓	◒	◓	◓	NA	NA
Corrosion damage	○	○	◓	○	◓	◓	○	NA	NA
Freeze-thaw/ pulverized/ cracks	○	○	◒	◒	◓	◒	○	NA	NA
Delamination in soffit[1]	○	○	◓	◓	◒	◓	NA	NA	NA
ASR	○	○	◒	○	◒	◒	○	NA	NA
Active corrosion/ corrosion potential	○	◒	○	○	◒	○	○	◓	●

[1] NDE technologies applied to the soffit surface.

Table D7. Concrete deck overlays.

Damage Mode or Mechanism	Routine Visual	Hands-On Visual	Sounding	IR	GPR	Impact Echo	Chain Drag	SAW
Spalling/patches	◒	●	◓	◓	◓	◓	◓	◓
Delamination/ debonding	○	○	●	◓	◒	◓	◓	◓
Overlay cracking	○	○	◓	○	○	◒	○	●

Table D8. Inspection methods for steel members.

Damage Mode or Mechanism	Routine Visual	Hands-on Visual[1]	PT[2]	MT[2]	UT[2]	UT-T	ET[2]
Fatigue cracks – primary stress	○	◓	●	●	●	NA	●
Out of plane distortion cracking	○	◓	●	●	●	○	●
Section loss	◒	◓	NA	NA	●	●	NA
Coatings failure	◒	●	NA	NA	NA	○	NA
Steel pins – pack rust	◒	●	NA	NA	◒	◒	NA
Cracks in steel pins	○	○	NA	NA	●	○	NA

[1] Assumes inspectors have been adequately trained.

[2] Assumes Level II certification; Level III procedure development.

Table D9. Inspection methods for open prestressed girders.

Damage Mode or Mechanism	Routine Visual	Hands-on Visual	Sounding	IR	GPR	IE	MFL	RT	UPV	Chloride Ion Content
Spalling/patches	◒	●	◓	◓	◓	◓	○	○	◒	NA
Delamination	○	○	◓	◓	◒	◓	○	○	◓	NA
Strand corrosion	○	◒	○	○	○	○	◒	◓	○	NA
Freeze-thaw/ pulverized/ cracks	◒	●	●	◓	◓	◓	○	○	◓	NA○
Delamination in soffit[1]	○	○	◓	◓	◒	◓	○	○	○	NA
ASR	◒	●	◒	◒	◒	◒	○	○	◓	NA
Active corrosion/ corrosion potential	○	◒	◒	○	◒	○	○	○	○	●

[1] NDE technologies applied to the soffit surface.

APPENDIX E

Attribute Index and Commentary

Introduction

This section includes suggested attributes for the reliability assessment of bridges. Users can select attributes from this listing. It is also recommended that users develop additional attributes that meet the needs of their individual agencies. This commentary is organized into four sections: Screening, Design, Loading, and Condition. The Screening section describes attributes that may be used to quickly identify bridges that should not be included in a particular analysis, either because they already have significant damage or they have attributes that are outside the scope of the analysis being developed. In many cases, these attributes may require engineering analysis beyond that which is typically conducted during a reliability assessment using this Guideline.

Screening attributes are typically attributes that:

- Make the likelihood of failure very high.
- Make the likelihood of failure unusually uncertain.
- Identify a bridge with different anticipated deterioration patterns than other bridges in a group.

Design attributes are characteristics of a bridge element that are part of the element's design. Design attributes are frequently intrinsic characteristics of the element that do not change over time, such as the amount of concrete cover or material of construction [concrete, high performance concrete (HPC), etc.]. In some cases, preservation or maintenance activities that contribute to the durability of the bridge element may be a design attribute, such as the use of penetrating sealers as a preservation strategy.

Loading attributes are characteristics that describe the loads applied to the bridge element. These may include structural loading, traffic loading, or environmental loading. Environmental loading may be described in macro terms, such as the general environment in which the bridge is located, or on a local basis, such as the rate of de-icing chemical application on a bridge deck. Loading attributes describe key loading characteristics that contribute to the damage modes and deterioration mechanisms under consideration.

Condition attributes describe the relevant bridge element conditions that are indicative of its future reliability. These can include its current element or component level rating, or may be a specific condition that will affect the durability of the element. For example, if the deterioration mechanism under consideration is corrosion at the bearing areas, the condition of the bridge joint may be a key attribute in determining the likelihood that corrosion will occur in the bearing area.

The listing of attributes included here is *not* intended to be comprehensive or mandatory. Users should consider adding attributes that are important to their specific inventory. Users are encouraged to document the rationale for including additional attributes in the reliability assessment, along with an appropriate scoring scheme. Users may also wish to omit certain attributes if they are not relevant to their inventory or do not contribute to the reliability and durability of bridges within their inventory. The suggested weightings are also exemplary in nature and may need to be adjusted to meet the needs of a particular bridge inventory.

Scoring Scheme

Attributes are assigned points based on the importance or contribution of the attribute in terms of the durability and the reliability of the element being assessed. In general, the scoring scheme utilizes a three-stage assessment of the importance of the attribute as shown in Table E1. The Ranking Descriptor is intended to provide some verbal description of the weight associated with each score. As shown, three relative course levels are presented: Low, Moderate, and High. The RAP may wish to modify the suggested scoring for a given attribute, based on local conditions, past experience, and previous performance within its bridge inventory and operational

Table E1. Suggested rank scoring for attributes.

Ranking Descriptor	Total Points
High	20
Moderate	15
Low	10

environment. The scoring scheme should effectively develop sound engineering rationale to support risk-based inspection practices.

Screening Attributes

S.1 Current Condition Rating

Reason(s) for Attribute. The current condition rating characterizes the overall condition of the component being rated according to the NBIS rating scale. Bridge components that have condition ratings of 4 or less have been rated to be in poor condition. In some cases, these components may already be on a reduced (12 month or less) inspection frequency. Users may wish to use this criterion to screen bridges that are already in poor condition and, as a result, require more in-depth analysis to identify their inspection needs. Users could also assign the OF of "high" without further assessment, since the component is already in poor condition.

For element-level inspection approaches, National Bridge Elements (NBEs) or Bridge Management Elements (BMEs) could be utilized within the screening criteria, as appropriate for specific bridge inventories and inspection practices. Generally, elements indicated with condition states of 4 would be appropriate for consideration as a screening tool for elements selected to match the needs and practices within the specific bridge inventory.

Assessment Procedure. This screening attribute is scored based on whether the current condition rating is 4 or less or greater than 4. The current condition rating from the most recent inspection report should be used. If using an element-level approach, the RAP should identify appropriate elements and condition states for screening.

Current condition rating is less than or equal to 4	Component is screened from general reliability assessment
Current condition rating is greater than 4	Continue with procedure

S.2 Fire Damage

Reason(s) for Attribute. Incidences of fire on or below a highway bridge are not uncommon. This type of damage is most frequently caused by vehicular accidents that result in fire, but secondary causes such as vandalism, terrorism, or other damage initiators should not be discounted. If fire does occur on or below a bridge, an appropriate follow-up assessment should be conducted to determine how the fire has affected the load carrying capacity and the durability characteristics of the main structural members and the deck. This assessment is typically performed during a damage inspection immediately following the incident.

Damage to bridge components resulting from a fire is either immediately apparent during the damage inspection, or may manifest within the first 12- to 24-month interval following the fire. Based on this observation, bridges that have experienced a fire may be screened from the reliability assessment until an inspection, which has been conducted approximately 12 months or more after the fire, confirms that the fire has not affected the typical durability characteristics of the bridge components. The purpose of this screening is to ensure that damage from the fire has not manifested after the damage inspection.

Assessment Procedure. This attribute is scored based only on the occurrence of a fire on or below the structure being assessed. It is assumed that an appropriate assessment immediately following the fire incident (i.e., damage inspection) has been performed.

Fire incident has occurred and an inspection 12 months after the fire has not occurred	Bridge is not eligible for reliability assessment until inspection confirms that the bridge is undamaged
There have been no incidence of fire on or below the bridge, or inspections conducted approximately 12 months or more an after the fire have confirmed that the bridge is undamaged	Continue with procedure

S.3 Susceptible to Collision

Reason(s) for Attribute. This screening attribute can be used to screen an inventory or a family of bridges to identify those bridges with specific vulnerabilities to random or near-random damage from collision. This attribute is intended to apply to a limited number of bridges for which the risk of collision is unusually high or special. Simply because a bridge could be subjected to impact does not mean the likelihood of impact is high, and, in fact, it could actually be quite remote. However, there are some structures that have been impacted many times in the past, where a channel or a roadway is particularly difficult to navigate, vertical clearance is inadequate, etc. that are much more likely to be struck. Examples include collisions from barges, debris, or heavy trucks. This attribute would typically be used to screen specific bridges that have an unusual or a unique risk of collision damage than a larger group or family of bridges which do not. In such cases, individual reliability analysis may be required.

Assessment Procedure. This screening attribute should be assessed based on sound engineering judgment and is intended to screen bridges with unusual or special collision risks from an assessment of a group of bridges that do not.

Highly susceptible to collisions	Requires specialized assessment and/or mitigation
Structure is not susceptible to collisions	Continue with procedure

S.4 Flexural Cracking

Reason(s) for Attribute. When the primary load-bearing members in a concrete bridge exhibit flexural cracking, it may indicate that the members were either inadequately designed for the required loading, that overloads have occurred, or that deterioration has occurred that has reduced the load-bearing capacity of the members. In any case, large flexural cracks can be indicative of an inadequate load-bearing capacity that may require an engineering analysis in order to determine the cause of the cracking and the resulting effect on the load capacity of the structure. As a result, bridges exhibiting moderate to severe flexural cracking should be screened from the general reliability assessment unless appropriate engineering analysis indicates that the cracking is benign or corrective repairs have been made.

The effects on the strength and the durability of a prestressed element due to flexural cracking are generally more significant than for a reinforced concrete element.

Assessment Procedure. Flexural cracks will typically present themselves with a vertical orientation either near the bottom flange at mid-span or near the top flange over intermediate supports, if the member is continuous.

Engineering judgment must be exercised in determining whether any present flexural cracking is moderate to severe. Crack widths in reinforced concrete bridges exceeding 0.006 inches to 0.012 inches reflect the lower bound of "moderate cracking." The American Concrete Institute Committee Report 224R-01 (*1*) presents guidance on what could be considered reasonable or tolerable crack widths at the tensile face of reinforced concrete structures for typical conditions. These values range from 0.006 inches for marine or seawater spray environments to 0.007 inches for structures exposed to de-icing chemicals, to 0.012 inches for structures in humid, moist environments. In prestressed concrete bridge structural elements, tolerable crack width criteria have been adopted in the Precast/Prestressed Concrete Institute (PCI) *Manual for the Evaluation and Repair of Precast, Prestressed Concrete Bridge Products* (MNL-37-06). The PCI Bridge Committee recommends that flexural cracks greater in width than 0.006 inches should be evaluated to affirm adequate design and performance.

Presence of moderate to severe flexural cracking in reinforced or prestressed concrete bridge elements	Assess individually to determine source, extent, and effect of cracking
Flexural cracking is not present, or it has been determined to be benign or repaired	Continue with procedure

S.5 Shear Cracking

Reason(s) for Attribute. If the primary load-bearing members in a reinforced or a prestressed concrete bridge exhibit shear cracking, it may indicate that the members were either inadequately designed for the required loading, an overload has occurred, or that deterioration has occurred that has reduced the load-bearing capacity of the members. In any case, shear cracks can be indicative of an inadequate load-bearing capacity requiring an engineering analysis in order to determine the cause of the cracking and the resulting effect on the load capacity of the structure. As a result, bridges exhibiting cracking attributable to a deficiency in shear strength should be screened from the reliability assessment unless appropriate engineering analysis indicates that the cracking is benign or corrective repairs have been made.

Assessment Procedure. Engineering judgment must be exercised in determining whether any present shear cracking is attributed to a shear strength deficiency. Shear cracks will typically present themselves with a roughly 45 degree diagonal orientation for conventionally reinforced concrete and down to roughly 30 degrees for prestressed elements, and will generally radiate toward the mid-span of the member. The ends of the member and any sections located over piers should be checked for this type of cracking.

Presence of unresolved shear cracking	Assess individually to determine source and extent of cracking
Shear cracking is not present or it has been determined to be benign	Continue with procedure

S.6 Longitudinal Cracking in Prestressed Elements

Reason(s) for Attribute. This attribute is for the assessment of prestressed bridge elements. Longitudinal cracking in prestressed elements can be indicative of corrosion or fracture of the embedded prestressing strands. As a result, prestressed elements with reported longitudinal cracking should be individually assessed to determine the source of the cracking and the condition of the prestressing strands.

Assessment Procedure. This attribute is assessed based on data in the inspection report and engineering judgment. If longitudinal cracking is reported, further assessment may be required.

Significant longitudinal cracking is present	Assess individually to determine source and extent of cracking and condition of strand
No significant longitudinal cracking	Continue with procedure

S.7 Active Fatigue Cracks Due to Primary Stress Ranges

Reason(s) for Attribute. Active fatigue cracks in steel bridge elements due to primary stresses can propagate quickly and potentially lead to a fracture in the element. These cracks are distinguished from distortion cracks or out-of-plane fatigue cracks, which are more commonly observed, but generally less critical.

Assessment Procedure. If any active fatigue cracks due to primary stresses are found in the element, it is *strongly recommended* that the element be retrofitted before continuing with this procedure. It is noted that a "stable" fatigue crack can potentially propagate to brittle fracture depending on the toughness of the material, the total applied stress, and the temperature. A fatigue crack can be considered "not active" if previous inspection reports show that the crack has not grown over a set period of time (e.g., the longest inspection interval plus 1 year). Primary stresses are those stresses (i.e., stress ranges) that are readily calculated using traditional mechanics principles (e.g., MC/I or P/A) and are typically obtained during design or rating.

Active fatigue crack(s) due to primary stresses present	Retrofit before continuing
No active fatigue crack(s) due to primary stresses present	Continue with procedure

S.8 Details Susceptible to Constraint-Induced Fracture (CIF)

Reason(s) for Attribute. Details that are susceptible to CIF can lead to brittle fracture in the absence of any observable cracking. An example of this is the failure of the Hoan Bridge in December 2000 in Milwaukee, WI (*2*). The bridge had been in service for approximately 25 years before two of the three girders experienced full-depth fractures and the third girder had a crack that arrested in the flange. Inspection is not a valid method to prevent these types of failures from occurring (the Hoan Bridge was inspected a few days prior to the failure). Hence, the attribute is included as a screening criterion.

Assessment Procedure. Details susceptible to CIF have a much higher probability of fracture failure than other types of details. It is recommended that CIF details be retrofitted or examined more closely before continuing with this process.

Structure contains details susceptible to CIF	Retrofit before continuing
Structure does not contain details susceptible to CIF	Continue with procedure

S.9 Significant Level of Active Corrosion or Section Loss

Reason(s) for Attribute. This attribute is intended to be used to screen bridges that have a significant level of existing or active corrosion sites that make the likelihood of severe corrosion damage relatively high. A significant amount of active corrosion and/or section loss in an element increases the probability of severe corrosion damage developing in the near future. As a result, individual engineering assessments may be required to effectively assess the reliability characteristics for the element. Significant section loss would normally be visible for steel structural members.

Assessment Procedure. If a significant amount of active corrosion with section loss is found on a steel element it is recommended that the element be repaired before continuing with this process. Engineering judgment must be used to determine what is defined as a *significant* amount of active corrosion with section loss and assess its effects. Previous inspection reports and engineering judgment must also be used to determine whether or not the corrosion is active.

Corrosion damage in steel elements that is inactive is explicitly distinguished from corrosion that is active. For example, section loss on a girder web that was the result of a leaking expansion joint that was corrected (the joint was replaced and the girder was repainted), could be classified as inactive corrosion if the expansion joint repair eliminates the vulnerability to corrosion. It is assumed that the owner has either determined that the existing section loss is insignificant or has taken it into account in the rating procedures and load posting, if needed, is in place.

Significant level of active corrosion and section loss	Repair before proceeding
Active corrosion or section loss is not significant or has been repaired	Continue with procedure

S.10 Design Features

Reason(s) for Attribute. This attribute is intended to be used to screen bridges that have unusual or unique design features that make the likelihood of serious damage either usually high or unusually uncertain, relative to other bridge in the same family, or identify bridges with different anticipated deterioration patterns than other bridges in a group or family. This attribute can be used to subdivide a family of bridges into two or more groups with similarly anticipated deterioration patterns, based on specific design features that are not common to each sub-group. Design features for use as screening items should be identified by the RAP. Two examples below are provided to illustrate the way in which this attribute might be used.

Bridges with pin and hanger connections: Pin and hanger connections generally have a history of presenting maintenance challenges. As such, it may be desirable to screen a bridge that includes this particular type of connection from a larger family, such as a family of steel multi-girder bridges.

Jointless bridges: Jointless bridges are typically less susceptible to corrosion-related damage associated with leaking joints in the bearing areas. As such, the deterioration patterns may differ from other bridges of similar materials and general overall design.

Assessment Procedure. Unique or unusual design features should be identified through review of bridge plans or other documentation describing the design features of a bridge.

Bridge has unique or unusual design feature	Screen
Bridge does not have unique or unusual design features	Proceed

Design Attributes

D.1 Joint Type

Reason(s) for Attribute. Bridge joint types can be categorized as either closed systems or open systems. Compared to open joint systems, closed joint systems provide for higher durability based on the way their designs shield the inner workings of the joint from dirt and debris. This, in turn, increases the amount of time before a joint begins to leak onto other bridge components. The presence of open-type deck joints increases the probability of chloride-contaminated water leaking onto bridge elements below the deck, thus increasing the likelihood of corrosion-related damage.

Assessment Procedure. This attribute is rated based on the presence of open joints.

Open joint system	10 points
Closed joint system	0 points

D.2 Load Posting

Reason(s) for Attribute. The presence of a load posting typically indicates that the given bridge was either not designed to carry modern loading or that the bridge has become damaged and its structural capacity has been reduced. A structure of this type may be more likely to experience damage from heavy traffic and dynamic loading. This attribute is intended to consider the contribution of high and possibly even excessive loads on accelerating damage generally for a given bridge or a family of bridges. Engineering judgment is necessary to evaluate if this attribute is applicable. Considerations include the likelihood of the applied loading being higher than (i.e., illegal) or near the load posting. In some cases, traffic patterns are such that the fact that the bridge is load posted will not affect the rate of damage accumulation on the bridge. For example, a bridge is load posted for the state's legal truck load, but is located on a parkway where trucks are prohibited.

Assessment Procedure. This attribute is scored based only on whether or not a bridge has been load posted; the level of the rating does not need to be considered. This assessment should consider if the load posting has a significant effect on the durability of the bridge.

Structure is load posted	20 points
Structure is not load posted	0 points

D.3 Minimum Vertical Clearance

Reason(s) for Attribute. This attribute is intended to consider the likelihood that a bridge may be impacted by an over-height vehicle and damaged such that the deterioration rate of the superstructure elements may be increased. For concrete bridges, impacts may damage the embedded reinforcement or the prestressing strands, or damage the typical concrete cover exposing the steel to the environment. For steel bridges, impacts can deform members and damage coating systems in the areas of the impact. Impact damage that affects the structural capacity of the bridge requires a damage inspection and an assessment beyond the scope of a typical reliability assessment. Users may wish to use this attribute to include the potential for increased deterioration rates for bridges that experience frequent impact damage.

The bridge superstructure's minimum vertical clearance influences on how often it will be impacted. A bridge with a lower vertical clearance will be more likely to experience impact damage than a bridge with higher vertical clearance. The likelihood of being hit may also

Table E2. FHWA coding guide minimum vertical underclearance provisions.

Underclearance Code	Minimum Vertical Underclearance			
	Functional Class			Railroad
	Interstate and Other Freeway	Other Principal and Minor Arterials	Major and Minor Collectors and Locals	
9	>17 ft	>16.5 ft	>16.5 ft	>23 ft
8	17 ft	16.5 ft	16.5 ft	23 ft
7	16.75 ft	15.5 ft	15.5 ft	22.5 ft
6	16.5 ft	14.5 ft	14.5 ft	22 ft
5	15.75 ft	14.25 ft	14.25 ft	21 ft
4	15 ft	14 ft	14 ft	20 ft
3	Rating <4 and requiring corrective action			
2	Rating <4 and requiring replacement			
1	*No value indicated*			
0	Bridge closed			

depend on the traffic composition of the roadway below, such as the average daily truck traffic (ADTT).

This attribute is generally based on the total vertical clearance between the bottom of the girders and the riding surface of the roadway below. The functional classification of the roadway below the bridge may also be a consideration. NBIS data fields record the vertical clearance and the functional classification of the route passing under the bridge, and are rated using the model provided in the coding guide (*3*), which is provided in Table E2.

Assessment Procedure. This attribute should be scored based on appropriate measurements or on the information stored in the bridge file. The suggested scoring models shown below consider only the vertical clearance of the bridges. Users may wish to consider the functional classification or the typical traffic pattens below the bridge in their assessment. In the scoring models shown, increased importance is given to over height clearances for prestressed concrete bridges relative to steel and conventionally reinforced bridges. This is due to the potential for strand corrosion when the concrete cover is damaged by impact, and the increased rate of deterioration for strands relative to mild steel.

Prestressed Concrete Girders

Vertical clearance is 15 feet or less	20 points
Vertical clearance is between 15 feet and 16 feet	15 points
Vertical clearance is between 16 feet and 17 feet	10 points
Vertical clearance is greater than 17 feet or no under traffic present	0 points

Steel and Concrete Girders

Vertical clearance is 14 feet or less	15 points
Vertical clearance is between 14 feet and 15 feet	12 points
Vertical clearance is between 15 feet and 17 feet	7 points
Vertical clearance is greater than 17 feet or no under traffic present	0 points

D.4 Poor Deck Drainage and Ponding

Reason(s) for Attribute. This attribute is intended to consider the adverse effects of poorly designed deck drainage systems and the possibility of ponding on the deck surface, as well as for inadequate provisions for preventing scuppers and drains from splashing de-icing chemicals onto the superstructure below. Ineffective deck drainage increases the likelihood of bridge elements developing corrosion related damage. This results from drainage onto the superstructure and the substructure elements. Both concrete and steel elements will have an increased susceptibility to corrosion damage when exposed to prolonged periods of wetness and/or frequent wet-dry cycles. The presence of chlorides from de-icing chemicals applied to the deck also increases the likelihood of corrosion damage to these elements.

This attribute can also be used to characterize decks with ponding or with drain diversion issues. When water is allowed to sit on the surface of the deck, there is an increase in the likelihood that corrosion of the reinforcing steel will initiate and damage will propagate. Water and chlorides are more likely to penetrate to the level of the reinforcement when periods of wetness are prolonged and chloride concentrations at the surface are high.

Assessment Procedure. This attribute is scored based on the drainage design of the bridge and any known ponding or drainage issues, as noted in the inspection report. Drainage systems which normally allow water to run off onto the components below the bridge deck are considered ineffective, regardless of whether they have sustained any damage or not. Deck drains through curb openings, where the water from the decks typically drains onto superstructure elements, are an example of poor deck drainage. Decks with ponding issues may need to be individually scored.

Ponding or ineffective drainage	10 points
No problems noted	0 points

D.5 Use of Open Decking

Reason(s) for Attribute. The presence of an open deck increases the likelihood that corrosion of the steel superstructure will occur. An open deck allows water, de-icing chemicals, and other debris to fall directly onto the superstructure instead of running into deck drains and then to downspout pipes, as they would in a closed deck system. As a result, the likelihood of damage occurring in superstructure elements, bearing, and substructure elements is greatly increased. Users may also use this as a screening attribute.

Assessment Procedure. The attribute is scored based on whether or not the bridge contains an open deck. Common types of open decks include timber or open grating decks.

Bridge has an open deck	20 points
Bridge does not have an open deck	0 points

D.6 Year of Construction

Reason(s) for Attribute. This attribute reflects the influence of bridge age and historic design on the most prevalent aging mechanisms in highway bridges—deterioration of concrete associated with corrosion of embedded reinforcement, and corrosion damage and/or fatigue and fracture for steel structures.

The corrosion of embedded reinforcing steel occurs due to the penetration of chlorides, water, and oxygen to the level of the reinforcement. For intact concrete, the penetration of the chlorides is presently modeled as a diffusion process, using Fick's Law, which depends on time, temperature, the permeability of the concrete, and the concentration of chlorides at the component's surface. Additionally, if the concrete has suffered damage, such as cracking or spalling, chlorides can more easily concentrate at the reinforcement, effectively expediting the corrosion process.

The quality of the concrete used in bridge construction has generally improved over time due to concrete technology innovation, improvements in quality control, and in better supplier understanding of optimal material selection for strength and durability. Therefore, it is reasonable to expect that a concrete component constructed to modern standards is likely to have improved corrosion resistance characteristics compared to older components. Additionally, older structures have been exposed to the surrounding environment for a longer period of time, and are therefore more likely to be affected by corrosion.

With respect to steel girders, the year the bridge was designed can provide valuable information about the susceptibility of the bridge to fatigue cracking and fracture. Over the years, there have been numerous changes in design specifications that have resulted in the improved fatigue and fracture resistance of bridges. Four key dates have been identified; 1975, 1985, 1994, and 2009, with regard to changes in design specifications. These dates were selected for the following reasons:

1975

Fatigue

The "modern" fatigue design provisions, based on the research of Fisher and others, were fully incorporated into the AASHTO Specifications with the 1974 Interims. The basic detail categories have not changed significantly since their introduction. Hence, 1975 was selected as a differentiator regarding fatigue design of steel bridges. Prior to 1975, fatigue design was based on principles that were not generally appropriate for welded structures. Although these early provisions appeared in the 1965 version of the specifications and were in place through 1976, it was felt that it was reasonably conservative to ignore the earlier provisions and set the cutoff date at 1975.

Fracture

In 1974, partly in response to the Point Pleasant Bridge collapse (1967), mandatory Charpy V-Notch (CVN) requirements were set in place for welds and base metals as a part of the AASHTO/AWS Fracture Control Plan. The purpose of these CVN requirements was to ensure adequate fracture toughness of materials used in bridges. Furthermore, "modern" fatigue design provisions, based on the research of Fisher and others, were fully incorporated into the AASHTO Specifications as previously discussed. Hence, 1975 was selected as a differentiator regarding fatigue and fracture design of steel bridges.

1985

In 1985, AASHTO introduced changes to address and to prevent distortion-induced fatigue cracking. A common example of distortion-induced fatigue cracking is web-gap cracking. Hence, considering the specifications introduced in 1975 and 1985, bridges designed after 1985 are less likely to be susceptible to fatigue due to primary or secondary stress ranges than bridges built prior to these revisions.

1994

In 1994, the AASHTO design specifications changed from load factor design (LFD) to load and resistance factor design (LRFD). The LRFD method is intended to ensure greater reliability in bridge design. There were several changes regarding the load models and the load distribution factors used for the fatigue limit state. These changes were intended to result in a more realistic and reliable fatigue design. Hence, for the fatigue limit state, bridges designed after 1994 would be expected to have improved reliability.

2009

In 2008, language was introduced into the *AASHTO LRFD Bridge Design Specifications* which directly addressed the issue of CIF. The article provided prescriptive guidance to ensure that details susceptible to CIF are avoided. It is included in the 2009 and later versions of the *AASHTO LRFD Bridge Design Specifications.*

Assessment Procedure. The year of construction is intended to characterize the years of environmental exposure a component has experienced or the fatigue susceptibility of the design. The suggested values are intended to put elements into four broad classes that range from very old to relatively new. For elements that have been replaced, the year of the replacement should be used. Elements that have been rehabilitated should use the original construction date. These ranges are advisory; users may consider modifying these categories based on experience with their bridge inventory or significant changes to construction practices that may have occurred within their state. For steel-girder categories, users should consider if the design specification used in the design of the bridge matched the contemporary specifications at the time, as described above. If, for example, the LRFD provisions of 1994 were not implemented in the state until 2000, then the ranges should be adjusted accordingly.

Concrete Bridge Decks, Prestressed Girders, Substructures

Built before 1950	10 points
Built between 1950 and 1970	6 points
Built between 1970 and 1990	3 points
Bridge is less than 20 years old	0 points

Steel Girders, Fatigue

Bridge designed before 1975/unknown	20 points
Bridge designed between 1976 and 1984	10 points
Bridge designed between 1985 and 1993	5 points
Bridge designed after 1994	0 points

Steel Girders, Fracture

Bridge designed before 1975/unknown	20 points
Bridge designed between 1975 and 1984	10 points
Bridge designed between 1985 and 1993	5 points
Bridge designed between 1994 and 2008	3 points
Bridge designed after 2009	0 points

D.7 Application of Protective Systems

Reason(s) for Attribute. Protective systems such as membranes, overlays, or sealers may be applied to the surface of a concrete element to reduce the ingress of water, which may contain dissolved chlorides or other corrosive substances. When these corrosive materials diffuse to the level of the reinforcement, the likelihood of reinforcement corrosion increases, which may lead to the propagation of damage. Protective systems delay or prevent this process from occurring thereby reducing the likelihood for future corrosion damage. Some overlays have also been shown to delay the development of spalling as a result of an increased resistance to cracking and an increased ability to confine delamination damage (*4*).

An overlay is defined herein as an additional layer of protective material, which is applied on top of the concrete deck and that also serves as the riding surface. Overlays may consist of asphalt, latex-modified concrete, low-slump dense concrete, silica fume concrete, polymer concrete, or other materials.

A membrane is defined herein as a barrier that is placed on top of the concrete deck and is then covered by another material, which serves as the riding surface. Common membranes may consist of hot-rubberized asphalt, resin, bitumen-based liquid, or prefabricated sheets.

Sealers are somewhat different from overlays and membranes in that they are applied thinly to concrete surfaces and penetrate the porosity of the concrete to seal it from moisture. Initially, sealers were used to counteract freeze-thaw damage and de-icing chemical-application related scaling. With the proper use of air-entraining admixtures, the primary purpose of sealers changed to preventing or slowing the ingress of chlorides (*5*). Types of sealers include silanes, siloxanes, silicates, epoxies, resins, and linseed oil.

Surface coatings such as epoxy, polyurethane, or polyurea may also be applied to the concrete elements of a bridge in order to increase their resistance to water intrusion and consequently reduce their probability of developing corrosion damage. The application of these coatings can improve the durability and corrosion resistance of concrete elements.

Each of these protective systems is intended to delay or prevent corrosion damage in concrete bridge elements. If the protective systems are effective, then the likelihood of corrosion-related damage will be reduced compared to unprotected elements of similar design characteristics and environmental conditions. As a result, the application of protective systems may be considered in the reliability assessment.

Assessment Procedure. If protective systems such as membranes, overlays, or sealers have been applied to a concrete element, their effectiveness should be evaluated based on engineering judgment and local experience or test data along with any documented research and field testing data that is available. Important factors to consider include the effectiveness of the applied system as well as how often that system is applied or maintained. This attribute assumes that overlays and sealers generally have similar effects in terms of corrosion protection for the deck. Based on their experience, users may wish to separate certain overlays or membrane systems. For example, an owner may have experience that indicates that low-slump overlays are having a significant effect on extending the service life of bridge decks. In that case, the owner may wish to increase the importance of this attribute to a moderate or high level, and distribute the scoring appropriately. The suggested scoring assumes the protective system has a low importance relative to other design characteristics.

Never applied, poor functioning, or non-functioning	10 points
Yes, penetrating sealer, crack sealer, limited effectiveness	5 points
Yes, periodically applied, effective	0 points

D.8 Concrete Mix Design

Reason(s) for Attribute. Concrete mix designs, such as those considered to be "HPC," typically have a lower permeability and a higher durability than other traditional concrete mixes. Therefore, high performance mixes provide an increased resistance to de-icer or marine environment-based chloride ion penetration. This in turn can increase the time to corrosion initiation in reinforcing steel. This design attribute is intended to consider the increased durability provided by HPC mixes.

The permeability of a concrete mix depends on several factors including the water to cementitious ratio, the use of densifying additives, and the use of mix-improving additives. Supplementary cementitious materials such as fly ash, ground-granulated blast furnace slag, and silica fume have been shown to reduce permeability. Additionally, a properly designed and placed concrete mix with a lower water to cementitious ratio will have a lower permeability.

Materials and criteria that have been identified as being beneficial in enhancing the performance of concrete bridge decks can be found *NCHRP Synthesis 333: Concrete Bridge Deck Performance* (5).

Assessment Procedure. The evaluation of a bridge's concrete mix design should be based on information contained in the bridge's design plans and on engineering judgment. Many different types of concrete mixtures can be considered to be high performance, therefore, users should consider the corrosion resistance characteristics of the particular mixture and assess if the concrete mix used is expected to provide an increased durability relative to a typical concrete mix design. Past experience with concrete mixes of similar characteristics should be considered.

The concrete used is not considered to be high performance	15 points
The concrete used satisfies high performance conditions	0 points

D.9 Deck Form Type

Reason(s) for Attribute. Concrete decks constructed with stay-in-place (SIP) forms have the surface of the deck soffit hidden from visual inspection. Signs of corrosion damage such as efflorescence, rust staining, and cracking in the deck soffit cannot typically be observed. As a result, there can be increased uncertainty in the condition of the deck determined through visual inspection. This attribute is intended to consider the increased level of uncertainly in the deck condition that may exist when SIP forms are used.

Assessment Procedure. This attribute is assessed based on whether the deck has SIP forms.

SIP forms	10 points
Removable forms	0 points

D.10 Deck Overlays

Reason(s) for Attribute. Similar to SIP forms, deck overlays prevent the visual observation of the deck condition. Signs of deterioration, corrosion damage, and cracking of the deck cannot typically be observed. As a result, there can be increased uncertainty in the condition of the deck determined through visual inspection. This attribute is intended to consider the increased level of uncertainty in the deck condition that may exist for decks with overlays.

Assessment Procedure. This attribute is assessed based on whether or not the deck has an overlay.

Deck has an overlay	10 points
Bare deck	0 points

D.11 Minimum Concrete Cover

Reason(s) for Attribute. This attribute is intended to consider the improved corrosion resistance and the increased durability associated with adequate concrete cover, and the historically poor performance of bridge elements with inadequate cover. The depth of concrete cover characterizes how far corrosive agents need to travel in order to reach the embedded steel reinforcement. Several studies have identified that the depth of concrete cover over the top reinforcing steel mat is the most significant factor contributing to the durability of decks (*5*). The importance of adequate concrete cover is also an important durability factor for other concrete elements. The value used for this attribute should be the actual amount of concrete cover, which may not necessarily be the design cover. If quality control procedures are adequate to ensure that the design cover matches the as-built cover, the design cover may be used. If such quality control procedures have not been utilized or have historically been inadequate, it may be necessary to assess the as-built cover.

In 1970, the general recommendation for concrete cover was a minimum clear concrete cover of 2 inches over the top-most steel. Currently, the *AASHTO Standard Specifications for Highway Bridges* (2002) requires a minimum concrete cover of 2.5 inches for decks that have no positive corrosion protection and are frequently exposed to de-icing chemicals. Positive corrosion protection may include epoxy coated bars, concrete overlays, and impervious membranes. The *AASHTO LRFD Bridge Design Specifications* (2004) also requires a minimum concrete cover of 2.5 inches for concrete that is exposed to de-icing chemicals or on deck surfaces that are subject to stud or chain wear. The concrete cover may be decreased to 1.5 inches when epoxy coated reinforcement is used.

It is also important to note that the type of damage and the rate of damage development vary with the amount of concrete cover. It has been reported that the type of damage changes from cracks and small, localized surface spalls to larger delaminations and spalling as the concrete cover increases (*4*). There is also an increase in the time to corrosion initiation and a reduction in the rate of damage development when cover increases, as shown schematically in Figure E1. In summary, as concrete cover increases, the time to corrosion initiation increases due to the increased depth that chloride ions must penetrate to initiate the corrosion process. As corrosion progresses, an increased concrete cover provides confinement that reduces the rate and the type of damage that develops at the surface of the concrete element.

It should be noted that concrete cover greater than 3 inches can result in increased cracking, providing pathways for the intrusion of water and chlorides. This may be a consideration in special cases in which the concrete cover is unusually large.

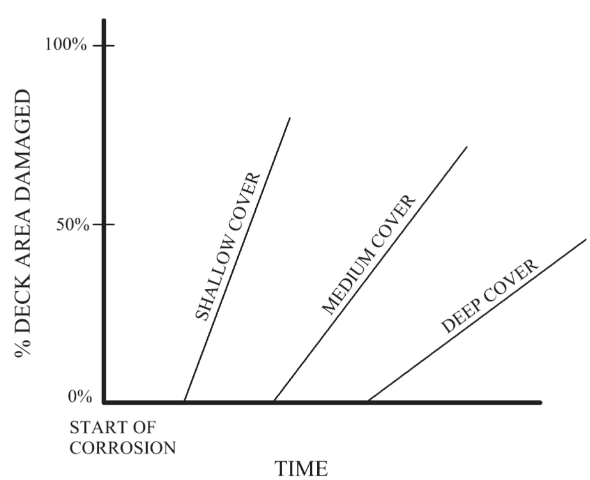

Figure E1. Effect of concrete cover on the time to corrosion initiation and development of damage (4).

Assessment Procedure. This attribute is scored based on the actual, physical clear cover which with the specified bridge element operates. The user should consider whether quality control practices used at the time of construction were adequate to provide confidence that the as-built concrete cover conforms to the design concrete cover, or if there are indications that the concrete cover may not be adequate. In these cases, the as-built concrete cover may be required and can be easily obtained using a covermeter.

1.5 inches or less, unknown	20 points
Between 1.5 inches and 2.5 inches	10 points
Greater than or equal 2.5 inches	0 points

D.12 Reinforcement Type

Reason(s) for Attribute. This attribute is intended to characterize whether or not the embedded reinforcing steel has a barrier to protect it against corrosion. The most commonly used barrier is an epoxy coating; however, galvanized bars and stainless steel, either as cladding or as solid bars, have also been used.

Uncoated steel reinforcement will corrode easily and significantly when under attack from corrosive elements such as chloride ions, oxygen, and water. Since this exposure is inevitable in an operating structure, one way to slow the corrosion process is to coat the mild steel bars with either an organic or a metallic coating or to use an alternate solid metal bar, such as stainless steel. These coatings or alternate bars help slow the corrosion process by providing either a physical or a metallurgical barrier against the action of the corrosive elements.

The most commonly used barrier coating is fusion-bonded epoxy powder. This type of coating has been used since 1973 and has been the subject of a significant body of research. It has been shown that, in reinforced concrete decks, if only the top mat is coated, for every year required to consume a given amount of mild steel, it will take 12 years for the epoxy coated bar to lose that same amount of metal. If both the top and bottom mats are coated, it may take up to 46 years (*6*). This significant increase when both mats are coated is due to increased electrical resistance, which further slows corrosion.

Two of the more common metallic coatings used are zinc and stainless steel. Zinc coated bars are also known as galvanized bars. Conflicting reports have been given on the performance of galvanized bars, mostly with respect to varying levels of the water to cement ratio and to whether or not galvanized bars are used in conjunction with mild steel bars. Research suggests that galvanized bars may add 5 more years to the 10 to 15 years required for corrosion-induced stress to manifest in unprotected bridge decks (*6*).

Solid stainless steel or stainless steel clad mild steel bars have also been used, although to a lesser extent due to their higher costs. Research conducted by the State of Virginia compared the performance of stainless steel clad and stainless steel bars with uncoated carbon steel bars. The research concluded that defect-free stainless steel clad bars performed nearly identically to the solid stainless steel bars. These types of bars were determined to tolerate at least 15 times more chloride than the carbon steel bars (*6*).

Regardless of the specific coating or reinforcement material used, protected bars generally have a higher resistance to corrosion damage than uncoated, mild steel bars. As such, the scoring for this attribute considers only if the rebar is protected by one of these methods, or if it is not.

Assessment Procedure. The type of reinforcement is scored based on the presence of barrier coatings or the use of alternative metal for the embedded reinforcement. This information can typically be identified from the structure's design plans. If suitable information is unavailable, engineering judgment should be used.

Reinforcement is uncoated carbon steel	15 points
Reinforcement has a protective coating or is produced from an alternate corrosion resistant metal (e.g., stainless steel)	0 points

D.13 Built-Up Member

Reason(s) for Attribute. Many bridges, especially older structures, contain built-up members. These built-up members are sometimes more susceptible to corrosion than normal rolled steel sections because they contain pockets or crevices, which can retain water, salt, debris, etc. This has been known to result in an accelerated corrosion rate since debris and moisture can remain trapped. Bridge washing, if thoroughly performed, can mitigate these effects.

Assessment Procedure. For this attribute, a built-up member refers to riveted or bolted members. Welded members should not be included in this assessment because they do not contain the type of pockets or crevices that can trap corrosion inducing materials.

Element is a built-up member	15 points
Element is not a built-up member	0 points

D.14 Constructed of High Performance Steel

Reason(s) for Attribute. In addition to possessing higher yield strengths than normal steels, high performance steels (HPSs) generally have greater fracture toughness than that required by ASTM A709, and of other common bridge steels. Improved fracture toughness results in steel that is more resistant to fracture than normal steels. This is because it is more likely that cracks will propagate at a slower rate, and could even arrest, in HPS compared to normal steels.

At this time, the CVN levels required for HPS in ASTM A709 are not established with the objective of achieving any particular level of fracture resistance or crack tolerance. Hence, the benefits provided by using HPS, if the steel just meets the ASTM A709 specification, are limited. Therefore, the suggested ranking of HPS is low in terms of contribution to durability and reliability (10 pts), relative to normal steel. This may change as future research becomes available and the minimum required CVN values increase for HPS.

Assessment Procedure. This attribute should be scored based on whether or not the element is constructed out of HPS. If there is no documentation or it is unknown if the element is constructed of HPS, the attribute should be scored accordingly.

Element is not constructed of HPS/unknown	10 points
Element is constructed of HPS	0 points

D.15 Constructed of Weathering Steel

Reason(s) for Attribute. Weathering steel is a type of steel that contains alloying elements that increase the inherent corrosion resistance of the steel. For this reason, weathering steels are less susceptible to corrosion than normal black steels. However, this is only true if the steel is used in the proper environment and is detailed properly.

Assessment Procedure. This attribute is scored based on whether or not the element is constructed using weathering steel and is detailed and located in a manner that minimizes the contact of the steel with de-icing chemicals and moisture. If it is unknown if the element is composed of weathering steel, the element should be scored accordingly. The assessment procedure assumes that the steel is used in the proper environment and is detailed properly. Guidance on the appropriate application of uncoated weathering steel can be found in FHWA Technical Advisory T-5140.22 (*7*). The document also includes recommendations for maintenance to ensure continued successful performance of the steel.

Element is not constructed of weathering steel or location and detailing may allow impact of ambient or de-icing chemicals on steel surfaces	10 points
Element is constructed of weathering steel and properly detailed consistent with FHWA Technical Advisory T-5140.22	0 points

D.16 Element Connection Type

Reason(s) for Attribute. Welded connections are usually more susceptible to the effects of fatigue damage than other types of connections, as there is a direct path for cracks to propagate between connected elements. For example, a crack in a flange can grow into the web through the web-to-flange weld. Fatigue cracking is generally of greatest concern for welded details that have low fatigue resistance, such as D, E, and E', along with residual stresses and weld toe defects.

Riveted connections, unlike welded connections, do not offer a direct path for cracks to propagate from one element to another. Using the web-to-flange connection example, cracks in an angle used to make up a flange are not able to grow directly into the web plate because the elements are not fused together. Hence, there is a certain amount of redundancy at the member level. Nevertheless, the quality of the rivet hole (e.g., punched vs. drilled) and a lack of consistent pretension in rivets results in these details being classified as category D.

Similar to riveted connections, high strength (HS) bolted connections are more resistant to a fatigue crack propagating from one component of a member to another, as compared to welded members. A properly tightened HS bolt generates very high compressive forces in the connection. The pretension force is much greater and is much more consistently achieved in a HS bolted connection than in a riveted connection. As a result of the significant pretention in a fully tightened A325 or A490 bolt, the quality of the hole itself has little or no effect on the fatigue resistance of the connection (in contrast to riveted joints). As a result, they are classified as category B details.

It is noted that considering the element connection type may appear to be a double penalty when considered in conjunction with D.17 Worst Fatigue Detail Category. However, it is clear that should cracking occur at a welded detail in a main member, it is more likely to become an issue than in, say, the equivalent bolted detail simply due to the fact that there is no direct path for cracks to grow from component to component in the bolted joint. Hence, it is considered a "better" condition even though both welded and bolted details may both be classified as category B. Riveted details, which do not have as high a fatigue resistance as HS bolted connections, but are not as susceptible to crack propagation as welded joints, have been arbitrarily scored in the middle.

Assessment Procedure. If the element has multiple types of connections, the worst type of connection should be scored for this attribute.

Element connected with welds	15 points
Element connected with rivets	7 points
Element connected with HS bolts	0 points

D.17 Worst Fatigue Detail Category

Reason(s) for Attribute. The likelihood of fatigue cracking is influenced by the type of fatigue detail category present. It is generally accepted that poor fatigue details are more likely to develop cracks than more fatigue resistance details. This is implied in the current *AASHTO LRFD Bridge Design Specifications*, which discourages the use of details lower than category C and encourages design for infinite life. Fortunately, since the introduction of the modern AASHTO fatigue provisions in 1975, the use of poor details (D, E, and E') has been greatly reduced. Hence, details in bridges designed over the past 30 years or so will typically be of higher fatigue resistance.

Assessment Procedure. The worst type of detail subjected to tensile stress ranges in the element or member should be used for this attribute. The AASHTO fatigue details A through E' should be used.

Fatigue detail category E or E'	20 points
Fatigue detail category D	15 points
Fatigue detail category C	5 points
Fatigue detail category A, B, or B'	0 points

If the element has multiple types of connections, the worst type of connection should be scored for this attribute.

Element connected with welds	15 points
Element connected with rivets	7 points
Element connected with HS bolts	0 points

D.18 Skew

Reason(s) for Attribute. Bridge skew can introduce unanticipated forces in a bridge deck, deck joints, and superstructures. Thermal expansion of the superstructure and deck may introduce uneven strain distributions and/or torsional forces. As a result, bridges with high skew angles may suffer atypical deterioration patterns including cracking in bridge decks, failure of joints and bearing, and distortion-induced cracking at diaphragms (*8–12*).

Assessment Procedure. This attribute is typically scored based on the recorded skew angles for a bridge. Angles of 30 degrees or greater may be used as a value for evaluating the potential for adverse skew angle effects. This attribute may also be used as a screening attribute.

Skew 30° or more	20 points
Skew 20–30°	10 points
Skew less than 20°	0 points

D.19 Presence of Cold Joints

Reason(s) for Attribute. Cold joints or construction joints within deck spans can sometimes result in leakage of water and de-icing chemicals through the deck and onto the supporting superstructure. This may result in accelerated deterioration patterns including coating failure and section loss for steel members, corrosion damage in concrete members, and/or corrosion damage in the deck.

Assessment Procedure. This attribute is typically scored based on the presence of known cold joints within the deck span. Data to support this assessment may come from inspection reports, because cold joints that are performing as designed may not be known.

Presence of cold joints	10 points
No known cold joints	0 points

D.20 Construction Techniques and Specifications

Reason(s) for Attribute. Construction techniques and specifications have evolved over time to improve the durability and performance characteristics of bridges. Certain construction techniques and specifications used during previous eras may be problematic, and result in deterioration and damage patterns that can be associated with the techniques or specification in use at the time of bridge construction. For example, reduced bridge deck thickness may have been typical during a certain era. Over time, the reduced deck thickness may be shown to reduce the durability of the bridge deck and result in deck damage such as punch-through. As a result, decks constructed during that era may be more likely affected by a certain damage mode than bridges constructed during other eras.

Assessment Procedure. This attribute will typically be identified by RAP members based on experience of bridge inspection and maintenance personnel. Historical records documenting the evolution of design standards and construction techniques may be necessary to identify the specific era, or estimates based on experience may be used. This attribute may also be used as a screening attribute.

Bridge constructed during identified era	20 points
Bridge not constructed during identified era	0 points

D.21 Footing Type

Reason(s) for Attribute. Spread-type footings may be susceptible to the adverse effects of scour, soil sliding, or rotations due to uneven settlement or subsidence. In contrast, pile foundations may be unaffected by these phenomena. As such, deterioration patterns and damage modes that affect spread footings may not be relevant for pile foundations.

Assessment Procedure. This attribute can typically be determined from the design drawing available in the bridge file. This attribute may be used as screening criteria for specific damage modes that affect spread footings, but would not affect pile foundations.

Spread-type footing	15 points
Pile foundation	0 points

D.22 Subsurface Soil Condition

Reason(s) for Attribute. Footings on certain soils may be susceptible to the effects of soil sliding or rotations due to uneven settlement or subsidence. This attribute is typically utilized in conjunction with D.21 to reflect the increased likelihood of damage modes such as substructure rotations, cracking, or displacements for bridges in certain geographic regions.

Assessment Procedure. Subsurface soil conditions susceptible to these effects are typically known to geotechnical engineers and/or maintenance personnel. This attribute may be identified based on soil testing results or experience.

Poor or unknown subsurface soil conditions	20 points
Acceptable soil condition or pile foundations	0 points

Loading Attributes

L.1 ADTT

Reason(s) for Attribute. The ADTT on a bridge is used to characterize the frequency of occurrence of large external loads on the bridge due to heavy vehicles. Large transport trucks or other heavy vehicles place stress on a bridge as static and dynamic loads, the latter reflecting impact and other dynamic amplification effects.

As ADTT levels increase, the rate of damage formation and accumulation in concrete is typically expected to increase. This is in part because the stresses caused by traffic loads accelerate the effects of the internal expansion forces from reinforcement corrosion (*4*). These loads, especially when placed on a bridge with existing deterioration, will open cracks and possibly allow corrosive elements to enter the cracks or increase the crack density. Experience has shown that bridge decks exposed to heavy truck traffic generally deteriorate at a much higher rate than decks with little or no truck traffic.

For steel girders, research has shown that trucks produce nearly all of the fatigue damage in highway bridges. Hence, a bridge with high truck traffic (high ADTT) will have a higher probability of fatigue damage. Of course, the converse is also true, bridges with little or no truck traffic (e.g., HOV bridges) are unlikely to experience fatigue cracking.

It is important to note that ADTT only considers the "load" side of the equation. The likelihood of fatigue cracking also depends on the "resistance" side of the equation, which is addressed by the D.16 Element Connection Type and D.17 Worst Fatigue Detail Categories. Although ADTT does not provide an exact correlation to the stress ranges an element will experience, it does provide a reasonably good understanding of how quickly fatigue damage may accumulate.

Assessment Procedure. This attribute should be scored based on the ADTT.

For steel structures, the scoring limits for ADTT were taken from a recent study on fracture critical bridges titled *A Method for Determining the Interval for Hands-On Inspection of Steel Bridges with Fracture Critical Members* (*13*). Although these limits were developed primarily with fracture critical bridges in mind, it was decided these limits could be applied to other highway bridges as well for the fatigue limit state. The reasoning behind the limits as documented in Parr and Connor's report is as follows:

"The ADTT limit of 15 comes from the fact that for bridges where the ADT is less than 100, the ADT is generally not reported in the NBIS. During the Purdue University Workshop, it was agreed than an ADTT of 15% (of the ADT) was a reasonably conservative estimate of the proportion of trucks crossing a typical low volume bridge. Hence, 15% of the lowest ADT reported in the NBIS (ADT = 100) yields an ADTT of 15.

The lower bound value of 100 was set such to separate bridges in rural areas versus 'moderately' traveled bridges. The upper bound limit of an ADTT equal to 1,000 was obtained by simply increasing the 'moderate' limit by a factor of 10. It was included simply to create a boundary between 'heavily' and 'moderately' traveled bridges."

For concrete bridges, high ADTT will likely have the most significant effect on the durability of the bridge deck. Superstructure components will be affected to a much lesser extent; if designed to modern standards, high ADTT may have little effect on the durability of superstructure components. Deck joints may also deteriorate more rapidly in the presence of high ADTT.

Users may wish to adopt different thresholds for the scoring model, depending on typical traffic patterns and needs.

Concrete Bridge Deck, Prestressed Concrete Girder

ADTT is greater than 5,000	20 points
ADTT is moderate	10 points
ADTT is minor	5 points
No heavy trucks	0 points

Steel Girders

ADTT is greater than 1,000	20 points
ADTT is between 100 and 1,000	15 points
ADTT is between 15 and 100	5 points
ADTT is less than 15	0 points

L.2 Dynamic Loading from Riding Surface

Reason(s) for Attribute. This attribute is intended to consider the detrimental effects of dynamic loading on the deterioration patterns for concrete bridge decks. This attribute would typically be used to adjust assessments to consider a reduction of the durability of bridge decks with high dynamic loads (i.e., high speed traffic and high ADTT). This attribute is included to consider cases where the riding surface or the deck joint becomes damaged, such as through the development of potholes, rough patches, or a bump at the end of the bridge, and increased dynamic forces are created due to the traffic loading. These forces place additional stress on the structure leading to a perpetual cycle of damage propagation that accelerates the rate of deterioration for the deck element (*14*).

Assessment Procedure. This attribute is based on engineering judgment. Considerations in assessing this attribute include the roughness of the riding surface, the existence of potholes and patches, durability of deck joints, ADTT, and traffic speeds.

Dynamic forces leading to increased rate of deterioration a significant consideration	15 points
Dynamic forces not a significant consideration	0 points

L.3 Exposure Environment

Reason(s) for Attribute. The environment surrounding a bridge can have a significant effect on the rate of deterioration, particularly for corrosion. This attribute is intended to characterize the macro-environment surrounding a bridge and account for the likelihood of increased deterioration rates in environments that are particularly aggressive, such as coastal or marine environments. Aggressive environments typically have high ambient levels of chlorides, high ambient moisture levels (high humidity or frequent wet/dry cycles, increased temperature), and the presence of other harmful chemicals (i.e., high levels of carbon dioxide, sulphates, etc.).

Assessment Procedure. The assessment procedure is similar to other environmental exposure classifications that are already in practice. Marine environments are deemed to be the most severe due to the high levels of ambient chlorides and moisture. "Moderate" environments are those in which corrosive agent levels (water and chlorides) are elevated but lower than those found in marine or other severe exposures. "Industrial" environments are less severe than marine but may contain other harmful chemicals. Under modern regulatory constraints, airborne pollutant levels associated with industrial environments are minimized, and this should be considered in the assessment of industrialized environments. "Benign" environments are those in which application of de-icing chemicals is minimal or nonexistent; the environments may be arid and atmospheric pollutants typical.

Severe/Marine	20 points
Moderate/Industrial	10 points
Benign	0 points

L.4 Likelihood of Overload

Reason(s) for Attribute. This attribute can be used when the likelihood of overload is a consideration for the bridge or a family of bridges being assessed. The likelihood of overload is used to characterize the chance that a bridge will be loaded beyond its inventory load rating. Such overloads generally increase the deterioration rate for structural elements. The probability of this occurring may be greater for bridges with a reduced capacity, such as those that have already been load posted.

Assessment Procedure. This attribute is scored based on how likely it is that a bridge will be overloaded. Sound engineering judgment should be used to assess this attribute.

High likelihood of overload	15 points
Moderate likelihood of overload	10 points
Low likelihood of overload	0 points

L.5 Rate of De-icing Chemical Application

Reason(s) for Attribute. This attribute is intended to characterize the volumes of the de-icing chemicals containing chloride ions that are being applied regularly to the surface of the deck. The detrimental effects of de-icing chemicals on the durability of bridge elements are well known. The intrusion of chloride ions to the level of the reinforcing steel provides an important driving force for corrosion of the reinforcing steel (*15*). When combined with oxygen and water, higher levels of de-icing chemical application generally lead to more rapid and severe reinforcement corrosion rates. The presence of increased chloride concentrations at the surface of the concrete increases chloride diffusion rates, shortening the time for the initiation of corrosion in the steel. If faulty deck joints or a substandard drainage system are present, which permit water seepage, bridge elements below the deck may also be affected by increased chloride ion levels. This will lead to increased levels of corrosion and consequently to corrosion-related damage.

Assessment Procedure. This attribute can be scored based on the average annual number of applications of de-icing chemicals to the deck surface. The application rates may either be expressed quantitatively, if the bridge owner keeps such records, or on a qualitative scale. Factors that could be used to help estimate the rate of salt application include the ADT of the roadway and the amount of snowfall the bridge experiences. Typically, bridges with high ADT lie along critical roadways that may receive the focus of local maintenance crews for the application of de-icing chemicals. Obviously, the more frequent the snowfall, the more often de-icing chemicals are likely to be applied. Users may have other data or information regarding the application of de-icing chemicals that can be used to develop rationales identifying those bridges exposed to high levels of de-icing chemicals and those where de-icing chemical use is minimal.

High (more than 100 applications per year)	20 points
Moderate	15 points
Low (less than 15 applications per year)	10 points
None	0 points

L.6 Subjected to Overspray

Reason(s) for Attribute. Overspray refers to the de-icing chemicals on a roadway that are being picked up and dispersed by traveling vehicles onto adjacent highway structures, including bridges and their substructures. Bridges that are located over roadways may receive overspray from the road below. Since overspray typically consists of salt or other de-icing chemicals, more exposure increases the likelihood of developing a corrosion problem.

It is noted that L.6 Subjected to Overspray is explicitly considered to be a separate item from L.5 Rate of De-icing Chemical Application. This because some bridges may not have de-icing chemicals directly applied to their decks, but still can be exposed to overspray from below. An example of this would be a rural road over an interstate. However, to address the more severe condition where de-icing chemicals are applied to the bridge directly *and* by overspray, the items are considered separately.

Assessment Procedure. Similar to the rate of de-icing chemical application, a quantitative estimate of overspray exposure may be difficult. The frequency of de-icing chemical application on the highway that the bridge crosses (if applicable) can be used to aid in estimating the overspray exposure. The vertical clearance of the bridge is also a consideration. For example, a bridge with greater than 20 feet of vertical clearance over the roadway below may experience minimal effects from overspray. In any case, sound engineering judgment should be used. The suggested scoring scheme is based on the generally more significant effect of overspray on steel bridge elements. These suggested scales should be modified appropriately based on local experience.

Concrete Bridge Deck, Prestressed Girder, Substructure

Severe overspray exposure	15 points
Moderate overspray exposure	7 points
Low exposure overspray or not over a roadway	0 points

Steel Girder

Severe overspray exposure	20 points
Moderate overspray exposure	10 points
Low exposure overspray or not over a roadway	0 points

L.7 Remaining Fatigue Life

Reason(s) for Attribute. The remaining fatigue life of an element is somewhat related to the probability of a fatigue crack propagating to the point of brittle fracture. Obviously, for elements that have longer remaining fatigue lives, there is a lower probability of failure due to fatigue cracking than for elements with shorter remaining fatigue lives.

Assessment Procedure. The remaining fatigue life of an element can be determined using any established method. *Insufficient fatigue life* refers to a fatigue life that is less than the required service life or some other interval defined by the owner (e.g., less than 10 years). It is noted that it is possible to calculate a life of less than the length of time the bridge has been in service (i.e., a "*negative fatigue life*"). In many cases, although a negative fatigue life has been calculated, there is no evidence of fatigue cracking on the structure. Although a negative fatigue life does not make physical sense, it does suggest that the probability of failure due to fatigue cracking is greater. In such cases, more in-depth evaluation efforts are justified, such as field testing or monitoring to obtain in-service stress range histograms or a more accurate finite element model of the structure. Often, the more in-depth evaluations reveal that there is significant remaining fatigue life.

Sufficient fatigue life refers to a fatigue life that exceeds the expected service life, or a defined life required by the owner (e.g., 10 years until replacement) of the element, but is not infinite. *Infinite life* is the case in which fatigue cracking is not expected to propagate during the life of the structure. It is noted that a greater penalty is placed on not having any knowledge of the remaining fatigue life than on having performed a fatigue analysis that determined a negative fatigue life.

Unknown remaining fatigue life	10 points
Insufficient remaining fatigue life	7 points
Sufficient remaining fatigue life	3 points
Infinite remaining fatigue life	0 points

L.8 Overtopping/High Water

Reason(s) for Attribute. Certain bridges are susceptible to periodic overtopping or high water condition in which the bridge superstructure is partially or totally immersed in water. Such condition may not adversely affect the loading carrying capacity of the structure; however, this condition may increase the likelihood that A) the structure is impacted by debris or ice in the water, or B) debris is deposited on the flanges and surrounding the bearing areas of the bridge. Impact from debris or ice in the water may increase the likelihood that a certain bridge suffers impact damage, even though the structure is not over a roadway. Debris deposited on the superstructure or at the bearing will retain moisture and may accelerate corrosion damage.

Assessment Procedure. Bridges that are likely to be overtopped during periods of high water are typically documented in the NBIS data submitted annually to the FHWA. Experience may also be used to identify bridges susceptible to the adverse effects of high water. Scoring of this attribute may be different values for conditions A and B.

Periodic overtopping/high water	20 points
No overtopping/high water	0 points

Condition Attributes

C.1 Current Condition Rating

Reason(s) for Attribute. The condition rating for a bridge component describes the existing, in-place bridge as compared with the as-built condition. The condition ratings provide an overall characterization of the general condition of the entire component. It is reasonable to assume that a given element that has already shown signs of damage is more likely to deteriorate to a serious condition than an element showing little or no signs of damage. It is typical for a concrete component with a condition rating of 5 or less to have observable corrosion damage in the form of cracking or spalling (either as open spalls or patched spalls). Such damage provides pathways for the increased penetration of chlorides ions and for increased rates of damage accumulation. For steel elements, low condition ratings are frequently emblematic of significant corrosion damage. Fatigue cracking or member distortions due to unexpected settlement, etc. may be present. Conversely, components with a high condition rating (6 or above) typically have lower levels of existing deterioration. Consequently, some consideration should be given to the overall component rating when assessing the durability of the bridge element.

Assessment Procedure. For this attribute, a condition rating of 5 or less is considered to have a much higher likelihood for accelerated damage than component with higher condition ratings. A condition rating of 6 is considered to have a smaller likelihood of accelerated damage.

Condition rating is 5 or less	20 points
Condition rating is 6	5 points
Condition rating is 7 or greater	0 points

C.2 Current Element Condition State

Reason(s) for Attribute. When element-level inspections are conducted under the *AASHTO Bridge Element Inspection Manual*, element condition states (CS) that are linked to specific evidence of damage or deterioration to the subject bridge element are defined. Elements or portions of elements in CS 1 typically have very little or no evidence of deterioration. Elements or portions of elements in CS 2 have some evidence of damage. As such, it is reasonable to assume that if a given element is entirely in CS 1, the likelihood of severe damage occurring in the near future is lower than an element with portions of the element in CS 2, 3, or 4. This attribute is intended to consider the positive attributes of an element in CS 1.

Assessment Procedure. For this attribute, the current CS for a given bridge element is considered. For elements entirely in CS 1, the scoring of 0 points is suggested, for elements where CS 3 is indicated for any portion of the element, a score of 20 points is suggested. Users may wish to utilize appropriate gradations for elements with conditions indicated as CS 2. The severity and the significance of CS 2 vary by element, and the RAP may wish to develop alternative scoring schemes based on specific elements and CS apportionment. Element-level inspection implementation varies at the owner level, and therefore appropriate scoring should be considered by the RAP according to existing inspection practices.

CS 2 is indicated for a significant portion of the element, or CS 3 is indicated for any portion of the element	20 points
Condition State 2 is indicated for a minor portion of the element	10 points
Condition State 1 is indicated for entire element	0 points

C.3 Evidence of Rotation or Settlement

Reason(s) for Attribute. This attribute is intended to consider the effects of unexpected rotation or settlement of abutments and piers. Use of this attribute is for minor settlements or rotations that do not affect the structural capacity, but may result in atypical or accelerated deterioration patterns. Significant rotations or settlements may require engineering analysis. The rotation of a bridge substructure beyond its design tolerances may result in damage that is manifested by cracking, skewing, and/or misaligned bridge components. Unexpected settlements may result in cracking that provides pathways for intrusion of water and chlorides, leading to accelerated corrosion of reinforcing steel.

Assessment Procedure. Evidence of rotation or settlement should be rated based on their severity using engineering judgment.

Rotation or settlement resulting in cracking of concrete, misaligned joints, or misaligned members	15 points
Minor evidence of rotation or settlement with the potential to result in unexpected cracking or poor joint performance	5 points
No evidence of rotation	0 points

C.4 Joint Condition

Reason(s) for Attribute. The presence of one or more leaking joints will dramatically increase the possibility for corrosion related deterioration on the elements below the deck. This is because joints that are leaking will usually leak chloride-contaminated water directly onto other bridge components such as the superstructure, substructure, and bearing areas. This allows corrosion to initiate and propagate at a faster rate in the affected elements.

Assessment Procedure. This attribute should be rated based on either visual observation or on information contained in bridge inspection reports. For this attribute, the presence of a leaking joint is considered to be severe. If a joint has become debris filled, there is an increased probability that that joint will become damaged and start to leak in the near future. Users should consider historical experience with typical joints in their inventory in evaluating this attribute. For example, if certain typical joint types are expected to have a service life of less than 5 years, it may be appropriate to assume that this joint is a leaking joint, because even if it is not leaking currently, it is expected to leak in near future. Open joints should be expected to allow for the passage of water and debris, and thus should be scored accordingly if this effect is unmitigated. For bridges that are jointless, it is assumed that the bridge is performing as intended and deck drainage is not affecting the bearing areas.

Significant amount of leakage at joints	20 points
Joints have moderate leakage or are debris filled	15 points
Joints are present but not leaking	5 points
Bridge is jointless	0 points

C.5 Maintenance Cycle

Reason(s) for Attribute. This attribute is intended to consider the positive benefits of consistent maintenance and preservation activities on the durability and the reliability of bridge elements. Activities such as deck cleaning, maintenance of drainage, debris removal, washing out joints, and periodic application of the sealers help preserve bridge elements and extend their service lives. Conversely, a bridge that does not receive periodic maintenance and preservation activities is likely to experience damage and deterioration much earlier in its service life, and deteriorate at a higher rate relative to a bridge receiving consistent, periodic maintenance.

Assessment Procedure. This attribute is scored based on the bridge maintenance policies and practices within the particular inventory being assessed. The RAP panel should consider the policies and practices within its state with regard to the intensity of maintenance activities within particular regions, districts, or municipalities. For example, state-owned bridges typically receive more consistent and thorough maintenance than locally-owned bridges. Bridges located in rural areas may receive less intense maintenance than those located near population centers, etc. The RAP should consider specific situations within its bridge inventory when assessing this attribute, and develop criteria for establishing which bridges receive regular maintenance that can be expected to prevent deterioration, and those bridges which do not.

Bridge does not receive routine maintenance	20 points
Some limited maintenance activities	10 points
Bridge is regularly maintained	0 points

C.6 Previously Impacted

Reason(s) for Attribute. If a bridge has been previously struck or impacted by a vehicle, it is reasonable to assume that there is an increased probability of further impact damage. The element could also have been damaged as a result of previous impact, which has been shown to decrease, for example, a steel girder's resistance to brittle fracture (*16*). For concrete bridge elements, impacts can compromise the concrete cover, resulting in the exposure of embedded steel elements. The occurrence of previous impacts should be considered in the analysis for potential impact damage.

Assessment Procedure. This attribute is scored based only on whether or not the bridge has been previously impacted. If the impact risks have been mitigated, this should be considered in the analysis.

Bridge has been previously impacted	20 points
Bridge has not been previously impacted	0 points

C.7 Quality of Deck Drainage System

Reason(s) for Attribute. The purpose of the deck drainage system is to get water, de-icing chemicals, and debris off of the bridge deck effectively, without draining directly onto other elements of the bridge, such as the superstructure and the substructure elements. This attribute is intended to address leakage or deck drainage onto other bridge elements as a result of damage, deterioration, or the ineffective performance of a deck drainage system. Deck drainage systems with ineffective designs would typically be address using attribute D.4 Poor Deck Drainage and Ponding.

Assessment Procedure. This attribute is based on the performance of the drainage system in place on the bridge deck. Since estimating the quality of the drainage system is subjective, it should be based on experience, engineering judgment, and common sense. Some key factors to consider when scoring this attribute include build-up at the deck inlet grates, clogged drains or pipes, section loss in pipes, etc.

Deck drains directly onto superstructure or substructure components, or ponding on deck results from poor drainage	20 points
Drainage issues resulting in drainage onto superstructure or substructure components, or moderate ponding on deck; effects may be localized	10 points
Adequate quality	0 points

C.8 Corrosion-Induced Cracking

Reason(s) for Attribute. This attribute considers the presence of corrosion-induced cracking in concrete bridge elements. Corrosion-induced cracking typically occurs due to the expansion of reinforcing steel caused by the development of corrosion by-products on the surface of the bar. This expansion leads to cracking of the concrete, providing pathways for water and chlorides to penetrate to the reinforcement level. Frequently, this type of cracking is accompanied by rust staining. Such evidence of active corrosion would typically be detected during a typical visual inspection of a bridge. The presence of active corrosion increases the likelihood for corrosion damage to occur to a severe extent in the future.

Assessment Procedure. This attribute is scored based on the presence and the severity of corrosion-induced cracking in concrete bridge elements. The determination of the significance of the cracking should be based on engineering judgment.

Significant corrosion-induced cracking	20 points
Moderate corrosion-induced cracking	10 points
Minor corrosion-induced cracking	5 points
No corrosion-induced cracking	0 points

C.9 General Cracking

Reason(s) for Assessment. This attribute is used to characterize the presence non-structural cracks in concrete. These cracks may result from shrinkage, thermal forces, or other non-structural effects. These cracks can provide pathways for the intrusion of chlorides to the level of the reinforcement. It is generally recognized that cracks perpendicular to the reinforcing bars hasten the corrosion of the intersected reinforcement by facilitating the ingress of moisture, oxygen, and chloride ions. Cracks that follow the line of a reinforcing bar are much more serious, since the length of the bar equal to the length of the crack is exposed to corrosive elements. The presence of cracking also reduces the concrete's ability to contain spalling as the reinforcement corrodes. This attribute is generally used for cracking other than corrosion-induced cracking, which is described in attribute C.8.

Assessment Procedure. The rating of this attribute depends on engineering judgment. More specific guidance to classifying crack sizes and density can be found in the 2010 edition of the *AASHTO Bridge Element Inspection Manual.*

Widespread or severe cracking	15 points
Moderate cracking present	10 points
Minor or no cracking present	0 points

C.10 Delaminations

Reason(s) for Attribute. Delaminations are subsurface cracks in concrete generally parallel to the concrete surface. Delaminations are caused by the formation of horizontal cracking as a result of volumetric expansion of the reinforcing steel during the corrosion process. Delaminations are typically emblematic of the corrosion of embedded steel, and thus provide an early indicator of where future spalling is likely to occur. This attribute is intended to consider that concrete elements with delaminations are more likely to experience deterioration and damage in the future, relative to elements in which delaminations are not present. The detection of delaminations in concrete can reduce the uncertainty in determining if there is active corrosion that is manifesting in damage to the concrete.

This attribute may also be used to characterize conditions for a deck overlay. Under these conditions, delaminations are indicative of a loss of bond between the overlay and the substrate. Overlays that are debonding are likely to deteriorate more rapidly than an overlay with good bonding characteristics.

It is implied that some form of NDE has been conducted to address this attribute, as delaminations are not visibly detectable. This typically includes hammer sounding or chain drag, but may include other techniques such as infrared thermography, impact echo, or other methods.

Assessment Procedure. This attribute is scored based on inspection results that indicate the extent of delaminations present in a given concrete element. This attribute should be scored based on the amount of surface area of the structure that includes delaminations. Suggested values for the significant levels of delamination are indicated below.

Significant amount of delaminations present (greater than 20% by area) or unknown	20 points
Moderate amount of delaminations present (5% to 20% by area)	10 points
Minor, localized delaminations (less than 5% by area)	5 points
No delaminations present	0 points

C.11 Presence of Repaired Areas

Reason(s) for Attribute. Repaired spalls and patches are a way to temporarily seal reinforcement exposed as a result of damaged concrete. However, even though the reinforcement is again sealed from the environment, the existing corrosion can continue to propagate. Patches frequently have a relatively short service life, especially when traffic loading is high.

The service life of deck patches ranges from 4 years to 10 years (*17*), although an FHWA TechBrief indicates that the service life of a patch ranges from 4 years to only 7 years (*18*). The service life of the patch depends largely on the corrosivity of the surrounding concrete and the development of the halo effect. When concrete is contaminated with chlorides in concentrations greater than the threshold level in the area surrounding the patches, inadvertent acceleration of the corrosion rate can occur. The patched area acts as a large non-corroding site (i.e., cathodic area) adjacent to corroding sites (i.e., anodic areas), and thus corrosion cells are created.

Assessment Procedure. The presence of repaired areas should be scored based on the total surface area of the bridge that has repaired areas. Engineering judgment should be exercised. If the repaired areas result from impact damage or other non-corrosion–related damage, and chlorides levels for the intact concrete are expected to be nominal, a reduced score may be assigned.

Significant amount of repaired areas	15 points
Moderate amount of repaired areas	10 points
Minor amount of repaired areas	5 points
No repaired areas	0 points

C.12 Presence of Spalling

Reason(s) for Attribute. This attribute is intended to consider the presence of spalling on concrete bridge elements. Open spalls are sections of concrete that have separated from the larger mass of concrete and fallen off of the structure, usually exposing the underlying reinforcement. Unrepaired spalling allows corrosive elements to directly contact the exposed reinforcement and prestressing steel, if present. This will lead to accelerated rates of corrosion damage in the area surrounding the spall.

Users may wish to include repaired spalls under this attribute, or utilize the attribute C.11 Presence of Repaired Areas.

Assessment Procedure. This attribute is scored based on the severity and the extent of spalling as reported in bridge inspection reports. Users should consider the importance of the spalling in terms of the structural performance of the element under consideration in developing their scoring methodology. Spalling that leads to the exposure of prestressing strands is considered significantly more important than spalling in a reinforced element exposing the mild steel bars.

Significant spalling (greater than 10% of area with spalling, rebar or strands exposed)	20 points
Moderate spalling (greater than 1 inch deep or 6 inches in diameter or exposed reinforcement)	15 points
Minor spalling (less than 1 inch deep or 6 inches in diameter)	5 points
No spalling present	0 points

C.13 Efflorescence/Staining

Reason(s) for Attribute. This attribute is intended to consider the increased likelihood of corrosion damage associated with the presence of efflorescence on the surface of concrete elements. Efflorescence is a white stain on the face of a concrete component which results from the crystallization of dissolved salts. While efflorescence is typically considered an aesthetic problem, it may be indicative of a problem with the concrete mix and may contribute to corrosion initiation. Efflorescence on the soffit of a bridge deck typically indicates that water is passing freely through the deck, likely carrying with it chlorides that may cause corrosion of the reinforcing steel. When rust stains are present, the corrosion of reinforcing steel is assured.

Extensive leaching causes an increase in the porosity and the permeability of the concrete, thus lowering the strength of the concrete and making it more vulnerable to hostile environments (e.g., water saturation and frost damage, or chloride penetration and the corrosion of embedded steel). Those concretes that are produced using a low water-cement ratio, adequate cement content, proper compaction, and curing are the most resistant to leaching that results in efflorescence on the surface of the concrete (*19*).

Assessment Procedure. This attribute is scored based on inspection results. The scoring for this attribute is based on the existence of efflorescence stains and whether or not rust stains have also been deposited from corroding reinforcement.

Moderate to severe efflorescence with rust staining; severe efflorescence without rust staining	20 points
Moderate efflorescence without rust staining	10 points
Minor efflorescence	5 points
No efflorescence	0 points

C.14 Flexural Cracking

Reason(s) for Attribute. When the primary load-bearing members in a concrete bridge exhibit flexural cracking, it may indicate that the members were either inadequately designed for the required loading, that overloads have occurred, or that deterioration has occurred that has reduced the load-bearing capacity of the members. In any case, large flexural cracks can be indicative of an inadequate load-bearing capacity that may require an engineering analysis in order to determine the cause of the cracking and the resulting effect on the load capacity of the structure. As a result, bridges exhibiting moderate to severe flexural cracking should be screened from the general reliability assessment unless appropriate engineering analysis indicates that the cracking is benign. Flexural cracking in a prestressed element is generally more significant than in a reinforced concrete element.

In cases where flexural cracking is minor or appropriate assessment has indicated that the cracking is not affecting the adequate load capacity of the element, the cracking nonetheless may provide pathways for the ingress of moisture and chlorides that may cause corrosion of the embedded steel. This attribute is intended to consider the increased likelihood of corrosion resulting from the cracking in the concrete.

Assessment Procedure. Flexural cracks will typically present themselves with a vertical orientation either near the bottom flange at mid-span or near the top flange over intermediate supports, if the member is continuous.

Engineering judgment must be exercised in determining whether any present flexural cracking is moderate to severe. Crack widths in reinforced concrete bridges exceeding 0.006 inches to 0.012 inches reflect the lower bound of "moderate cracking." The American Concrete Institute Committee Report 224R-01 (*1*) presents guidance for what could be considered reasonable or tolerable crack widths at the tensile face of reinforced concrete structures for typical conditions. These range from 0.006 inches for marine or seawater spray environments to 0.007 inches for structures exposed to de-icing chemicals, to 0.012 inches for structures in a humid, moist environment.

In prestressed concrete bridge structural elements, tolerable crack width criteria have been adopted in the PCI MNL-37-06 *Manual for the Evaluation and Repair of Precast Prestressed Concrete Bridge Products* (*20*). The PCI Bridge Committee recommends that flexural cracks greater in width than 0.006 inches should be evaluated to affirm adequate design and performance.

Note that this attribute is a companion to the screening attribute S.4 Flexural Cracking, in which any moderate to severe flexural cracking should exclude the bridge from a risk-based assessment unless appropriate engineering analysis has been completed showing that the cracking is benign or has been repaired. Generally, cracking in prestressed elements is more problematic than cracking in reinforced concrete elements.

Crack widths equal to or less than 0.006 inches to 0.012 inches, depending on environment for reinforced concrete; crack widths equal to or less than 0.006 inches for prestressed concrete	10 points
No flexural cracking	0 points

C.15 Shear Cracking

Reason(s) for Attribute. Similar to flexural cracking, if the primary load-bearing members in a concrete bridge exhibit shear cracking, it can be assumed that the members were either inadequately designed for the required loading or that deterioration has occurred, which has reduced the load-bearing capacity of the members. In either case, large shear cracks can be indicative of an inadequate load-bearing capacity, which may require an engineering analysis in order to determine the cause of the cracking and the resulting effect on the load capacity. As a result, bridges exhibiting moderate to severe shear cracking should be screened from the reliability assessment unless appropriate engineering analysis indicates that the cracking is benign in terms of the load-bearing capacity.

Assessment Procedure. Engineering judgment must be exercised in determining the severity of any present shear cracking. Shear cracks will typically present themselves with a roughly 45 degree diagonal orientation and will radiate towards the mid-span of the member for conventionally reinforced concrete. For prestressed concrete, angles down to roughly 30 degrees may be observed. The ends of the member and any sections located over piers should be checked for this type of cracking. Note that this attribute is a companion to the screening attribute S.5 Shear Cracking, where any moderate to severe flexural cracking should exclude the bridge from a risk-based assessment until adequate assessments have been conducted.

Minor, hairline to less than 0.0625 inch shear cracking	10 points
No shear cracking	0 points

C.16 Longitudinal Cracking in Prestressed Elements

Reason(s) for Attribute. This attribute is for the assessment of prestressed concrete bridge elements. Longitudinal cracking in prestressed elements can be indicative of the corrosion or the fracture of the embedded prestressing strands. As a result, elements with reported longitudinal cracking in the soffit, web, or flange should be individually assessed to determine the source of the cracking and to assess the condition of the prestressing strands (*21*).

Assessment Procedure. Longitudinal cracking in prestressed elements can be indicative of strand corrosion and damage, and, as such, significant longitudinal cracking is a screening attribute. The use of longitudinal cracking in prestressed elements as a condition attribute assumes the cracking in question is minor in nature, and significant strand corrosion is not currently present. In this case, the longitudinal cracking provides pathways for the intrusion of moisture and chlorides to the prestressing strands and the mild steel bars. As a result, a prestressed element with minor longitudinal cracking is more likely to experience deterioration and damage than an uncracked element. This attribute is scored based on inspection results.

Minor longitudinal cracking in beam soffit	15 points
No longitudinal cracking in beam soffit	0 points

C.17 Coating Condition

Reason(s) for Attribute. This attribute considers the effect of the coating condition on the likelihood of corrosion damage occurring in steel bridge elements. Coatings are applied to steel elements to provide protection from corrosion and for aesthetic reasons. Elements with coatings in good condition, and performing as intended, are generally less susceptible to corrosion damage. Elements with significant rusting and corrosion in areas in which that paint system has failed are more likely to experience further corrosion damage in the future.

Assessment Procedure. Depending on the condition of the coating, the likelihood of corrosion damage varies. Coatings typically deteriorate more rapidly where drainage from the bridge deck is allowed to flow onto the steel surface. As a result, conditions for the accelerated corrosion of steel may already exist. If the coating is already in poor condition, the likelihood of severe corrosion damage is greater than for a coating in good condition. If the element is constructed with weathering steel (assuming it is placed in the proper environment and is detailed correctly), it should be scored as though the coating is in good condition. The development of an effective patina for the weathering steel should be confirmed.

Coating system in very poor condition, limited or no effectiveness for corrosion protection, greater than 3% rusting	10 points
Coating system is in poor condition, 1% to 3% rusting, substantially effective for corrosion protection	5 points
Coating is in fair to good condition, effective for corrosion protection	0 points

C.18 Condition of Fatigue Cracks

Reason(s) for Attribute. Active fatigue cracks due to primary stress ranges will continue to grow until the failure of the member, either by brittle or by ductile fracture. An arrested or repaired fatigue crack is better than having an active crack, but it is still worse than having no crack at all, as it suggests that the conditions necessary for cracking to initiate were or still may be present in the structure. In other words, other similar details (that have not been preemptively retrofitted) may be susceptible to cracking in the future.

Assessment Procedure. To determine whether or not a fatigue crack is arrested, a comparison must be made between previous inspection reports. In order to be considered arrested, a crack must have not grown in a specified amount of time (e.g., the inspection interval plus one year). It is noted that although no fatigue cracks may have been observed, a detail still may be highly susceptible to fatigue. Hence, other attributes such as D.16 Element Connection Type, D.17 Worst Fatigue Detail Category, and L.1 ADTT are included in the assessment procedure to address the susceptibility to cracking.

Fatigue crack exists and is active/unknown	20 points (see S.7)
Fatigue crack exists and has arrested or been retrofitted	10 points
No fatigue cracks are present	0 points

C.19 Presence of Fatigue Cracks Due to Secondary or Out-of-Plane Stress

Reason(s) for Attribute. Fatigue cracks due to secondary or out-of-plane stresses are the most common type of fatigue cracks found on highway bridges. Most of these cracks occur due to incompatibility or relative movement between bridge components.

Assessment Procedure. The scoring for this attribute is based on the existence or non-existence of fatigue cracks. Some common types of fatigue cracks due to secondary stresses include web-gap cracks, deck plate cracking in orthotropic bridge decks, and floor beam connections.

Fatigue cracks are present and are active/unknown	15 points
Fatigue cracks are present but have been arrested or have been retrofitted	5 points
No fatigue cracks are present	0 points

C.20 Non-Fatigue-Related Cracks or Defects

Reason(s) for Attribute. This attribute refers to steel bridge elements that may be susceptible to fatigue-induced cracking. Fatigue cracks generally start from some initial crack or defect. As a result of this, fatigue and brittle fracture is less likely if there are no cracks or defects from which cracks can propagate.

Assessment Procedure. This attribute should be scored based on whether or not cracks or other defects are found in the element. Previous inspection reports should be used when evaluating this attribute.

Non-fatigue-related cracks or defects are present	10 points
Non-fatigue-related cracks or defects are not present	0 points

C.21 Presence of Active Corrosion

Reason(s) for Attribute. The presence of visible active corrosion on steel bridge elements indicates that severe corrosion damage in the future is possible, since the environment and the bridge features are vulnerable to the initiation and the propagation of corrosion. It is also well known that corrosion damage typically propagates at an accelerated rate, once initiated, and that elements that show no signs of active corrosion are very unlikely to develop severe corrosion damage during the assessment interval of 72 months. Maximum rates of section loss under the most severe marine conditions typically do not exceed 10 mils/year (0.010 inches/year). For moderate conditions, rates are typically on the order of 4 mils/year (0.004 inches/year) or less.

Corrosion damage that is inactive is explicitly distinguished from corrosion that is active. For example, section loss on a girder web that was the result of a leaking expansion joint that was corrected (the joint was replaced *and* the girder was repainted), may be assumed to have inactive corrosion. It is assumed that the owner has determined that the existing section loss is either insignificant or has taken it into account in the rating procedures and that load posting, if needed, is in place.

Assessment Procedure. This attribute should be scored based on the amount of active corrosion present on the element. Engineering judgment should be used in determining whether or not the corrosion is active. This attribute may also be used as a screening tool in a reliability assessment.

Significant amount of active corrosion present	20 points
Moderate amount of active corrosion present	15 points
Minor amount of active corrosion present	7 points
No active corrosion present	0 points

C.22 Presence of Debris

Reason(s) for Attribute. The presence of debris on bridge elements can substantially increase the probability of corrosion damage by maintaining a moisture-rich environment on the surface of the steel. Debris can be especially damaging if it is allowed to remain on the bridge without maintenance action, such as washing or cleaning. This attribute is intended to characterize bridges susceptible to having debris deposited on the flanges, bearings, connections, or other details that results in atypical (e.g., accelerated) deterioration patterns.

Assessment Procedure. This attribute should be assessed based on if debris is present or likely to be present on the element, resulting in an atypical deterioration pattern.

Debris is or is likely to be present	15 points
Debris not likely to be present	0 points

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APPENDIX F

Illustrative Examples

F 1 Introduction

This section provides three illustrative examples of applying reliability-based analysis to establish an inspection interval and strategy. The first is an example of a bridge constructed with a superstructure composed of prestressed girders, the second example is a bridge with a multi-girder steel superstructure, and the third example is a multi-girder reinforced concrete superstructure. The RAP assembled by a bridge owner would typically conduct this analysis. For these examples, typical attributes that could be identified by a RAP have been selected for illustrative purposes. Attribute scoring sheets are shown to illustrate the process of applying a numerical scoring process for identified attributes to estimate the reliability of bridge elements, and to develop rationale for determining the appropriate inspection interval.

In the examples shown, Occurrence Factor (OF) categories were determined by applying the following equation:

$$X = \frac{\sum S_i}{\sum S_o} * 4$$

Where S_i is the score recorded for each attribute and S_o is the maximum score for each attribute, such that the ratio $\sum S_i / \sum S_o$ is a value between 0 and 1. OFs were then applied such that values of X between 0 and <1 were identified as "Remote," values 1 or greater but less than 2 "Low," etc. This provides a simple methodology for ranking bridges according to their important attributes that contribute to the durability and reliability of the bridge, and estimating the appropriate OF. This scoring methodology should be calibrated by the RAP for its specific bridge inventory to ensure results are consistent with sound engineering judgment.

The examples also describe the Consequence Factors that were selected for each bridge, along with the rationale for selection. Based on these results, an appropriate inspection interval is identified for each bridge based on the risk matrix (Figure C1). The IPN for each damage mode is also calculated to illustrate how the process prioritizes damage modes to support inspection procedures for that bridge.

F 2 Example 1: Prestressed Concrete Bridge

F 2.1 Bridge Profile

F 2.1.1 Overview

This example bridge is constructed of prestressed girders with a composite concrete deck (Figure F1). The bridge has a typical reinforced concrete deck, seven prestressed AASHTO Type IV girders, and a reinforced concrete substructure. The bridge was constructed in 2006. Epoxy-coated reinforcement has been used in the deck and in parts of the prestressed girders. The substructure contains regular, uncoated reinforcement. The rate of de-icing chemical application is moderate, and the environment is also moderate. The reported ADTT is 210 vehicles. An element-level inspection had been conducted on the bridge, and data from the element-level inspection including inspector notes were used in determining values for the condition attributes. All elements were rated 100% in Condition State (CS) 1.

Figure F1. Elevation view of Example Bridge 1.

F 2.1.2 Concrete Bridge Deck

The deck for this structure was cast-in-place and constructed with normal concrete and epoxy-coated rebar. From the design plans, the concrete cover for the top of the deck is 1-½ inches. Asphaltic plug joints in the deck are in good condition.

Some transverse cracks, spaced 2 to 3 feet apart, have been noted on the underside of the deck. Efflorescence is present near these cracks, though there is no rust staining. No other damage has been observed. The current condition rating is 7-Good Condition, based on the most recent inspection.

F 2.1.3 Prestressed Girders

The superstructure of this bridge consists of 7 AASHTO Type IV prestressed concrete girders. There is at least 2 inches of clear cover for all surfaces as determined from the design plans, and the mild reinforcing is epoxy coated. No sealers or coatings have been applied to the girders. The maximum span length is 99 feet. The superstructure has no observed spalling or cracking and was most recently rated as being condition 8-Very Good Condition.

F 2.1.4 Substructure

The substructure was constructed of normal concrete with uncoated carbon steel reinforcement. The minimum design cover was determined to be 2 inches. Water from the deck does not contact the substructure either through the drainage system or through the joints. There are no observed signs of cracking or spalling. No evidence of unusual rotation or settlement has been noted, and the bridge is founded on rock. The substructure is rated to have a condition rating of 8-Very Good Condition based on the most recent inspection report.

F 2.2 Assessment

This section will show how the methodology is applied to determine the OFs, the Consequence Factors, and the corresponding inspection intervals for this bridge. A detailed scoring of each damage mode will be presented with written descriptions of how the consequence of damage was considered. The results are then summarized in a table that provides the maximum inspection interval based on the risk matrix and the IPN determined from the analysis.

The primary elements of this bridge are a concrete bridge deck, prestressed concrete girders, piers, and abutments. For the concrete bridge deck element, the RAP identified typical damage modes of widespread corrosion-induced cracking and spalling. Since each of these damage mode results from the effects of corrosion, these damage modes were combined into a single damage mode named "Corrosion Damage."

For the prestressed concrete girders, the RAP identified the following damage modes:

- Bearing Area Damage,
- Corrosion Between Beam Ends,
- Flexural and Shear Cracking, and
- Strand Fracture.

For the substructure, the damage mode considered was:

- Corrosion Damage (cracking and spalling due to the effects of corrosion).

Considering the damage modes identified for each element, attributes relating to each damage mode were identified and ranked, as described in the Guideline. The following sections contain illustrative examples of attribute scoring sheets developed for the different elements and damage modes for the bridge and the estimated OFs based on the attribute scoring.

F 2.2.1 Concrete Bridge Deck

The RAP determined that certain attributes of a bridge deck that contribute to the likelihood of corrosion damage are common and well known, and that these same attributes would generally apply to other bridge decks in its inventory, as well as other typical concrete elements. Additionally, because corrosion will affect most concrete elements and associated damage modes, repetition of certain common attributes could be reduced by having a single corrosion profile for an element. This corrosion profile could then be applied to all damage modes stemming from corrosion for a given element more efficiently. As such, a corrosion profile was developed to assess the corrosion-resistance characteristics of a concrete bridge deck or other concrete element. This profile included typical attributes that were well known to affect the durability of concrete, but did not depend on the current condition or individual characteristics of an element. The attributes identified included:

- Poor Deck Drainage and Ponding,
- Years of Construction,
- Application of Protective Systems,
- Concrete Mix Design,
- Minimum Concrete Cover,
- Reinforcement Type,
- Exposure Environment,
- Rate of De-icing Chemical Application, and
- Maintenance Cycle.

Supporting rationale for each of these attributes from the commentary (Appendix E) was used. Utilizing these corrosion profile attributes and the suggested rankings in the commentary, the RAP developed a simple scoring sheet to calculate the corrosion profile for a bridge deck as shown in the table below.

Corrosion Profile, Concrete Bridge Deck	
Attribute	Score
D.4 Poor Deck Drainage and Ponding • The deck drainage system is of modern design and is effective	0
D.6 Year of Construction • Bridge constructed in 2006	0
D.7 Application of Protective Systems • Protective systems never applied to deck	10
D.8 Concrete Mix Design • Constructed of normal grade concrete, no admixtures	15
D.11 Minimum Concrete Cover • Design cover is 1.5 inches	10
D.12 Reinforcement Type • Epoxy-coated reinforcement used	0
L.3 Exposure Environment • Deck environment is moderate	10
L.5 Rate of De-icing Chemical Application • Rate of de-icing chemical application is moderate	15
C.5 Maintenance Cycle • Bridge receives regular, periodic maintenance	0
Corrosion Profile score	**60 out of 140**

Attributes were identified by the RAP that affected the reliability and durability of a bare concrete deck. These attributes include the corrosion profile score, plus attributes based on the loading and the condition of a particular deck. The RAP identified screening criteria of the Current Condition Rating and Fire Damage for concrete bridge decks, to identify decks that may require further assessment. Other attributes of bare concrete decks were identified and ranked. The scoring plan was then applied to the subject concrete deck.

Corrosion Damage, Concrete Bridge Deck	
Attribute	Score
S.1 Current Condition Rating • Current deck condition rating is greater than 4	Pass
S.2 Fire Damage • No fire damage in the past 12 months	Pass
Corrosion Profile score	60
L.1 ADTT • ADTT is moderate (210 vehicles)	10
C.1 Current Condition Rating • Current deck condition rating is 7	0
C.8 Corrosion-Induced Cracking • Minor corrosion-induced cracking noted	5

Corrosion Damage, Concrete Bridge Deck	
Attribute	Score
C.9 General Cracking • No general cracking observed	0
C.10 Delaminations • No delaminations found	0
C.11 Presence of Repaired Areas • No repaired areas	0
C.12 Presence of Spalling • No spalling noted	0
C.13 Efflorescence/Staining • Minor efflorescence without rust observed	5
Corrosion Damage total	**80 out of 290**
Corrosion Damage ranking	**1.1 Low**

This bridge deck is still relatively new, was built to modern standards for durability and corrosion resistance, and has very little damage accumulation. As a result, the deck received very low scores for the attributes identified. Based on the attribute score, the RAP estimated that the likelihood of the failure for the deck (based on the criteria described in Section 2.1) in the next 72 months was low, i.e., the OF was *Low* (OF = 2).

F 2.2.2 Prestressed Girder

For the assessment of a prestressed girder, the corrosion profile scoring model was also used. As with the corrosion profile for bridge decks, this basic profile can be applied across many concrete elements. In this case, the prestressed girder scored the same as the deck.

Corrosion Profile, Prestressed Girder	
Attribute	Score
D.4 Poor Deck Drainage and Ponding • The deck drainage system is of modern design and is effective	0
D.6 Year of Construction • Bridge constructed in 2006	0
D.7 Application of Protective Systems • Protective systems never applied	10
D.8 Concrete Mix Design • Constructed of normal grade concrete	15
D.11 Minimum Concrete Cover • Minimum concrete cover is 2 inches	10
D.12 Reinforcement Type • Reinforcement is epoxy coated	0
L.3 Exposure Environment • Superstructure environment is moderate	10
L.5 Rate of De-icing Chemical Application • Rate of salt application is moderate	15

Corrosion Profile, Prestressed Girder	
Attribute	Score
C.5 Maintenance Cycle • Bridge receives regular, periodic maintenance	0
Corrosion Profile point total	**60 out of 140**

The RAP then considered the identified damage modes for a prestressed girder element, identified and ranked attributes, and applied the scoring model for each damage mode as shown below.

Bearing Area Damage, Prestressed Girder	
Attribute	Score
Corrosion Profile score	60
D.1 Joint Type • Bridge contains a closed joint system	0
C.4 Joint Condition • Joints are not leaking	5
C.8 Corrosion-Induced Cracking • No corrosion-induced cracking noted	0
C.9 General Cracking • No general cracking observed	0
C.11 Presence of Repaired Areas • No repaired areas	0
C.12 Presence of Spalling • No areas of spalling noted	0
Bearing Area Damage point total	**65 out of 240**
Bearing Area Damage ranking	**1.08 Low**

Corrosion Between Beam Ends, Prestressed Girder	
Attribute	Score
Corrosion Profile score	60
C.8 Corrosion-Induced Cracking • No corrosion-induced cracking noted	0
C.10 Delaminations • No delaminations found	0
C.11 Presence of Repaired Areas • No repaired areas	0
C.12 Presence of Spalling • No spalling present	0
C.13 Efflorescence/Staining • No signs of efflorescence	0

Corrosion Between Beam Ends, Prestressed Girder	
Attribute	Score
Corrosion Between Beam Ends point total	**60 out of 235**
Corrosion Between Beam Ends ranking	**1.02 Low**

Flexural/Shear Cracking, Prestressed Girder	
Attribute	Score
S.4 Flexural Cracking • No flexural cracking	Pass
S.5 Shear Cracking • No shear cracking	Pass
D.2 Load Posting • Bridge is not load posted	0
L.4 Likelihood of Overload • Likelihood of overload is low	0
C.14 Flexural Cracking • No flexural cracking	0
C.15 Shear Cracking • No shear cracking	0
Flexural/Shear Cracking point total	**0 out of 55**
Flexural/Shear Cracking ranking	**0 Remote**

Strand Fracture, Prestressed Girder	
Attribute	Score
S.1 Current Condition Rating • Superstructure condition rating is greater than 4	Pass
S.6 Longitudinal Cracking in Prestressed Elements • Significant cracking is not present	Pass
Corrosion Profile score	60
L.6 Subjected to Overspray • Bridge not over a roadway, not exposed to overspray	0
C.1 Current Condition Rating • Superstructure condition rating is 8	0
C.4 Joint Condition • Joints are present but not leaking	5
C.8 Corrosion-Induced Cracking • No corrosion-induced cracking noted	0
C.10 Delaminations • No delaminations found	0
C.11 Presence of Repaired Areas • No repaired areas	0
C.12 Presence of Spalling • No spalling present	0

Strand Fracture, Prestressed Girder	
Attribute	Score
C.16 Longitudinal Cracking in Prestressed Elements • No longitudinal cracking in the girders	0
Strand Fracture point total	**65 out of 285**
Strand Fracture ranking	**0.91 Remote**

Based on the attributes identified by the RAP, the OF for the bearing area damage and corrosion between the beam ends was estimated to be *Low* (OF = 2). For the damage modes of shear cracking, flexural cracking and strand fracture, the OF was *Remote* (OF = 1).

F 2.2.3 Substructure

For the piers and abutments, the RAP considered that the most likely damage modes were corrosion-induced cracking and spalling, or a settlement or rotation of one of the substructure elements. However, settlement and rotations were determined to not be relevant damage modes because the bridge substructure is founded on rock. To estimate the likelihood for the corrosion damage mode, the panel once again used the generalized corrosion profile scoring. The panel then considered appropriate attributes for estimating the OF for the corrosion damage mode, identified and ranked key attributes, and scored the piers and abutments for the bridge, as shown below.

Corrosion Profile, Substructure	
Attribute	Score
D.4 Poor Deck Drainage and Ponding • Deck does not drain onto the substructure	0
D.6 Year of Construction • Bridge constructed in 2006	0
D.7 Application of Protective Systems • Protective systems never applied	10
D.8 Concrete Mix Design • Substructure constructed with normal grade concrete	15
D.11 Minimum Concrete Cover • Minimum design cover is 2 inches	10
D.12 Reinforcement Type • Reinforcement is uncoated carbon steel	15
L.3 Exposure Environment • Environment is rated as moderate	10
L.5 Rate of De-icing Chemical Application • Rate of de-icing chemical application is moderate	15
C.5 Maintenance Cycle • Bridge receives regular, periodic maintenance	0
Corrosion Profile point total	**75 out of 140**

Corrosion Damage—Piers and Abutments, Substructure	
Attribute	Score
Corrosion Profile score	75
C.1 Current Condition Rating • Current substructure condition rating is 8	0
C.4 Joint Condition • Joints present but not leaking	5
C.8 Corrosion-Induced Cracking • No corrosion-induced cracking noted	0
C.9 General Cracking • No cracking observed	0
C.10 Delaminations • No delaminations found	0
C.11 Presence of Repaired Areas • No repaired areas present	0
C.12 Presence of Spalling • No spalling noted	0
C.13 Efflorescence/Staining • No signs of efflorescence	0
Corrosion Damage point total	**80 out of 290**
Corrosion Damage ranking	**1.10 Low**

Based on the attribute scoring, the OF for the damage mode of "Corrosion Damage" was assessed to be *Low* (OF = 2).

F 2.3 Consequence Assessment

Once the likelihood for each damage mode has been ranked, the RAP must perform a consequence analysis for each damage mode considered. For the concrete bridge deck, based on the damage mode of corrosion damage, the RAP considered the scenario of significant spalling of the deck as a result of extensive corrosion damage. Since the bridge is over a non-navigable waterway, spalling of concrete from the soffit would have a low consequence. Considering the ADT and the posted speed limit, spalling on the deck surface was determined to have only a moderate effect on serviceability for the bridge and a planned repair. The consensus of the RAP was that the appropriate Consequence Factor was *Moderate* (CF = 2). The RAP's consequence assessment will be included in the file for the bridge.

For the prestressed girder superstructure, in order to determine the consequence of failure, the RAP considered the scenario that one of the prestressed beams lost 100% of its load carrying capacity due to the damage modes of strand fracture, flexural and shear cracking, or corrosion between the beam ends. The RAP reviewed data from two very similar bridges for which truck impacts severely damaged one or more of the prestressed girders. The RAP determined that these two bridges could be considered "very similar" as their span lengths were within 10% of the bridge under consideration, and had nearly identical girder spacing and deck configuration. In both cases, the impact severely damaged at least one of the girders such that its load carrying capacity was effectively reduced to 0. The bridges exhibited little or no additional dead load deflection and were capable of carrying normal live loads. Temporary barriers were installed to shift traffic away from the shoulder area above the fascia girders that were damaged. Further, the load rating information for this bridge was reviewed and the bridge possessed a capacity far in excess of the required Inventory and Operating ratings. Hence, the RAP concluded that the loss of one girder would at most have a *Moderate* (CF = 2) consequence based on the following rationale:

- The bridge is redundant, based on AASHTO definitions;
- The bridge is very similar to other bridges for which a member failure has occurred, but did not result in collapse of the bridge or excessive deflection;
- The bridge capacity far exceeds required Inventory and Operating ratings;
- The bridge has low ADT, such that there will not be a major impact on traffic; and
- The bridge is located over a non-navigable stream. Thus, the risks to people or property under the bridge are minimal.

For the damage mode of bearing area damage, two scenarios were considered. The first scenario considered that the bearing area damage was sufficient to result in a downward displacement of the bridge deck. The most likely consequence was assessed by the panel to be *Moderate*, because such a displacement would result in only moderate disruption of service and require a planned repair. This was based on the rationale that the deck is composite with the superstructure and the bridge is a multi-girder bridge with normal beam spacing, such that any displacement would be minor and localized in nature, because loads could transfer to adjacent girders and the composite deck would limit displacements. The second scenario considered was that the bearing area damage resulted in severe cracking in the shear area of the beam, resulting in damage to the development length of the strands or shear cracking. The RAP considered that such a scenario would, at worst, result in 100% loss in load carrying capacity, as was considered for the damage modes of strand fracture, flexural or shear cracking, and corrosion between the beams ends in the previous scenario. Based on these two scenarios, the CF of *Moderate* (CF = 2) was selected for this damage mode. The RAP's consequence assessment will be included in the file for the bridge.

For the reinforced concrete substructure, the RAP considered the scenario that there was widespread corrosion damage (cracking and spalling) to the piers and abutments. The bridge is over a small creek, and hence there is little concern of injury from spalling concrete. The piers and abutments are short. Past experience of the panel with many piers and abutments of similar characteristics indicated that serious corrosion damage has a benign immediate effect on serviceability and safety. Therefore, the consensus of the panel was that the appropriate consequence category was *Low* (CF = 1).

The data from the RAP assessment was then applied to the appropriate risk matrix (Figure C1) to determine the maximum inspection interval for the bridge. A summary of the scoring and maximum inspection interval for the bridge are shown below.

F 2.4 Scoring Summary

Table F1 shows a summary of the analysis for this bridge. The maximum inspection interval based on the RAP analysis was determined to be 72 months, based on the low likelihood of serious damage (failure) to the elements of the bridge, and the moderate consequences associated with that damage.

F 2.5 Criteria for a Family of Bridges

The RAP assessed that it has many bridges in its inventory of very similar design characteristics. Based on the key attributes developed by the RAP, the panel identified a series of criteria to apply to a family of bridges to extend this analysis to other bridges in its inventory. These criteria describe bridges of the same design type and characteristics, with similarly adequate load ratings, and similar environmental loading. Condition attributes were mapped to suitable surrogates in the element-level bridge inspection data that were being collected for the bridge. For example, for the prestressed concrete girders, the panel identified that the individual condition attributes identified

Table F1. Reliability assessment scoring summary for Example Bridge 1.

Element	Damage	Occurrence Factor (OF)	Consequence Factor (CF)	Maximum Interval	OF x CF (IPN)
Deck	Corrosion Damage	Low (2)	Moderate (2)	72 months	4
Prestressed Girders	Bearing Area Damage	Low (2)	Moderate (2)	72 months	4
	Corrosion Between Beam Ends	Low (2)	Moderate (2)	72 months	4
	Flexural/Shear Cracking	Remote (1)	Moderate (2)	72 months	2
	Strand Fracture	Remote (1)	Moderate (2)	72 months	2
Substructure	Corrosion Damage	Low (2)	Low (1)	72 months	2

by the analysis, such as shear or flexural cracking, corrosion-induced cracking, spalling, or efflorescence, were either not present or minimal if the CS ratings for the element were CS 1 or CS 2. Therefore, for the prestressed girder element, elements that are rated as CS 1 and CS 2 would not have the damage characteristics the panel identified as key to the potential for serious damage to develop. Bridges with any portion of the prestressed element rated as CS 3 would likely have one or more of these condition attributes present, and therefore would require reanalysis and possibly a reduced inspection interval. Similar criteria were developed for each of the elements assessed by the RAP.

The RAP also identified that "longitudinal cracking in prestressed elements" was a key condition attribute not adequately represented in its element-level inspection scheme. As a result, the RBI procedure for bridges in this family needed to include a requirement that longitudinal cracking be assessed during the inspection. This requirement was included in the RBI procedure as a special emphasis area for this family of bridges.

The RAP developed a listing of criteria, including design characteristics and using surrogate element data for certain condition attributes, to apply to the overall family of similar bridges in its inventory. These criteria are based on the engineering assessment documented through the RAP analysis. Example criteria to identify the family of bridges included:

- Maximum span length less than 120 feet;
- Four or more AASHTO prestressed girders;
- Beam spacing of 10 feet or less;
- ADTT less than 1000;
- Constructed in 1995 or later;
- No structural element with CS 3 reported;
- No joint element with CS 3 reported;
- Load rating exceeds requirements;
- No significant flexural, shear, or longitudinal cracking in the prestressed element; and
- Bridge receives RBI-based inspections.

The RAP determined that bridges meeting these criteria will be treated as a family under the RBI methodology. If a particular bridge violates any of these criteria, it must be reassessed according to the attribute scoring criteria developed for this family of bridges.

Table F2 summarizes the information from the RAP analysis to be included in the RBI procedure for these bridges. Longitudinal cracking in the prestressed elements is indicated as a special emphasis area for the inspection, to ensure this key damage mode is assessed during subsequent inspections. Other IPNs for identified damage modes are low, indicating a standard RBI inspection is required for the bridge.

Table F2. Table of information to be included in the RBI procedure.

Maximum Inspection Interval: 72 months		
Special Emphasis Items		
S.6 Longitudinal Cracking in Prestressed Elements		
RBI Damage Modes		
Element	**Damage Mode**	**IPN**
Deck	Corrosion Damage	4
Prestressed Girder	Bearing Area Damage	4
	Corrosion Between Beam Ends	4
	Flexural/Shear Cracking	2
	Strand Fracture	2
Substructure	Corrosion Damage	2

F 3 Example 2: Steel Girder Bridge

F 3.1 Bridge Profile

F 3.1.1 Overview

This example bridge carries a state highway over a non-navigable river. The bridge was constructed in 1954 with a continuous steel girder superstructure, a non-composite reinforced concrete deck, and a reinforced concrete substructure (Figure F2). All steel reinforcement used in this bridge is regular uncoated mild carbon steel. The observed ADTT is 130 vehicles. The rate of salt application is determined to be high by the RAP, with more than 100 applications of de-icing chemicals per year. The exposure environment is considered moderate.

F 3.1.2 Concrete Bridge Deck

The reinforced concrete bridge deck was constructed of cast-in-place normal concrete. From the design plans, the minimum cover was determined to be 1-9⁄16 inches. The deck has a bituminous wearing surface of unknown thickness which was assessed to be in fair condition. In some locations the wearing surface has come off the deck. No membranes or sealers have been applied. The deck has no reported drainage or ponding problems.

Figure F2. Elevation view of Example Bridge 2.

The most recent inspection rated the deck condition as 6-Satisfactory. According to the inspection report, the underside of the deck has hairline transverse cracks, spaced 2 to 3 feet apart, with efflorescence stains. The underside of the approach span at abutment 1 has heavy efflorescence stains on the left side.

F 3.1.3 Steel Girders

The continuous steel girder superstructure is constructed from four painted steel girders with steel diaphragms. These girders are riveted at the connection plates. No problems were found at the connection plates during a recent in-depth inspection. The bottom flanges of the girders have corrosion with missing paint. These locations have some pack rust formation. The superstructure was assessed to have a condition rating of 6-Satisfactory.

Based on the inspection report, no fatigue or fracture related damage is present. Based on the provided design plans, it was determined that the girders are riveted built-up members, so the worst fatigue detail category is D.

F 3.1.4 Substructure

The substructure was constructed of normal grade reinforced concrete with uncoated carbon steel reinforcement. The minimum cover was determined to be 3-⅜ inches. Drainage from the deck is leaking onto the substructure from the deck due to leaking joints.

There is no observed evidence of rotation or settlement. The concrete piers have random hairline cracks with some moderate surface scaling below the high water line. Hairline to 1⁄32 inch (0.03125 inch) diagonal and vertical cracks with minor efflorescence stains have been reported on the concrete abutments. The concrete pier caps have some hairline cracks but appear to be in good condition. There is spalling in the concrete piers exposing rebar. The substructure condition was assessed to be 6-Satisfactory.

F 3.2 Assessment

The primary elements of this bridge are a concrete bridge deck with an asphalt overlay, riveted steel girders, deck joints, piers, and abutments. For the concrete bridge deck element the typical damage modes identified were concrete cracking and spalling. Since each of these damage modes results from the effects of corrosion, these damage modes were again grouped into a single damage mode termed "Corrosion Damage." The same corrosion profile as developed for the previous example was used for the deck. The asphalt overlay for the deck was assessed individually for debonding and spalling/potholes. For the steel girders, the damage modes considered were:

- Corrosion Damage,
- Fatigue Damage, and
- Fracture Damage.

For the substructure, the damage mode considered was:

- Corrosion Damage (cracking and spalling due to the effects of corrosion).

The RAP determined through consensus that tilting of the piers or unexpected settlement were not credible damage modes. This was based on the rationale that the bridge had been in service for more than 50 years without any signs of tilt or rotation, the geographic area was not susceptible to subsurface erosion or unexpected settlements, and the roller bearings were insensitive to moderate displacements of the substructure.

F 3.2.1 Concrete Bridge Deck

The concrete deck was assessed for the damage mode of corrosion damage, using the corrosion profile for concrete elements and attributes identified for the deck, as shown below.

Corrosion Profile, Concrete Bridge Deck	
Attribute	Score
D.4 Poor Deck Drainage and Ponding • No drainage problems noted	0
D.6 Year of Construction • Bridge constructed in 1954	6
D.7 Application of Protective Systems • Protective systems never applied to deck	10
D.8 Concrete Mix Design • Constructed of normal grade concrete, no admixtures	15
D.11 Minimum Concrete Cover • Design cover is between 1.5 inches and 2.5 inches	10
D.12 Reinforcement Type • Uncoated carbon steel reinforcement	15
L.3 Exposure Environment • Deck environment is moderate	10
L.5 Rate of De-icing Chemical Application • Rate of de-icing chemical application is high (100 times per year)	20
C.5 Maintenance Cycle • Maintenance cycle is at least limited	10
Corrosion Profile score	**96 out of 140**

Corrosion Damage, Concrete Bridge Deck	
Attribute	Score
S.1 Current Condition Rating • Current deck condition rating is greater than 4	Pass
S.2 Fire Damage • No fire damage in the past 12 months	Pass
Corrosion Profile score	96
L.1 ADTT • ADTT is minor (130 vehicles)	5
C.1 Current Condition Rating • Current deck condition rating is 6	5
C.8 Corrosion-Induced Cracking • Minor corrosion-induced cracking noted	5
C.9 General Cracking • No general cracking observed	0
C.10 Delaminations • Unknown—Asphalt overlay prevents effective sounding	20
C.11 Presence of Repaired Areas • No repaired areas	0
C.12 Presence of Spalling • No spalling noted	0

Corrosion Damage, Concrete Bridge Deck	
Attribute	Score
C.13 Efflorescence/Staining • Moderate efflorescence without rust observed	10
Extent of Damage total	**141 out of 290**
Corrosion damage ranking	**1.94 Low**

Based on the attributes identified by the RAP, the OF for corrosion damage was assessed to be *Low* (OF = 2).

F 3.2.2 Asphalt Overlay

The asphalt overlay was assessed by the panel using a simple expert elicitation. The general consensus of the panel was that the typical service life of an asphalt overlay was less than 10 years. The RAP agreed that the likelihood of failure of the asphalt overlay was greater than 1% over a 72-month interval, given that the overlay was already in service. The OF for the overlay failure was determined to be *High* (OF = 4) by consensus of the panel.

F 3.2.3 Steel Girders

The steel girders were assessed for three damage modes: Fatigue Damage, Corrosion Damage, and Fracture Damage. Key attributes were identified by the RAP as shown below. Supporting data and rationale for each attribute are included in the commentary.

Fatigue Damage, Steel Girder	
Attribute	Score
S.7 Active Fatigue Cracks due to Primary Stress Ranges • No active fatigue cracks due to primary stress	Pass
D.6 Year of Construction • Bridge was built in 1954	20
D.16 Element Connection Type • Element is connected by rivets	7
D.17 Worst Fatigue Detail Category • Worst fatigue detail category is D	15
L.1 ADTT • ADTT is 130 vehicles	15
L.7 Remaining Fatigue Life • Remaining fatigue life is unknown	10
C.18 Condition of Fatigue Cracks • No fatigue cracks present	0
C.19 Presence of Fatigue Cracks due to Secondary or Out-of-Plane Stress • No fatigue cracks due to secondary or out of plane stress	0
Fatigue Damage point total	**67 out of 110**
Fatigue Damage ranking	**2.44 Moderate**

Corrosion Damage, Steel Girder	
Attribute	Score
S.9 Significant Level of Active Corrosion or Section Loss • Active corrosion present is not alarming	Pass
D.5 Use of Open Decking • Bridge does not have an open deck	0
D.13 Built-Up Member • Element is built up	15
D.15 Constructed of Weathering Steel • Element not constructed with weathering steel	10
L.3 Exposure Environment • Exposure environment is moderate	10
L.5 Rate of De-icing Chemical Application • Rate of de-icing chemical application is high (100 times per year)	20
L.6 Subjected to Overspray • Superstructure is not subjected to overspray	0
C.4 Joint Condition • Joints are moderately leaking	15
C.7 Quality of Deck Drainage System • Drainage system is of adequate quality	0
C.17 Coating Condition • Element is painted, with steel exposed on bottom flanges	10
C.21 Presence of Active Corrosion • Significant active corrosion is present	20
C.22 Presence of Debris • Element has no debris	0
Corrosion Damage point total	**100 out of 190**
Corrosion Damage ranking	**2.1 Moderate**

Fracture Damage, Steel Girder	
Attribute	Score
S.7 Active Fatigue Cracks due to Primary Stress Ranges • No active fatigue cracks due to primary stress	Pass
S.8 Details Susceptible to Constraint-Induced Fracture • No details susceptible to constraint induced fracture	Pass
D.3 Minimum Vertical Clearance • Bridge is not over a roadway, max vertical clearance	0
D.6 Year of Construction • Bridge constructed in 1954	20
D.14 Constructed of High Performance Steel • Element is not constructed of HPS/unknown	10
L.1 ADTT • ADTT is 130 vehicles	15

Fracture Damage, Steel Girder	
Attribute	Score
L.7 Remaining Fatigue Life • Remaining fatigue life is unknown	10
C.6 Previously Impacted • Bridge has not been impacted before	0
C.19 Presence of Fatigue Cracks due to Secondary or Out-of-Plane Stress • No fatigue cracks present	0
C.20 Non-Fatigue-Related Cracks or Defects • No fatigue cracks present	0
Fracture Damage point total	**55 out of 125**
Fracture Damage ranking	**1.76 Low**

The RAP analysis of key attributes for the damage modes indicated that the steel superstructure has a moderate likelihood of fatigue damage (OF = 3), a moderate likelihood of developing corrosion damage (OF = 3), and a low likelihood of fracture (OF = 2).

F 3.2.4 Substructure

The substructure was assessed for the damage mode of corrosion damage, using the corrosion profile for concrete elements and attributes identified for the piers and abutments.

Corrosion Profile, Substructure	
Attribute	Score
D.4 Poor Deck Drainage and Ponding • No drainage problems noted	0
D.6 Year of Construction • Bridge constructed in 1954	6
D.7 Application of Protective Systems • Protective systems have not been applied	10
D.8 Concrete Mix Design • Substructure constructed with normal grade concrete, no admixtures	15
D.11 Minimum Concrete Cover • Minimum design concrete cover is 3-3⁄8″	0
D.12 Reinforcement Type • Reinforcement is uncoated carbon steel	15
L.3 Exposure Environment • Exposure environment is moderate	10
L.5 Rate of De-icing Chemical Application • Rate of de-icing chemical application is high (100 times per year)	20
C.5 Maintenance Cycle • Maintenance cycle is at least limited	10
Corrosion Profile point total	**86 out of 140**

Corrosion Damage—Piers and Abutments, Substructure	
Attribute	Score
Corrosion Profile score	86
C.1 Current Condition Rating • Current substructure condition rating is six	5
C.4 Joint Condition • Joints are significantly leaking onto substructure	20
C.8 Corrosion-Induced Cracking • Moderate corrosion-induced cracking noted	10
C.9 General Cracking • Presence of minor general cracking	5
C.10 Delaminations • Minor localized delaminations on footings	5
C.11 Presence of Repaired Areas • No repaired areas present	0
C.12 Presence of Spalling • Significant spalling with exposed reinforcement present on piers	20
C.13 Efflorescence/Staining • Moderate efflorescence without rust staining	10
Substructure Elements point total	**161 out of 290**
Substructure Elements ranking	**2.22 Moderate**

Based on the attribute scoring, the RAP estimated the OF was *Moderate* (OF = 3) for corrosion damage for the piers and abutments. A considerable amount of damage has already accumulated in the form of spalling with exposed reinforcement and moderate cracking.

F 3.3 Consequence Assessment

Since the bridge carries a state highway over a non-navigable river, key damage to the bridge deck is likely to be in the form of spalling on the riding surface of the bridge deck. The most likely consequence of severe damage to the deck is *Moderate* (CF = 2) because there may be some disruption of service or reduction in posted speed. The bridge is a four girder bridge with typical girder spacing, such that even a through-thickness punch-through is likely to be local in nature and not represent a high consequence. The assignment of a moderate consequence is based on common experience with bridge decks of similar design characteristics.

The consequence of the asphalt overlay failing was determined to be *Low*, because failure of the asphalt overlay was a maintenance need and would not necessitate increased inspection or monitoring.

The superstructure consists of four steel girders with diaphragms spaced at 20 to 25 feet. Although fatigue damage is the most likely damage mode, the worst outcome associated with fatigue would be the fracture of one of the girders. Hence, the consequence scenario evaluated was the fracture of one of the girders. Note that this analysis does not depend on the damage failure mode, thus, failure could also be due to corrosion. As stated, the cross section is made up of four identical built-up members. In evaluating the most likely consequence, the RAP identified several similar designs where full-depth fractures of steel girders occurred. These bridges had spans greater than or equal to this bridge, had similar skew, had similar girder spacing, and had a non-composite deck. In all cases, none of the bridges collapsed, though some displayed minor sagging. The bridges carried full service load up until the time that fracture was detected in later inspections. Hence, the RAP determined that the consequence associated with fracture of one of the girders should be set as *High* (CF = 3) based on the following rationale:

- The bridge is redundant, based on AASHTO definitions;
- The bridge is very similar to other bridges where full-depth girder fractures occurred, but did not result in collapse of the bridge or excessive deflection;
- The bridge meets required Inventory and Operating ratings;
- Fracture in a member will have a major impact on travel, since the member failure would result in a lane closure; and
- The bridge is located over a non-navigable river. Thus, the risks to people or property under the bridge are minimal.

The RAP's consequence assessment will be included in the bridge file along with appropriate references to the other bridges cited in the consequence scenario evaluation.

Engineering calculations showing that the effects of a girder fracture would result in a Moderate consequence (CF = 2) would be required to reduce the consequence category for this scenario. Based on the above and the fact that the bridge is not fracture-critical, the consequence category of Severe was not considered a plausible outcome for girder fracture.

For the substructure, the scenario considered for damage to the piers and abutments of the bridge was severe corrosion damage and spalling. The most likely consequence of this scenario is a Low consequence (OF = 1), because severe corrosion damage of this type would typically require monitoring and assessment, but would not affect the serviceability of the bridge.

The summary of the RAP assessment is shown in Table F3. Based on this assessment, the maximum inspection interval for this bridge is 24 months, due to the likelihood and high consequence associated with the development of fatigue cracking. This is due to in part to the fact that the bridge has fatigue-prone details (category D), the bridge was constructed before modern fracture control requirements were in place, and there is truck traffic on the bridge. Even though the bridge has not developed any fatigue cracks in more than 50 years of service, the rational assessment performed by the RAP indicates that the potential for cracking exists, and should be treated appropriately. Additionally, the bridge is susceptible to serious corrosion damage, because its current condition includes active corrosion, the applications of de-icing chemical are high, the members are built up, and the joints are leaking. As such, the required maximum interval for an RBI is 24 months.

F 3.4 Scoring Summary

The scoring summary for this bridge is shown in Table F3. Based on the reliability assessment, the maximum inspection interval was determined to be 24 months.

Table F3. Reliability assessment scoring summary for Example Bridge 2.

Element	Damage	Occurrence Factor (OF)	Consequence Factor (CF)	Interval	OF x CF (IPN)
Deck	Corrosion Damage	Low (2)	Moderate (2)	72 months	4
Overlay	Debonding/Spalling	High (4)	Low (1)	48 months	4
Steel Girders	Fatigue	Moderate (3)	High (3)	24 months	9
	Corrosion	Moderate (3)	High (3)	24 months	9
	Fracture	Low (2)	High (3)	48 months	6
Substructure	Corrosion Damage	Moderate (3)	Low (1)	48 months	3

Table F4. Table of information to be included in the RBI.

Maximum Inspection Interval: 24 Months		
Special Emphasis Items		
S.7 Active Fatigue Cracks due to Primary Stress Ranges S.9 Significant Level of Active Corrosion or Section Loss		
RBI Damage Modes		
Element	**Damage Mode**	**IPN**
Deck	Corrosion Damage	4
Steel Girder	Fatigue Cracking	9
	Corrosion Damage	9
	Fracture	6
Substructure	Corrosion Damage	3

F 3.5 Inspection Data

Table F4 summarizes the information from the RAP analysis to be included in the RBI procedure to be used for this bridge. The identified screening criteria for fatigue cracking due to primary stresses and significant section loss are included as special emphasis items. The data in the table also indicates that fatigue cracking and corrosion damage are priority items for inspection of the steel girders, based on their IPN of 9. Because of the high IPN for corrosion damage, the RAP recommends utilizing an ultrasonic thickness gauge (UT-T) to assess the areas of section loss in the steel girder. This will ensure accurate reporting of the remaining section and mitigate the risks associated with severe section loss, which was identified through the RAP analysis as being moderately likely to occur, and resulting in a high consequence. For fatigue cracking, the high IPN number will prioritize fatigue cracking for the inspection team conducting the RBI on the bridge. The IPN of 9 indicates to the inspector that the bridge has the potential for fatigue cracking, and the consequences of that cracking are potentially high were it to go undetected. The RAP could recommend NDE, such as Magnetic Particle Testing (MT) or Dye Penetrant Testing (PT), be applied to a sampling of locations during periodic inspections to ensure that fatigue cracking is detected and enhance the reliability of the inspection.

F 4 Example 3: Reinforced Concrete Bridge

F 4.1 Bridge Profile

F 4.1.1 Overview

This example bridge is a typical, simply-supported three-span reinforced concrete bridge with a bare cast-in-place deck. The bridge owner's inventory includes more than 100 bridges of similar span length and design characteristics, and, as such, is developing the RAP analysis for application to a family of bridges, using this bridge as an example of the family. The specific bridge was constructed in 1963 and carries highway traffic over a local road. The estimated ADT on the bridge is 22,000 vehicles, while the ADT on the local road under the bridge is 60 vehicles. Both the rate of salt application and the surrounding environment are considered to be moderate. A photograph of the bridge is shown in Figure F3.

F 4.1.2 Concrete Bridge Deck

For this bridge, the deck was constructed with normal grade cast-in-place concrete and uncoated mild steel reinforcement. The asphalt has been removed from the top of the deck and a water proof sealant has been applied. Hairline to 1⁄16-inch cracks have been observed on the top of the deck near the abutments. Hairline diagonal cracks with efflorescence stains have been observed on the soffit of the deck near the abutments. No delaminations or spalling are noted on the deck.

Figure F3. Elevation view of Example Bridge 3.

From the design plans, the minimum cover was determined to be 1-13⁄16 inches. Based on the most recent inspection report, the deck is considered to be in CS 6-Satisfactory. This deck contains concrete edge joints with silicon sealant. The seals are considered to be in good condition but are leaking water. No other ponding or drainage issues are noted.

F 4.1.3 Reinforced Concrete Girders

The superstructure for this bridge consists of seven reinforced concrete girders that are constructed from normal grade concrete and uncoated mild steel reinforcement. Each girder, per span, has hairline vertical flexure cracking. The right exterior girder has a spall on the bottom end which measures 12 inches tall by 3 inches wide by 5 inches deep due to impact.

One of the exterior girders has an 8-inch diameter spall resulting from an over-height vehicle collision. Girders five and six also have scrapes and spalls from an over-height vehicle collision. The superstructure is considered to be in CS 5-Fair. From the design plans, the minimum concrete cover is 3-5⁄8 inches.

F 4.1.4 Substructure

The substructure for this bridge is also constructed of normal grade concrete with uncoated mild steel reinforcement. From the design plans, the minimum cover was determined to be 2-½ inches. The columns have random hairline cracks and the top of column four has an area of delamination that is 29 inches tall by 21 inches wide. Both abutments have hairline to 1⁄16-inch vertical cracks and spalling with exposed reinforcement on their right sides.

All bents have water staining resulting from leaking joints. Bent cap one, span one, has horizontal cracks with delamination in the bottom left corner. Bent cap two, span two, has an area of cracking and delamination that is 16 inches wide by 8 inches tall near girder six. Bent cap two, span three, also has an area of cracking and delamination that is 27 inches wide by 4 inches tall near girder six.

The substructure has neoprene pad bearings which have curled on the ends but are still in satisfactory condition. The overall condition rating for the substructure is "5-Fair." There are no signs of settlement or rotation and the substructure itself is founded on rock.

F 4.2 Assessment

The primary elements of this bridge are a concrete bridge deck, reinforced concrete girders, and piers and abutments. For the concrete bridge deck element, the typical damage mode identified was corrosion damage (concrete cracking and spalling). The same corrosion profile developed for the previous examples was also used for this deck. For the reinforced concrete girders, the damage modes considered were:

- Bearing Area Damage,
- Corrosion Between Beam Ends, and
- Flexural and Shear Cracking.

Based on the owner's inventory data and experience, there has been no occurrences of significant shear cracking in bridges of similar design to the one being analyzed. However, there have been isolated cases of cracking due to flexural stresses, possibly resulting from overloaded trucks. Based on this experience, the RAP determines that flexural cracking is an important damage mode, while the likelihood of shear cracking is more remote, generally. To provide focus on the flexural cracking experience in this particular inventory, the RAP determines that shear cracking and flexural cracking should be separated into distinct damage modes. Additionally, the RAP determined through consensus that the likelihood of overload would have the greatest influence on the likelihood of flexural cracking progressing; existing flexural cracking had moderate effect, and the fact that bridge may be load posted has only a small effect. As such the RAP assigns 20 points to L.4, Likelihood of Overload, only 10 points to D.2, Load Posting and 15 points to C.14, Flexural Cracking. The key attributes for flexural cracking were therefore determined by the RAP to be as follows:

- S.4 Flexural Cracking (screening criteria),
- D.2 Load Posting,
- L.4 Likelihood of Overload, and
- C.14 Flexural Cracking.

The screening criteria for Flexural Cracking (S.4) was also utilized to identify bridges with significant flexural cracking, which may require individual engineering assessment. For shear cracking, the relevant attributes identified by the RAP were:

- S.5 Shear Cracking (screening),
- D.2 Load Posting,
- L.4 Likelihood of Overload, and
- C.15 Shear Cracking.

Again, the screening attribute S.5 for unresolved shear cracking is utilized to identify any bridges with shear cracking that may require engineering assessment.

For the substructure, the damage mode considered was:

- Corrosion Damage (cracking and spalling due to the effects of corrosion).

F 4.2.1 Concrete Bridge Deck

The concrete deck was assessed for the damage mode of corrosion damage, using the corrosion profile for concrete elements and attributes identified for the deck, as shown below

Corrosion Profile, Concrete Bridge Deck	
Attribute	Score
D.4 Poor Deck Drainage and Ponding • No drainage problems noted	 0

Corrosion Profile, Concrete Bridge Deck	
Attribute	Score
D.6 Year of Construction • Bridge constructed in 1963	6
D.7 Application of Protective Systems • Waterproof penetrating sealer applied, frequency unknown	5
D.8 Concrete Mix Design • Constructed of normal grade concrete, no admixtures	15
D.11 Minimum Concrete Cover • Design cover is between 1.5 inches and 2.5 inches	10
D.12 Reinforcement Type • Uncoated carbon steel reinforcement	15
L.3 Exposure Environment • Deck environment is moderate	10
L.5 Rate of De-icing Chemical Application • Rate of de-icing chemical application is moderate	15
C.5 Maintenance Cycle • Maintenance cycle is at least limited	10
Corrosion Profile score	**86 out of 140**

Corrosion Damage, Concrete Bridge Deck	
Attribute	Score
S.1 Current Condition Rating • Current deck condition rating is greater than four	Pass
S.2 Fire Damage • No fire damage in the past 12 months	Pass
Corrosion Profile score	86
L.1 ADTT • ADTT is high (5,500 vehicles)	20
C.1 Current Condition Rating • Current deck condition rating is 6	5
C.8 Corrosion-Induced Cracking • Moderate corrosion-induced cracking noted	10
C.9 General Cracking • Moderate general cracking observed	10
C.10 Delaminations • No delaminations noted	0
C.11 Presence of Repaired Areas • No repaired areas	0
C.12 Presence of Spalling • No spalling noted	0

Corrosion Damage, Concrete Bridge Deck	
Attribute	Score
C.13 Efflorescence/Staining • Minor efflorescence without rust observed	5
Corrosion Damage total	**136 out of 290**
Corrosion Damage ranking	**1.88 Low**

Based on the attributes identified by the RAP, the OF for corrosion damage in the deck was estimated as *Low* (OF = 2).

F 4.2.2 Reinforced Concrete Girders

The reinforced concrete girders were assessed for the damage modes of bearing area damage, corrosion between the beam ends, and flexural and shear cracking.

Corrosion Profile, Reinforced Concrete Girder	
Attribute	Score
D.4 Poor Deck Drainage and Ponding • No drainage problems noted.	0
D.6 Year of Construction • Bridge constructed in 1963	6
D.7 Application of Protective Systems • Protective systems never applied	10
D.8 Concrete Mix Design • Constructed of normal grade concrete	15
D.11 Minimum Concrete Cover • Minimum concrete cover is greater than 2.5 inches	0
D.12 Reinforcement Type • Reinforcement is uncoated mild steel	15
L.3 Exposure Environment • Superstructure environment is moderate	10
L.5 Rate of De-icing Chemical Application • Rate of salt application is moderate	15
C.5 Maintenance Cycle • Bridge maintenance is at least limited	10
Corrosion Profile point total	**81 out of 140**

Bearing Area Damage, Reinforced Concrete Girder	
Attribute	Score
Corrosion Profile score	81
D.1 Joint Type • Bridge has closed joints	0

Bearing Area Damage, Reinforced Concrete Girder	
Attribute	Score
C.4 Joint Condition • Joints are leaking but sealant is still in fair condition	15
C.8 Corrosion-Induced Cracking • No corrosion-induced cracking noted	0
C.9 General Cracking • No general cracking observed	0
C.11 Presence of Repaired Areas • No repaired areas	0
C.12 Presence of Spalling • Moderate spalling in several locations, no exposed reinforcement noted.	15
Bearing Area Damage point total	**111 out of 240**
Bearing Area Damage ranking	**1.85 Low**

Corrosion Between Beam Ends, Reinforced Concrete Girder	
Attribute	Score
Corrosion Profile score	81
C.1 Current Condition Rating • Current condition rating is 5	20
C.6 Previously Impacted	20
C.8 Corrosion-Induced Cracking • No corrosion-induced cracking noted	0
C.9 General Cracking • No general cracking observed	0
C.10 Delaminations • Unknown	20
C.11 Presence of Repaired Areas • No repaired areas	0
C.12 Presence of Spalling • Moderate spalling in several locations (due to impact), no exposed reinforcement noted	15
C.13 Efflorescence/Staining • No signs of efflorescence	0
Corrosion Between Beam Ends point total	**156 out of 290**
Corrosion Between Beam Ends ranking	**2.15 Moderate**

Flexural Cracking, Reinforced Concrete Girder	
Attribute	Score
S.4 Flexural Cracking • Hairline flexural cracking noted, determined to be benign	Pass

Flexural Cracking, Reinforced Concrete Girder	
Attribute	Score
D.2 Load Posting • Bridge is not load posted	0
L.4 Likelihood of Overload • Likelihood of overload is moderate	10
C.14 Flexural Cracking • Hairline flexural cracking noted	15
Flexural Cracking point total	**25 out of 45**
Flexural Cracking ranking	**2.22 Moderate**

Shear Cracking, Reinforced Concrete Girder	
Attribute	Score
S.5 Shear Cracking • No shear cracking present	Pass
D.2 Load Posting • Bridge is not load posted	0
L.4 Likelihood of Overload • Likelihood of overload is moderate	10
C.15 Shear Cracking • No shear cracking	0
Shear Cracking point total	**10 out of 45**
Shear Cracking ranking	**0.88 Remote**

The attribute scoring indicated an OF of *Moderate* (OF = 3) for corrosion between beam ends and flexural cracking, an OF of *Low* (OF = 2) for bearing area damage, and an OF of *Remote* (OF = 1) for shear cracking.

F 4.2.3 Substructure

The substructure was assessed for the damage mode of corrosion damage, using the corrosion profile for concrete elements and attributes identified for the piers and abutments.

Corrosion Profile, Substructure	
Attribute	Score
D.4 Poor Deck Drainage and Ponding • No drainage problems noted	0
D.6 Year of Construction • Bridge constructed in 1963	6
D.7 Application of Protective Systems • Protective systems have not been applied	10
D.8 Concrete Mix Design • Substructure constructed with normal grade concrete, no admixtures	15

Corrosion Profile, Substructure	
Attribute	Score
D.11 Minimum Concrete Cover • Minimum design concrete cover is 2-½ inches	0
D.12 Reinforcement Type • Reinforcement is uncoated carbon steel	15
L.3 Exposure Environment • Exposure environment is moderate	10
L.5 Rate of De-icing Chemical Application • Rate of de-icing chemical application is moderate	15
C.5 Maintenance Cycle • Maintenance cycle is at least limited	10
Corrosion Profile point total	**81 out of 140**

Corrosion Damage—Piers and Abutments, Substructure	
Attribute	Score
Corrosion Profile score	81
C.1 Current Condition Rating • Current substructure condition rating is five	20
C.4 Joint Condition • Joints are moderately leaking onto substructure	15
C.8 Corrosion-Induced Cracking • Localized cracking near delaminations noted	5
C.9 General Cracking • Presence of moderate general cracking	10
C.10 Delaminations • Unknown	20
C.11 Presence of Repaired Areas • No repaired areas present	0
C.12 Presence of Spalling • Moderate spalling with exposed reinforcement present	15
C.13 Efflorescence/Staining • No efflorescence noted	0
Concrete Elements point total	**166 out of 290**
Concrete Elements ranking	**2.28 Moderate**

Based on their analysis, the RAP assessed that the likelihood of failure due to corrosion damage was moderate for the pier and abutments (OF = 3). Already, a considerable amount of damage has accumulated in the form of localized delaminations and spalling resulting in exposed reinforcement.

F 4.3 Consequence

For the concrete bridge deck, the RAP considered the scenario that the corrosion damage in the deck resulted in spalling of either the driving surface of the deck or deck soffit. In this case, the bridge carries a high-volume highway over another, lower-volume roadway. The roadway on the bridge carries 22,000 vehicles a day, and the roadway below the bridge carries 60 vehicles a day. Based on this information, any spalling from the deck soffit has the potential to fall into the roadway below and strike a motorist. However, given the low traffic volume and speed on the roadway below, the RAP considered the likelihood of this occurring to be relatively small. Therefore, the consensus of the RAP was that the appropriate Consequence Factor was *High* (CF = 3). For spalling of the riding surface, the panel determined that such a scenario was likely to have an effect on serviceability of the deck, and may require a reduction in the posted traffic speed. Therefore, the consensus of the RAP was that this represented a Consequence Factor of *Moderate* (CF = 2). For this case, the scenario of concrete falling into the roadway below the bridge provides the Consequence Factor for corrosion damage in the bridge deck.

To determine the Consequence Factor for the concrete beams, the RAP considered the scenario that one of the reinforced concrete beams lost 100% of its load carrying capacity due to corrosion damage between the beam ends, flexural or shear cracking, or bearing area damage. The RAP considered that the superstructure is reinforced concrete with a composite deck such that redundancy in the structure would prevent the total collapse of a girder. The RAP also reviewed data from two very similar bridges for which corrosion damage had resulted in loss of load carrying capacity in one girder of a multi-girder, reinforced concrete bridge with a composite deck. The RAP determined that these two bridges could be considered "very similar" to the bridge being analyzed because their span lengths were within 10% of the bridge under consideration and they utilized a nearly identical girder spacing and deck configuration. In both cases, the corrosion damage had reduced a single girder's load carrying capacity effectively to zero, however, the bridge exhibited little or no additional dead load deflection and was capable of carrying normal live loads. Lane closures were required on the bridges as the result of the faulted girder, resulting in a significant impact on traffic.

The load rating information for the bridge was reviewed and the bridge possessed a capacity far in excess of the required Inventory and Operating ratings. However, the bridge carries a high ADT, such that a lane closure would have a major impact on traffic. Additionally, the roadway under the bridge is a low-volume road that may be impacted by the shoring required or debris. As a result, the Consequences Factor was determined to be *High* (CF = 3) based on the following rationale:

- The bridge is redundant, based on AASHTO definitions;
- The bridge is very similar to other bridges for which a member failure has occurred, but did not result in collapse of the bridge or excessive deflection;
- The bridge capacity far exceeds required Inventory and Operating ratings;
- The bridge has high ADT, such that there could be a major impact on traffic; and
- The bridge is located over a low-volume roadway such that there would be some risks to traffic on the roadway below.

For the damage mode of bearing area damage, two scenarios were considered. The first scenario considered that the bearing area damage was sufficient to result in a downward displacement of the bridge deck. The most likely consequence was assessed by the panel to be *Moderate* (CF = 2), resulting in only a minor disruption of service, since the deck is composite with the superstructure and it is a multi-girder bridge with normal beam spacing. The second scenario considered was that the bearing area damage resulted in severe cracking in the shear area of the beam, resulting in a loss of load-carry capacity. As above, the Consequence Factor of *High* (CF = 3) was assigned.

Table F5. Reliability assessment scoring summary for Example Bridge 3.

Element	Damage	Occurrence Factor (OF)	Consequence Factor (CF)	Maximum Interval	OF x CF (IPN)
Deck	Corrosion Damage	Low (2)	High (3)	48 months	6
Reinforced Concrete Girders	Bearing Area Damage	Low (2)	High (3)	48 months	6
	Corrosion Between Beam Ends	Moderate (3)	High (3)	24 months	9
	Flexural Cracking	Moderate (3)	High (3)	24 months	9
	Shear Cracking	Remote (1)	High (3)	48 Months	3
Substructure	Corrosion Damage	Moderate (3)	High (3)	24 months	9

For the reinforced concrete substructure, areas of delaminations are present in several locations and both abutments have areas of spalling with exposed reinforcement. Here, the most likely damage mode will result in spalling of the concrete. The RAP considers that this bridge is located over a roadway, and the piers are immediately adjacent to the roadway such there is a chance that concrete spalling off of a pier could strike a passing motorist. Based on this factor, the consequence scenario for this damage mode was assessed to be *High* (CF = 3).

F 4.4 Scoring Summary

Table F5 shows a summary of the scoring for this bridge. Based on the likelihood of corrosion damage between the beam ends, flexural cracking and corrosion damage to the substructure, and the associated consequences, the maximum inspection interval was determined to be 24 months.

F 4.5 Inspection Data

Table F6 summarizes the information from the RAP analysis to be included in the RBI procedure for this bridge. This information includes the identified screening criteria of inspection for flexural cracking as a special emphasis item for the RBI inspection.

Based on the RAP assessment, this particular bridge has high IPNs for corrosion between the beam ends (9), corrosion damage in the substructure (9), and corrosion damage in the deck (6),

Table F6. Table of information to be included in RBI procedure.

Maximum Inspection Interval: 24 Months		
Special Emphasis Items		
S.4 Flexural Cracking		
RBI Damage Modes		
Element	**Damage Mode**	**IPN**
Deck	Corrosion Damage	6
Reinforced Concrete Girder	Bearing Area Damage	6
	Corrosion Damage between the beam ends	9
	Flexural Cracking	9
	Shear Cracking	3
Substructure	Corrosion Damage	9

each of which have the potential to result in debris falling into the roadway below the bridge. As a result, the RAP determined that enhanced inspection for corrosion damage was needed as part of the RBI procedure. Available technologies to complete the delamination survey include hammer sounding, infrared thermography (IR) and Impact Echo (IE). The RAP recommends delamination surveys be completed during the periodic inspections to mitigate the risk of debris falling into the roadway below the bridge unexpectedly.

Flexural cracking also has a high IPN, indicating that this damage mode is of high importance and needs to be prioritized during subsequent RBIs for the bridge. Flexural cracking is included as a special emphasis item for subsequent inspections.

PART II

Final Research Report: Developing Reliability-Based Inspection Practices

SUMMARY

Final Research Report: Developing Reliability-Based Inspection Practices

Introduction

The National Bridge Inspection Standards (NBIS) mandate the frequency and methods used for the safety inspection of highway bridges. The inspection frequency specified in the National Bridge Inventory (NBI) is calendar-based and generally requires routine inspections to be conducted at a maximum interval of 24 months. The calendar-based inspection interval applied uniformly across the bridge inventory results in the same inspection interval for new bridges as for aging and deteriorated bridges. Such a uniform inspection practice does not recognize that a newly constructed bridge, with improved durability characteristics and a few years of exposure to the service environment, may be much less likely to develop serious damage over a given time period than an older bridge that has been exposed to the service environment for many years. As such, inspection needs may be less for the newer bridge, and greater for the aging structure, relative to the uniform interval currently applied. Bridges that are in benign, arid operating environments are currently inspected at the same interval as bridges in aggressive marine environments, in which significant damage from corrosion may develop much more rapidly, resulting in increased inspection needs. Further, bridges that are known to possess “good” characteristics or details are treated the same as those with characteristics or details known to perform poorly. Current practices make it difficult to recognize if the same or improved safety and reliability could be achieved by varying inspection methods and frequencies to meet the needs of a specific bridge, based on its design, condition, and operational environment. A more rational approach to inspection planning would better match inspection requirements to inspection needs through reliability-based analysis that considers the design, materials, condition, and operational environment of a bridge.

As such, the goals of this project were to develop reliability-based inspection practices to meet the goals of:

1. Improving the safety and reliability of bridges and
2. Optimizing resources for bridge inspection.

The objective of this project was to develop a suggested bridge inspection practice for consideration for adoption by AASHTO. The practices developed through the project are based on rational, reliability-based methods to ensure bridge safety, serviceability, and effective use of resources.

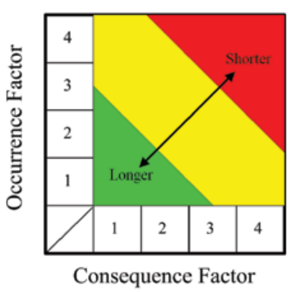
Findings

Reliability theories and practices were applied through the research to develop a guideline for Risk-Based Inspection (RBI) that provides a new approach for bridge inspection. The methodology consists of bridge owners performing a reliability assessment of bridges within their inventories to identify those bridges that are most in need of inspection to ensure bridge safety, and those for which inspection needs are less. This assessment is conducted by an expert panel at the owner level known as a Reliability Assessment Panel (RAP). The RAP conducts a reliability-based engineering assessment of the likelihood of serious damage resulting from common deterioration mechanisms, over a specified time period, and the likely outcome or consequences if that damage were to occur. The reliability-based assessment can be described by a simple, three-step process:

Step 1: What can go wrong, and how likely is it? Identify possible damage modes for the elements of a selected bridge type. Consider design, loading, and condition characteristics (attributes), and then categorize the likelihood of serious damage occurring into one of four *Occurrence Factors* (OFs) ranging from remote (very unlikely) to high (very likely).

Step 2: What are the consequences? Assess the consequences in terms of safety and serviceability, assuming the given damage modes occur. Categorize the potential consequences into one of four *Consequence Factor*s (CFs) ranging from low (minor effect on serviceability) through severe (i.e., bridge collapse, loss of life).

Step 3: Determine the inspection interval and scope. Prioritize inspection needs and assign an inspection interval for the bridge, based on the results of Steps 1 and 2.



Reliability matrix for determining maximum inspection intervals for bridges.

This assessment is based on common and well-known design, loading, and condition attributes that affect the durability characteristics of bridges. The attributes are identified and prioritized through expert elicitation processes. A simple reliability matrix, shown in the figure to the left, is used to identify the appropriate inspection interval for the bridge, based on the reliability analysis. Damage modes that tend toward the upper right corner of the matrix, meaning they are likely to occur and have high consequences if they did occur, require shorter inspection intervals and possibly more intense or focused inspections. Damage modes that tend toward the lower left corner, meaning they are unlikely occur and/or consequences are low if they did occur, require less frequent inspection.

Inspection intervals determined through the RBI process may be longer or shorter than those specified by traditional uniform, calendar-based approaches, depending on needs identified by the reliability-based engineering assessment. Inspections conducted under the RBI process are typically more intense and thorough than traditional inspection practices, and require condition assessment of bridge elements to meet the needs of the reliability-based assessment. Inspection needs are prioritized to improve the reliability of the inspection process, and bridge-specific inspection procedures can be developed based on the reliability analysis. The methodology developed is intended for typical highway bridges of common design characteristics.

The methodology developed through the research capitalizes on the extensive body of knowledge and experience with in-service bridge behavior, and the common deterioration mechanisms that cause bridges to deteriorate during their service lives. The process allows for the integration of emerging technologies such as improved data on long-term bridge performance and advanced modeling and analysis techniques, when available. The methodology was developed with suitable flexibility to address owner-specific needs and conditions, while providing systematic processes and methods to support consistent application of the technology.

The methodology developed through the research was tested using two case studies in different states. During these case studies, the processes described in the Guideline for RBI analysis were implemented using state forces to develop RBI intervals for typical highway bridges with superstructures constructed of steel and prestressed members. The RBI intervals determined through the RBI were verified through analysis of historical records for a sample of bridges in each state.

The reliability-based inspection practices developed through the research differ from traditional, calendar-based approaches. The new approach to bridge inspection provides a methodology to improve the safety and reliability of bridges by focusing inspection resources where most needed. This also leads to optimized allocation of resources, as inspection requirements are better matched to inspection needs through a reliability-based engineering assessment.

Conclusions

This research developed inspection practices to meet the goals of (1) improving the safety and reliability of bridges and (2) optimizing resources for bridge inspection. The goals of the research have been achieved through the development of a new guideline document entitled "Proposed Guideline for Reliability-Based Bridge Inspection Practices," Part I of this report, which has been developed based on the application of reliability theories. This document meets the project objective of developing a suggested practice for consideration for adoption by AASHTO, based on rational methods to ensure bridge safety, serviceability, and effective use of resources. A reliability-based approach was fully developed and documented through the Guideline. This new inspection paradigm could transform the calendar-based, uniform inspection strategies currently implemented for bridge inspection to a new, reliability-based approach that will better allocate inspection resources and improve the safety and reliability of bridges.

The implementation of the Guideline developed through the research was tested by conducting case studies in two states. These studies demonstrated and verified the effectiveness of the procedures developed in the research for identifying appropriate inspection intervals for typical highway bridges. It was shown through these studies that the RBI practices identified appropriate inspection intervals of up to 72 months. It was concluded from these studies that implementation of the RBI practices did not adversely affect the safety and serviceability of the bridges analyzed in the study, based on the analysis of historical inspection records. These studies also successfully demonstrated the implementation of the Guideline and the procedures therein using state DOT personnel.

The results reported herein demonstrated and verified that inspection intervals of up to 72 months were suitable for certain bridges. Such extended inspection intervals would allow the reallocation of inspection resources toward bridges requiring more frequent and in-depth inspections, resulting in improved safety and reliability of bridges. As such, the project goals of developing a reliability-based bridge inspection practice that could improve the safety and reliability of bridges, and optimizes the use of resources, were achieved through the research.

Suggestions

The research reported herein has demonstrated the effectiveness of the RBI procedures for determining suitable inspection intervals for typical highway bridges, and as such, broader implementation of the technology is suggested.

The procedure, methods, and approach described herein can be applied for atypical bridges as well. For example, non-redundant bridge members can be assessed using this approach, as illustrated in previous research (*60*). The approach can also be applied to complex bridges, or to bridges with advanced deterioration. Analysis requirements may be more detailed and advanced; development of such analysis may be pursued to provide a uniform strategy for bridge inspection across the entire bridge inventory.

Finally, the back-casting procedure utilized herein may be considered for implementation when RBI practices are to be used. Back-casting provides a means for verification of models developed by the RAP and quality assurance of the RBI process. As such, the back-casting procedure provides a critical tool for the implementation of RBI technology.

CHAPTER 1

Background

The periodic inspection of highway bridges in the United States plays a critical role in ensuring the safety, serviceability, and reliability of bridges. Inspection processes have developed over time to meet the requirements of the National Bridge Inspections Standards (NBIS) (*1*) and to meet the needs of individual bridge owners in terms of managing and maintaining bridge inventories. The inspection frequency mandated by the NBIS requires the inspection interval (maximum time period between inspections) not to exceed 24 months. Based on certain criteria, that interval may be extended up to 48 months with approval from the Federal Highway Administration (FHWA) (*2*). Maximum inspection intervals of less than 24 months are utilized for certain bridges according to criteria developed by the bridge owner, typically based on age and known deficiencies. Most bridge owners utilize the maximum inspection interval of 24 months, as mandated by the NBIS, for the majority of the bridges in an inventory, and the reduced intervals for bridges with known deficiencies. The uniform inspection interval of 24 months was specified at the origination of the National Bridge Inspection Program in 1971 based on experience, engineering judgment, and the best information available at the time.

The uniform approach provides a single maximum inspection interval for most bridges, regardless of the bridge age, design, or environment. To date, this mandated inspection interval has provided an adequate level of safety and reliability for the bridge inventory nationwide. However, such a uniform inspection practice does not recognize that a newly constructed bridge with improved durability characteristics and a few years of exposure to the service environment may be much less likely to developed serious damage over a given time interval than an older bridge that has been exposed to the service environment for many years. As such, inspection needs may be less for the newer bridge, and greater for the aging bridge, relative to the uniform interval currently required. Bridges that are in benign, arid operating environments are inspected at the same interval as bridges in aggressive marine environments, where significant damage from corrosion may develop much more rapidly, requiring increased inspection to ensure that safety and serviceability is maintained. Fracture critical members designed under modern criteria have vastly improved resistance to fatigue than older bridges, and as such, the likelihood of fatigue damage for modern bridges is much lower than for older bridges. Newer bridges in general are designed to higher standards with more durable materials such that their resistance to loading and environmental effects is much greater than older bridges. Current practices make it difficult to recognize if the same or improved safety and reliability could be achieved by varying inspection methods or frequencies to meet the needs of a specific bridge based on its design, condition, and operational environment.

Recognizing the variability in the design, condition, and operating environments of bridges would provide for inspection requirements that better meet the needs of individual bridges and improves both bridge and inspection reliability. A more rational approach to inspection planning would determine the interval and scope of an inspection according to the condition of the bridge and the likelihood that damage would occur. This would allow for resources to be focused where most needed to ensure the safety and reliability. Such inspection planning tools are highly developed in other industries, using the principles of reliability and risk assessment to match inspection requirements to inspection needs. These methodologies evaluate the specific characteristics of components, such as resistance to damage modes, anticipated deterioration mechanisms, current condition, and loading, to evaluate the reliability of the component. Appropriate inspection requirements are determined based on this evaluation, such that the safety and operation of the component is maintained over its service life, and resources are allocated efficiently.

As such, the goals of this project were to develop reliability-based inspection practices to meet the goals of:

(1) Improving the safety and reliability of bridges and
(2) Optimizing resources for bridge inspection.

The objective of this project was to develop a proposed bridge inspection practice for consideration for adoption by AASHTO. The practices developed through the project are based on rational methods that ensure bridge safety, serviceability, and effective use of resources. This report includes an overview of the inspection planning process that is based on the reliability principles developed during this project, and is documented in Part I of this report: "Proposed Guideline for Reliability-Based Bridge Inspection Practices."

CHAPTER 2

Research Approach

The Guideline were developed in consideration of modern industrial practices and is the result of an exhaustive review and analysis of current methodologies and practices for the inspection and management of structures and facilities, assessment of needs and capabilities, and the development of methodologies focused on the unique needs of highway bridges. The research was aimed at identifying the most effective strategy regarding development of a reliability-based bridge inspection practice. Through this investigation, a systematic process for determining the frequency and scope of highway bridge inspections has been developed based on reliability concepts.

Theories and practices for applying "reliability concepts" are increasingly popular as a basis for design codes as a means of adopting a more scientific basis for estimating variations in loading and resistance (strength) of components. Applying reliability theories in this context typically includes probabilistic analysis to deal with uncertainties in the design parameters and loading. There have been attempts to apply these design reliability concepts to maintenance and inspection activities, and some of this prior work will be discussed in this report. Unfortunately, such probabilistic approaches are, in most cases, found to be exceptionally complex and often require assumptions regarding the future behavior and performance of bridges that are difficult to verify. Additionally, probabilistic methods are typically focused on predicting strength, and do not address the serviceability requirements that are important in terms of bridge inspection. As such, alternative methodologies were sought through the course of the research.

In industrial applications, the more common terminology for inspection practices that use reliability theories for development of inspection and maintenance strategies is "risk-based," with reliability being one component of a risk analysis that also includes consideration of the consequences of some type of failure or loss of service. Sometimes, reliability and risk terms are used interchangeably. An extensive study of the current state-of-the-practice and state-of-the-art for reliability and RBI practice was conducted as part of this project to determine the most applicable methodologies for the inspection of highway bridges. The best practices and the successful implementations of these inspection practices were reviewed, analyzed, and considered by the Research Team. An expert panel meeting/workshop was held that included bridge inspection experts from state departments of transportation to provide bridge-owner perspective on the tools being developed through the research.

Several different approaches for developing a reliability-based inspection practice for highway bridges were considered, ranging from "pure" probabilistic structural reliability theories to fully qualitative risk analysis. The system that was developed is intended to incorporate the best practices and concepts from both schools of thought. The resulting methodology provides a reliability-based inspection practice that is implementable within the existing bridge inspection programs in the United States. Important consideration in developing the methodology included:

- The approach should be practically implementable and realistic.
- The approach needs to be sufficiently flexible to meet the needs of states with different inspection programs and bridge management approaches.
- The approach must be effective in ensuring bridge safety.
- The approach should match inspection requirements with inspection needs.
- The approach should capitalize on the existing body of knowledge regarding in-service bridge behavior.

Based on these considerations, a reliability-based methodology was developed for risk-based bridge inspection. In summary, the methodology developed has its foundation based on risk analysis that includes both the anticipated reliability of bridges (and their elements) and the consequences of damage to a bridge. The methodology is strongly grounded in existing industrial practice.

The methodology described in this report has been developed based on the well-established methods used in other industries for practical inspection planning. Such industrial standards, which are discussed in detail in the project interim report (*3*), provide a technical foundation for the methodology developed. The approach has been customized to provide a practical, implementable tool that can be expanded and developed over time. The research resulted in the development of the Guideline, which documents the tools, methodologies, and requirements for RBI practices.

CHAPTER 3

Findings and Applications

3.1 Introduction

The Guideline developed under this project describes the methodology for RBI practices for typical highway bridges. The goal of the methodology is to improve the safety and reliability of bridges by focusing inspection efforts where most needed and optimizing the use of resources. The Guideline provides a framework and procedures for developing suitable inspection strategies, based on a rational, reliability-based engineering assessment of inspection needs. The methodology considers the structure type, age, condition, environment, loading, prior problems, and other characteristics that contribute to the reliability and durability of highway bridges.

Generally, the methodology involves bridge owners performing a "reliability assessment" of bridges within their bridge inventory to identify those bridges that are most in need of inspection to ensure bridge safety, and those for which inspection needs are less. The assessment is conducted by considering the reliability and safety attributes of bridges and bridge elements. This reliability assessment is conducted by an expert panel assembled by a bridge owner (e.g., state) known as an RAP. This panel conducts an engineering assessment of the likelihood of serious damage resulting from common deterioration mechanisms, over a specified time period, applied to key elements of a bridge. This assessment is based on common and well-known design, loading, and condition attributes that affect the reliability characteristics of bridge elements. These attributes influence the likelihood that a particular element will fail over a given time period, i.e., its reliability. The attributes are identified and prioritized through an expert elicitation process. This process capitalizes on the experience and knowledge of bridge owners regarding the performance of the bridges within specific operational environments, given typical loading patterns, ambient environmental conditions, construction quality, etc.

The reliability estimate is combined with an evaluation of the potential outcomes or consequences, in terms of safety and serviceability, of damage progressing to a defined failure state. These data are then used to determine and prioritize inspection needs for specific bridges, or families of bridges with very similar design and condition characteristics. This includes determining a suitable inspection interval and scope, or procedures, to be used in the inspection. Under this process, the inspection interval is not fixed, such as it is in a uniform, calendar-based system, but rather is adjusted to meet the anticipated needs of the specific bridge or bridges in a family. Therefore, bridges with highly reliable characteristics, which are unlikely to have serious deterioration over a specified time, typically have a longer inspection interval than a bridge with less reliable characteristics, or for which the consequences of a failure may be more severe. For example, a bridge in good condition with highly durable and redundant design characteristics may have a longer inspection interval than a bridge in poor condition, lacking modern durability characteristics, and/or having a non-redundant design. Through this process, inspection resources can be focused where most needed to ensure the safety and serviceability of bridges. Inspection needs are prioritized to improve the safety and reliability of the bridge inventory overall.

The approach developed under the research is a risk-based approach that differs from purely reliability-based approaches in that the likelihood of failure is combined explicitly with the consequences of that failure. Risk can be defined generally as the product of the probability of an event and the associated consequences:

$$Risk = Probability \times Consequence$$

Probability in this equation is the likelihood of an adverse event or failure occurring during a given time period. This is sometimes expressed quantitatively as a probability of failure (POF) estimate for a given time interval, or as a qualitative assessment of the likelihood of an adverse event based on experience and engineering judgment. Generally, this probability is the complement of the reliability. *Consequence* is a measure of the impact of the event occurring, which may be measured in terms of economic, social, safety, or environmental impacts.

The Guideline developed through this research was focused on the inspection of typical highway bridges of common design characteristics. Atypical structures, such as long-span truss bridges, cable-stayed bridges, suspension bridges, and other unique or unusual bridge designs may require certain considerations not presently captured in the Guideline. Scour and underwater inspections have existing methodologies for evaluation, and as such are not included in the Guideline. Bridges assessed using this methodology are assumed to have a current load rating that indicates that the structural capacity is sufficient to carry allowable loads.

3.2 Overview of Methodology

The RBI process involves an owner (e.g., state) establishing an expert panel (RAP) to define and assess the durability and reliability characteristics of bridges within that state. The RAP uses engineering rationale, experience, and typical deterioration patterns to evaluate the reliability characteristics of bridges and the potential outcomes of damage. This is done through a relatively simple process that consists of three primary steps:

Step 1: What can go wrong, and how likely is it? Identify possible damage modes for the elements of a selected bridge type. Considering design, loading, and condition characteristics (attributes), categorize the likelihood of serious damage occurring into one of four *Occurrence Factors* (OFs) ranging from remote (very unlikely) to high (very likely).

Step 2: What are the consequences? Assess the consequences in terms of safety and serviceability assuming the given damage modes occur. Categorize the potential consequences into one of four *Consequence Factors* (CFs) ranging from low (minor effect on serviceability) through severe (i.e., bridge collapse, loss of life).

Step 3: Determine the inspection interval and scope. Use a simple reliability matrix to prioritize inspection needs and assign an inspection interval for the bridge based on the results of Steps 1 and 2. Damage modes that are likely to occur and have high consequences are prioritized over damage modes that are unlikely to occur or are of little consequence in terms of safety. An RBI procedure is developed based on the assessment of typical damage modes for the bridges being assessed that specifies the maximum inspection interval.

Inspections are conducted according to the RBI procedure developed through this process. The RBI procedure differs from current inspection practices generally, because the damage modes typical for the specific bridge are identified and prioritized. The inspection is required to be capable of assessing each of these damage modes sufficiently to support the assessment of future needs. As a result, the inspections may be more thorough than traditional practices, including hands-on access to key portions of a bridge such that damage is effectively identified to support the RBI assessment. The results of the inspection are assessed to determine if the existing RBI procedure needs to be modified or updated as a result of findings from the inspection. For example, as a bridge deteriorates over time and damage develops, as reported in the inspection results, inspection intervals may be reduced to address the inspection needs for the bridge as it ages.

The overall process for assessment under the developed Guideline is shown schematically in Figure 1. The process begins with the selection of a bridge or family of similar bridges to be analyzed. For the selected bridge or bridges, the RAP identifies common damage modes for elements of the bridge given the design, materials, and operational environment. Key attributes are identified and ranked to assess OFs that categorize the likelihood of serious damage developing over a specified time interval. CFs that categorize the poten-

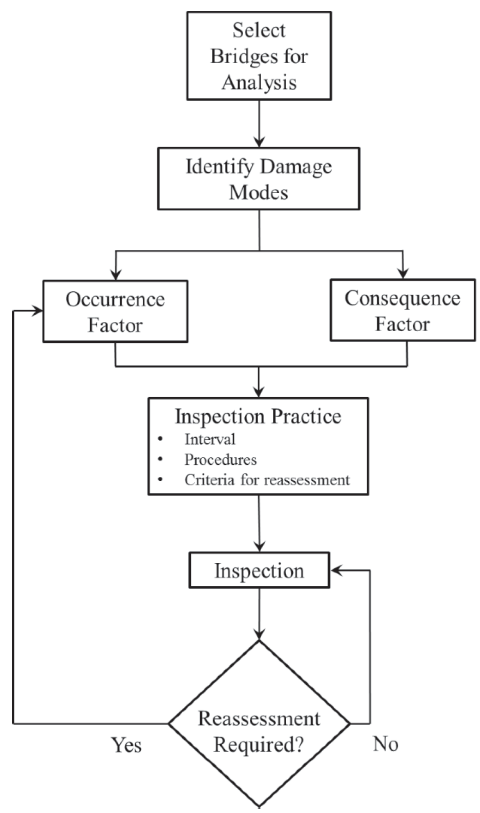


Figure 1. Schematic diagram of the RBI process.

tial outcomes or consequences of damage are also assessed. Based on the assessment of the OFs and CFs for the various elements of the bridge, an inspection procedure is established, including the interval and scope for the inspection. Criteria for reassessment of the inspection procedure are also developed based on the assessment. The criteria for reassessment are typically based on conditions that may change as a result of deterioration or damage, and that may affect the OFs for the bridge. The RBI practice is then implemented in the subsequent inspection of the bridge. Inspection results are assessed to determine if any established criteria have been violated, or if conditions have changed that may require a reassessment of the OFs. If such changes exist, a reassessment of the OFs is completed and the inspection practice modified accordingly.

The method of determining the inspection interval, or time period between inspections, is shown schematically in Figure 2. The interval is based on the RAP assessment of the OFs and the CFs, plotted on a simple two-dimensional reliability matrix as shown in the figure. The OFs and CFs are used to place typical damage modes in an appropriate location on the matrix. In this figure, the horizontal axis represents the CF as determined for a particular damage mode for a given bridge element. The vertical axis represents the outcome of the OF assessment for a given damage mode for the given element. Damage modes that tend toward the upper right corner of the matrix, meaning they are likely to occur and have high consequences if they did occur, require shorter inspection intervals and possibly more intense or focused inspections. Damage modes that tend toward the lower left corner, meaning they are unlikely to occur, and/or consequences are low if they did occur, require less frequent inspection. This is simply a rational approach to focusing inspection efforts; inspections are most beneficial when damage is likely to occur and important to the safety of the bridge; inspections are less beneficial for things that are very unlikely to occur, or are not important to the safety or serviceability of the bridge.

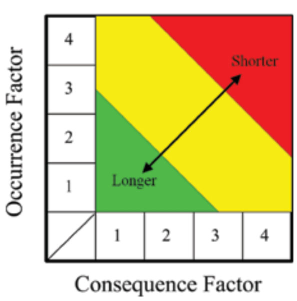


Figure 2. Reliability matrix for determining maximum inspection intervals for bridges.

Through this process, individual bridges, or families of bridges of similar design characteristics, can be assessed to evaluate inspection needs from a reliability-based engineering assessment of the likelihood of serious damage occurring, and the effect of that damage on the safety of the bridge. The methodology can be applied throughout a bridge inventory, or to portions of a bridge inventory. Suitable Quality Control (QC) and Quality Assurance (QA) procedures should be utilized to ensure consistency.

The RBI approach considers the structure type, age, condition, and operational environment in a systematic manner to provide a rational assessment process for inspection planning. A documented rationale for the inspection strategy utilized for a given bridge is developed. The damage modes most important to ensuring the safety of the bridge are identified such that inspection efforts can be focused to improve the reliability of the inspection results.

The sections that follow describe the key elements of the RBI practices for bridge inspection. Section 3.3 provides background data underlying the RBI process, including reliability concepts such as POF, the reliability theory applied with the RBI process, damage modes and deterioration mechanisms considered in the analysis, and typical lifetime behavior characteristics that support the RBI approach. This section also highlights the differences between the reliability theory applied for inspection planning, and those traditionally applied for structural design codes. Section 3.4 discusses key elements of the Guideline developed under the research and initial testing of some of the processes developed. Finally, Section 3.5 describes data needs and resources to support the RBI analysis.

3.3 Reliability

A key element in the RBI process is to understand the meaning and role of *reliability* in the context of determining inspection needs and inspection planning. This section of the report provides supporting data and background information regarding important aspects of reliability and its underlying theories, and how these support RBI.

Reliability is defined as the *ability of an item to operate safely under designated operating conditions for a designated period of time or number of cycles.* The inspection practices documented in the Guideline are based on the concepts and theories of reliability. The *reliability* of a bridge element is defined in terms of its safe operation and adequate condition to support the serviceability requirements for bridges. This definition is broader and more applicable to determining bridge inspection needs than structural reliability estimates, which are typically defined as a function of the load-carrying capacity of the structure and notional POF estimates. The challenge with applying theoretical structural reliability concepts, such as those used in modern design specifications, is that the envisioned damage mode (loss of load-carrying capacity) represents only a portion of the required information needed from a bridge inspection. From the perspective of practical bridge inspection, safe operation includes strength considerations, but also includes a variety of serviceability limit states that may be related in some way to strength considerations, but are not direct measures of strength. Serviceability considerations such as local damage that can affect traffic, deflections and cracking, and loss of durability characteristics need to be assessed through periodic inspections, even if the effect on structural capacity, and therefore structural reliability, is nominal. Additionally, existing required load ratings provide structural analysis in terms of load capacity for bridges (*4*). These ratings generally provide limited insight into the inspection needs for a bridge, although the engineering analysis considers certain inspection results, such as section loss, in the analysis.

Several methods and processes have been suggested for the assessment of in-service bridge reliability and the estimation of inspection requirements based on structural reliability, and these were studied during the course of the research. Research based on structural reliability theory for the development of inspection strategies, repair optimization, and updating bridge reliability estimates based on visual inspections has been performed (*5–8*). Significant work in the area of applying structural reliability theory to highway bridges was reviewed during the course of the research, and detailed review is included in the project interim report (*3, 7, 9–15*). The conclusion reached based on the review of this literature was that these approaches were not currently implementable for highway bridge inspection, due to several factors. First, structural reliability models and probabilistic analysis does not typically capture the serviceability limit states critical to identifying in-service bridge inspection needs. Second, structural reliability models are highly theoretical in nature, and the complexity of analysis required for even a simple structure makes application to the diversified bridge inventory in the United States impractical. Finally, the results of the structural reliability assessments are often based on POF estimates that are notional and design-based, such that significant uncertainty would result from mapping these results to inspection needs for specific bridges.

However, the underlying concepts of reliability could be applied for the purpose of bridge inspection if appropriate and implementable methodologies for estimating reliability of bridges or bridge elements were developed. These methodologies need to consider the serviceability requirements for bridges and bridge inspection, and define reliability appropriately such that it can be assessed based on inspection results and anticipated future deterioration. This analysis could then be applied as one component of an inspection planning process that includes an assessment of the consequences associated with failure due to specific damage modes (*16–19*).

Based on the analysis of the research on reliability methods, the research team pursued a path to develop a semi-quantitative, reliability-based framework for inspection practices. The key elements of developing that methodology included identifying the reliability theories to be implemented to evaluate bridges, and an appropriate description of "failure" to assess when a bridge element is no longer performing adequately, and hence has reduced reliability. The following sections describe briefly the underlying reliability theory utilized in the RBI Guideline, and the definition of failure used. Damage modes and deterioration mechanisms that cause a bridge element to deteriorate into the defined failure state are discussed, and the overall concept of matching inspection needs to bridges during different stages of typical in-service behavior are described.

3.3.1 Reliability Theory

Reliability is defined as the *ability of an item to operate safely under designated operating conditions for a designated period of time or number of cycles.* For bridges and bridge elements, reliability typically decreases as a function of time due to deterioration and damage accumulated during the service life of a bridge, for example, corrosion of steel elements in a bridge that develops over the service life of the bridge, resulting in increasing damage over that service period. The likelihood of failure typically increases with time such that the reliability of a bridge or bridge element can be expressed as:

$$R(t) = \Pr(T \geq t)$$

Where $R(t)$ is the reliability, T is the time to failure for the item, and t is the designated period of time for the item's operation. In other words, the reliability is the *probability* (Pr) or *likelihood* that the failure time exceeds the operation time. Sometimes, the likelihood is expressed as a probability density function (pdf) that expresses the time to failure of an item (T) as some generic distribution, such as normal, log normal, etc. (*13, 15, 20*). This distribution can be used to calculate a POF function, $F(t)$, to express the probability that the item will fail sometime up to time t. This time-varying function describes likelihood of failure up to some given time, or the *unreliability* of the item, and the reliability is then:

$$R(t) = 1 - F(t)$$

In other words, the reliability is the probability that the item *will not* fail during the time period of interest. The challenge for RBI was to determine an appropriate and practical method of estimating the probability, or likelihood, of failure described by the function $F(t)$. This requires a definition of what is meant by "failure" for a bridge element or structure. It also requires an appropriate time interval over which an effective and meaningful assessment can be accomplished given the diversity in materials, designs, and operational environments included across the bridge inventory.

When a large population of test data for identical or near identical components exposed to the same operational environment are available, a probability function describing the failure characteristics of the component may be determined and verified based on test results. This can provide a quantitative frequency-based estimate of the POF that indicates the number of events (failures) expected during a given time period. However, such test data are generally unavailable for bridges, because design, construction quality, and operational environments vary widely, and failures are rare. A suitable probability distribution may be assumed when test data are not available, but verifying the accuracy of such a distribution can be difficult for complex systems like highway bridges, where design and construction methods are constantly evolving, operational environments vary, and performance characteristics are also evolving. As a result, past performance of similar elements of a bridge may not be indicative of future performance, and the applicability of an assumed function to a specific bridge is unverifiable, since the lifetime failure characteristics described by the assumed function describe events that have not yet occurred. If designs, construction practices, and materials were not evolving over time, this might be more practical, but this is not the case for highway bridges.

Under conditions for which data to adequately characterize anticipated future behavior is limited, or where failure is rare, engineering judgment and experience can be used to estimate the expected reliability of a specific bridge within a given operational environment (*21–23*). Under these circumstances, the POF is determined based on qualitative or semi-quantitative analysis and the probability is based on degree of belief, rather than frequency. To make such decisions, individuals with expertise and experience with typical performance characteristics, under a specific set of operational environments, is required. Utilizing expert judgment and expert elicitation is a common method of characterizing the reliability of components or systems for the purpose of assessing inspection needs (*21–24*). Such engineering judgment and knowledge provides data when quantitative data are missing, incomplete, or inadequate. In the RBI methodology, expert elicitation is used as a process for estimating the anticipated likelihood of failure for bridge elements, and hence their reliability, over a given time period of 72 months. The following sections describe the definition of failure, damage modes and deterioration mechanisms, and typical lifetime performance characteristics that are underlying the RBI process analysis.

3.3.2 Failure

A key step in assessing the reliability of a bridge element is understanding how and why elements "fail," and the typical deterioration mechanisms that cause the elements to "fail." The damage modes and deterioration mechanisms that typically affect bridge elements are well known, in most cases. For example, corrosion is obviously a significant deterioration mechanism in concrete and steel bridge elements that causes them to "fail." The likelihood of the failure occurring in some future time interval depends on attributes of the element, such as its materials of construction, design, durability, and current condition, as well as what conditions are used to describe an element as "failed." For bridges, catastrophic collapse would be one obvious condition that could be used to define failure, but such failures are very rare. Important concerns for bridge inspections extend well beyond simply avoiding rare catastrophic failures. Ensuring the safety of the bridge, in terms of structural capacity, serviceability, and safety of the traveling public are important factors in determining the inspection needs of a bridge.

Therefore, *failure* requires a suitable definition that captures the need to ensure the structural safety of the bridge, the safety of travelers on or below the bridge, and the serviceability of the bridge. *Failure*, utilized in this context, is defined as when an element is *no longer performing its intended function to safely and reliably carry normal loads and maintain serviceability.* For example, a bridge deck with severe spalling may represent a "failed" condition for the bridge deck even though the deck may have adequate load-carrying capacity, because the ability of the deck to reliably carry traffic is compromised. Therefore, for the case of reliability assessments for determining bridge inspection needs, it was necessary to adopt a commonly understood definition of failure that considers common deterioration patterns in bridges and that can effectively be assessed through the inspection process. Additionally, failure must be defined in a commonly understood manner that can be readily assessed, is consistent with the historical experiences of bridge managers, and is sufficiently general to be easily applied across the broad spectrum of design characteristics and elements that exists across the bridge inventory. To meet this need, the NBIS condition rating of 3, "serious condition," was chosen as a general, durable, and readily understood definition of failure. Bridge elements that have deteriorated to this extent may no longer be performing their intended function, and remedial actions are typically planned to address such conditions. It is not envisioned that any bridges or bridge elements assessed using a reliability-based approach are allowed to deteriorate to this condition. *Rather, inspection intervals are adjusted to ensure that the likelihood of failure in the time intervals between inspections always remains low.*

The subjective condition rating of 3 is defined within the *Recording and Coding Guide* (*25*) as follows:

> *NBIS Condition Rating 3: SERIOUS CONDITION: Loss of section, deterioration, spalling or scour have seriously affected primary structure components. Local Failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.*

In terms of the *AASHTO Bridge Element Inspection Guide*, this condition generally aligns with elements in Condition State (CS) 4, "serious" (*26*).

These condition descriptions are widely understood and there is significant past experience in the conditions warranting a rating of 3 throughout the bridge inventory for the myriad of different materials and design characteristics that exist. This condition description provides a practical frame of reference for assessing the likelihood of failure in some future time period. For example, one could readily assess if a bridge deck that currently has a condition rating of 7, and has durable attributes such as adequate concrete cover and epoxy-coated reinforcing steel, was *very likely*, or *very unlikely* to deteriorate to a condition rating of 3 in the next 72 months. If the deck is very unlikely to deteriorate to a failed state during that time interval, repeated inspections of the deck may yield little or no benefit. On the other hand, if the deck were in poor condition, with a condition rating of 4, it may be more likely to fail during this time period, and more frequent inspections are necessary to monitor the deterioration and identify repair needs.

3.3.3 Damage Modes and Deterioration Mechanisms

The failure state described above is typically reached as the result of the accumulation of one or more forms of damage. For example, a deck may reach the "failed" state because of widespread spalling; a steel beam may reach that state as the result of severe section loss. These typical forms of deterioration in bridges are observable in a visual assessment of the bridge, or sometimes with the assistance of a nondestructive evaluation technology (NDE). The observable effects on which a condition assessment is normally based are forms of damage, or *damage modes.* Damage modes are typically assessable through the inspection process and their extent or degree recorded in the inspection results. Spalling, cracking, scaling, sagging, etc. are *damage modes.*

Damage modes are normally the result or manifestation of a *deterioration mechanism*, such as corrosion or fatigue. Deterioration mechanisms describe the path to failure, and may occur at different rates depending on factors such as operational environment and loading patterns. For example, a concrete bridge deck may fail due to the damage mode of concrete spalling, and the deterioration mechanism is corrosion. If the deck is located in an aggressive environment, the corrosion mechanism may be fast acting, if in a benign environment, the mechanism may be slow acting. Similarly, if the damage mode is cracking in a steel element, and the cracking results from the deterioration mechanism of fatigue, then the rate at which the damage mode will progress depends on the cyclic loading of the bridge. If the bridge has very low average daily truck traffic (ADTT), then the likelihood of the damage mode progressing is lower than if the ADTT were high. However, if the damage mode is cracking and the deterioration mechanism is constraint-induced fracture (CIF), the progression of the damage mode may only depend on the susceptibility of the weld detail to CIF.

Within the RBI process, it is important to separate the damage mode from the deterioration mechanisms such that suitable attributes or characteristics can be appropriately identified. For example, if the damage mode is spalling in a bridge deck, the deterioration mechanism could be corrosion of embedded reinforcing steel, or could be debonding of an overlay. Obviously, the attributes affecting how likely it is that debonding will occur differ from those that affect how likely it is that corrosion damage may occur, even though the resulting damage may have very similar effects on the serviceability of the deck.

3.3.4 Lifetime Performance Characteristics

Part of the overall assessment of the reliability of a bridge element is an understanding of the typical lifetime behavior of engineering components. Generally, failure patterns can be described by a "bathtub" curve such as that shown in Figure 3, which represents the failure rate, or POF, as a function of the time. The "bathtub curve" shows the initial failure of new components due to defect (infant mortality), the useful life period, and the wear-out period. For bridges, the infant mortality portion of the bathtub curve illustrates the effects of construction errors or flaws, which typically become evident in the early life of a bridge. One of the purposes of QCs and inspections during the construction phase of a bridge is to reduce the infant mortality rate, that is, to ensure there are not defects in the structure from construction errors that

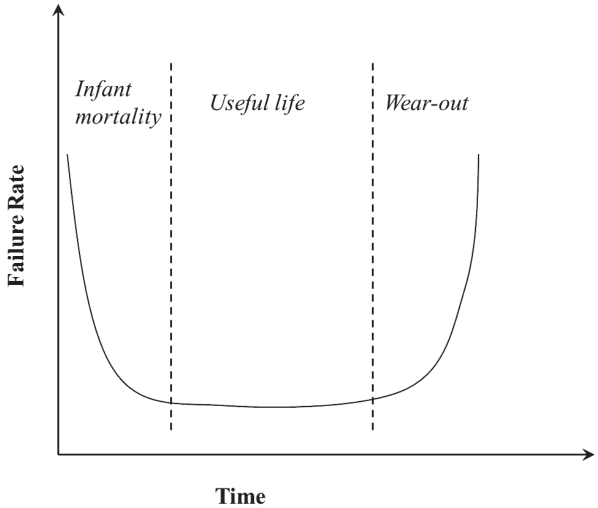


Figure 3. Plot of the "bathtub" probability curve.

will lead to a shorter than expected service life. Following the period in which infant mortality may occur, elements typically have long service lives and failures are rare. Toward the end of the service life, when elements are in advanced stages of deterioration, the likelihood of failure can increase substantially. As a result, more frequent and thorough inspections may be necessary to monitor deterioration and identify repair needs. The bathtub curve shows schematically the typical performance of engineered components: the shape and timeline of the curve for specific bridge elements obviously depends on the attributes of the element, including the design characteristics, typical construction quality, operational environment, management and maintenance practices, etc. Among the purposes of RBI or any other life-cycle management system that includes inspection is the reduction of the wear-out rate by finding and repairing or replacing components before they fail, reducing unnecessary or unjustified inspection efforts, and optimizing the utilization of inspection resources. Inspection needs are typically lower during the useful life of elements, when failures are rare, and increase as the failure rate increases as the result of deterioration mechanisms that manifest in damage.

Many different methods are available to model failure processes and determine failure rate characteristics such as those shown in Figure 3, from qualitative to quantitative including hybrid methods. Qualitative methods would include expert judgment; hybrid methods would include methods like Markov Chain models, which use expert opinions and empirical data to estimate transition probabilities (*27*). Quantitative methods can range from fully empirical (using statistical fits to test or field data) to fully physics-based (using physical models of failure processes). Weibull and log-normal statistics have both been used to describe failure processes that are driven by forces such as fatigue, wear, and/or corrosion. Given a sufficiently large population of engineered structures and the same driving forces, their rate of failure (or equivalently the POF at any time) can often be described by Weibull or log-normal statistics. Thus, if items are cheap and easy to test, a statistical description of their failures can be created and used to predict the behavior of similar items in the future. However, for bridges, characteristics of the elements and their environment vary widely and are difficult to capture within such models, particularly when considering the needs of a specific bridge. For example, Table 1 shows variables used for probabilistic modeling of bridge reliability from some common literature resources and indicates the level of data resources that need to be either determined empirically, or estimated using statistical tools and probability functions (*9, 10, 14, 28*). As this table indicates, the magnitude of data that needs to be either collected or assumed is significant. The assumptions required to effectively estimate such a large number of properties and characteristics require verification, and may vary widely across different

Table 1. Variables used for probabilistic estimates of time-varying reliability.

• Concrete cover	• Corrosion rate	• Time to corrosion initiation
• Workmanship	• Crack width	• Prestress steel strength and modulus
• Concrete strength and modulus	• Critical crack width	• Prestress losses
• Reinforcing steel strength and modulus	• Crack depth	• Impact factor
• Shrinkage of concrete	• Cracking density	• Area of reinforcing steel in concrete
• Thickness	• Loading rate	• Flexural forces
• Dead load	• Surface chloride concentration	• Shear forces
• Truck live load	• Critical chloride concentration	• Load distribution factors
• Water-cement ratio	• Chloride diffusion	• Reinforcement spacing

bridge design, operational environments, and construction practices. Verification of the assumption requires observation of bridge performance over its service life, and therefore, by definition, cannot be determined in time to be usefully applied. It is also notable that among the many parameters assembled to estimate time-dependent reliability, bridge joint condition is not among them. However, practically, this factor alone may outweigh all of the others in terms of assessing the expected deterioration patterns and rate for a bridge.

Additionally, because design and construction processes are evolving, elements that have the same role in different bridges often do not share key design features or operational environments that could affect their long-term performance. This makes estimating the many factors shown in Table 1 even more challenging and impractical across an inventory that includes 600,000 bridges and a multitude of operational environments. Therefore, expert judgment is required to consider the role and significance of specific design and environmental features for specific bridges, and to estimate future performance effectively.

3.4 Key Elements of RBI

This section of the report provides an overview of key elements of the RBI process described and detailed in the "Proposed Guideline for Reliability-Based Inspection Practices" developed through the research. This includes a description of the OF, the CF, inspection procedures for RBI, and the RAP.

3.4.1 The OF

Within the RBI process, an estimate of the POF for a given bridge element is expressed as an OF. This factor is an estimate of the likelihood of severe damage occurring in a specified time interval, considering the likely damage modes and deterioration mechanism acting on the element. Key attributes of the element that affect the likelihood are considered and documented to support the estimate. This section describes the approach and methodology for estimating the probability, or likelihood, of failure for bridge elements for the purpose of inspection planning.

There are a variety of methodologies for estimating the expected performance of components or elements. These range from fully quantitative methods to fully qualitative methods. For example, the American Petroleum Institute's Recommended Practice 581 has, for certain critical components, empirical equations that estimate the POF for the component given certain attributes of the component and its operational environment (*29*). These empirical equations include factors associated with the attributes of specific components and are used to calculate the expected POF over some defined time period. In other cases, physics-based models for damage such as fatigue cracking are combined with industrial modeling tools to estimate the POF for specific components or systems (*17, 30–33*). For cases in which historical data may be scarce, where systems are complex and/or evolving such that relevant historical data are unavailable, expert judgment and expert elicitations can be used (*21*).

To develop an estimate of the POF over a certain period, several factors need to be considered, including what constitutes a practical definition of failure, as described above, over what time period the assessment can be made, and what resolution is required for the estimate. Often, estimates utilized in reliability analysis are simply order of magnitude estimates, or even ranges, over which the POF is expected to fall. For example, ASME guidelines suggest first-level qualitative analysis can be achieved using a simple three-level scale shown in Table 2 (*21*). An estimate of the annual POF associated with a qualitative ranking is also provided. In this context (for this industry) a high POF is intended to represent failure rates on the order of a 0.01 or 1 in 100, while moderate probability (likelihood) is intended to cover 2 orders of magnitude from 0.01 to 0.0001, with low probably being less than 0.0001. In moving from totally qualitative to semi-quantitative analysis, the order of magnitude of the failure rate may be estimated, and these numerical values provide a mapping of qualitative to quantitative rankings. In practical applications, even if quantitative methods are used, the estimated POFs are typically considered to be, at best, order of magnitude estimates, due to the inherent variation and uncertainty in engineered systems.

For application for the RBI assessment for highway bridges, existing industrial approaches were considered as a basis for developing appropriate methodologies for estimating reliability for highway bridges elements. This required that an appropriate time interval be determined over which an assessment for the POF could be made, based on available data and engineering factors. Appropriate categorizations or qualitative scales to effectively describe that reliability were developed for use as part of a reliability-based assessment.

3.4.1.1 Assessment Interval

Given the typically long service life of a bridge and the slow rate of deterioration mechanism such as corrosion, annual POF estimates such as those described above may have little

Table 2. ASME POF rankings using a three level scale.

Possible Qualitative Rank	Annual Failure Probability
Low	<0.0001 (1/10,000)
Moderate	0.0001-0.01 (1/10,000 – 1/100)
High	>0.01 (1/100)

meaning and vary widely according to the assumption made in a particular analysis. Additionally, "failure" is not typically well defined as is the case, for example, with a pipe or valve. If the pipe leaks, it is failed, if the valve fails to open when required, it is failed. But with elements in a bridge, the majority of deterioration mechanisms extend over long time periods, fracture being an exception, and the "failure" state itself can be subjective (*34*). Elements may reach a state that meets the definition of "failure" and stay in that condition for some number of years. Therefore, it is more appropriate to describe how likely it is for deterioration or damage to occur to the extent that an element deteriorates into a "serious" or "failed" condition. For the RBI process for bridge inspection, an OF is used to represent a qualitative measure of this likelihood over a time interval of 72 months. This time period was determined based on engineering factors that included prior research, experience, expert judgment, and data from corrosion, damage and deterioration models (*13, 35–43*). For example, commonly available corrosion models indicate that significant periods of time transpire between construction of a bridge and initiation of corrosion, particularly in environments that are not aggressive (i.e., little or no use of de-icing chemicals and no marine exposure). Once initiated, corrosion may take a significant period of time to manifest in damage, depending on factors such as bar spacing, cover, concrete material properties, and environment. Estimates for damage progression typically range from 6 years on the low end, for uncoated rebar in typical concrete structures, to 20 years or more for epoxy-coated bars. For steel elements, although corrosion damage can be severe, the rate at which corrosion damage occurs is actually very slow, typically less than 0.004–0.006 in./year, even in moderately aggressive environments (*37, 44, 45*). Therefore, the amount of section loss that could occur during a 72-month interval is nominally less than 1⁄16 of an inch, assuming two sides of a steel plate were corroding equally, at a relatively high rate of 0.005 in./yr. Section loss on this order of magnitude would not be considered serious. Therefore, it is practical to assess the likelihood of damage progression occurring over a time frame of 72 months, because the likelihood is low that these deterioration mechanism could result in a bridge element deteriorating from a "good" condition to a "serious" condition during such a time interval. It should be noted that this interval of 72 months is an assessment interval over which the reliability of an element is estimated for the purpose of assessing inspection needs. Inspection intervals may be significantly less than 72 months when existing damage is present, or the attributes of an element suggest the likelihood of damage developing is high.

The time period of 72 months is also considered a time period for which an engineer could reasonably estimate future performance within four fairly broad categories, ranging from "remote" to "high," based on key attributes that describe the design, loading, and condition of a bridge or bridge element. The interval provides a suitable balance between shorter intervals, when the POF could be unrealistically low do to the typically slow progression of damage in bridges, or longer intervals, where uncertainty would be increasingly high. For example, if an engineer was asked to predict if a deck currently rated in good condition was likely to progress to a serious state in 1 year, that estimate would be very low, since deterioration mechanisms are slow acting. However, if the time period of 10 years were used, the uncertainty could be very high. The time interval was selected in part to provide a suitable balance over which damage progression could be reasonably predicted based on engineering assessment and rationale.

3.4.1.2 OF Categorization

The OF is a qualitative ranking or categorization of the likelihood that an element will fail during a specified time interval. A four category, qualitative scale was developed for estimating the OF for RBI practices. The scale ranges from remote, when the likelihood is extremely small such that it would be unreasonable to expect failure, to high, where the likelihood of the event is increased. The categories and associated verbal descriptions are shown in Table 3.

The OF is determined by expert judgment considering key characteristics, or *attributes*, of bridge elements. "Attributes" are characteristics of a bridge element that contribute to the element's reliability, durability, or performance. These attributes are typically well-known parameters affecting the performance of a bridge element during its service life. These includes relevant design, loading, and condition characteristics that are known or expected to affect the durability and reliability of the element. These attributes are identified and assessed through the expert elicitation process.

Numerical ranges that could be used to describe the OF scale quantitatively are shown in Table 4. Such numerical values provide ranges or target values for the qualitative rankings that could be used to map quantitative data, if it is available, to the qualitative rating scales. Failure of a bridge element is a relatively rare event, and design and construction details vary widely. As a result, relevant and verifiable frequency-based probability data are scarce, as previously discussed. The numerical values shown in Table 4 are target values that

Table 3. OF rating scale for RBI.

Level	Category	Description
1	Remote	Remote likelihood of occurrence, unreasonable to expect failure to occur
2	Low	Low likelihood of occurrence
3	Moderate	Moderate likelihood of occurrence
4	High	High likelihood of occurrence

Table 4. OF categories and associated interval estimates of POF.

Level	Qualitative Rating	Description	Likelihood	Expressed as a Percentage
1	Remote	Remote probability of occurrence, unreasonable to expect failure to occur	≤1/10,000	0.01% or less
2	Low	Low likelihood of occurrence	1/1000-1/10,000	0.1% or less
3	Medium	Moderate likelihood of occurrence	1/100-1/1,000	1% or less
4	High	High likelihood of occurrence	>1/100	> 1%

can be used to map such data or models to the qualitative scales used in the analysis. For example, data from PONTIS deterioration curves or from probabilistic analysis, or other deterioration models, could be incorporated directly into the assessment of the OF using these scales. These numerical categories can also provide a framework for future development of models or data derived from analysis of the deterioration patterns in a particular bridge inventory.

The quantitative description can be also be used as a vehicle for expert elicitation by using common language equivalents for engineering estimates. For example, if you asked an expert to estimate the probability of serious corrosion damage (widespread spalling, for example) for a particular bridge deck given its current condition, a common engineering response might include a percentage estimate, for example, less than 0.1% chance or less than 1 in a thousand. This estimate can then be mapped to the qualitative scale as being "low." Such estimates are typically very conservative, particularly for lower, less likely events.

3.4.1.3 Method of Assessing OFs

OFs are determined through expert elicitation by the RAP assembled by a bridge owner. The RAP provides experience and knowledge of the performance of materials, designs, and construction quality and methods within a specific operational environment. This knowledge and experience is used to categorize the OF considering credible damage modes and deterioration mechanisms for bridge elements.

The assessment is conducted by identifying critical design, loading, and condition characteristics, or *attributes*, that affect the reliability and durability of the elements. For example, consider the damage mode of spalling due to corrosion damage in a concrete bridge deck. A bridge deck may have "good" attributes, such as being in very good condition, having adequate concrete cover, epoxy-coated steel reinforcing, and minimal application of de-icing chemicals. Given these attributes of the deck, it may be very unlikely that severe damage (i.e., failure) would occur in the next 72 months. This is based on the rationale that the deck is presently in good condition, and has attributes that are well known to provide resistance to corrosion damage. As such, an OF of "Low" or "Remote" might be used to describe the likelihood of failure due to this damage mode. Alternatively, suppose the deck is in an environment where de-icing chemicals are frequently used, the reinforcement is uncoated, and the current rating for the deck is a 5, Fair Condition, indicating that there are signs of distress in the deck. Based on this rationale, the likelihood of serious damage developing would be much greater, resulting in an OF rating of "Moderate" or "High." Past experience with decks of a similar design, combined with engineering judgment, can be used to support the assessment of the specific OF for a given deck.

These attributes can be generally grouped into three categories: Design, Loading, and Condition attributes. *Design attributes* of a bridge element are those characteristics of the element that describe its design. Design attributes are frequently intrinsic characteristics of the element that do not change over time, such as the amount of concrete cover or material of construction. In some cases, preservation or regular maintenance activities that contribute to the durability of the bridge element may be a design attribute, such as the use of penetrating sealers as a preservation strategy.

Loading attributes are characteristics that describe the loads applied to the bridge element. This may include structural loading, traffic loading, or environmental loading. Environmental loading may be described in macro terms, such as the general environment in which the bridge is located, or on a local basis, such as the rate of de-icing chemical application on a bridge deck. Loading attributes describe key loading characteristics that contribute to the damage modes and deterioration mechanisms under consideration.

Condition attributes describe the relevant bridge element conditions that are indicative of its future reliability. These can include the current element or component-level rating, or a specific condition that will affect the durability of the element. For example, if the damage mode under consideration is concrete damage at the bearing, the condition of the bridge joint may be a key attribute in determining the likelihood that corrosion will occur in the bearing area.

3.4.1.4 Screening Attributes

Attributes can also be identified as screening criteria that identify certain characteristics that have a predominate effect on the reliability of an element. Attributes used for screening may be design, loading, or condition attributes. Screening attributes are used to quickly identify bridges that should not be included in a particular analysis, either because they already have significant damage or they have attributes that are outside the scope of the analysis being developed. Screening attributes are typically attributes that:

- Make the likelihood of serious damage occurring very high,
- Make the likelihood of serious damage occurring unusually uncertain, and
- Identify a bridge with different anticipated deterioration patterns than other bridges in a group.

The RAP must identify the appropriate value/condition for the attribute to use as a screening tool. For example, if considering the likelihood that the steel bridge will suffer corrosion damage that reduces its rating to a 3, and the current rating is 4, the RAP may consider that such condition indicates that there is an unusually high likelihood of further damage developing over the next 72-month period, and as such, use the condition rating of 4 as a screen. In such a case, the analysis can move forward to an assessment of the consequences of the damage without further evaluation of the attributes that contribute to the likelihood of damage, based on the screening item. Another example would be to screen steel beam elements in bridges that have open decking. Since the open decking allows drainage directly onto the steel beams, the deterioration of these bridges would not be similar to steel beams with typical concrete decks; these bridges are screened from the analysis of steel beam bridges, as they may require separate analysis. It may be appropriate to treat these bridges as a separate group, developing the analysis to consider key attributes of those bridges with open decking.

3.4.1.5 Ranking Attributes

Key attributes for a bridge element are identified by the RAP and used to assess the appropriate OF for the given element and damage mode being considered. This assessment is supported through an empirical scoring procedure that provides a rational method of estimating the OF category. The attributes identified are ranked according to the importance of each attribute in assessing the reliability of a certain bridge element. For example, for attributes that play a primary role in determining the likelihood of damage, a scale of 20 points could be used, 15 points for an attribute that has a moderate role, and 10 points for an attribute that plays a minor role. For the damage mode of corrosion in a steel beam, for example, a leaking joint which results in drainage of de-icing chemicals directly onto the superstructure is highly important in assessing the likelihood of serious corrosion damage occurring. Therefore, this attribute may be assigned a 20 point scale by the RAP. The RAP may consider age of the structure to contribute to the likelihood of corrosion damage, but to a much lesser extent relative to a leaking joint, and assign a 10 point scale.

Once the overall importance of the attribute is identified, different conditions or situations may be described to distribute points appropriately based on the engineering judgment of the RAP. Again using the joint as an illustration, if the joint is leaking or can reasonably be expected to be leaking during the time interval, it will have the highest effect and be scored the full 20 points. If the joint is debris-filled or exhibiting moderate leakage, a score of 15 points may be appropriate, if there is a joint, but it is not leaking, a score of 5 points may be assigned. If the subject bridge is jointless, a score of 0 points may be used. The exact scoring for a given attribute may vary according to the design characteristics or operational environment of a particular bridge inventory. The key attributes and ranking scores are then used to develop a simple scoring process that ranks the reliability characteristics of a particular element, for a given damage mode, as a rational means of assessing the appropriate OF.

The scoring methodology is intentionally flexible to adjust to the needs and requirements of different bridge inventories, while still providing a systematic process to document rationale for the OF assessment. It is not a "one size fits all" approach, but rather intended to conform to the varying needs of different operational environments and bridge inventory characteristics. The commentary section of the Guideline, Appendix E, provides suggested scoring and rationale for more than 50 common attributes that might be identified by a RAP assessment of concrete and steel bridges. Alternatively, the RAP may identify additional attributes that meet the needs of a particular inventory, and develop rationale explaining the purpose and assessment process for the attribute. Suggested scoring weights for the attributes may also vary according to the needs and experiences within particular operational environments. Calibration of the scoring process is obviously required to ensure the overall assessment of attributes is consistent with engineering judgment.

Certain key attributes should be identified as part of criteria for reassessment of bridge inspection requirements, following subsequent inspections. These attributes are typically associated with condition, which may change over the service life of the bridge as deterioration occurs. When changes in these condition attributes can result in a change in the likelihood of a given damage mode resulting in failure (i.e., the OF), reassessment of the inspection requirements is necessary.

Several illustrative examples of attribute scoring are also provided as guidance in making the assessments. This includes scoring sheets for tabulating scores for different elements of bridges, and using those scores to determine the OF. However, once attributes and attribute rankings for bridge elements are determined by an RAP, the scoring may be more readily accomplished by integrating or developing software for scoring characteristics of bridge elements more efficiently. Some of the attributes identified by the RAP may already be stored in existing databases and bridge management systems; others may need to be acquired from inspection reports, bridge plans, and other sources.

3.4.1.6 Use of Surrogate Data

For many bridges, the use of "surrogates" for the attributes identified in the reliability analysis may be considered to improve the efficiency of the analysis for larger families of bridges. As used herein, "surrogate" refers to specific data that can be used to either infer or determine another piece of information that is required for the reliability assessment. For example, assume a fracture critical bridge was designed and built in the year 2000, which is well after the implementation of the AASHTO/AWS Fracture Control Plan. This information can be used to determine that the steel must at least meet certain minimum toughness requirements, and the bridge meets modern fatigue design requirements. Note that this was determined only from the date of construction and with no detailed review of the design calculations or specifications.

As stated, the use of surrogates is particularly attractive when identifying and assessing a family of bridges. Design and loading attributes identified by the RAP are typically static in nature, that is, they do not change over time. The condition attributes will typically change over time, as damage accumulates and deterioration mechanisms manifest. However, when elements are in generally good condition, specific condition attributes identified by the RAP may not require individual assessment for each bridge or family of bridges; the previous inspection results can simply be used as a surrogate for the individual attributes. This will typically allow for larger groups of bridges of similar design to be grouped into a particular inspection interval, based on the criteria developed by the RAP. For example, again considering steel bridges built to modern design standards, it is known that the design attributes that would increase the likelihood of fatigue cracking and fracture have been mitigated through improvements in the design, fabrication, and construction process. The condition attributes that are required to assess the reliability of the element would include the presence of fatigue cracks due to out-of-plane distortions, fatigue cracking due to primary stresses, and corrosion damage. However, if the component rating is 7, in good condition according the NBIS scale, or CS 1 in an element-level scheme, the existing ratings can be used as a surrogate for the condition attributes. *Note: This assumes the inspection result is from an RBI procedure, i.e., the inspection was capable of identifying if fatigue cracks existed.* This allows all bridges that are of this same rating (and similar design and condition attributes) to be treated collectively in a process that does not require much detailed analysis of individual bridges. If the condition rating or condition state changes, then the bridges can be reevaluated according to the RAP criteria. If the condition does not change between periodic inspections, reassessment may not be necessary.

It is important to note that this process is significantly different than assigning an inspection interval based simply on the current condition of the bridge; for example, deciding to inspect all steel bridges with rating of 7 on a longer interval than all of those rated a 6. The RAP analysis forms a rationale that identifies not only the current condition attributes that affect the reliability of the element, but also the design and loading attributes of the bridge or bridge element that affect the *potential* for damage to occur. In other words this RAP evaluation forms an engineering rationale for the decision-making process that considers not only the condition of the element, but also the damage modes and the potential for that damage to occur.

For element-level inspection schemes, the attributes identified by the RAP may map directly to an element and element condition state. For example, consider that the RAP identifies leaking joints as an attribute driving the likelihood of section loss in the bearing area of a steel beam. The element condition state (joint leaking) is recorded in the inspection process and can be used as a criterion for that attribute score. In some cases, all of the attributes identified by the RAP as being critical to the likelihood of failure of an element may be included in a comprehensive element-level inspection process; in other cases, they may not.

For NBI-based inspection schemes, attributes identified by the RAP may map to sub-element data collected in addition to the required condition ratings for the primary components of the bridge. This data could be used if it is collected under a standardized scheme for rating and data collection for the sub-elements. For the primary components, the generalized nature of the component rating makes this more difficult for specific attributes.

3.4.2 CFs

The second factor to be assessed under the RBI process is the CF, a categorization of the likely outcome presuming a given damage mode were to result in failure of the element being considered. The assessment of consequence is geared toward assessing and differentiating elements in terms of the consequences, assuming that failure of the element occurs. It should be noted that *failure of an element is not an anticipated event when using an RBI approach*, rather the process of assessing the consequences of a failure is merely a tool to rank the importance of a given element relative to other elements for the purpose of prioritizing inspection needs.

The CF is used to categorize the consequences of the failure of an element into one of four categories, based on the anticipated or the expected outcome. Failure scenarios are considered based on the physical environment of the bridge, typical or expected traffic patterns and loading, the structural characteristics of the bridge, and the materials involved. These scenarios are assessed either qualitatively, through necessary analysis and testing, or based on past experience with similar failure scenarios. The four-level scale used to assign the CF is shown in Table 5. The CF ranges from low, used to describe failure scenarios that are benign and very unlikely to have a significant effect on safety and serviceability, through catastrophic scenarios, where the threat to safety and life is significant. Thus, both short-term (generally safety related) and long-term (generally serviceability related) consequences can be considered.

In assessing the consequences of a given damage mode for a given element, the RAP must establish which outcome characterized by the CFs in Table 5 is the most likely. In other words, which scenario does he or she have the most confidence will result if the damage were to occur. Using the illustration of brittle fracture in a girder, it is obvious that the most likely consequence scenario would (*and should*) be different for a 150 foot span two-girder bridge than for a 50 foot span multi-girder bridge. For the short-span, multi-girder bridge, an engineer may state with confidence that the most likely consequence scenario is "High" and that "Severe" consequences are very remote for a multi-girder bridge, based on his/her experience and the observed behavior of multi-girder bridges. For the two-girder bridge, the consequence scenario is likely to be "Severe." As this example illustrates, the CF simply ranks the importance of the damage mode as being higher for a two-girder bridge than for a multi-girder bridge. For many scenarios, qualitative assessments based on engineering judgment and documented experience are sufficient to assess the appropriate CF for a given scenario; for others, analysis may be necessary using suitable analytical models or other methods. A series of more detailed criteria for specific elements (i.e., decks, steel girders, P/S girders, etc.) are provided in the Guideline that can be utilized during the assessment to determine the appropriate CF for a given element failure scenario. These criteria, combined with owner-specific requirements developed in the RAP or from other rational sources for assessing bridges and bridge redundancy, are then used to determine the appropriate CF for a given scenario.

3.4.3 Inspection Procedures in RBI

Conducting a reliability-based assessment of the inspection needs for bridges requires specific information regarding the current condition of bridge elements that allows for the assessment of expected future performance. For example, to determine the appropriate OF for corrosion damage in a steel bridge element, one would have to know if corrosion damage were currently present and to what degree or extent. Without this information, it would not be possible to assess the likelihood of severe damage developing over the next 72 months. Therefore, it is necessary under the RBI approach to perform inspections that are capable of detecting and evaluating relevant damage modes in a bridge. The relevant damage modes for specific bridge elements are identified through the RAP analysis of the OF, and this assessment provides foundation for the inspection scope and procedures to be used in the field for future inspections. The thoroughness of the inspection process is typically increased relative to, for example, component-level approaches that require only a single rating for a component (superstructure, substructure or deck).

The methods or procedures used to conduct the inspection must be capable of reliably assessing the current condition of the bridge elements for the specific damage modes identified through the RBI process. In many cases, visual inspection or visual inspection supplemented with sounding may be adequate for conducting RBI. The inspections may be hands-on, such that damage is effectively identified to support the reliability assessment. For example, when assessing the likelihood of severe fatigue cracking in a bridge (the OF), it would be necessary to know if there were *currently* fatigue cracks. To make

Table 5. CFs for RBI.

Level	Category	Consequence on Safety	Consequence on Serviceability	Summary Description
1	**Low**	None	Minor	Minor effect on serviceability, no effect on safety
2	**Moderate**	Minor	Moderate	Moderate effect on serviceability, minor effect on safety
3	**High**	Moderate	Major	Major effect on serviceability, moderate effect on safety
4	**Severe**	Major	Major	Structural collapse/loss of life

that assessment, sufficient access to the superstructure of a bridge is required to determine if fatigue cracking is currently present, obviously, and the inspection procedure must include reporting the presence or absence of fatigue cracks. In some cases, NDE techniques may be required within the inspection procedure to allow for reliable detection of certain damage modes identified through the RBI analysis. For example, if the RAP identifies cracking in a bridge pin as a credible damage mode because a bridge has pin and hanger connections, a visual inspection is inadequate. Because the surface of the pin where cracking is likely to occur is not accessible, due to interference from the hanger plates, beam web and reinforcements, ultrasonic testing (UT) or other suitable NDE technology is necessary to allow for the cracking to be assessed.

The RAP analysis of the OFs and CFs provide a basis for the inspection requirements to be used in the field, by identifying credible damage modes and prioritizing these damage modes based on their potential effect on safety and serviceability. Based on the assessment of the OFs and the CFs, damage modes for a bridge can be prioritized based on the product of these factors:

$$IPN = OF \times CF$$

Where *IPN = Inspection Priority Number*. For example, if the fatigue cracking has a moderate likelihood of occurring and the consequence is severe, then the IPN would be $3 \times 4 = 12$. If fatigue cracking were moderately likely, but the consequence were only moderate (minor service disruption), for example, if the bridge in question is a short-span, multi-girder bridge with known redundancy, the IPN for that damage mode would only be $3 \times 2 = 6$. This process highlights the damage modes that are most important, that is, most likely to occur, and have the greater associated consequences if they did occur. This information is included in the inspection procedure for the bridge, providing guidance to the inspectors on emphasis areas for the inspection, based on the engineering analysis and rationale developed by the RAP.

It should be noted that the calculation of the IPN for each damage mode identified in the process does not limit the scope of the inspection to only those damage modes. However, it provides a simple method of prioritization of damage modes that are most important, based on a rational assessment that incorporates bridge type, age, design details, condition, etc., as well as the associated consequences. The resulting outcome from the RAP analysis provides inspection requirements that are tailored to the specific needs of the bridge and include a prioritization of the damage modes for that bridge. This provides a more focused inspection practice that is based on an engineering assessment of the specific bridge or bridge type in order to improve the effectiveness and reliability of the inspection.

3.4.3.1 Reliability of Inspection Methods

For most RBI planning processes, such as those used for assessing cracking in nuclear power plants or oil and gas facilities, the reliability of different inspection strategies or methods is considered the assessment (*21, 23, 29*). For inspection technologies, reliability is typically defined by a measure of the ability of the technology to perform its intended function. Reliable and effective inspection methodologies reduce the uncertainty in the current condition of components, and therefore can affect future POF estimates and rationale for a given inspection interval. The reliability of specific inspection methods may be quantified using probability of detection (POD) or other reliability analysis for a limited number of especially high-risk components and damage scenarios. This may be justified based on the significant risk associated with these facilities, including both the high cost and high environmental consequences of certain failure modes. However, for more general assessments of risk, the effectiveness of inspections is qualitatively described to rank various inspection approaches on a relative scale using engineering judgment. For example, API has created a five-category rating system used for several components described in API 581 (*29*). Inspection methods are qualitatively categorized on a scale that ranges from A to E, with A being "highly effective" and E being "ineffective."

A similar approach was taken to develop guidance on the reliability or effectiveness of inspection methods for typical damage modes anticipated for common bridge elements. Tables included in the Guideline indicate the reliability of NDE technology for various damage modes for specific bridge elements, such as steel beams, concrete decks, etc. The reliability of the inspection method is described on a four-level qualitative scale and represented symbolically. Methods that are generally unreliable for a given damage mode or mechanism are described as "Low" and methods expected to provide high reliability and effectiveness are "High." The assessments of the reliability of inspection methods were made using expert judgment, literature review, experience, and data from other industries, where available (*46*). Information on the relative costs of different methods is also included as guidance. The Technical Readiness Level (TRL) of different methodologies is also provided and describes if the methodology is a commonly available tool that is readily accessible, if the method is specialized such that specialized expertise is required for implementation, or if the method is experimental in nature.

Presently, there is somewhat limited reliability data available for many bridge inspection techniques and NDE technologies applied for bridge inspection. In part this is because historically there has been little motivation to conduct such testing, since the inspection intervals are uniform and generally do not require any formal demonstration of effectiveness of the inspection procedure. However, in an RBI approach, where inspection intervals may be longer based on rational assessments of potential damage, inspection scopes may need to be appropriately adjusted. As a result, determination of the reliability of the inspection method becomes a factor in the overall approach to the inspection process. Reliability data such as that provided in the Guideline is expected to be refined and developed over time, as the reliability-based approach is implemented for existing bridge inventories. The tables provided in the Guideline provide the framework for including such analysis in the RBI methodology. These tables provide user guidance for identifying appropriate inspection methods and/or NDE technologies to address specific anticipated damage modes.

3.4.3.2 Element-Level vs. Component-Level Inspections

There exists under the current implementation of the NBIS a variety of approaches to collecting, documenting, and storing data on bridge inventories within individual states. While many states are licensed to use the PONTIS bridge management system, which is an element-level process for storing inspection information and evaluating future programmatic needs, the degree to which states fully implement the element-level inspection process varies. Other states use the component-based system that is required under the NBIS; still others use a span-by-span approach. However, to implement the RBI process, more detailed information than that typically required for a component-based system is needed. A component-level approach, which is intended to provide a single average or overall rating for the three major bridge components, does not provide sufficient data for assessing the likelihood of future damage developing for most cases, and as such will not support an RBI analysis. Information on the specific damage modes present on the bridge, their location, and their extent are needed to assess inspection needs. As a result, inspection needs under an RBI process are more closely aligned with more detailed, element-level systems. The key characteristics that are needed to support the RBI assessment are as follows:

- Report the damage mode or modes affecting key elements of the bridge,
- Report the location and extent of the damage, and
- Report on key damage precursors as developed through the RAP assessment.

Precursors identified through the RAP process may include evaluating specific elements of the bridge such as the joints or drainage systems. Specific conditions that are precursors necessary to assess the likelihood of damage in the future will also be needed, such as the presence of rust-stained efflorescence or fatigue cracking. Many of these may be found in the current *AASHTO Bridge Element Inspection Manual* (*26*), in many cases as bridge management elements or defect flags. The bridge management elements and defect flags may need to be more fully developed under the RBI process as needs develop for specific inventories.

3.4.4 RAP

The RAP is an expert panel assembled at the owner level to conduct analysis to support RBI by assessing the reliability characteristics of bridges within a particular operational environment and the potential consequences of damage. The performance characteristics of bridges and bridge elements vary widely across the bridge inventory due to a number of factors. Variations in the ambient environmental conditions obviously have a significant effect, since some states have significant snowfall, and, as a result, apply de-icing chemicals to bridges frequently, while other states are arid and warm, such that de-icing chemicals may be infrequently or never applied. Design and construction specifications vary between states. Typical details such as drainage features, and use of protective coating or other deterioration inhibitors, for example, sealers for concrete, vary between bridge owners as do traditional construction practices, construction details, and materials of construction. In terms of consequences, redundancy rules and traditional policies vary somewhat between bridge owners, with some bridge owners requiring four members to be considered redundant, while others require only three, for example. Owners may also have policies specifying girder spacing or other configuration requirements. All of these factors contribute to the operational environment of a bridge that affects the likelihood and rate of deterioration of bridges and bridge elements, and, to a lesser extent, the assessment of the potential consequences of that damage. As a result, knowledge and expertise of the operational environment, historical performance characteristics, bridge management and maintenance practices, and design requirements for bridges and bridge elements are essential for conducting reliability-based assessments.

The role of such expert knowledge of a specific operational environment is a typical component for reliability or risk-based assessments of inspection needs. It is necessary that individuals with historical knowledge of the operational environment and typical deterioration patterns within that environment participate in the process. This participation is needed to effectively assess reliability characteristics of bridge elements and to identify and prioritize key attributes and factors that support the rational characterization of the OFs and CFs. To utilize this expert knowledge, which is inherently local to a specific bridge inventory, a RAP is formed at the owner level to conduct the reliability-based assessment. The RAP panel typically will consist of four to six experts from the bridge-owning agency. This team should include an inspection team leader or program manager that is familiar with the inspection procedures and practices as they are implemented for the inventory of bridges being analyzed. The team should include a structural engineer who is familiar with the common load paths and the overall structural behavior of bridges, and a materials engineer who is familiar with the behavior of materials in the particular environment of the state and has past experience with materials quality issues. Experts from outside the bridge-owning agency, such as academics or consultants, may be used to fill technical gaps, provide independent review, or simply supplement the RAP knowledge base as needed. A facilitator may also be used to assist in the RAP process.

3.4.4.1 RAP Expert Elicitation

Expert elicitation is a method of gathering insight into the probability or likelihood of failure of a component, or of evaluating associated consequences when insufficient operational data exists to make a quantitative, frequency-based estimate. When failures are rare, or it is necessary to predict future failures, expert elicitation is used to provide quantitative or qualitative estimates (categories) for use in assessing inspection needs or the likelihood of adverse future events. Processes for expert elicitations are common in nuclear applications and other safety-critical industries for performing risk assessments of operating events and assessing in-service inspection needs (*21, 22, 47, 48*). Key elements of the elicitation process include assembling appropriate subject matter experts and framing the problem to be assessed for the experts in order to elicit objective judgments. Consensus processes are used to aggregate expert judgments and ensure contributions from all of the experts involved (*21*). For RBI for bridges, expert elicitation is used to:

- Categorize the OF based on expert judgment:
  - Determine credible damage modes for bridge elements and
  - Identify and prioritize key attributes that contribute to the reliability and durability of bridge elements.
- Assess likely consequence scenarios and categorize the CF.

The processes to elicit expert judgment from the RAP are simple and relatively straight-forward. The primary purpose of the processes is to provide a systematic framework that allows for efficient, objective analysis, and allows for input from all members the RAP. This allows for their expertise to be utilized and for dissenting judgments or views to be resolved such that issues are addressed as comprehensively as possible. For example, to identify the credible damage modes that are specific to the type of bridge and the element being considered, the problem is framed for the panel by describing the element under consideration and its operational environment. The following question is then posed to the RAP: "The inspection report indicates that the element is in serious condition. In your expert judgment, what is the most likely cause (i.e., damage mode) that has produced/resulted in this condition?" This elicits from the panel a listing of damage modes that are likely to occur for that element.

Each expert is asked to independently list the damage modes he/she judges are most likely to have resulted in a failure of the element. The expert records each damage mode and provides an estimate of the relative likelihood that each damage mode would have resulted in the element being in serious condition. The expert does this by assigning relative probabilities to each damage mode, typically with a minimum precision of 10% (the sum of the ratings should be 100%). The expert may note supporting rationale for the estimate. The individual results from each member of the RAP are then aggregated to evaluate consensus among the panel on the most likely damage modes for the element. An iterative process may be necessary to develop consensus on the credible damage modes for a given bridge element and identify damage modes that are not credible. However, for many elements, the damage modes are well known and consensus may be reached quickly.

Attributes are then identified through a follow-up process. In most cases, the key attributes for a given damage mode can be identified by posing the following question to the RAP:

- Consider damage mode X for the subject bridge element. If you were asked to assess the likelihood of serious damage occurring in the next 72 months, what information would you need to know to make that judgment?

This generates input from the RAP on what attributes of the element are critical for decision making regarding future expected behavior. The resulting input from the RAP can be categorized appropriately and ranked according to the relative importance of the attribute for predicting future damage for the identified damage mode and element. While there are potentially many attributes that contribute to the durability and reliability of a bridge element, it is necessary to identify those attributes that have the greatest influence on the future performance of an element. Rationale for each attribute is documented, either by using rationale already provided in the Guideline, or developing suitable rationale through a variety of means including past performance, experience with the given bridge element, input from the RAP members, previous and contemporary research, analysis of historical performance, etc.

Expert elicitation is also used for assessment of the CF by providing different potential failure and consequence scenarios and asking the RAP to assign relative likelihood to the outcome of the failure according to the CF scale. This is a useful tool for evaluating the appropriate CF for situations that are not well-matched to the examples and criteria provided in the Guideline, or to establish basic ground rules for the assessment of common situations. The process involves a few basic, but critical steps as follows:

1. *Statement of the Problem*: The RAP is presented with a clear statement of the problem and supporting information to allow for expert judgment to be made. Care should be taken to ensure the problem statement does not contain information that could lead to a biased decision. The problem statement typically includes data regarding the bridge design, location, typical traffic patterns, and the failure scenario under consideration.
2. *Expert Elicitation*: Independently, each member of the RAP is asked, based on judgment, experience, available data, and given the scenario presented, to determine the most likely consequence resulting from the damage mode under consideration. The expert is asked to express this as a percentage of the likelihood, with the smallest unit of estimate typically being 10%. The experts may provide a statement on what factors they considered in making the estimate.
3. *Comparison of results*: Once each member of the RAP has rated the situation, the results of the elicitation are aggregated. Generally, there will be consensus regarding the most likely consequence. However, in some cases, the most likely choice may not be clear and there will not be consensus.
4. *Identify CF*: If there is consensus among the panel regarding the appropriate CF, then the rationale for making the determination is recorded. This rationale should be consistent with criteria provided in the Guideline and if not, the panel documents the deviation or changes and associated rationale.

For cases in which consensus is not reached in the initial elicitation, the experts should discuss their rankings and their assumptions and rationale for their specific judgments. The members of the RAP should then be given the opportunity to discuss the various judgments and to revise their scores based on the discussion. In some cases, additional information may be needed to support developing a consensus regarding the appropriate CF. If consensus cannot be reached, a potential approach would be to adopt the most conservative consequence scenario that was included among the revised scores. Exceptions to the selected likelihood scenario should also be documented.

The RAP may determine that additional analysis is required to determine the appropriate consequence for a given damage scenario. In some cases, additional data collection may be required in order to reach a consensus. Individual RAPs have the flexibility to develop effective methodologies to address cases in which consensus cannot be reached. However, the method must result in the selection of the most appropriate CF, based on the Guideline provided and sound engineering judgment.

3.4.4.2 Example of Expert Elicitation

This section provides an example expert elicitation as an illustration of the RAP process. As part of the research for NCHRP Project 12-82, an expert panel was assembled of state bridge engineers and inspection experts from seven different states and an engineer from the FHWA. The goal of the two-day meeting was to have experts from several state DOTs contribute to the development of reliability and RBI practices for highway bridges by providing owner perspective on the approach and tools being developed. The participants in the meeting represented a good cross section of personnel from state departments of transportation, ranging from personnel responsible for overseeing bridge inspection activities at the district level through the state-wide programs for inspection and maintenance.

The meeting covered many of the topics necessary to operate a RAP at the state level, including identifying key damage modes for certain bridge elements, identifying and weighting bridge element attributes that contribute to the durability/reliability of the element, and evaluating the consequences of various damage modes. Among the activities at the meeting was a trial of the suggested expert elicitations processed utilized in the Guideline for conducting the reliability analysis needed as part of RBI practices. This section of the report provides example results from this workshop to illustrate the elicitation process and sample data provided by a cross section of practicing engineers. Although this panel included individuals from a variety of operational environments, and results of the elicitation process would likely have differences within a specific environment, the results are included here to illustrate the process and provide typical results. The example presented here includes the results for a steel bridge superstructure. These same processes were used during RAP meetings held as part of two case studies of the technology, reported in Section 3.6.

3.4.4.2.1 Identifying Damage Modes. The process for determining credible damage modes based on an expert elicitation was conducted during the workshop. The goal of the exercise was to identify the most likely and credible damage modes for the element and establish the consensus (or lack of consensus) of the panel regarding the most common damage modes for that element. The panel was asked to perform this assessment for a steel girder. The following question was

Table 6. Example of expert elicitation worksheet for steel girder damage modes.

Damage Mode	Likelihood (in 10% increments)
Corrosion / Severe Section Loss	● ● ● ● ● ○ ○ ○ ○ ○
Fatigue Cracking	● ● ○ ○ ○ ○ ○ ○ ○ ○
Impact Damage/ Fire	● ● ○ ○ ○ ○ ○ ○ ○ ○
Overload	● ○ ○ ○ ○ ○ ○ ○ ○ ○
Stress Corrosion Cracking	○ ○ ○ ○ ○ ○ ○ ○ ○ ○
	10% 20% 30% 40% 50% 60% 70% 80% 90% 100%

posed to the panel, "You are told a steel girder is condition rating 3, serious condition, according to the current NBIS rating scale. Based on your experience, what damage is likely to be present?" The expert was provided a form similar to that shown in Table 6, except that the damage modes and likelihood indicators were blank. Each member of the panel completed the form, identifying the damage modes and relative likelihood with a precision of 10%. Table 6 illustrates the results provided by one of the panel members. As shown in the table, this member rated corrosion/section loss as the most likely damage mode to be present, with fatigue cracking and impact damage as less likely, and overload as a possible damage mode. In this case, the panel member identified stress corrosion cracking as a possible damage mode, but one that was very unlikely such that no likelihood was assigned for that damage mode.

Figure 4 shows the results from each of the panel members for this elicitation exercise. It was the consensus of the panel that the most common damage mode for a steel girder was corrosion damage/severe section loss. This damage mode was selected by everyone on the panel, typically with values of greater than 50% likelihood.

The methodology is simple for many bridge elements for which damage modes are well known, and it establishes the consensus of the panel in regards to the most likely damage modes. It also helps to identify damage modes that may be less well known, but of concern for the particular state or bridge inventory. For example, one member of the expert panel had a different view of the most likely damage modes for a steel girder, marking impact damage (40%) as the most likely damage mode in his/her state. The particular state has large areas of arid environment, and hence a different perspective

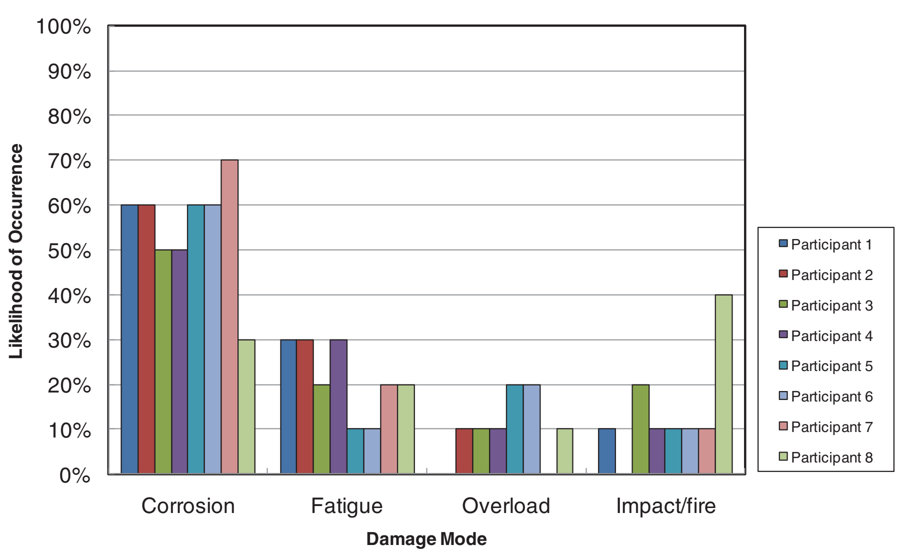


Figure 4. Results of expert elicitation on steel girder damage modes.

on the most likely damage modes. This illustrates how RAPs in different operational environments may identify and prioritize damage modes differently, depending on their operational environment and experiences managing their bridge inventory. This is an advantage of the methodology, as damage modes that are most important to a given bridge inventory are identified through the process; these damage modes are not necessarily the same across the diversified operational environments of bridges across the country.

It should be noted that this type of expert elicitation is a process for identifying and prioritizing likely damage modes for a given bridge or family of bridges based on expert judgment. It is not necessarily repeated over and over again for cases in which damage modes are well known. Rather, it is a tool for establishing that there is agreement on the most likely damage modes, capturing the expert judgment of the panel, and ensuring that the analysis is comprehensive and considers all *credible* damage modes.

Through this process, damage modes for which the likelihood is very small or essentially zero can be sorted out from more common damage modes through a rational process. In most cases, as was shown here, likely damage modes are expected to be well known by experienced bridge engineers and consensus can be readily achieved.

3.4.4.2.2 Attributes. Once the primary damage modes were identified, the panel considered the individual damage modes identified and the element attribute that contributed to the reliability of bridge element. For example, for the damage mode of corrosion/section loss, the expert elicitation consisted of posing the question to the panel, "For the steel girder, you are asked to estimate how long it will be before significant corrosion/section loss would occur for that bridge. What information would you need to know to make that estimate?" A group discussion was held to identify and discuss the key attributes, and discuss their relative importance to determining the future deterioration pattern for the steel girder. The panel suggested that one of the most important attributes was the maintenance cycle for the bridge, or the maintenance activities that were typically performed as part of normal operations. This includes such activities as bridge washing, cleaning away of debris that may accumulate, and maintenance of joints. The consensus of the panel was that this was a highly important attribute that should contribute to the rationale. The panel also identified that the bridge deck type was an important attribute that could potentially be a screening criteria for those bridges that have, for example, open-grated or timber decks. The panel identified that built-up members with the potential for crevice-type corrosion, micro-environments associated with traffic overspray, and condition history (trend data) were other attributes that could be considered in assessing the future performance of steel bridge elements in terms of corrosion.

The attributes were ranked according to their importance as high (H), medium (M), or low (L), and if the attribute was potentially a screening criteria (S). Table 7 summarizes the results of the discussion.

The attributes identified by a particular RAP in a specific operational environment may differ from those indicated in Table 7; however, these results are provided as an illustration of the process of eliciting expert judgments from a RAP. Once the attributes are identified and ranked appropriately, a simple scoring regime can be developed based on the results and used to categorize the OF based on these attributes.

3.4.4.2.3 Consequence Scenarios. Expert elicitation to determine CFs was also demonstrated. An overview of the process for selecting the appropriate consequence category for a given damage mode was presented to the panel. This overview included several examples of different consequence scenarios that might be experienced during the evaluation process, and a review of the draft criteria for assessing the CF within an RBI process.

An exercise was conducted to illustrate and test the use of expert elicitation for evaluating the likelihood of different

Table 7. Summary of attributes identified by the expert panel for steel superstructures.

Design Attributes		Loading Attributes		Condition Attributes	
Attribute	Rank	Attribute	Rank	Attribute	Rank
Deck Joints/Drainage	H	Macro Env.	H,S	Existing Condition	H,S
Built-Up Members	M	Micro Env.	H	Joint Condition	H,S
Deck Type	M,S			Maintenance Cycle	H
Material Type	L,S			Condition History Trend	M
Age	L			Debris Accum.	M

consequence scenarios. The purpose of this exercise was to determine if, given a certain damage scenario, there could be consensus on the most likely outcome of that damage, based on the defined consequence scenarios and applied to a specific bridge. This process can be used by an RAP to develop and illustrate consensus and agreement with the Guideline for assigning consequence categories, to address situations that may not be sufficiently addressed or unclear, or to address unique situations for which expert judgment is required. The bridge presented to the panel was a multi-girder steel bridge with an Average Daily Traffic (ADT) of 2000 and spanning a divided state highway. Photographs of the bridge and descriptions of its structural configuration were provided to each member of the panel in written form for use in assessing different damage modes and associated consequence scenarios.

Each panel member was provided with a handout that included basic directions, a bridge description, photographs of the bridge, and nine different damage scenarios to evaluate independently. The panel members were asked to complete the bubble chart for each damage scenario, as shown in Table 8. The results were collected and reduced to summary charts showing the average assigned likelihoods.

There were nine damage scenarios presented to the panel, ranging from fracture of primary member to delamination and spalling of piers and abutments. The results of this exercise indicated that for certain scenarios, there was strong consensus on the most likely consequence of the indicated damage. For example, for the following scenario:

"*The overlay is debonding; approximately 20% of the deck is spalled.*"

The assessment of the panel was distributed as shown in the Figure 5. As shown in the Figure, there was consensus from the panel's independent assessments that this scenario represented a low to moderate consequence. Discussion of this scenario indicated that some panel members judged that the consequences could be high, based on their interpretation of the failure scenario presented. Discussion of the assessments quickly yielded assessment that the appropriate CF was moderate.

Table 8. Sample table for assessing likelihood for damage scenarios.

Consequence Category	Likelihood (%)
Low	○○○○○○○○○○○
Moderate	○○○○○○○○○○○
High	○○○○○○○○○○○
Severe	○○○○○○○○○○○

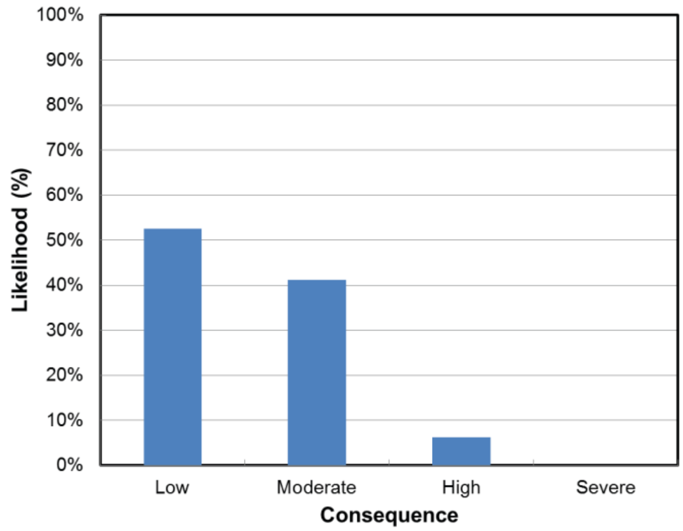


Figure 5. Likely consequences of general deck spalling.

A second scenario of interest was a comparison between fatigue cracking due to out-of-plane distortion vs. fatigue cracking due to primary stress. For the former, the panel rated the most likely consequence as moderate (~50% likelihood), for the latter, the most likely consequence as high (>60% likelihood)—for the multi-girder bridge utilized in the exercise. It was also interesting that a scenario of one beam fracturing was similar to a primary stress fatigue crack, >60% likelihood that the consequence would be "high" according to the consequence categories provided. Figure 6 shows the average outcome for the fracture of one of the steel beams, and as indicated in the figure, the elicitation indicated that the most likely outcome/consequence for this scenario would

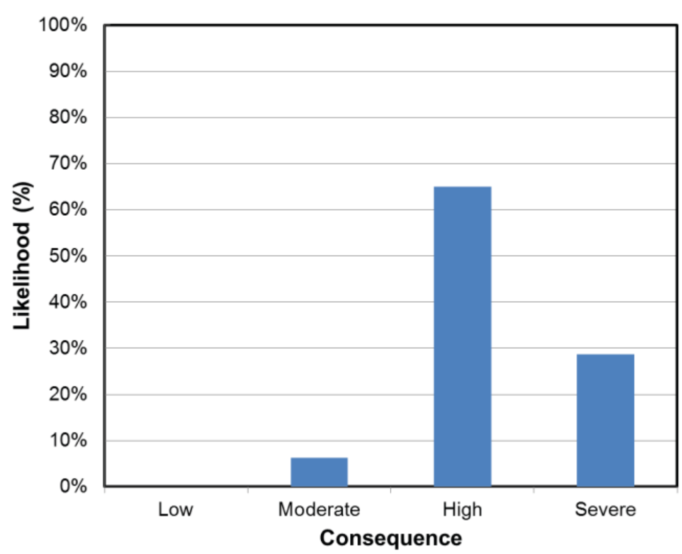


Figure 6. Likely consequences of beam fracture.

be "high." These data, which illustrate the consensus formed from the independent assessments of individuals from a number of different states, would be refined when applied within a specific bridge inventory and operational environment. A series of criteria and requirements are provided in the Guideline to assist in this process, and in many cases the CFs may be governed simply by the criteria in the Guideline or owner policies regarding the treatment of redundancy or other factors. In other cases, additional analysis or testing needed may be identified through the process. For cases not easily addressed or well defined, this type of expert elicitation is especially useful as a tool for developing rationale to support the categorization of the CF or identified specific analysis needs.

These examples illustrate the process and feasibility of expert elicitation for determining the key factors required in the RAP assessment. The decisions regarding the likely damage modes and potential consequences are very similar to decision processes currently utilized by bridge engineers to determine the urgency of repair needs, anticipate future repair needs, and manage bridge inventories to ensure safety and serviceability of bridges. These decision processes are simply collected and aggregated systematically to provide rationale for decision making regarding bridge inspection requirements. Additional testing of the processes and evaluation of the consistency of the elicitation outcomes were conducted through case studies reported in Section 3.6.

3.5 Data to Support RBI Analysis

There are a number of resources available or that could be developed to support the RAP assessment of the OF for bridge elements by providing data to support decision making. While none of these sources for data provide perfect solutions, for example, for calculating quantitatively the OF, they can provide data that supports decision making and rationale developed through the RBI process. This section of the report describes a few of these resources, as well as important consideration for utilizing these data for the reliability assessment of bridges. First, use and application of the qualitative and quantitative data is described.

3.5.1 Quantitative vs. Qualitative Analysis

Industrial standards for reliability and risk assessment recognize both quantitative and qualitative methods for estimating the POF and consequences of failure. *Qualitative* data typically are composed of information developed from past experience, expertise, and engineering judgment. Inputs are often expressed in data ranges instead of discrete values and/or given in qualitative terms such as high, medium, and low (although numerical values may be associated with these levels) (*21, 23, 29, 49*). *Quantitative* data are data developed through specific probabilistic models, databases of failure rates, or past performance data such as deterioration rate models. These data are typically more in-depth and detailed than qualitative data. This can provide valuable insight and uniformity in approach, but developing such data can be impractical for realistic situations that are too complex to be modeled effectively. Data on past performance are frequently incomplete or inaccurate, and in some cases can provide ineffective estimates of future performance (*50*). Additionally, the effort required to collect and analyze the data may far outweigh the value of the data in estimating future performance, particularly when the data are sparse, include a large uncertainty, or design characteristics are evolving.

Qualitative data enables the completion of assessments in the absence of detailed quantitative data. This qualitative data can be augmented with quantitative data when and where available, forming a continuum of data as shown in Figure 7 (*23*).

The accuracy of results from a qualitative assessment depends on background and expertise of the analyst (*21*). Quantitative data, such as deterioration rate information measured from NBI or bridge management software (BMS) data, can provide supporting rationale for decision making, if handled appropriately. Estimates of precise numerical values (quantitative analysis) can imply a higher level of accuracy when compared to qualitative analysis, though this is not necessarily the case, particularly when there is a high degree of uncertainty or variation. It is the quality of the data that is most important to support an analysis, and the fact that data are quantitative does not necessarily mean they are more accurate. Difficulty in effectively representing past experience, expert knowledge, and bridge-specific conditions can result in quantitative data that are biased and inaccurate, or whose applicability to a specific situation is unknown due to a complex array of assumptions utilized in developing the data. As a result, careful elicitation of expert knowledge from those most familiar with the operating environments, historical performance characteristics within those environments, and the expected future performance is used for RBI (*21*). Formal methods for eliciting expert opinion for the purpose of risk assessment are included in the Guideline and in the literature, as previously discussed (*47, 51, 52*).

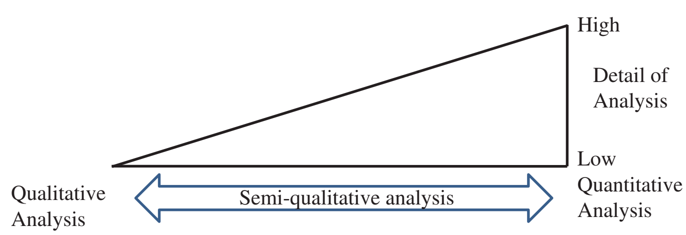


Figure 7. Continuum of data needed for qualitative to quantitative analysis (23).

3.5.2 Data Needed for Assessment

To perform a reliability-based assessment, the primary data required include data on bridge design characteristics and details, materials, environment, and current condition. Bridge inventory data describing the overall characteristics of a bridge, such as can be developed from existing NBI data tables, can provide some information. Data on materials and design characteristics are generally available in the bridge files, typical design and detailing practices, and local knowledge of construction practices. Damage data describe the deterioration active or expected on a structure and estimate its effects on the structure and rate of development. For damage data, sources include general data available including the NBI database, inspection reports and supporting data within a DOT, element-level data for many states, and industrial data such as the experience of bridge owners, previous research, historical data, and historical experience. In some case, deterioration rate data or trends may be available and used as part of the assessment of future performance. Additional data on the anticipated performance of bridge elements is developed through the RAP process based on expert judgment.

3.5.2.1 Deterioration Rate Data and Previous Failure Histories

Deterioration rate data such as that developed by Agrawal (*35*) and others (*40, 53, 54*) can be used to support estimates of future performance of bridge elements. However, there are challenges to applying these data exclusively to determine appropriate inspection strategies for bridges. First, data on bridge deterioration is often not specific, expressed normally in subjective condition ratings that may not capture specific characteristics of the bridge or the deterioration mechanisms that led to a certain condition rating. As a result, making accurate predictions regarding future performance can be challenging. Second, variation in the data is high, such that estimating deterioration curves typically requires advanced probabilistic analysis that develops mean estimates for the population. These mean or average values provide information on expected average performance of an overall population, but not for a specific item within that population. Deterioration rate data may need to be modified to adjust the data to local operating and management conditions to be used effectively to estimate the future performance of specific bridges or bridge elements within a population.

However, deterioration curves and probabilistic failure estimates are valuable to the RBI analysis process in several ways. Deterioration curves can provide background and support rationale for engineering judgment regarding future performance of bridge elements, based on past performance when combined with an assessment of the key attributes for the elements identified through the RAP process. If a bridge owner had a population of bridge elements that were very similar in design, and constructed at the same time and to the same specifications and quality, and exposed to the same environment, then accurate probabilistic estimates of future performance could be developed. Generally, this would be atypical of the bridge inventory. Consequently, the method developed for RBI practices provides a means for incorporating such analysis, but does not rely on these data alone.

Considerations for utilization of deterioration rate data include:

- **Similarity of operational environment**: The RAP should consider if the particular bridge under consideration shares the same operational environment as the elements from which data was obtained. Key elements of the operational environment include the ADT, ADTT, macro-environment of the bridges (severe environment vs. benign environment), micro-environment (salt application, joint and drainage conditions, exposure to overspray), and typical maintenance and management (among others).
- **Similarity of Key Attributes**: Key attributes that affect the damage modes and mechanisms for the bridge element should be similar for the bridge under consideration to those from which deterioration rate data was obtained. This may include materials of construction, design attributes, and condition attributes. Quality of construction and years in service may also be a factor.

Component ratings for superstructure, substructure, and deck (and culverts) are provided for all bridges under the NBIS scheme providing general information on the deterioration of the structural components over time, based on visual observations. Element-level data are documented for states using PONTIS or other element-level inspection schemes. Obviously, these condition data are an important component to evaluating the current condition of a bridge, at least in a general way, and identifying bridges with low or high condition ratings. These data can also be used to construct the deterioration curve data to support assessments, or to make estimates of typical performance characteristics for bridges of a particular design, as described below.

3.5.2.2 Inventory Data Analysis

Data from the NBI database can be analyzed to support the rationale for bridge inspection intervals developed through the RBI process. For example, historical NBI data can be analyzed to determine the average period of time a particular bridge element remains in a certain condition rating. These data can be utilized to support rational decision making and the use of surrogate data, such as utilizing condition ratings

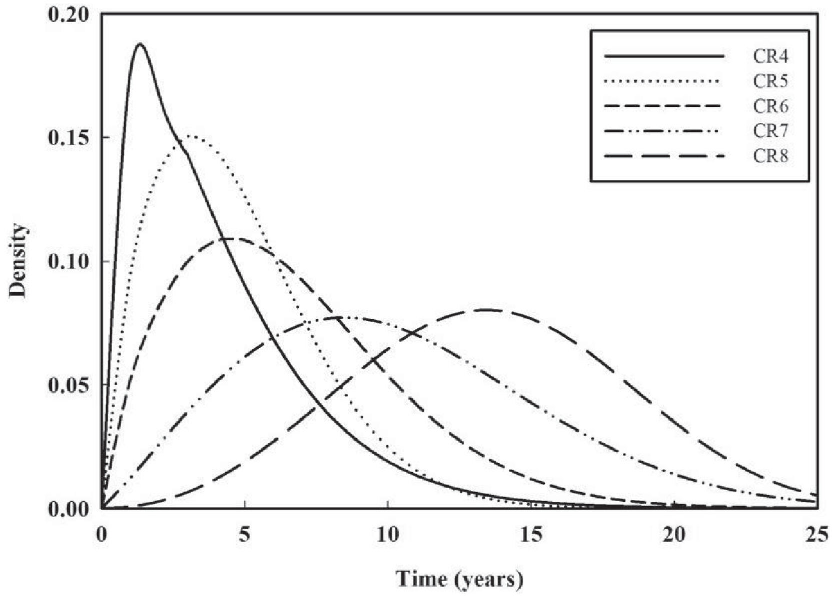


Figure 8. Graph showing Weibull distributions for time-in-condition for prestressed bridges in Oregon.

of 7, "Good Condition," as a surrogate for condition attributes associated with a certain bridge type. For example, Figure 8 shows the time-in-condition for prestressed bridge superstructures in the state of Oregon. These data were developed by examining 20 years of NBI data, and determining from these data the time period (no. of years) individual bridges remained in a certain condition rating, according to the inspection results documented in the NBI. Weibull distributions were used to characterize the distribution of years in rating for this population of bridges, and these Weibull distributions are shown in the figure. Simply summing the mean (average) number of years, historically, that a prestressed bridge has remained in each certain condition rating, assuming a bridge component is currently rated a 7 and changes to a 6 immediately, the average number of years to progress to a condition rating of 3 is ~15 years. Given a maximum inspection interval of 72 months (6 years), at least two inspection cycles would be completed within this 15 year period. During these inspections, if deterioration occurs more rapidly than initially envisioned, the inspection interval is appropriately reduced. These data support the rationale that significant margin exist when considering a bridge currently in a condition rating of 7. When considered within an RBI process, which identifies attributes of bridges that are likely to cause more rapid deterioration, such rationale is well-founded and based on quantitative, historical data.

More complex analysis of such data may also be used, including deterioration curves, probability calculations, etc. BMS, such as the PONTIS program, may provide data on transition probabilities or lifetime estimates based on Weibull statistics, which can be utilized to provide quantitative data to support the RAP analysis. These data can be used to complement the RAP analysis. However, to effectively use these data, information provided through the RAP process is needed to ensure the relevance of the data as discussed in Section 3.5.2.

3.5.3 Industry Data

The RBI practices rely on engineering judgment and experience with performance of engineered structures under actual conditions to estimate future performance. So-called "industry data" are developed from the existing body of knowledge across the industry, frequently contained in the body of research literature available, to inform and support expert judgments. These data may include specific, quantitative data such as would be provided from models, or the combined or collective knowledge based on the existing body of research and past experience across the industry. This section provides two examples of "industry data" that can be used to support analysis under the RBI process: a simple, commonly available modeling example and a collective knowledge example.

There exists a significant body of research concerning the degradation of highway bridges by common deterioration modes. There are two primary modes of deterioration that cause bridge damage—corrosion of reinforcing steel in concrete, and corrosion of steel bridge components. Certainly there are others, such as fatigue cracking, but corrosion and its effects can be associated with much of the damage occurring in bridges over time. Methods of determining the remaining life of elements and details based on fatigue mechanisms are documented and well known. Because of the significant importance of corrosion-based deterioration modes to the degradation of bridges, there exists a significant foundation of knowledge regarding corrosion and its effects on bridges, which can be leveraged to develop estimates of future behavior based on the age, current condition, and design attributes of a bridge. The rate of corrosion of steel and steel embedded in concrete varies widely according to localized conditions, with the local environment being a key factor. Geographical regions where de-icing chemicals are regularly applied generally have significantly higher corrosion rates than regions where de-icing chemical use is low or even nonexistent. The local environment at the bridge, such as leaking joints or poor deck drainage, also has a significant effect. This section discusses generalized data regarding the corrosion rates in steel, for both steel members and steel embedded in concrete. This data is provided to illustrate the type of "industry data" that can be used to support the rationale used by an RAP during the assessment process, and could be further developed if needed to address specific situations, or utilized as current industrial knowledge for general cases.

3.5.3.1 Corrosion in Concrete Structures

The rate at which corrosion damage may develop varies widely for different geographical regions, depending on the level of exposure of the concrete to corrosive agents such as air-borne chlorides, marine environments, and the use of de-icing chemicals. The main factors that contribute to steel corrosion are the presence and amount of chloride ions, oxygen, and moisture. To illustrate how these factors affect structures located in different geographical regions, commercial software was used to generate benchmark corrosion effects models for different regions of the country.

One of the objectives of the modeling was to illustrate the variation in the likelihood of corrosion damage occurring in different geographical locations across the United States. Given that the inspection interval is uniform under the existing system, and that corrosion presents one of the most common and significant forms of damage to bridges, this study was intended to examine how much variation there might be in corrosion rates, and hence inspection needs, to assess corrosion damage across the United States. The results of the study are reported in terms of time to the initiation of corrosion. The time to the propagation of damage varies somewhat but can be considered to be on the order of 6 years for uncoated reinforcement to 20 years for epoxy-coated reinforcement, based on the rate that damage is expected to propagate once initiated in the reinforcing steel (*36*). Design parameters such as the amount of concrete cover, rebar spacing, and concrete material properties obviously affect the rate at which damage will propagate for a specific concrete component. These factors were assumed constant for the purposes of evaluating how quickly the effects of corrosion might be realized across different geographic regions.

Fick's second law of diffusion was used as the governing equation to account for differences between geographic locations, such as temperature levels and ambient chloride concentrations. Fick's second law of diffusion is generally stated as:

$$\frac{dC}{dt} = D * \frac{d^2C}{dx^2}$$

Where

C = the chloride content
D = the apparent diffusion coefficient
x = the depth from the exposed surface, and
t = time

The chloride diffusion coefficient, D, is modeled as a function of both time and temperature, which represents the rate at which chloride ions travel through uncracked concrete. Higher temperatures allow for an increase in chloride diffusion as the ions have more energy to move, as compared to those in cooler temperatures.

For the modeling, the benchmark concrete mixture assumed contained only Portland cement with no special corrosion protection strategies. The value of 0.05 percent by weight of concrete was used as the threshold chloride level for corrosion initiation for the uncoated rebar. This was done to represent a worst case scenario for corrosion initiation, given that no corrosion mitigation strategies were employed. Complete details on the analysis process are available in the literature.

Six states across the United States that represented different geographical regions and thus different chloride build-up rates on the surface of the concrete, resulting from chlorides in the environment and de-icing chemical application, were modeled. These states included Arizona, Arkansas, Florida, New York, Washington, and Wisconsin. For each state, chloride diffusion rates were modeled for rural highway bridges, urban highway bridges, and also for marine zones, where appropriate. Cover depths of 1 inch and 3 inches were used to illustrate the effect of concrete cover over the range of typical cover. Representative results of the analysis for an urban highway bridges are presented here.

Figure 9 visually illustrates the difference in the modeled time to corrosion initiation for different geographic regions. As shown in Figure 9, there are vast differences in the model time to corrosion initiation for different locations across the country. For aggressive climates, such as New York and Wisconsin, corrosion initiated in as little as ~7 years, while in less aggressive environments, such as Arizona, corrosion

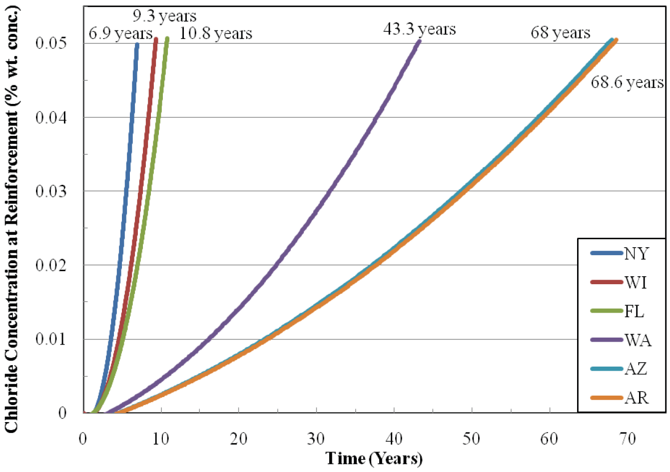


Figure 9. Time to corrosion initiation for different states based on a diffusion model.

initiation is not anticipated for almost 70 years. While this model does not consider localized effects, such as cracking of the concrete that can greatly increase the rate of chlorides intrusion into the concrete, it does illustrate that the time to corrosion for a generic, uncracked case varies significantly across geographic regions.

As shown in the figure, New York, Wisconsin, and Florida have very similar behavior in terms of time to corrosion initiation. These environments would fall more toward the severe or aggressive side of the exposure environment scale. Washington falls within a more moderate exposure environment. Arizona and Arkansas, with the slowest chloride diffusion rates, are more mild environments. What is most notable in this data is that the time to corrosion for the simple, benchmark situation varies over an order of magnitude across the different geographic regions modeled. This data illustrates that given the important role of corrosion in the time-dependent deterioration of bridges, uniform inspection intervals are unlikely to be the most efficient solution to the inspection problem. Bridges in aggressive environments are likely to deteriorate more rapidly, and thus require more frequent inspections than bridges located in benign environments. This is only one among a multitude of factors that contribute to the need for inspections; however, it is one of the most important and widespread. Data such as those provided through this simple modeling can be used, among other inputs, to provide supporting rationale for categorizing the OF with the RBI system. Element attributes that contribute to increased corrosion resistance, such as the use of epoxy-coated rebar or concrete mixes intended to resist the effects of corrosion are also needed for the analysis. This is particularly true in aggressive environments in which corrosion mitigation strategies might greatly increase the time to corrosion if they were used, supporting rationale for a lower OF, or conversely a higher factor if they were not used. Such modeling is relatively simple, widely available (the application used was available free-ware), and can include other relevant attributes to provide quantitative data to support the RAP assessment.

3.5.3.2 Corrosion in Steel

There is also a significant amount of available literature related to the corrosion of steel bridges and the use and performance of protective coatings for steel bridge corrosion control (*37, 44, 45, 55–57*). During periods of active corrosion, it is generally accepted that corrosion rates of steels under most natural exposure conditions follow a linear rate to a point where the corrosion rate slows and flattens to a steady state rate less than that of the initial few years of corrosion. During the initial stages of corrosion, the rust scale builds up at the steel surface at a fairly consistent rate. Once the scale covers the entire exposed surface in a uniform manner, the rate of corrosion is limited by the rate of oxygen diffusion through the intact rust layer. Although this pattern of a "deteriorating linear" corrosion rate is dominant for boldly exposed steel, the rate itself is highly dependent upon the specific exposure conditions. The corrosion rate tends to abate over time for many environments, but for the most aggressive environments (marine) this reduction in corrosion rate may not occur. Also, the corrosion rate at localized areas on the same structure, or even the same steel member, can vary. Therefore, it is prudent to view long-term corrosion rates as maintaining a near linear corrosion rate over time and to assume corrosion rates that are in the range documented for steel exposed to high moisture, high chloride environments. These corrosion rates tend to be in the range of 0.004 inches to 0.006 inches per year, per side of exposed steel, and these values can be used as a conservative planning rate to predict the impact of corrosion on a deteriorating member. Because of this relatively slow rate of corrosion section loss in steel, the accumulation of damage in the near future is predictable, particularly in a relatively short time frame such as the next 72 months. The condition of the structural steel and protective coatings relative to corrosion can be easily assessed during inspections. If the current condition is not well understood, for example, the amount of section loss present in the bridge is not known, an effective assessment may not be possible. However, under an RBI scheme, the inspection process to be used must ascertain the level of section loss present, enabling the effective assessment of the likelihood and severity of future damage occurring. This data provides an example of the collective knowledge available and easily accessible that can be used to provide a basis for RAP assessments.

3.6 Case Studies of the Methodology

Two case studies were conducted to evaluate the effectiveness of the RBI method. The objectives of the case studies were as follows:

- Demonstrate the implementation of the methodologies with state DOT personnel and
- Verify the effectiveness of RBI analysis in determining suitable inspection intervals for typical highway bridges.

To demonstrate the implementation of the methodologies for RBI, two state DOTs were selected to be trained for and execute an RAP analysis for a family of bridges in their states. This included training in RBI technologies and executing expert elicitation according the procedures described in the Guideline. These RAP meetings resulted in data models for determining the RBI requirements for a family of bridges. These results were then tested to verify that the RBI practice developed through the RAP process was effective in determining an acceptable inspection interval for the subject bridges. This was achieved through a back-casting process that utilized historical inspection records for specific bridges. These inspection records were used to assess if the inspection intervals identified through RBI would have been effective in maintaining the safety and serviceability of the bridge, were the RBI procedures applied in the past. This process compared the outcome of the RBI analysis with actual performance data for specific bridges, providing a validation of the RBI approach.

The first case study was conducted for a sample of prestressed bridges in Oregon and the second one for steel bridges in Texas. In each case a group of bridge experts were gathered to conduct the RBI analysis during a 1.5 day RAP meeting in the host state. The composition of the RAP panels consisted primarily of state department of transportation engineers involved in the inspection, maintenance, and management of bridges within the state. The goals of RAP meetings were to develop RBI practices for the subject family of bridges. The objectives of the meeting were to identify and rank damage modes for each bridge component (deck, superstructure, and substructure), discuss deterioration mechanisms that lead to those damage modes, and identify related attributes. These attributes were then ranked according to their impact on the likelihood of severe damage occurring within a specified time interval. CFs associated with these damage modes were also assessed.

This section of the report describes the outcome of the case studies conducted in Oregon and Texas. This includes an overview of the RAP meeting agenda, resources used in the RAP meetings, and the results of back-casting completed to verify the RBI approach.

3.6.1 Summary Overview of RAP Meeting

The RAP meeting consisted of a series of designed expert elicitations intended to develop comprehensive data models for RBI. Processes implemented during the case studies were as described in Section 3.4. During the RAP, credible damage modes pertaining to the family of bridges being analyzed were identified through consensus of the RAP. Relevant attributes that contribute to likelihood of those damage modes progressing or occurring were also developed through the designed elicitations. Following the identification of the damage modes and relevant attributes, these attributes were ranked according to their impact on the likelihood for that damage mode (high, medium, or low) as a means of establishing an initial scoring approach. CFs for each damage mode and bridge component are also developed through a designed elicitation and consensus of the panel. Data from the RAP meetings were subsequently analyzed by the research team, organized into scoring models for each damage mode based on the RAP results, and utilized in the back-casting procedure to verify the effectiveness of the RAP results.

3.6.2 RAP Meeting Attendees

The RAP meetings were attended by a variety of individuals from participating states, as shown in Table 9. The RAP meeting in Oregon was attended by nine individuals, including DOT engineers, academics, and industrial representatives. The industrial representative participating on the Oregon RAP was from a fabricator that provided precast members for bridge projects in the state. The RAP also included a university professor with active research in the area of bridge evaluation and condition assessment. There were three individuals with Ph.D.'s.

In contrast, the RAP in Texas was comprised of only five individuals, and all of the participants were employed by the Texas DOT. The participants generally held Bachelor of Science (B.S.) degrees, with one member holding a Master of Engineering (M.E.) degree.

Most of the participants in the RAP meeting held at least B.S. degrees in civil engineering. A little more than 70% of the participants were registered Professional Engineers (P.E.).

3.6.3 Schedule and Agenda

The RAP meeting in each state consisted of a 1.5 day face to face meeting in the host state. A webinar was presented approximately 1 week prior to the RAP meeting, to familiarize participants with the overall process, field any questions participants may have, and identify any resources that may be needed for the meeting. This teleconference consisted of presenting overview slides introducing the concepts and

Table 9. Listing of RAP meeting attendees in Oregon and Texas.

Name	Emp.	Current Position	Highest Degree	P.E.
Oregon				
Participant A	Oregon DOT	Bridge Program Unit Manager	B.S.M.E.	Y
Participant B	Oregon DOT	Structural Service Engineer	B.S.C.E.	Y
Participant C	Oregon DOT	Senior Engineer	Ph.D.	Y
Participant D	Oregon DOT	Bridge Operation and Standards Managing Engineer	B.S.C.E.	Y
Participant E	Oregon St U.	Professor	Ph.D. Str. Eng.	N
Participant F	Knife River Corp	Chief Engineer	Ph.D.	Y
Participant G	Oregon DOT	Bridge Maintenance	-	N
Participant H	Oregon DOT	Bridge Planner & Financial Analyst	M.S. of Economics	N
Participant I	Oregon DOT	Senior Bridge Inspector	B.S.C.E., AE Struct. Eng.	Y
Texas				
Participant A	TX DOT	Director of Field Operations-Bridge Division	B.S.C.E.	Y
Participant B	TX DOT	State Bridge Constr/Maint Engr	B.S.C.E.	Y
Participant C	TX DOT	Senior Bridge Const. and Maint. Engr	M.E.C.E.	Y
Participant D	TX DOT	State Inspection Engineer	B.S.C.E.	
Participant E	TX DOT	Bridge management Engineer	B.S.C.E.	Y

approach of the research and the planned activities during the RAP meeting. Most of the individuals that participated in the RAP meeting also attended this webinar to be introduced to the technology and prepare themselves for participation.

3.6.3.1 RAP Meeting Agenda

The meeting agenda was developed to establish an effective training pattern for the reliability assessment to be conducted. The previous expert panel meeting held during the initial phase of the project acted as the model for the RAP meeting agenda to be carried out in each state. However, in developing the RAP agenda, it was decided that the training goals would be best met by reorganizing the session into distinct training and execution phases. In other words, training associated with each of the aspects of the analysis, such as CFs, OFs, etc. were provided for the entire process *before* tasks to identify the parameters specifically for the family of bridges to be examined in the case study. This is in contrast to the expert panel meeting held during the initial phases of the research, during which the elicitations for each factor were conducted following training for that particular factor. The primary motivation for this decision was to ensure that the participants had a full and complete picture of how data would fit together in the final analysis, before making any decisions on what the parameters or attributes should be for the particular family of bridges being analyzed.

The meeting began with an overview of the research approach, describing the goals and objectives of the RAP of the workshop and the overall research approach. This overview session was followed by a training session on how to identify damage modes and attributes for bridge elements, for the purpose of estimating the OF required for the analysis. This session includes three exercises to illustrate the process to be undertaken in the expert elicitation for identifying damage modes and key attributes, and ranking the importance of those attributes in terms of the reliability of the element under consideration. In these exercises, a typical two-span steel bridge was presented as the example to pose questions regarding the typical damage modes that would be anticipated for this element. The members of the RAP recorded their responses on the bubble sheets and subsequently discussed the identified damage modes as a group. During these discussions, credible damage modes were identified for further analysis.

This exercise was followed by an elicitation of attributes related to the reliability/durability associated with the primary damage modes identified by the group and prioritization of those attributes from high to low. This exercise illustrated the process of the developing attributes and a semi-quantitative scoring scheme for a particular family of bridges as a means of identifying the OF for the RBI analysis. The process illustrated in this example is later repeated by the RAP for the superstructure, substructure, and deck components for the subject family of bridges (i.e., prestressed superstructures in Oregon and steel superstructures in Texas).

Training was also provided on the CF categories that are part of the analysis. A group exercise expert elicitation for consequences was administered to illustrate the process of identifying a consequence ranking for a particular damage mode scenario. During this exercise, panel members considered the likely consequences of an identified damage mode progressing to the defined failure state (e.g., serious condition) in terms of safety and serviceability of the bridge.

Following these exercises, the expert elicitation for the family of bridges under consideration was conducted. Separate sections of the meeting address the superstructure, substructure, and deck components of the bridge. The same process implemented in the illustrative examples was conducted for each component to identify the likely damage modes, attributes contributing to the reliability considering those damage modes, and prioritization of the attributes. These data were used to identify criteria and develop the initial scoring scheme to be implemented for assessing the OF for the various damage modes identified through the process.

Consequence scenarios for each damage mode were also developed through group discussions. During this task, each damage mode identified in the earlier exercises was considered, and an expert elicitation was conducted to identify the appropriate CF for each damage mode, and key factors that affect the factor selected. For example, if the damage mode is spalling damage on a deck, the CF may be high or even severe if ADT and traffic speeds are high, but moderate if the ADT and traffic speeds are low. Group discussion was used to develop consensus on these factors. Policies and common practices in the particular state also contributed to these discussions.

The balance of the agenda was used to refine and complete the criteria and rankings for attributes, OFs, and CFs for the subject family of bridges. Screening criteria, surrogate data, and available data on attributes from existing inspection practices were identified. For example, if the subject state collects element-level data, how do various element ratings and damage flags correspond to the attributes and damage modes identified through the RAP process.

At the completion of the meeting, it was anticipated that the damage modes, ranking for attributes, and basic scoring approach would be completed, as well as the CFs for various scenarios. However, discussions of the CFs revealed that certain descriptions of the various CF levels were problematic, and these descriptions were subsequently modified to address these concerns. As a result, the RAP meetings provided preliminary data on the CFs to be used for the analysis, and these were later refined during the analysis process.

The data from the RAP meeting were compiled and analyzed by the research team following the meeting. These data were utilized to developed scoring models, or data models, reflecting the input from the RAP. These data models were then used in the back-casting process to evaluate the historical performance of a sample population of bridges in each state to verify the effectiveness of the data models developed through the RAP process.

3.6.3.2 RAP Participants Notebook

A participant's notebook was prepared for distribution to members of the RAP. This notebook provided a reference for use during the meetings. This notebook included standard information regarding the meeting, such as the agenda and copies of the slides to be presented during the training portions of the meeting, including space for participant's personal notes. In addition, copies of the forms to be completed during the meeting are included for future reference following the meeting. The notebook also included color copies of the risk matrices to be used in determining the inspection interval based on the RBI analysis conducted by the RAP.

The notebook also included key appendices from the Guideline. These appendices include the guidance for identifying damage modes and attributes (i.e., OFs), CFs, determining the inspection interval, and the complete index and commentary of attributes identified in the Guideline. These portions of the handbook were included to act as references for the RAP participants to use during the RAP meeting for conducting the RBI analysis.

3.6.3.3 Software Development

A software application was developed to support the RBI analysis of bridges based on the results of the RAP meetings. This application was developed within a spreadsheet program, and provides a simple and rapid means of implementing the damage modes, attributes, and scoring methodology for estimating the OF.

	Design Attributes	
☑	D.1	Joint Type
☐	D.2	Load Posting
☐	D.3	Minimum Vertical Clearance
☑	D.4	Poor Deck Drainage and Ponding
☐	D.5	Use of Open Decking
☑	D.6	Year of Construction
☑	D.7	Application of Protective Systems
☑	D.8	Concrete Mix Design
☐	D.9	Deck Form Type
☐	D.10	Deck Overlays
☑	D.11	Minimum Concrete Cover
☑	D.12	Reinforcement Type
☐	D.13	Built-Up Member
☐	D.14	Constructed of High Performance Steel
☐	D.15	Constructed of Weathering Steel
☐	D.16	Element Connection Type
☐	D.17	Worst Fatigue Detail Category
☐	D.18	Reserved Item

Figure 10. Example screen from software application showing selection of attributes.

In this application, the user selects the attributes identified by the RAP for a particular damage mode, as shown in Figure 10. A check box is used to select screening, design, loading, and condition attributes as described in the Guideline. Reserved attributes are included so a user can easily add additional attributes that may not be included in the Guideline.

Once the attributes are selected from the appropriate listing, the application organizes the selected attributes into a scoring page as shown in Figure 11. On this screen, pull-down menus are used to score the individual attributes for a particular bridge according to the scoring scheme developed. These pull-down menus allow a user to quickly select the appropriate ranking for a particular attribute based on the criteria developed through the RAP.

The individual scoring for any attribute can be easily modified on an editing page to meet the requirements of a particular user. A hot-link is provided to the attributes commentary included in the Guideline, such that a user can easily refer to the rationale for a particular attribute and the envisioned scoring mechanism. After each attribute is scored, the OF score and guidance is automatically calculated for that damage mode.

This software application was developed for use in the case studies to implement the analysis of the RAP from each state, and for testing that analysis against the historical performance of bridges during the back-casting. Looking forward, this software application provides a model for future, more sophisticated computer applications to allow for efficient and simple application of the RBI technology. For example, such a software module could be an add-on to the PONTIS program or other BMS, where many aspects of the scoring could be automatically obtained based on element ratings already collected as part of a routine inspection.

3.6.4 Back-Casting Procedure

The case studies conducted in Texas and Oregon developed a set of criteria and attributes for determining the OF and the CF, resulting in inspection intervals based on the risk matrix. These criteria and attributes produced a risk-based

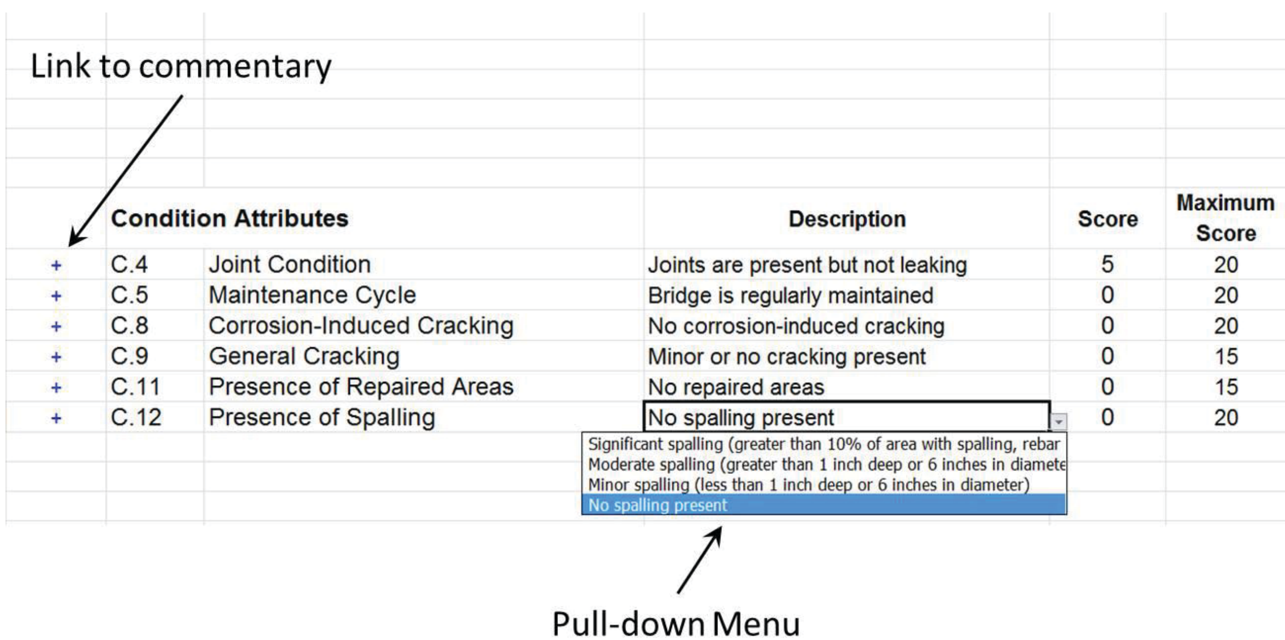


Figure 11. Example screen from software application showing pull-down menus for scoring attributes.

data model to be used to determine the appropriate maximum inspection interval for a specific bridge or family of bridges. To verify if the use of these models provided a suitable inspection interval that did not compromise the safety and serviceability of bridges, a back-casting procedure was used. In the back-casting procedure, the data models developed by the RAP were applied to individual bridges based on historical inspection records. For example, the data model may be applied to a bridge based on the year 2000 inspection records for the bridge, resulting in an RBI interval that would have been determined in the year 2000, were RBI practices applied at that time. These results were then compared with the actual performance of the bridge, based on the inspection records for the years 2002, 2004, 2006, etc. to determine if the RBI inspection interval would have adequately addressed the inspection needs for the bridge. The criteria for determining the effectiveness of the data model included:

1. Did the condition rating for any component change significantly during the RBI interval in a manner that was not captured or anticipated effectively, but would have been captured (or detected sooner) by a standard, 24-month interval?
2. Were there any significant maintenance or repair actions completed that would have been delayed as a result of implementing an RBI interval (relative to a standard, 24-month interval)?
3. Were there any significant factors or criteria not identified through the RAP analysis that were needed in the data models to provide suitable results?

The procedure for back-casting consisted of obtaining the element-level inspection reports ranging back to approximately 1998, depending on the availability of data for the specific bridge. The data model was applied at each inspection year to assess the appropriate inspection interval based on the inspection data. As a result, the RBI interval may be consistent over the time period examined, decrease over that time period, or even increase during the time period as a result of a repair or improved condition rating or condition state.

The overall concept of back-casting is shown schematically in Figure 12. This figure shows NBI ratings for an example bridge component over time. The RBI data model is applied to the bridge component based on inspection results from 1998. Assuming this results in an inspection interval of 72 months, the inspection results from each biannual routine inspection (24 months) is examined to see if there were any significant changes to the condition, or other events or circumstances detected by the routine inspection that may have been missed or delayed due to the RBI interval of 72 months. The RBI interval is calculated for each year there is an inspection result, indicated by the numerical results shown on the diagram. A change in the RBI inspection interval to 48 months is also shown in the figure. Assessment of the results includes determining if the change of inspection interval identified through the RBI criteria was effective in capturing the appropriate inspection interval, considering changes in the condition of the component reflected in the inspection results. It should be noted that the RBI inspection interval does not necessarily reflect NBI condition rating changes; however, since both depend on the condition of the component, they may be similar.

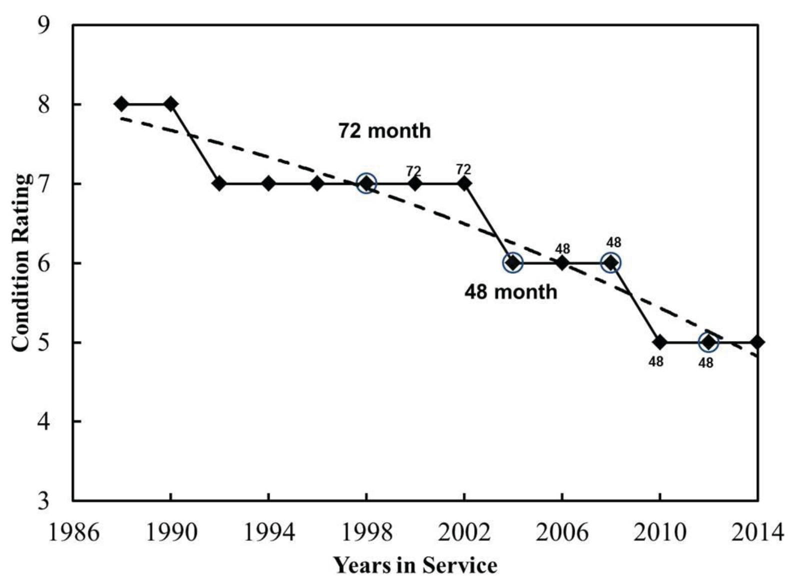


Figure 12. Graph of condition ratings for a bridge component over time, showing schematic example of the back-casting procedure.

3.6.4.1 Inspection Data for Back-Casting

Inspection data from each state were reviewed in detail to implement the data models developed through the RAP process, i.e., evaluation of the attributes identified by the RAP. This included design and loading attributes, which typically do not change over the life of the bridge and condition attributes that change as the bridge ages or undergoes repair or rehabilitation.

Inspection data from Oregon consisted of PONTIS data file outputs, including photographs, notes, and standard Structural Inventory and Appraisal (SI&A) sheets. Available inspection data from 1997 to present were assessed. In Texas, inspection data consisted of inspection reports that included standard SI&A sheets, NBI component rating sheets, and element-level data collected at the time of the inspection. Available data from 1999 through present were assessed for the Texas case study.

3.6.4.2 Review of Work

As part of the back-casting analysis, a database of work projects maintained by the Oregon DOT was queried to determine if any significant repairs had been completed on the subject bridges during the interval of the back-casting process, and results were provided to the research team. This was done to ensure no significant events occurred on the bridge that resulted in major work or repair between inspection intervals, which may have been missed due to an extended inspection interval or may not be reflected in the inspection reports.

In Texas, inspection records were more diversified. Work during the intervals between inspections was determined from the element-level data collected as part of the bridge inspection process. This was achieved by reviewing each inspection report for notes that would indicate that an improvement or repair was made to the bridge, or that an improvement or repair was urgently needed. Unexplained changes in the condition rating for a component were also investigated to determine if an urgent repair or rehabilitation activity was the source of the improvement.

3.6.4.3 Sampling

To complete the back-casting verification study of the result of the RAP assessment, a sample population of bridges was assessed over a time period dating back 15 to 17 years. To determine the number of bridges to be assessed to develop a statistically significant result, a statistical analysis of population sampling was completed. Generally, such statistical models require some a priori knowledge of the anticipated variance in the population to be sampled to estimate the number of samples required to represent the overall population, considering the parameter to be measured. It was anticipated that the RBI criteria developed by the RAPs would include the current condition rating for a bridge as one of the criteria (attributes). Therefore, it was desired to select a sampling of bridges that has the same variation as the population overall, namely, that the natural variation of the inspection results of the overall population is represented in the sampling selected, based on the condition ratings provided in the inspection files. Experimental data from the FHWA visual inspection study (*58*) was used as a basis for the estimate, assuming that the variance of condition rating for all components in the FHWA study. Based on population sampling statistics, assuming that the desired accuracy was ±0.5 condition ratings with 95.5% confidence resulted in a desired sample size of 17 bridges. For a confidence interval of 95%, the sample size for back-casting would be 10 bridges. Based on these results, the sampling of bridges included a minimum of at least 10 bridges; in the study, 17 bridges were selected from Texas and 22 bridges were selected in Oregon.

3.6.5 Statistical Analysis of NBI Data

Statistical analysis of NBI data for the participating states was conducted to identify the characteristics of the each state's inventory and to support the RBI analysis. Analysis of NBI data was completed with the following objectives:

1. To determine the typical characteristics of the bridge inventories in the participating states of Texas and Oregon.
2. To develop quantitative data based on NBI condition rating history to be used to support the RAP analysis and rationale for RAP-developed criteria.

The objective of providing quantitative statistical data to support anticipated criteria that may be developed by the RAP during the course of the case studies can be illustrated as follows. Consider that the RAP identifies an attribute/criteria (among others) that a bridge has a superstructure rating of 7, based on the rationale that such a condition rating would indicate little deterioration or damage presently, and a low likelihood (i.e., OF) that severe damage would occur over the ensuing 72-month period. Analysis of the time-in-condition data from the NBI records provides quantitative data to support this rationale, as discussed in Section 3.5.

To conduct these analyses, data from the NBI dating back to 1992 were obtained from the Federal Highway Administration (http://www.fhwa.dot.gov/bridge/nbi.cfm); these data are publicly available via the web site indicated. These data were used to develop data on the past performance of bridges in each of the participating states and to characterize the overall inventory in each state.

Table 10. Bridge population statistics for Texas and Oregon.

Description	No.	Length (m)	% of No.	% of Length	Average Age (year)
Bridge Inventory in Oregon					
Concrete	2,050	87,000	28	24	55.2
Steel	1,089	109,160	15	30	48.5
Prestressed concrete	3,612	154,877	49	43	28.9
Other	602	12,408	8	3	49.3
Total	7,353	363,444	100	100	40.8
Bridge Inventory in Texas					
Concrete	29,098	704,514	56	23.40	48.0
Steel	7,423	776,717	14	25.90	38.1
Prestressed concrete	13,781	1,392,706	27	46.30	23.6
Other	1,576	131,465	3	4.40	33.0
Total	51,878	3,005,403	100	100	39.6

3.6.6 Bridge Inventories in Texas and Oregon

The families of bridges selected for the two participating states were based on the bridge inventories in each state. It was as desirable to have a sufficient inventory as to have a large inventory from which to draw sample bridges, and a representative population of bridges in terms of age. Table 10 shows bridge population statistics for the participating states based on data available in the NBI. The bridge families selected are highlighted for both Oregon and Texas. Prestressed bridges were selected for analysis in Oregon because this bridge superstructure type made up almost 50% of the bridge inventory in that state, making it a significant population of bridges. This population of bridges has an average age of almost 29 years, consistent with the era of prestressed bridge construction, and there are more than 3,600 bridges of this material type. In Texas, the overall number of bridges is large, such that any family of bridges of similar superstructure materials would provide a suitable population of bridges for analysis. In this case, steel bridges were selected for analysis for three reasons; first, they provided suitable number of bridges for analysis, second, it was desirable to do one analysis for concrete and the other for steel bridges, and, finally, the average age of the population was much older than the prestressed bridge population in Oregon, providing diversity in the ages of populations in these states.

Figure 13 illustrates the age distribution for bridges in each state, as well as the age distribution for the bridge sample selected for analysis. Vertical lines on the figure indicate the mean ages for each population. As these distributions illustrate, the mean or average age of bridge selected for analysis were older than the overall populations. This was considered desirable, because relatively new bridges are generally less challenging for RBI analysis, because they are usually in good condition and have good durability attributes. Therefore, selecting a population that was slightly older than the overall population presented a greater challenge for testing the RBI processes.

Bridges included in the sample were generally randomly selected, with the exception that the desired sample of bridges for analysis had a geographic distribution across the subject state, and emphasis was placed on including bridges with sufficient historical data to make the back-casting meaningful. In Oregon, several bridges had limited historical data because the bridge was constructed after the year 2000; however, the sample of bridges was larger such that there were at least 17 bridges with the desired historical data available.

3.6.6.1 Bridge Sample Locations

Bridges selected for back-casting were distributed geographically within the states. Figure 14 shows the distribution of bridges in each state. As shown in the figure, bridges were selected from different regions of each state, although the geographic distributions of the sample bridges are affected by the population characteristics of each state. For example, in Oregon, population density is significantly higher in the western part of the state, and as such, the majority of bridges are in the western part of the state; the sample of bridge reflects this effect.

3.6.7 Time-in-Condition Rating

The NBI data for Texas and Oregon were analyzed to determine the typical lengths of time that a bridge component was in a particular condition rating. These data were derived from the NBI database, with some data trimming to accommodate the fact that the data sets are incomplete. That is, there are no data prior to 1992 or after 2011, so some trimming of these data are needed to improve the certainty of the derived time intervals. Data were trimmed from the data set if there were

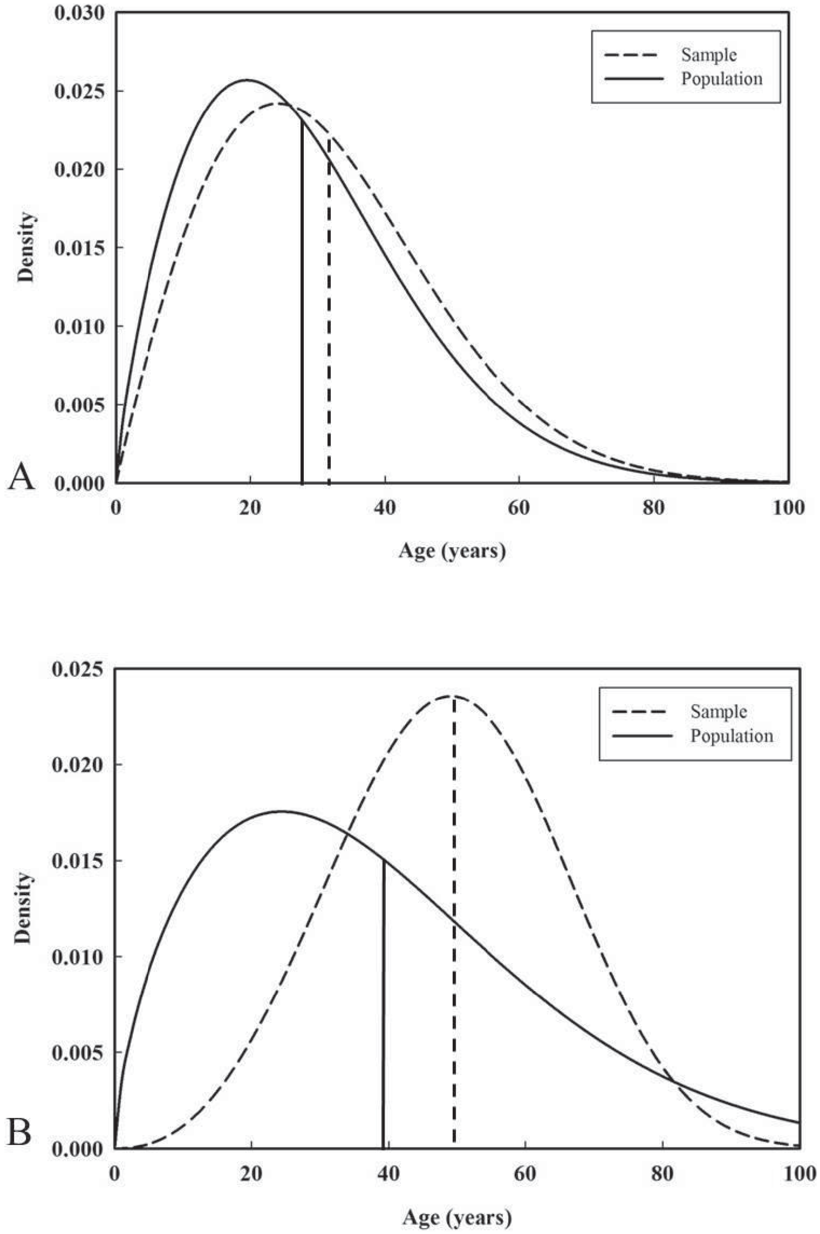


Figure 13. Age distributions of sample bridges and overall populations for (A) prestressed bridges in Oregon and (B) steel bridges in Texas.

5 years or less of consecutive data in a condition rating at the beginning or the end of the available time interval. The trimming value of 5 years was selected based on study of different possible trimming values, ranging from 3 to 7 years, performed by the research team. This study indicated that the specific trimming value only had modest effects on the outcome of the analysis, and as such, 5 was selected as an acceptable value that ensured sufficient data were available for a meaningful statistical analysis. This method of trimming the data provides a suitably conservative result, because the analysis indicates that time-in-condition ratings are typically much larger than 5 years for components in reasonably good condition (rated 6, 7, or 8). Data presented within this report include the superstructure and deck condition ratings; data for substructures were also analyzed. However, the deck and superstructure condition ratings typically change more frequently than substructure ratings, and as such, the deck and superstructure are the focus of the data reported herein.

Figure 15 shows the time-in-condition results for prestressed bridges and decks of prestressed bridges in the state of Oregon. As shown in the figure, bridge superstructures rated in good condition tend to have longer intervals in that rating; as

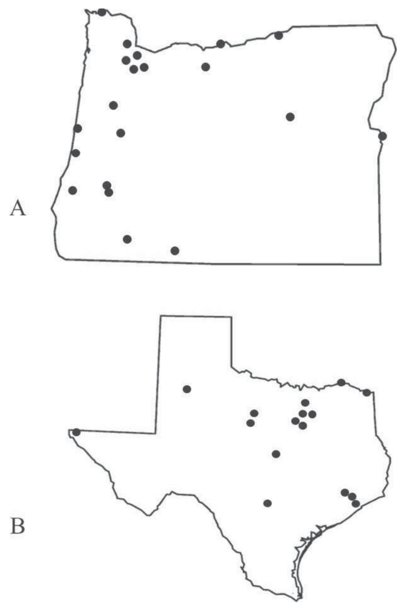


Figure 14. State maps showing geographic distribution of sample bridges.

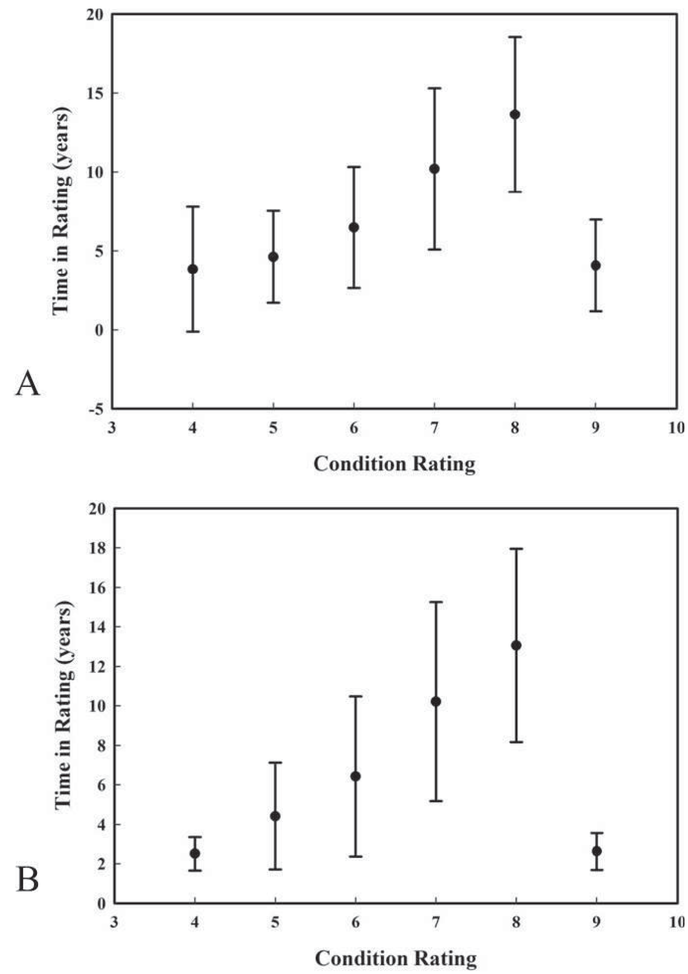


Figure 15. Time-in-condition rating for (A) prestressed bridge superstructures and (B) decks based on NBI data for Oregon.

the rating decreases, the time in a particular rating is reduced. For example, for the prestressed bridge superstructures illustrated in the figure, the average time period a superstructure was rated 8 was almost 14 years (σ = 4.9 years), but the time period a superstructure is rated a 5 is less than 5 years (σ = 2.7 years). The average time period for a prestressed superstructure is rated a 6 is 6.5 years (σ = 3.8 years). For bridges in condition ratings of 4 or 3, these data are not particularly useful for two reasons; first, there are very few bridges in this category, and second, the bridges get repaired, and as such, the time interval in the condition is really more representative of a measure of how quickly these bridges may be improved or repaired rather than how long they remain at this condition rating.

Figure 15B shows the time-in-condition rating for decks of bridges with prestressed superstructures. Similar observations can be made, as shown. For example, a deck remains in condition rating of 7 for 10.2 years (σ = 5.03 years); the time period a 6 remains a 6 is 6.4 years (σ = 4.8 years), on average.

Figure 16 shows the results of the trimming analysis for steel bridges in Texas. In this case, steel superstructures and bridge decks on steel superstructures were analyzed. For steel superstructures in Texas, the average time-in-condition rating of 7 was 10 years (σ = 5.4 years), for decks of steel bridges, the average time-in-condition rating was found to be 11 years (σ = 5.6 years).

These data are useful as they reinforce and support the supposition that a bridge in good condition tends to stay in good condition for a long time interval (i.e., longer than the maximum inspection interval recommended using the proposed methodologies). For example, if one used the surrogate data of condition rating of 7 for superstructure, substructure, and deck to identify bridges with an appropriate inspection interval of 72 months, these data provide quantitative evidence to support that rationale, as discussed in Section 3.5. These data were used in the case studies to support "surrogate data" analysis based on the data models developed by each RAP. In this analysis, the condition rating of 7 was used as "surrogate data" for the condition attributes to assume the OF would be low for condition-related damage modes. For these cases, the inspection interval of 72 months may be applied, based on these data.

3.6.7.1 Inspection Intervals

Inspection intervals were determined based on the reliability matrix introduced in the Guideline. Figure 17 shows the proposed reliability matrix that is used for typical highway bridges. This matrix illustrates the appropriate

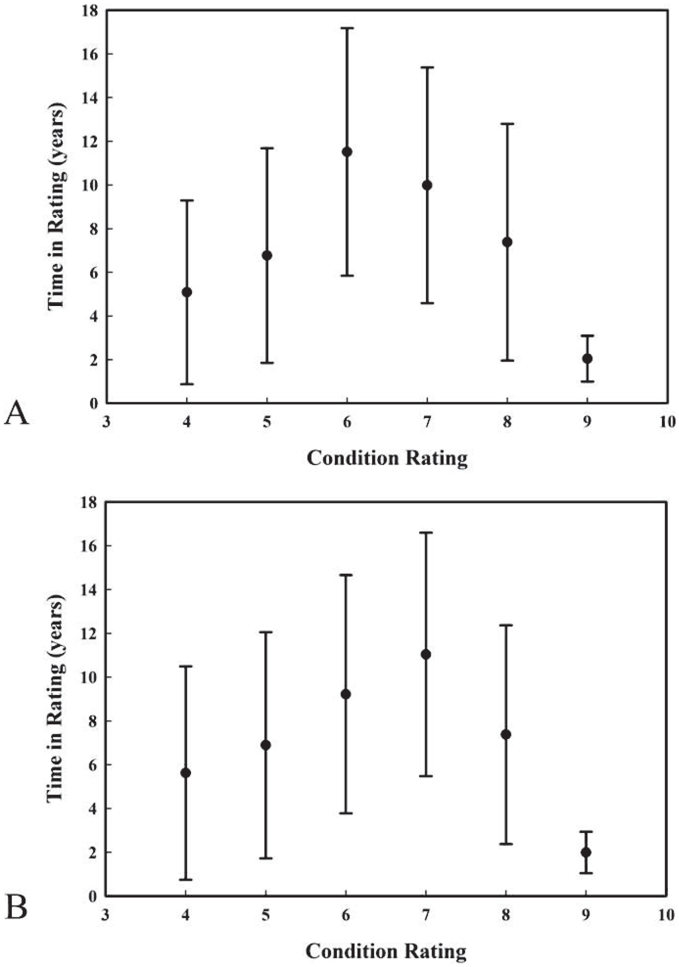


Figure 16. Time-in-condition rating for steel bridge (A) superstructures and (B) decks based on NBI data for Texas.

inspection intervals based on the estimates of the OF and the CF from the RAP analysis. In the figure, the inspection intervals are I =12 months, II = 24 months, III = 48 months, IV = 72 months, and V = 96 months. For example, when an OF is "Low" and CF is "High," the proposed inspection interval is 48 months. This matrix was applied to the results of the OF analysis, based on attribute scoring, and the appropriate

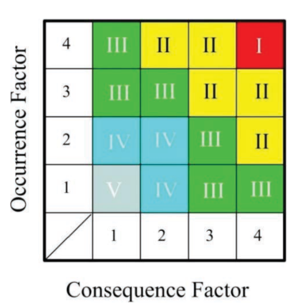


Figure 17. Reliability matrix for RBI.

CF for the given bridge component and damage mode. Each damage mode for each bridge component was analyzed using the RBI procedure, resulting in a data pair (OF, CF) for each damage mode for each component. These data were located on the risk matrix to determine the inspection interval for each bridge, as illustrated in the results section for each state.

3.6.8 Overview of Case Study Results

The objective of this section of the report is to provide an overview of the results of the RAP meetings in each state. This section includes a summary of the damage mode and attributes identified in each state, and the consequence analysis that was conducted during the RAP meetings.

3.6.8.1 Summary of Damage Modes and Attributes

This section summarizes the damage modes and attributes identified through the RAP process. These data provide the data model for assessing the OF as part of the RBI process, and as such, are documented here to illustrate how the data model was developed and what was considered. Due to the detailed nature of many of the attributes and description, most of these data have been placed in Appendix A for the Oregon case study and Appendix B for the Texas case study. These appendices document the attributes and attribute scoring for each damage mode that was used during the back-casting analysis.

3.6.8.2 Damage Modes and Attributes

The expert elicitation process described in the Guideline and implemented during the case studies generally worked effectively to ascertain credible damage modes and identify key attributes affecting those damage modes. The process consists of having participants complete forms identifying credible damage modes, and then using a consensus process to list the damage modes, pare down those that are repetitive or irrelevant, etc. During the consensus process, damage modes identified by participants were recorded on a white board, along with the data from the likelihood estimates made by the participants. An example of this process is shown in Figure 18. This figure illustrates the beginning of the expert elicitation process, when the data from each member of the RAP is collected for discussion. The orange numbers shown in the figure indicate the number of panel members recording a particular likelihood (10%, 20%, 30%, etc.) for a given damage mode. As shown in the figure, this initial process included a number of damage modes for decks, including rebar corrosion, delamination, and spalling, which were pared down through discussion to a corrosion-related damage mode of spalling.

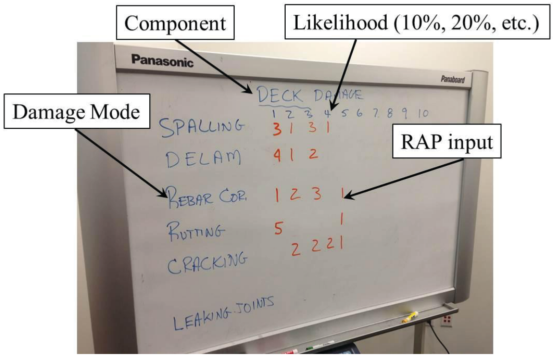


Figure 18. Example of RAP data for damage modes in decks.

Rutting was also identified as a credible damage mode for decks by the RAP in Oregon. This damage mode illustrates one benefit of a RAP consisting of bridge owners. Rutting of decks is related to the use or over-use of studded tires, and occurs along particular corridors in Oregon. It is unlikely many other states would identify this damage mode, but regardless, in Oregon such damage occurs and affects the serviceability of some bridges. It was the consensus of the panel that this damage mode was credible and required consideration in an RBI process.

Texas identified punch-through as a credible damage mode. In this case, punch-through is not a corrosion-related damage mode, but rather related to the construction of thin decks, sometimes with poor quality concrete, that results in punch-through as a result of repetitive loading and age. Because much of the state is relatively arid, and use of de-icing chemicals is minimal, decks may have longer lives than they might in an area where corrosion is a significant issue. If the deck is thin and concrete quality is poor, punch-through can occur. Like rutting, this damage mode is due to *local* (state) policies and construction practices, namely that very thin decks were used during certain historical time intervals, and concrete quality was not well controlled at the time. In a state where corrosion damage was more prevalent, such a deck would deteriorate severally due to corrosion before such punch-through could occur. Like rutting, this damage mode is not likely common in other states. These relatively unique damage modes illustrate the utility of the RAP approach.

A summary of identified damage modes is shown in Table 11 for Oregon and Texas. It can be seen that damage modes of concrete deck and substructure are similar for Oregon and Texas. For superstructures, only the impact damage mode was common between prestressed and steel bridges analyzed in the two states, as would be expected, since the superstructures are of different material types.

During the Oregon RAP, the panel expanded its assessment from open prestressed shapes, such as typical AASHTO shapes and Bulb-Tees, to include adjacent box girders bridges and prestressed slabs. The consensus of the panel was that the damage modes and attributes were

Table 11. Summary of damage modes in Oregon and Texas.

Bridge Element	Oregon (Prestressed or Post-Tensioned)	Texas (Steel)
Deck	• Spalling • Rutting • Cracking (Non-corrosion Induced)	• Spalling • Punch-Through • Cracking • Delamination
Superstructure	• Cracking (Shear) • Strand Corrosion • Fire Damage • Impact • Rebar Corrosion within the Span • Bearing Seat Problems **Adjacent Box Girders** • Rebar Corrosion/Section Loss • Strand Corrosion (Fracture) • Flexural Cracking • Shear Key Failure • Impact / Fire	• Fatigue Cracking • Section Loss • Fire Damage • Impact • Deflection Overload • Bearing Failure
Substructure	• Settlement • Corrosion Damages • Fire • Overload Damages • ASR	• Settlement • Corrosion Damage • Overload Damage • ASR

essentially identical for these families of bridges, with the exception that adjacent box girder bridges had a shear key damage mode that would need to be assessed as a screening tool. These data are reflected in the summary of damage modes shown in Table 11.

For each damage mode identified by the RAP, attributes that contributed to the likelihood of that damage mode occurring and progressing were identified through the RAP survey process and consensus of the panel. An example result of the consensus process is shown in Table 12 for deck spalling, summarized from the Oregon RAP. In this table, the attributes identified by the panel are shown in the left column, followed by the rank that each attribute was assigned by the panel. This rank shows the unanimous vote on the rank for each attribute; this represents consensus developed among the panel, not necessarily initial results of the elicitation process. In some cases, individual members may have ranked these attributes differently, but consensus was developed through discussion. Once consensus was developed, limits or parameters for scoring each attribute was developed through open discussion among the RAP member and results are shown in the table. For Oregon, which utilizes element-level inspection processes, many of the attribute parameters could be described using existing models from their element-level inspection manual.

For example, for deck cracking, the element manual already has quantitative description of condition states 1, 2, 3, and 4, and therefore additional description was not necessary. For other attributes, for example ADTT, limits for high, medium, and low were developed through discussion. A comprehensive listing of the damage modes, attributes, and limits/parameters used in the back-casting analysis are included in Appendix A. The potential source of the data, based on state-specific inspection processes, is also tabulated in Appendix A. It should be noted that in some cases, the RAP identified attributes that were later correlated with existing element data following a more detailed review of the element-level manual. In other words, the RAP identified a given attribute and appropriate scoring limits during the meeting, and these were later found to match existing element descriptions in the Oregon element manual. A similar process was followed for Texas, which collects more limited element data during inspections.

3.6.9 CFs

Designed expert elicitations were also used to develop CFs for each of the damage modes during the RAP meetings. For most damage modes, singular failure scenarios were assessed for each bridge component. The failure scenarios considered consisted of the component condition rating being serious (CR = 3), not necessarily structural failure. For decks, for example, the scenario considered in that the deck deterioration would typically be considered "serious" (CR = 3) during a normal inspection. For superstructure components (i.e., prestressed girders or steel girders), loss of load-carrying capacity for one member was considered. For Oregon, the CF for deck damage and substructure damage was considered to be generally Moderate. For superstructure components, the initial CF developed in the RAP was High for most damage modes (except bearing area damage); this factor was subsequently discretized during the analysis process.

For Texas, issues were identified during the RAP meeting with the CF descriptions, as previously described, and these CF descriptions were subsequently adjusted during the back-casting to address these issues. These revisions adjusted the descriptions of different CF levels, but not the levels themselves.

The CFs were subsequently assessed during the back-casting according to a series of scenarios to test and evaluate the influence of different parameters on the analysis. These focused largely on the CF assigned to the bridge superstructure. The scenarios included considering the CF as uniformly

Table 12. Example attributes rankings for deck spalling from the Oregon RAP.

Attributes	Rank			Limits		
	H	M	L	H	M	L
Cracking		8		Existing model		
Delamination	8			>25%	11%-24%	<10%
ADTT		8		>5000	501-4999	<500
Location / Environment	8			Coastal and Mountain	Valley (general environment)	Desert
Age			8	>50	10-49	<10
Dynamic Loading	8			Existing Model		
Rebar Corrosion	8			Rust/Black/Low Cover		No stains, Epoxy/high cover
De-icing		8		High		Low

High and considering the CF as uniformly Moderate, or determining the CF based on structural redundancy and feature under the bridge. For the latter, the CF was based on the following criteria:

The CF was Moderate for the superstructure if:

- Superstructure consisted of more than four members AND
- Beam spacing of 10 ft or less AND
- Bridge not over a roadway.

The CF was considered High if:

- Superstructure consisted of four members or fewer OR
- Beam spacing was greater than 10 ft OR
- Bridge was over a roadway.

These criteria were based in part of the result of previous NCHRP research on redundancy of bridges and on discussions with engineers from the RAP panel (*59*). These discussions included previous experience with impact damage on structures that resulted in loss of load-carrying capacity for a prestressed bridge member.

The feature under the bridge, i.e., if the bridge were over a roadway, was included as a factor to consider based on the perceived risk of affecting the feature under the bridge. For example, if a primary bridge member lost load-carrying capacity or deteriorated to a serious condition, consequences may be increased either as a result of falling debris or significant displacement, or emergency shoring that may be required that would affect the serviceability of the roadway below the bridge.

Additional factors considered for determining the CF included considering the traffic volumes; in these analyses, bridge decks with ADT greater than 10,000 were considered to have High CFs. This is intended to reflect a case where deck damage resulted in a major serviceability consequence.

3.7 Back-Casting Results for Oregon

This section summarizes the results of the back-casting analysis conducted as part of the study. The State of Oregon provided 22 bridges from around the state for the analysis, as shown in Figure 14A. As shown in this figure, bridges were obtained from across the state to represent different environmental conditions surrounding the sample bridges. The damage modes, attributes, and data scoring models used in the back-casting process are documented in Appendix A.

3.7.1 Environments

The environmental conditions considered in the analysis of bridges in Oregon differed according to the damage mode being considered. For example, for corrosion of superstructure metals (rebar or strands), the RAP identified three separate areas with coastal and mountainous regions being the most aggressive environment, while desert portions of the state represented the least aggressive environment, obviously. However, for spalling of bridge decks, the panel identified areas of the state where de-icing chemical use was highest because these areas are urban areas with high traffic volumes. For the damage mode of rutting, travel corridors that experience high traffic volumes likely to be using studded tires were identified. Generally, these corridors were identified because they connected major urban areas and resort locations. The environments identified by the Oregon RAP are summarized in Table 13.

3.7.2 CFs

There were six different CF cases considered in analyzing results in Oregon, as shown in Table 14. These different cases were selected to illustrate how different criteria established by an RAP might affect the outcome of the analysis. These included considering all superstructure damage modes as

Table 13. Environments identified by the Oregon RAP for different damage modes.

Damage Mode	Environment	Reason
Corrosion	Coastal and Mountainous	Aggressive environment, high humidity and/or use of de-icing chemicals
	Valley or General Environment	
	Desert	
Spalling	Portland	High application of de-icing chemicals
	Salem	
	Bend	
	La Grande	
Rutting	I-5	Presence of traveling traffic with studded tires
	I-84	

Table 14. CF cases used for back-casting in Oregon.

Case No.	Description
1	High consequence for superstructure damage modes
2	Moderate consequence for superstructure damage modes
3	Superstructure damage mode CF is determined by redundancy and facility under bridge (screening not used)
4	**Superstructure damage mode CF is determined by redundancy and facility under bridge – screening for CS 4 or 5 is used**
5	All criteria in scenario 3 plus deck damage has high consequence if ADT > 10000, screening not used
6	All criteria in scenario 5 plus considering screening factors for CS 4 or 5

"High" consequence, considering all superstructure damage modes as "Moderate" consequence, and determining the CF based on the redundancy of the bridge, as described in Section 3.6.9. Additional analysis was done to test the effect of including, or not including, the screening criteria of elements with a condition state identified as CS 4 or 5. It should be noted that including this screening factor affects the OF, making the likelihood "High" for any element with any portion of the element reported in CS 4 or 5, a failed condition. Using these screening criteria does not change the CF, but may change the inspection interval. This case, which includes redundancy, feature under, and condition screenings is applicable for the subject bridges, and is shown in bold in Table 14.

Finally, the CF was adjusted to consider the consequences for deck damage modes as "High" for bridges with high ADT, in this case determined by bridges with ADT of 10,000 or greater (according to NBI data). This case demonstrates the consideration of traffic volumes in the consequence analysis of a deck, which may be applicable in certain urban areas.

3.7.3 Back-Casting Results for Oregon

Figure 19 illustrates the results of the back-casting procedure as done on one of the subject bridges. Shown in this figure is the NBI condition rating history for the bridge, showing how the condition ratings have varied over the course of the back-casting period. On this graph, the inspection interval

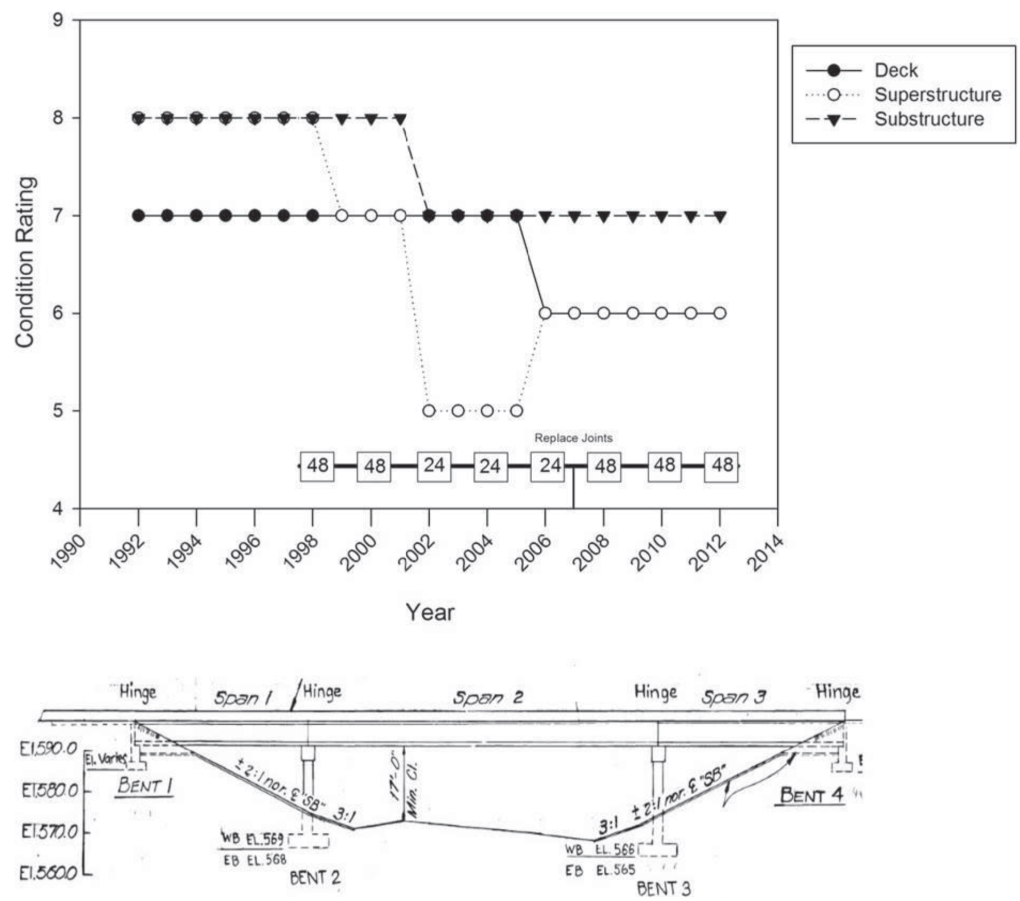


Figure 19. Example of the back-casting process showing NBI condition ratings over time and the inspection interval determined through RBI analysis.

determined through the RBI analysis for each year there was an available element-level inspection report is shown enclosed in a box near the bottom of the figure. In a few isolated cases, there were not element-level reports available for every year, though NBI data was available. This example was selected as an illustration of applying the RBI analysis for each historical inspection result, and how that outcome may vary over the course of the life of a bridge. In this case, the inspection interval was reduced and then later increases following a repair, based on the condition of the bridge. This was not common occurrence, but it is useful as an illustration of how the results of the back-casting are summarized in the figure, and how the RBI inspection interval could vary over the life of the bridge based on the RBI analysis. Also shown on the graph are any repairs that had been completed on the bridge, and the year that these repairs were completed.

It is very important to recognize that the RBI process is not intended to predict or track the NBI ratings. In some cases, changes in the inspection interval determined from the RBI analysis may coincide with changes in the NBI condition rating, because either these ratings are included in the analysis or the rating changes coincide with changes in the element condition states that are included in the analysis. In other cases, these may not coincide, because the RBI analysis depends not only on the current condition, but also the potential for serious damage to occur looking forward based on the bridge attributes (as expressed through the OFs) and the consequence of that damage. For example, a bridge rated in good condition according to the NBI condition rating may have a relatively short inspection interval, either because the potential for damage is high based on the attributes of the bridge, or the consequences are high based on the redundancy or other circumstances influencing the CF. The research team believes this feature, i.e., the ability to look forward with an RBI analysis, is a significant advantage over the present calendar-based system. At present, in the current calendar-based approach there is no rational way to attempt to address the negative (or positive) attributes associated with future condition of a given specific bridge or family of bridges.

Overall, the results of back-casting verified that the methodology was capable of determining an effective and safe inspection interval. There were no instances of bridge deteriorating to a serious condition during the RBI inspection intervals recommended using the proposed methodology. The process was effective in differentiating inspection intervals based on the risk profiles developed through the RAP process, i.e., the OFs stemming from attribute scoring and the CFs. In some cases, bridges that were in generally good condition according to the NBI ratings resulted in short inspection intervals, indicating that the process was sensitive to risk factors that are not necessarily revealed through condition ratings. In other words, even though the condition of the bridge at the present time was generally good, there was a high likelihood of deterioration based on the design, environment, or loading of the bridge. In other cases, bridges that included components rated in fair condition were assigned longer intervals.

Table 15 shows the overall results for each of the CF cases for the last inspection record analyzed for the 22 bridges, typically from an inspection conducted sometime between 2011 and 2013. The CF Case 4 is highlighted in the table because this case, which includes consideration of the redundancy of the bridge, traffic under the bridge, and screening any bridges with elements with CS 4 or 5 reported, is a durable and widely applicable category. These data are based on the consequence cases described above and the data models developed through the RAP. The year of construction, superstructure type (simple span or continuous), the facility under the bridge, and the scour rating are also shown in the table. These data were obtained from the NBI data for these bridges. This table also presents results for Cases 1 and 2, with CF for the superstructure always high or always moderate, respectively. These data represent the simplest analysis of the CF for a superstructure. Cases 5 and 6, which included an ADT criteria for deck CF are also shown, to illustrate how a more restrictive criteria for the deck would affect the analysis.

Scour ratings were not a part of the RBI analysis, as scour generally has its own evaluation procedures. Additionally, the scour rating was not considered in the overall analysis because this is a unique characteristic of the specific bridge, and therefore may skew the results for a population of bridges selected at random. A bridge owner may choose to screen bridges with poor scour ratings as a policy; however, screening bridges in this manner in the current analysis would not be beneficial in measuring the overall effectiveness of the RBI procedures.

Table 16 shows the summary of the RBI results for the Oregon bridges in terms of percentage of the sample population. Based on these analyses, again focusing on CF Case 4, approximately 41% of bridges would remain on a biennial inspection schedule, while just over 59% of bridges would have a larger interval of 48 or 72 months. These data illustrate the effect of using different criteria to identify the CF for the population of bridges, and results were as expected: relatively simple but conservative use of CF of "high" for the superstructure results in fewer bridges identified with extended intervals, using a less conservative "moderate" factor results in more bridges on extended intervals, etc.

The overall results of the back-casting, considering each of the analyses conducted at each existing inspection record, are shown in Table 17. These results include 157 separate analyses done based on the inspection records, and for each of the six cases for determining the CF and OF described in Table 14 above. CF Cases 5 and 6, which include consideration of the ADT on the bridge deck, show only a modest differ-

Table 15. Overall results for each of the CF cases in Oregon.

Bridge ID	Year Built	Facility Under	Simple span (SS) or Cont. (C)	Scour Rating	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
02376B	1975	Water	SS	3	48	48	48	**48**	24	24
07801A	1973	Highway	C	N	24	24	24	**24**	24	24
01741B	1962	Relief for waterway	SS	9	24	24	24	**24**	24	24
07935A	1973	Water	SS	3	24	48	48	**48**	48	48
07935B	1973	Water	SS	3	24	48	48	**48**	48	48
17451	1996	Water	C	8	48	48	48	**24**	48	24
16454	1987	Highway	SS	N	24	48	24	**24**	24	24
16453	1987	Highway	SS	N	48	48	48	**48**	48	48
9546	1967	Highway/ waterway	C	U	24	48	24	**24**	24	24
00988A	1967	Water	C	5	24	48	48	**48**	48	48
01056A	1970	Water	C	5	24	48	48	**24**	24	24
9358	1965	Highway	SS	N	24	48	24	**24**	24	24
16873	1991	Water	SS	8	48	72	72	**72**	48	48
18175	1999	Water	C	8	48	48	48	**48**	48	48
01895A	1995	Railroad waterway	C	8	24	48	48	**48**	48	48
9915	1970	Highway	C	N	24	48	24	**24**	24	24
8994	1962	Water	SS	U	24	48	24	**24**	24	24
8896	1963	Water	SS	3	48	48	48	**48**	48	48
20666	2009	Water	SS	8	48	72	72	**72**	48	48
19739	2007	Railroad waterway	C	5	24	48	48	**48**	24	24
19738	2006	Railroad waterway	C	5	24	48	48	**48**	24	24
19284	2005	Other	C	N	48	48	48	**48**	48	48

Note: N = not over waterway, U = bridge with "unknown" foundation.

ence. The results shown in this table are generally consistent with those shown in Table 16, considering that the bridges are aging with time, and consequently the inspection intervals may be reduced. For example, at the end of the back-casting period, 50% of the bridges had a 48-month inspection interval assigned, as shown in Table 16. However, 68% of the bridges had a 48-month interval assigned *at some point* in the back-casting period, and 57% of all of the analyses conducted indicated a 48-month interval, as shown in Table 17.

These data are significant in showing the consistency of the process when applied over 17 years of historical data through the back-casting process.

Significantly, there were no instances of bridge deteriorating to a serious condition between inspection intervals, and those with poor condition rating generally were assigned inspection intervals of 24 months based on the RBI analysis. For example, Figure 20 presents the condition rating history and RBI inspection interval for Bridge 16454. This bridge was

Table 16. Summary of final back-casting intervals for 22 bridges in Oregon.

CF Case No.	Inspection Interval		
	24 month	48 month	72 month
1	64%	36%	0%
2	9%	82%	9 %
3	32%	59%	9%
4	**41%**	**50%**	**9%**
5	45%	55%	0%
6	55%	45%	0%

Table 17. Summary of back-casting intervals for 22 bridges in Oregon (all analyses).

CF Case No.	Inspection Interval		
	24 month	48 month	72 month
1	62%	38%	0%
2	8%	82%	10%
3	28%	62%	10%
4	**34%**	**57%**	**9%**
5	39%	58%	3%
6	44%	53%	3%

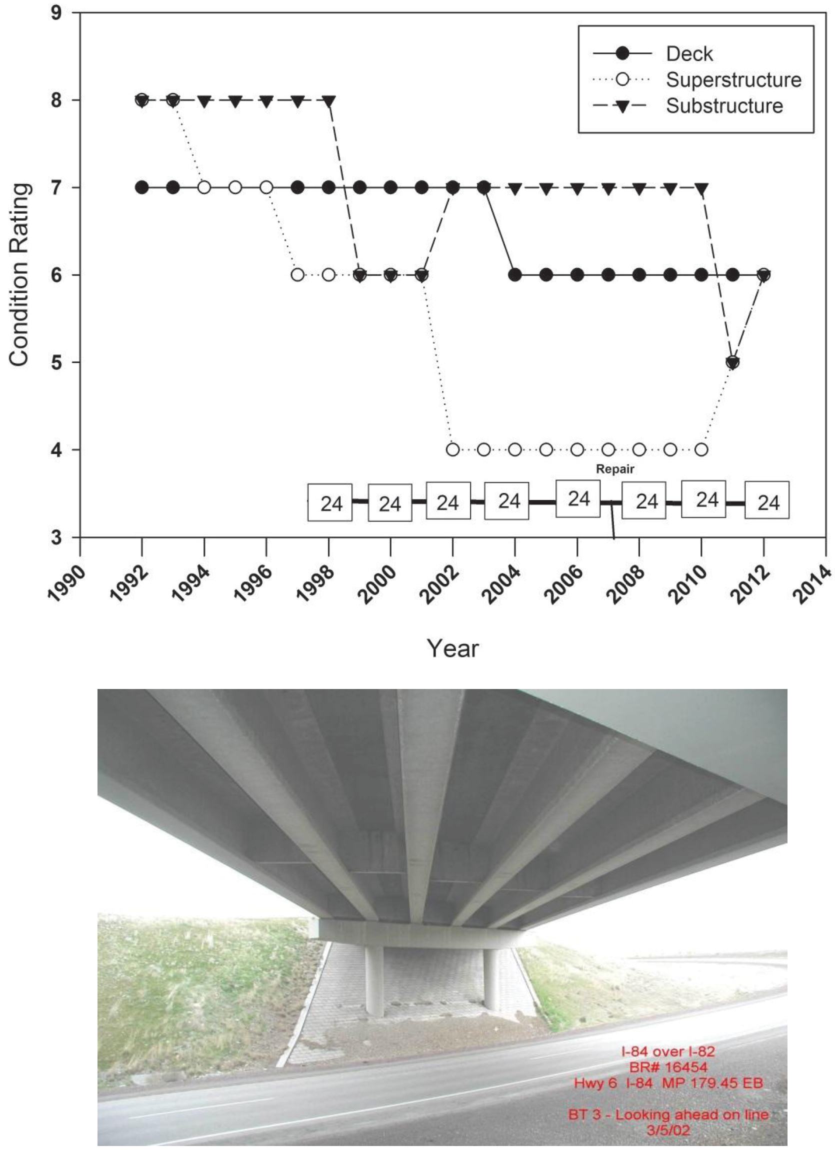
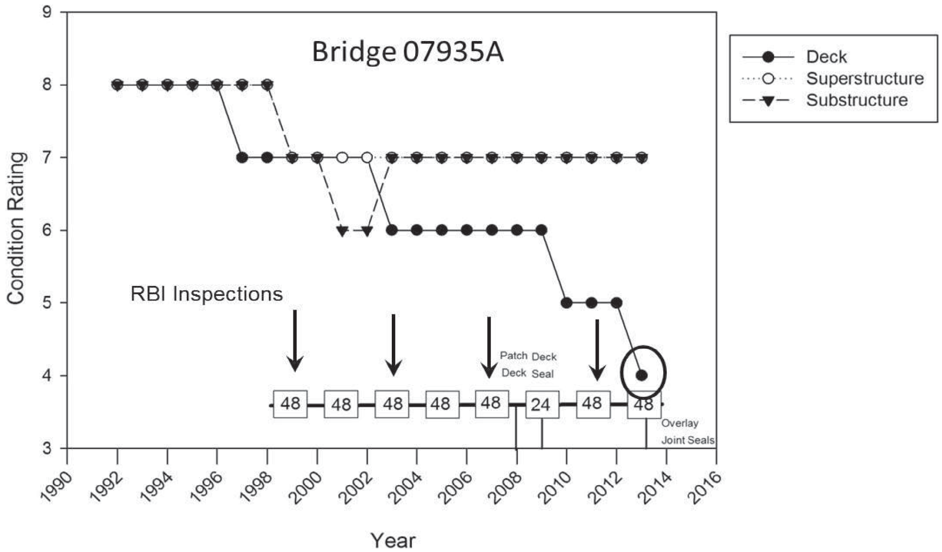


Figure 20. Condition rating history and RBI inspection interval for Bridge 16454.

constructed in 1987, less than 30 years ago, and the back-casting assessment for the bridge was initiated in 1998, when the bridge was only 11 years old. However, the RBI inspection interval was determined to be 24 months, due to damage modes related to corrosion susceptibility of the superstructure. For this bridge, cracking in the superstructure was present early in the service life, resulting in increased likelihood of corrosion damage to the strands in the prestressed members. A repair completed in 2007 consisted of epoxy injection of the superstructure cracking. Looking forward from 1998, the superstructure condition deteriorated relatively rapidly as the bridge aged. For this bridge, the RBI assessed interval was 24 months throughout the back-casting period, an appropriate interval given the susceptibility to corrosion damage for this bridge. The validity of the short interval is also supported by the fact that the CR decreased from 6 to 4 around 2002. Again, the ability of the RBI method to identify the attributes that would suggest the superstructure is susceptible to damage resulted in the shortened interval.

There were several bridges that had reported poor condition ratings, and typically those had inspection intervals of 24 months assigned. There were some exceptions: for example, Bridge 07935A (Figure 21) had a reported condition rating of poor (CR = 4) in 2013 and had an overlay installed, and the inspection interval assigned by RBI was 48 months. This may seem like a long inspection interval considering this deck apparently required an overlay. However, the element-level condition state for the deck was 100% in CS 2 (*CS 2 = Patched areas and/or spalls/delaminations exist on either side of the deck. The combined distressed area is 10% or less of the*



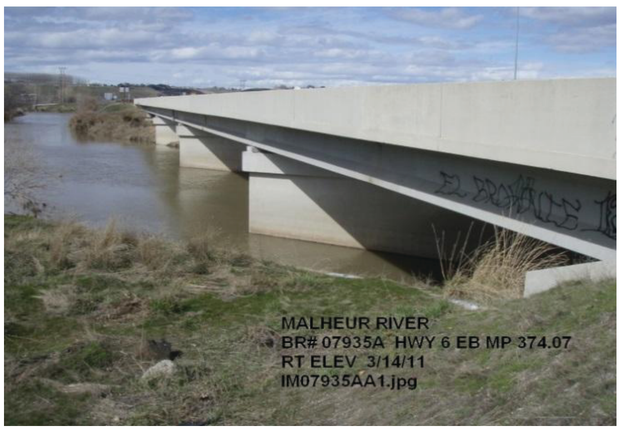


Figure 21. NBI condition rating history and RBI inspection intervals for Bridge 07935A.

total deck area); the soffit element was 95% in CS 1 and 5% in CS 2, and the deck cracking element was 100% in CS 1. In this case, the assigned NBI condition rating appears not to be well correlated with the element condition states. Given that the NBI condition rating typically have a variability of ±1, and in this case was not consistent with the element-level data, it may be that the condition rating is not reflective of the overall conditions. Since these element-level condition states contribute significantly to the likelihood estimate, a longer inspection interval was assigned.

Arrows superimposed on the figure illustrate when an RBI inspection would have been conducted, assuming the start year of 1999. In this case, the year of the RBI inspection would not coincide with the year that the condition rating of 4 occurred, though the schedule year is somewhat arbitrary, being based herein on the earliest date of available data. This example was the most problematic of the 22 sample bridges included in the back-casting, in terms of the RBI interval assigned for the bridge. However, as described above, the apparent incongruity between the RBI inspection interval and the condition rating was explained by the element-level inspection results.

3.7.3.1 Risk Matrices

The results of the analysis can be illustrated on the risk matrix to summarize the data and indicate the controlling damage modes, i.e., those damage modes representing the highest risk or IPN. Table 18 shows the damage modes assessed in the Oregon case study, along with an alpha-numeric

Table 18. Key to risk matrix summaries of RAP analysis.

Deck	Superstructure	Substructure
Spalling (D1) Rutting (D2) Cracking (D3)	Cracking (S1) Strand Corrosion (S2) Impact (S3) Rebar Corrosion Within the Span (S4) Bearing Seat Problems (S5)	Settlement (F1) Corrosion (F2)

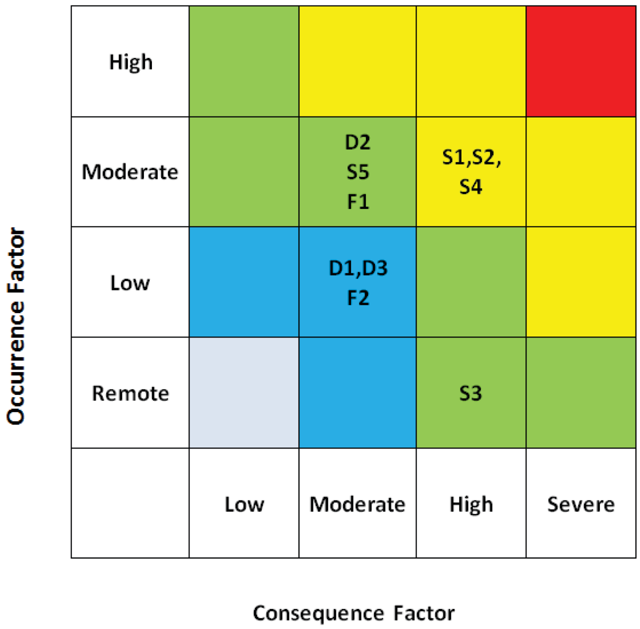


Figure 22. Risk matrix for Bridge 16454 illustrating results of the RBI analysis.

identifier (D1, D2, etc.). Figure 22 includes the risk matrix for Bridge 16454 with the damage modes located on the matrix according to the results of the RBI analysis (OF and CF). As shown in this figure, the results of the RBI analysis are plotted in appropriate locations on the diagram. The locations on these plots describe the inspection interval identified, and can also be used to calculate the IPN to identify the most important damage modes as identified through the RBI process. For example, in the plot shown, the IPN for S1, S2, and S4 = 9, indicating that these damage modes (cracking, strand corrosion, and rebar corrosion) have high importance related to the risk profile for the bridge. These data are useful for identifying emphasis areas for the inspection of the bridge, and could be included in inspection procedures or guidance as a normal outcome of the RBI assessment. Such risk-based inspection procedures may improve the reliability of inspection and communicate the engineering-based RBI assessment of the key damage modes for a bridge to inspectors in the field. Appendix C includes the controlling damage modes for the RBI analysis of bridges in Oregon. Frequently, several of the damage modes had similar risk profile, such that there is not a "controlling" damage mode. This is typical for bridges in good condition, such that inspection intervals are typically longer. These controlling damage modes evolve during the service life of the bridge, as damage develops and affects the OF.

3.7.3.2 Surrogate Data for a Family of Bridges

An analysis was conducted of the overall inventory in Oregon based on the results of the RAP analysis. The objective of this analysis was to identify the population of low-risk bridges that were in very good condition and that could be assessed in an entirely data-driven process that did not require individual assessments of a bridge. Such bridges could be considered for extended inspection intervals throughout the RBI analysis, based only on a screening process that utilized data in existing databases. These included a series of 22 items that were readily available, such as NBI items or bridge elements included in standard inspection reports. Table 19 indicates the individual items that were analyzed and the accepted values from the screening. Each of the criteria was based on attributes or items developed from the RAP analysis. Each of these items is shown in Table 19, along with the screening criteria used to analyze the inventory data. Generally, these criteria include bridges that have NBI condition ratings of 7 or higher and have no elements with any condition states of 3 or higher reported. In this case, scour ratings were considered as shown in the table, eliminating bridges with unknown foundations, bridges without scour analysis, or bridges that are scour critical.

Screening the Oregon databases was performed by the Oregon DOT, which provided a listing of all bridges meeting the element-level screening criteria included in the table. For the NBI criteria, filtering of the data was performed by the research team. Generally, these database searches and filtering took only a short time interval—a matter of 1 hour or less, consisting primarily of inputting screening or filtering criteria in search functions and yielding immediate results.

The results of the analysis indicated that 18% (652/~3600) of the prestressed bridge inventory met all of the criteria indicated in the table. For these bridges, the likelihood of serious damage developing in the next 72 months interval could be considered low or even remote, based on the RAP analysis. Assuming the CF to be moderate for this population of bridges, an inspection interval of 72 months could be assigned. If the effect of scour is not considered, or considered as a separate inspection requirement, the number of bridges meeting the other criteria was 970 bridges, or about 1 in 4 bridges.

These data indicate that the RAP process can be used to develop criteria for an entirely data-driven process for identifying bridges that are very low risk, and the number of bridges meeting these criteria is significant (almost 1 in 5 prestressed bridges in Oregon). Such analysis takes only a matter of a few hours to complete, once the data items are identified through the RAP process.

3.8 Back-Casting Results for Texas

This section of the report describes the results of back-casting for steel bridges in the state of Texas. This includes a description of the environments identified by the RAP for use in the OF analysis, the CF used in the back-casting analy-

Table 19. List of criteria for data-driven screening process based on RBI.

Prestressed Bridges (5, 6)*				
Deck				
No.	**Item**	**Criteria**	**Damage Mode**	**Notes**
1	#358 Deck Cracking SF	CS 2 or less	Deck Cracking	
2	#359 Soffit Cracking SF	CS 2 or less	Spalling	
3	Deck Elements	CS 2 or less	Spalling	
4	Age	Less than 50 years	Spalling	Deck Condition
5	NBI Item 58	7 or greater	Spalling	Deck Condition
6	# 325	CS 2 or less	Spalling	Dynamic Loading
7	#370-374	Coded 1 or uncoded	Fire	Fire or Incident
8	#326	CS 1 only	Rutting	Deck Wearing Surface Condition
Superstructure				
9	NBI Item 54	17 ft. or greater	Superstructure Impact	Bridge Height
10	NBI Item 70	Coded 5	Cracking	Legal Load Capacity
11	NBI Item 71	Coded 4 or greater	Impact	No Overtopping
12	NBI Item 41	Coded A	Cracking	Open, No Restrictions
13	#362 Impact(SF)	None	Rebar Corrosion	Traffic Impact Smart Flag
14	Superstructure Elements #104, 109, 115	CS 2 or less	Strand and Rebar Corrosion, Bearing	
15	NBI Item 59	7 or greater	Superstructure Condition	Superstructure Condition Rating
16	Deck Joint Items (All)	CS 2 or less	Bearing Area Damage	Failed Deck Joint
17	Bearing Elements (All)	CS 2 or less	Bearing Area Damage	Bearing Issues
18	NBI Item 34	30 degrees or less	Bearing Area Damage	Bridge Skew
Substructure				
19	#360 Settlement SF	CS 1 or uncoded	Settlement	Settlement
20	NBI Item 60	7 or greater	Corrosion Damage	Substructure Condition
21	NBI Item 113	Not U, 6 or 0-4	Settlement	Scour
22	Substructure Elements	CS 2 or less	Corrosion Damage	Sub. Element Conditions

* 5 = prestressed concrete, 6 = continuous prestressed concrete: from NBI database.

sis, overall results, and specific examples selected to illustrate implementation of the technology.

3.8.1 Environments

The environmental conditions considered in the analysis of bridges in Texas also differed depending on the damage mode being considered for the RAP. Generally, the RAP identified an east-west interstate highway, I-20, as dividing the state into areas where de-icing chemical were likely to be used (north) and areas where they are very unlikely to be used (south). These environments were applied for most damage modes, such as spalling of bridge decks. For the damage mode of section loss in steel members, the RAP identified that the coastal areas were the most aggressive environment, followed by areas north of I-20 and a moderately aggressive environment, and all other areas being the least aggressive environment.

3.8.2 CFs

There were four different CF cases considered in analyzing results in Texas, as shown in Table 20. These different cases were selected to illustrate how different criteria established by a RAP might affect the outcome of the analysis. These included considering all superstructure damage modes as "high"

Table 20. CF cases used for back-casting in Oregon.

Case No.	Description
1	High CF for superstructure
2	Superstructure CF is determined by redundancy and facility under bridge (screening not used)
3	**Superstructure damage mode CF is determined by redundancy and facility under bridge—screening for pin and hanger used**
4	All criteria in scenario 3 plus deck damage has high consequence if ADT > 10000

Table 21. List of bridges analyzed in Texas.

Bridge ID	Year Built	Facility Under	Structure Type	Scour Condition	Case 1	Case 2	Case 3	Case 4
01-139-0-0769-01-007	1956	Waterway	C	5	24	24	**24**	24
02-127-0-0014-03-194	1963	Highway	C	N	24	24	**24**	24
02-127-0-0094-04-057	1939	Waterway	SS	8	48	72	**72**	48
02-220-0-1068-02-058	1957	Highway	C	N	24	24	**24**	24
05-152-0-0067-11-188	1990	Highway, Railroad	C	N	48	48	**48**	48
08-030-0-AA01-31-001	1985	Waterway	SS	5	48	48	**48**	48
12-085-0-1911-01-003	1943	Waterway	SS	8	24	48	**48**	48
12-102-0-0027-13-195	1979	Highway	SS	N	48	48	**48**	48
12-102-0-0500-03-320	1990	Highway	C	N	48	48	**48**	24
15-015-0-0025-02-162	1967	Highway	C	N	48	48	**48**	48
15-015-0-B064-55-001	1964	Waterway	C	5	48	72	**72**	72
18-057-0-0092-14-210	1973	No Feature Under	C	N	48	48	**24**	24
18-061-0-0196-01-133	1960	Highway	C	N	24	24	**24**	24
19-019-0-0610-06-162	1971	Highway	C	N	24	24	**24**	24
23-141-0-0251-05-020	1934	Waterway	C	8	48	48	**48**	48
23-215-0-0011-07-056	1948	Waterway	C	8	48	48	**24**	24
24-072-0-0167-01-059	1970	Highway	C	N	48	48	**48**	48

consequence, and determining the CF based on the redundancy of the bridge, as described in Section 3.6.9. Additional analysis was done to test the effect of including, or not including, the screening criteria for a bridge with a pin and hanger connection. This screening criteria were not identified during the RAP process, although it would likely have been identified during the course of a full-scale implementation of RBI. Again, this screening factor affects the OF, making the likelihood "high" for any component containing a problematic detail such as a pin and hanger. Using these screening criteria does not change the CF, but may change the inspection interval. Finally, the CF was adjusted to consider the consequences for deck damage modes as "high" for bridges with high ADT, again determined by bridges with ADT of 10,000 or greater (according to NBI data). The CF Case 3 is considered the most appropriate case for the analysis, and is highlighted in the following tables.

3.8.3 Back-Casting Results for Texas

Table 21 lists the bridges analyzed in this portion of the study. This table includes data on the year of construction, the facility under the bridge, the structure span type (simple or continuous), etc. The CF Case 3 is highlighted in this table to illustrate the most likely or commonly applicable CF case that would be utilized to evaluate the bridges.

Data models developed through the RAP process were used to analyze each bridge and determine the appropriate RBI interval. Table 22 shows the results of the analysis for the most recent year for which inspection results were available. As shown in the table, for the most recent analysis year, 12% of the bridges had a 72-month inspection interval and 53% with a 48-month inspection interval, while 35% were found to have a 24-month maximum interval. The maximum interval found during the analysis indicated that 24% of the bridges had an RBI interval of 72 months at some point during the back-casting period, indicating that the RBI practice included shorter intervals as these bridges became older and deterioration progressed.

Appendix C includes the controlling damage modes for the RBI analysis of bridges in Texas. Frequently, several of the damage modes had similar risk profile, such that there is not a "controlling" damage mode. This is typical for bridges in good condition, such that inspection intervals are typically longer.

Table 23 shows the overall results from each of the 117 analyses conducted during the back-casting procedure. These data illustrate the relative consistency of the process and the application of the attribute criteria to the steel bridge population in Texas.

3.8.3.1 Examples

This section provides two examples from the analysis of bridges in Texas. The first example is a bridge that included

Table 22. Results of back-casting for bridges in Texas.

CF Case No.	Inspection Interval		
	24 month	48 month	72 month
1	35%	65%	0%
2	29%	59%	12%
3	**35%**	**53%**	**12%**
4	47%	47%	6%

Table 23. Results of back-casting including all analysis.

CF Case No.	Inspection Interval		
	24 month	48 month	72 month
1	29%	71%	0%
2	27%	58%	15%
3	**32%**	**53%**	**15%**
4	44%	50%	6%

a pin and hanger connection. In the back-casting analysis, the presence of a pin and hanger connection was used as a screening factor that made the OF high, regardless of other attributes of the bridge. This screening factor is based on the historical experience that pin and hanger connections frequently present maintenance challenges. Figure 23 indicates the inspection intervals determined for the structure during the back-casting, along with the NBI condition rating history. As can be seen in the figure, the superstructure condition rating dropped 3 ratings, from 7 to 4, over a single inspection interval. According to the inspection records reviewed during the back-casting, this reduction was due to damage to the pin and hanger connection. Rehabilitation of this pin and hanger joint was required and was ongoing in 2013.

This example is important for illustrating the importance of identifying screening factors in the RBI process. Screening factors are intended to identifying bridge attributes that make the likelihood of serious damage unusually high, unusually uncertain, or otherwise different than other bridges in a group. As shown in this example, the screening factor of bridges with pin and hanger connections was needed to capture the unusual behavior of this bridge.

The second example was a steel multi-girder short-span bridge constructed in 1943. For this bridge, located in a coastal environment, back-casting indicated an inspection interval of 72 months between the years of 2001 and 2006, changing to a 48-month interval based on the results of the 2008 inspection. The change in the inspection interval for this bridge resulted from corrosion-related deterioration of the superstructure, i.e., likelihood for severe section loss. As shown in Figure 24, even though this bridge was 70 years old, the overall condition of the superstructure was satisfactory at the beginning of the back-casting period, and subsequently reduced to fair, where the structure condition rating remained. The inspection interval is also reduced during this period. Again, the RBI practice does not necessarily reflect the NBI condition ratings, as the data model includes specific information regarding the

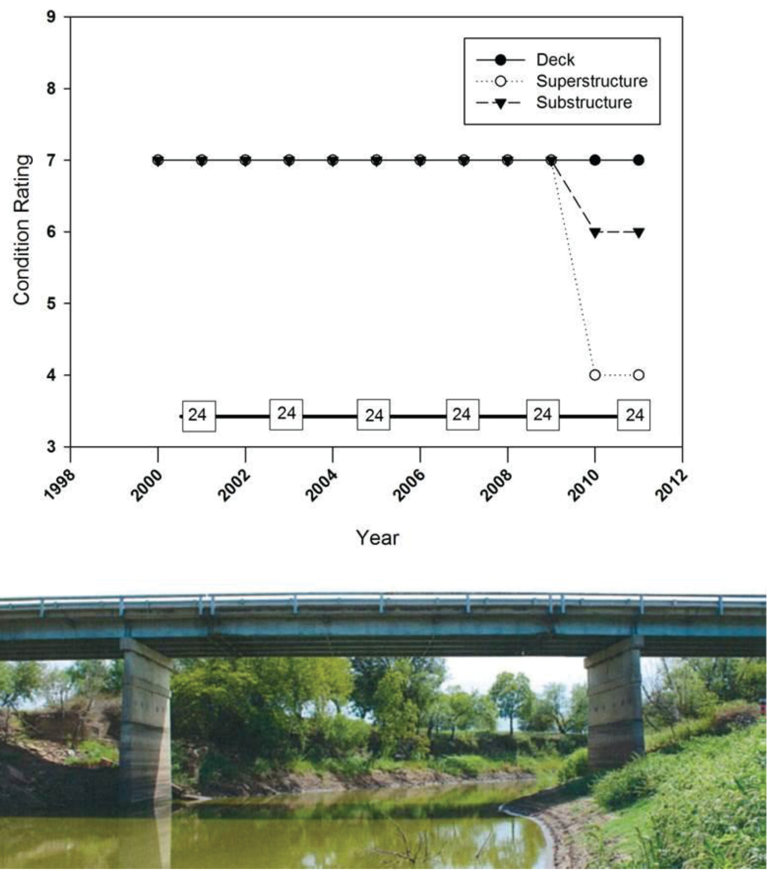


Figure 23. Historical NBI data and RBI inspection intervals for a steel bridge in Texas with a pin and hanger connection.

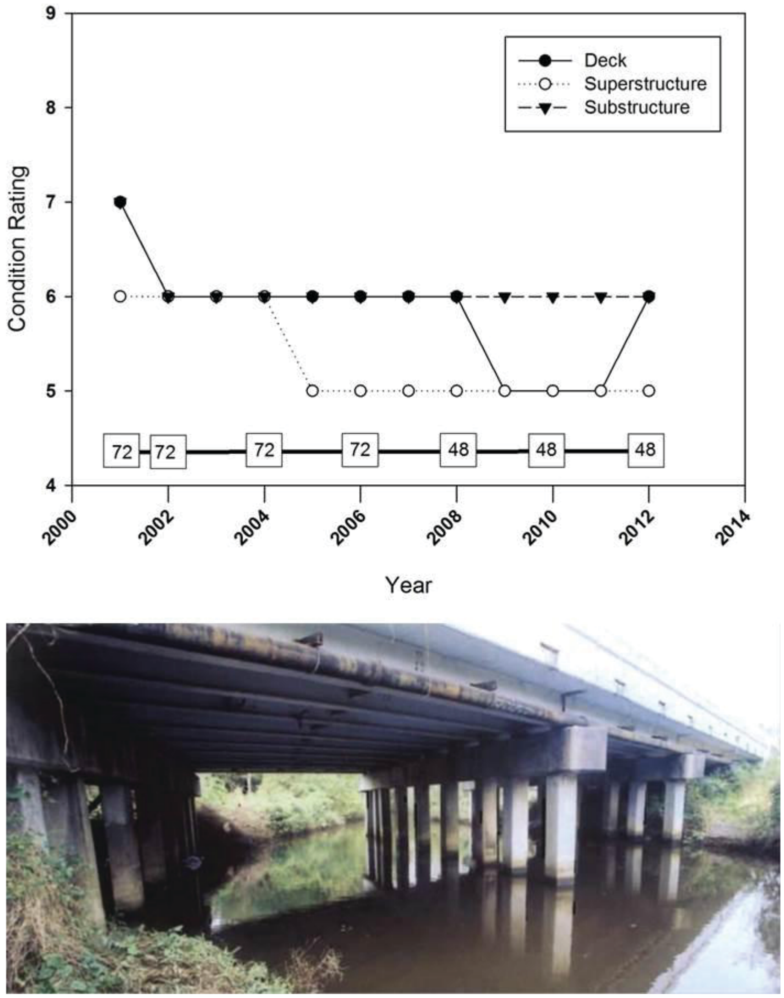


Figure 24. Example bridge in Texas with decreasing inspection interval resulting from section loss.

element condition state and other factors. It should also be noted that the deck and substructure are generally in satisfactory condition, and the superstructure is in "Fair" condition due to the damage mode of section loss caused by corrosion, known to be a slow-acting deterioration mechanism.

Table 24 indicates the damage modes evaluated for the steel bridges in Texas, including damage modes for the superstructure, deck, and substructure. Figure 25 indicates these damage modes plotted on the standard risk matrix, with Figure 25A being the risk matrix for the bridge including a pin and hanger connection, and Figure 25B the bridge with section loss. Considering Figure 25A, the data plotted on the figure illustrate that according to the damage modes identified, the inspection interval for this bridge would be 48 months. Recall that this bridge included a pin and hanger connection, used as a screening criteria to identify the OF as high for the superstructure. In other words, the data in Figure 25A indicates the inspection interval for the bridge if the bridge did not include a pin and hanger connection. This illustrates how screening criteria affect the analysis; for this bridge, the overall condition based on the condition rating, notes, and element-level data suggest an inspection interval of 48 months. However, the bridge includes an attribute, i.e., a pin and hanger connection, that makes the anticipated behavior of the bridge

Table 24. Damage modes for the steel bridges in Texas.

Deck	Superstructure	Substructure
Spalling (D1)	Section Loss (S1)	Settlement (F1)
Punch Through (D2)	Impact (S2)	Corrosion (F2)
Cracking (D3)	Fatigue Cracking (S3)	
Delamination (D4)	Overload Damages (S4)	

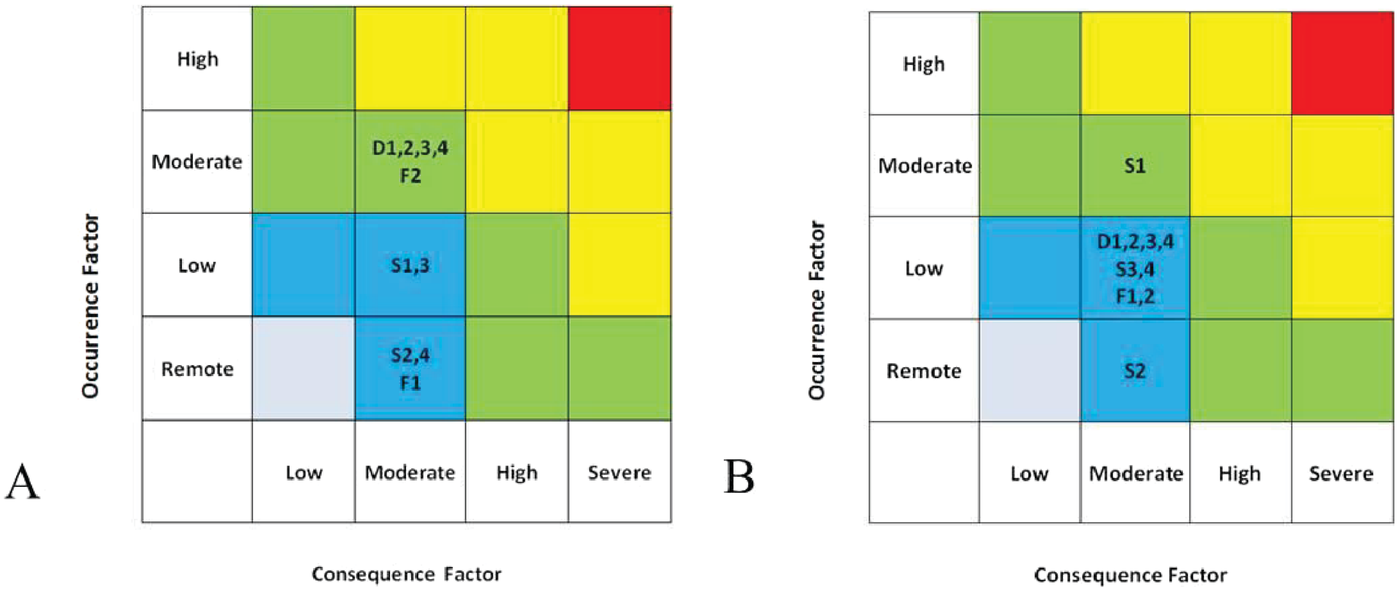


Figure 25. Risk matrices for steel bridges in Texas.

unusually uncertain, and not typical of other bridges in the family. As such, this screening criterion is critical to determining the effective interval for this bridge.

Figure 25B indicates the risk matrix for Bridge 1922-01-003. As shown in this figure, the damage mode of section loss controls the inspection interval for the bridge. These data illustrate how individual damage modes can control the inspection interval for the bridge. In this case, the bridge is 70 years old, and in a relatively aggressive coastal environment. As such, it is rational that a shorter inspection interval would be required than if the bridge were in an arid environment.

3.8.3.2 Surrogate Data

Surrogate data was analyzed for Texas based on the data models developed from the RAP. Because Texas has not traditionally used its element-level data for bridge management purposes, and as such these data are not maintained within a single database, the surrogate data relied solely on NBI data to scan the inventory and identify bridges for the extended interval of 72 months. Table 25 indicates the parameters used in the analysis.

Based on this analysis, it was found that 927 bridges, or 12.5% of the inventory in Texas, met all of these parameters

Table 25. List of criteria for data-driven screening process based on RBI for Texas.

Steel Bridges (3, 4)				
Deck				
No.	**Item**	**Criteria**	**Damage Mode**	**Notes**
4	Age	Less than 50 years	Spalling	Deck condition
5	NBI Item 58	7 or greater	Spalling, Cracking	Deck Condition
Superstructure				
10	NBI Item 70	Coded 5	Cracking	Legal Load Capacity
11	NBI Item 71	Coded 4 or greater	Impact	No Overtopping
12	NBI Item 41	Coded A	Cracking	Open, No Restrictions
7	NBI item 54	17 ft or greater	Superstructure Impact	Bridge Height
15	NBI Item 59	7 or greater	Superstructure Condition	Superstructure Condition Rating
18	NBI Item 34	30 degrees or less	Bearing Area Damage	Bridge Skew
Substructure				
20	NBI Item 60	7 or greater	Corrosion Damage	Substructure Condition
21	NBI Item 113	Not U, 6 or 0-4	Settlement	Scour

for the extended inspection interval. These data were also analyzed without regard to the age criteria identified in Table 25. This resulted in a slight increase: 1068/7423 = 14.38%.

3.9 Discussion of the Case Studies in Texas and Oregon

The case studies were used to verify the effectiveness of the RBI procedure developed through the research. Overall, the back-casting illustrated that the RBI process was effective in determining a suitable inspection interval for each bridge in the study.

3.9.1 Back-Casting Results

The back-casting procedure was used to verify the effectiveness of the RBI process, and there were three primary questions addressed, as discussed in Section 3.6.4. The following discusses each question individually in terms of the outcome of the back-casting.

1. *Did the condition rating for any component change significantly during the RBI interval in a manner that was not captured or anticipated effectively, but would have been captured (or detected sooner) by a standard, 24-month interval?*
   A detailed review of the condition ratings for each of the bridges included in the study was conducted, as illustrated in the examples presented herein. This review and analysis indicated that there were no cases where the condition rating changed unexpectedly in a manner that was not captured or reflected in the RBI inspection interval identified when screening criteria were used. Recall that screening criteria of CS 4 or 5 for prestressed bridges in Oregon, and a pin and hanger connection in Texas, were implemented in the analysis.
2. *Were there any significant maintenance or repair actions completed that would have been delayed as a result of implementing an RBI interval (relative to a standard, 24-month interval)?*
   Reviews of the repair histories for the subject population of bridges were conducted based on available records. This review did not indicate any instances where there were sudden or unexpected repairs required that would have been delayed as a result of RBI intervals. There were cases where routine maintenance or repair, such as a deck overlay, may not coincide with an RBI interval; however, this depends on when the RBI cycle was initiated. There were also several cases where repair or rehabilitation activities were performed during the back-casting window; however, the activities were generally consistent with the RBI analysis, and would not be adversely affected by the RBI implementation. For example, a bridge identified by RBI as being susceptible to corrosion damage had epoxy injection performed, consistent with the RBI analysis. In most cases, there were no significant repairs during the back-casting window.
3. *Were there any significant factors or criteria not identified through the RAP analysis that were needed in the data models to provide suitable results?*
   There was one case in each state where there were factors that were not identified through the RAP processes were needed for the data models. In Oregon, a screen for elements with CS 4 or 5 was needed in the data models; in Texas, a screen for pin and hanger connections was needed. In both cases, these are relatively obvious additions to the data models that were overlooked during the RAP meetings, but would likely be identified by anyone implementing the back-casting procedures.

The overall objectives of the case studies were to demonstrate the implementation of the methodologies with state DOT personnel, and verify the effectiveness of RBI analysis in determining suitable inspection intervals for typical highway bridges. In terms of these objectives, the RAP meeting in each state, and the effectiveness of the data models developed through that RAP process, indicated that the processes developed for RBI analysis were effective, practical, and implementable using state DOT personnel in Texas and Oregon. The results of the back-casting process described above verified the effectiveness of the RBI procedures, and demonstrated that implementation of the RBI practice did not adversely affect the safety and serviceability of the sample bridges analyzed.

It should also be noted that for bridges where an inspection interval of 72 months was proposed using the RBI analysis, there were no cases of sudden repair, unexpected progression of damage, or sudden changes to the condition ratings for the bridge. These results indicated that the RBI procedures were effective in identifying a portion of the inventory, typically on the order of 10% of the sample bridge population, where an inspection interval of 72 months provided a suitable inspection interval that did not compromise the safety and serviceability of these bridges. It should be noted that the sample population of bridges was older than the average age of the inventories in each state, such that the identified rate (~10%) would likely be higher for a population of bridges constructed more recently.

CHAPTER 4

Conclusions, Recommendations, and Suggested Research

This research developed inspection practices to meet the goals of (1) improving the safety and reliability of bridges and (2) optimizing resources for bridge inspection. The goals of the research have been achieved through the development of a new guideline document entitled "Proposed Guideline for Realibility-Based Bridge Inspection Practices," which has been developed based on the application of reliability theories. This document meets the project objective of developing a recommended practice for consideration for adoption by AASHTO, which is based on rational methods to ensure bridge safety, serviceability, and effective use of resources. A reliability-based approach was fully developed and documented through the Guideline. Background information and foundation for key elements of the process have been further expanded in the present report, to provide additional details and perspectives on the research conducted as part of the project. However, the primary outcome of the study is the comprehensive Guideline developed, which provides a new paradigm for bridge inspection. This new paradigm could transform the calendar-based, uniform inspection strategies currently implemented for bridge inspection to a new, reliability-based approach that will better allocate inspection resources and improve the safety and reliability of bridges.

The implementation of the Guideline developed through the research was tested by conducting case studies in two states. The objectives of the case studies were to demonstrate the implementation of the methodologies with state DOT personnel, and verify the effectiveness of RBI analysis in determining suitable inspection intervals for typical highway bridges. The verification of the methodology was analyzed using a backcasting procedure that compared historical inspection records and the results of RBI analysis. These studies demonstrated and verified the effectiveness of the procedures developed in the research for identifying appropriate inspection intervals for typical highway bridges. It was shown through these studies that the RBI practices identified appropriate inspection intervals of up to 72 months. It was concluded from these studies that implementation of the RBI practices did not adversely affect the safety and serviceability of the bridges analyzed in the study, based on the analysis of historical inspection records. These studies also successfully demonstrated the implementation of the Guideline and the procedures therein using state DOT personnel.

The results reported herein demonstrated and verified that inspection intervals of up to 72 months were suitable for certain bridges. Such extended inspection intervals would allow the reallocation of inspection resources toward bridges requiring more frequent and in-depth inspections, resulting in improved safety and reliability of bridges. As such, the project goals of developing a reliability-based bridge inspection practice that could improve the safety and reliability of bridges, and optimizes the use of resources, were achieved through the research.

The following sections describe specific recommendations and suggested research, including detailed suggestions for conducting key elements of an implementation strategy intended to support the broader implementation of the research.

4.1 Recommendations

The research reported herein has demonstrated the effectiveness of the RBI procedures for determining suitable inspection intervals for typical highway bridges, and as such, implementation of the RBI technology is recommended. The research also demonstrated that inspection intervals of up to 72 months were suitable for certain bridges and did not affect the safety and serviceability of bridges analyzed in the study. Such extended inspection intervals would allow the reallocation of inspection resources toward bridges requiring more frequent and/or in-depth inspections, resulting in improved safety and reliability of bridges. Based on these results, implementation of RBI technology and inspection intervals of up to 72 months for certain bridges should be pursued.

The procedure, methods, and approach described herein can be applied for atypical bridges as well. For example, non-redundant bridge members can be assessed using this approach, as illustrated in previous research (*60*). Specific attributes may differ for such an application; examples and illustrations of applying the RBI technology for these applications should be pursued. The approach can also be applied to complex bridges, or to bridges with advanced deterioration. Analysis requirements may be more detailed and advanced; development of such analysis should be pursued to provide a uniform strategy for bridge inspection across the entire bridge inventory. Additional research and testing may be used to broaden the application of the RBI technology.

Finally, the back-casting procedure utilized herein should be considered for implementation when RBI practices are to be used. This recommendation is based on the research result that indicated a screening criteria in each state was not identified during the RAP. Additionally, the RAP process may be subject to variability not observed in the research when applied over a broader platform. Back-casting provides a means for verification of models developed by the RAP and QA tool for assessing the RBI process. As such, the back-casting procedure provides a critical tool for the implementation of RBI technology.

4.2 Suggested Research

Suggested research stemming from this project includes developing applications of the technology for atypical highway bridges, including non-redundant members, complex bridges, and bridges with advanced deterioration, as described above. Additional research to demonstrate the consistency of the process across a larger population of bridge owners, and for families of bridges not examined herein, should be undertaken.

In addition, efforts will be required to support implementation of the technology. A comprehensive implementation plan, which includes additional research on such factors as economics of applying the methodology, is included below.

4.2.1 Implementation Strategy

The implementation of a reliability-based inspection planning process such as described herein will be a difficult challenge. As with any existing established procedures, specifications, or policy, change is difficult. The current U.S. bridge inspection program and associated procedures have been the standard since the early 1970's, and considerable "infrastructure" has been developed to support the program. Well-established training, experience, and organizational structure within state departments of transportation will need to be modified to meet the needs of an RBI practice. The workforce will need to be retrained to meet the needs of the new approach to inspection and inspection planning. Existing legislative requirements, including the NBIS will need to be modified to allow for the new methodology to be implemented. Therefore, a strategy for converting the established bridge inspection programs from a uniform, calendar-based system to a reliability-based system is required. This section of the report describes implementation strategies and tasks to establish a new paradigm for bridge inspection based on the RBI processes described in the Guideline.

A number of implementation challenges exist looking forward toward the adoption of the RBI methodology. Inspection program organizational structures and personnel may need to be modified to accommodate the larger role of engineering and inspection planning required for RBI compared to a uniform, calendar-based approach. Personnel with suitable experience and knowledge to effectively conduct the necessary assessment will be required. In an era where government agencies are suffering significant fiscal challenges, often resulting in staff reductions, developing and retaining the necessary resources may be a challenge. A strong technical foundation for RBI will need to be developed to justify maintenance of the resources needed.

Training and knowledge development to support RBI will also be needed to implement the technology on a widespread basis. Developing the necessary tools to train individuals in the various aspects of the technology and processes will be an important part of technology transfer and implementation.

There will also be a significant political challenge to modifying an existing inspection system, which has been in effect for many years, with a process that may result in fewer inspections for certain bridges, even if the process results in an improvement in the safety of bridges overall. Engineers, inspectors. and maintenance personnel are likely to perceive the benefits of a more rational system, but the non-technical audience may be more difficult to convince. Data from additional case studies or pilot implementation, economic impacts, and safety analysis will be required to provide evidence to support the new approach for inspection planning. However, because deterioration patterns for bridges typically require a long time period to manifest, and failures are rare, generating empirical data to measure improvements in safety will be a significant challenge. The implementation strategy described in this section is designed to address these issues, and will require some investment of resources to execute and complete effectively.

Given these challenges, the implementation strategy has been developed to meet the following goals:

- Provide a technical foundation for widespread implementation of the technology and
- Develop community support for the new inspection approach.

Activities in the implementation strategy to provide a technical foundation for widespread implementation of the technology include conducting additional case studies in certain states to test and develop the technology further, developing training modules and software to support the technologies, and conducting a study focused on the economic and safety impacts of transitioning to the new inspection approach. To develop community support, the implementation plan proposes developing an oversight committee to monitor and develop the Guideline and address bridge-owner needs, and developing an effective communication strategy. Throughout the implementation activities described herein, the FHWA can play an important role in assisting with moving the technology toward eventual acceptance.

4.2.1.1 Implementation Tasks

The strategy developed for implementing RBI for bridges in the United States will require a number of steps be completed to test and refine the technology, develop support for transition to a new approach to inspection planning, and eventually gain widespread acceptance of the new technology. The implementation plan developed for the project consists of a number of individual tasks to be completed to achieve the desired goals.

Task 1. Establishment of an oversight committee.
Task 2. Additional case studies.
Task 3. Development of training modules.
Task 4. Develop a communications strategy.
Task 5. Economic and safety impact study.
Task 6. Software development and integration

The sections that follow address each of the implementation tasks to be completed toward widespread adoption of the RBI technology.

4.2.1.2 Task 1. Establishment of an Oversight Committee

An important element of the longer-term implementation of RBI practices will be the establishment of a committee structure to oversee the development and maintenance of the technology. Implementation of the proposed methodology will require a significant shift in paradigm for inspection planning for highway bridges. Consequently, there will need to be a long-term commitment with respect to maintaining and implementing the new methodologies contained in the RBI Guideline. As is common with many design codes and standards, a committee is needed to oversee, maintain, and further develop the Guideline. This committee may be a subcommittee of the AASHTO's Standing Committee on Bridges, or a subcommittee of the existing T-18 committee on *Bridge Management, Evaluation and Rehabilitation*, or even the committee itself.

The committee should have the goal of providing objective oversight and management of the Guideline and requirements for RBI. Committee membership should be diversified, and include representatives from states in different geographic regions and with different types of bridge inventories. Participation of the FHWA in this committee would be desirable. During the transitional stages, which should be anticipated, the committee should include both states implementing or developing RBI processes, and states that are not yet utilizing RBI. Care should be taken to ensure the participation of the community as a whole in the committee, including both bridge owners adopting the RBI Guidelines and those who have not yet made the transition. An important aspect of the proposed methodology is transparency, and any committee overseeing progress will require critical voices to be effective.

The role of the committee should be as follows:

1. Oversee implementation of the technology across different states and act as a focal point for information interchange regarding states' experience, research, and developments.
2. Identify and recommend research and development needs to support the technology.
3. Recommend and approve changes to the RBI Guideline document.

It is envisioned that the oversight committee or subcommittee will be a long-term or even permanent organization that will serve the larger bridge community.

4.2.1.3 Task 2. Additional Case Studies

Additional case studies may be needed to test the application of the Guideline, identify implementation challenges, and provide additional data on the impact of transitioning to an RBI approach. The objectives of the case study should include:

- Assess the effect of RBI outcomes on the inspection practice for different families of bridges.
- Evaluate implementation challenges.
- Assess the repeatability and consistency of the process.
- Provide baseline data for economic and safety impact study.
- Revise the RBI guideline as needed.

The focus of these case studies will be to evaluate processes and methods described in the Guideline. To meet the objectives shown above, case studies should be conducted in several states to evaluate different families of common bridge types.

4.2.1.4 Task 3. Development of Training Modules

Implementation of the RBI process will obviously require the development of training modules for those that will be involved in the process. The inspection planning process is more involved and complex under an RBI scheme relative to a calendar-based inspection planning process. The assessment of reliability characteristics requires an understanding of the approach and the assessment needs. Therefore, training for both members of the RAP and for inspectors that will implement the results of the RBI planning process may be necessary. This section provides an overview of the training needs for implementing RBI practices.

4.2.1.4.1 Training for RAP Member. The development of an expert panel like the RAP is relatively new to the industry, and will require a substantial commitment from stakeholders to identify and train individuals to participate in the process. Individuals that can provide expertise objectively are needed, and training in the tools and mechanism for providing that objective expertise will be required to ensure the methodology is effectively implemented. Although these panels must be objective, they should also collectively possess an intimate knowledge of the inventory of the given agency or DOT. This group must also be isolated from the political and management pressures that could undermine the objectivity and effectiveness of the process (e.g., pressure to use the RAP process to simply extend intervals to save money).

Effective training for RAP members will be one of the most critical parts of the implementation of RBI on a broad scale. This training should provide sufficient knowledge in the theory and underlying approach to RBI planning, deterioration and reliability science, methodologies for expert elicitation, and processes for determining the factors required for the analysis. Training modules developed during the case studies that were conducted as part of the research were shown to be effective, based on the results of the case studies, and they provide a strong foundation for the development of more formal training for widespread implementation of the technology.

4.2.1.4.2 Training of Inspectors. Implementation of RBI practices will require training for bridge inspectors to develop the necessary understanding of the RBI process. RBI assessments for inspection planning provide a prioritization of inspection needs for a bridge based on the anticipated or expected damage modes and the importance of that damage in terms of safety of the bridge. Criteria developed through the RAP process identify key condition attributes used to determine the reliability of individual elements of the bridge and related criteria for reassessment of the inspection interval and scope. Additionally, the IPN identified through the RAP analysis prioritizes damage modes for inspection in a manner that is significantly different than the traditional, "detect all the defects" approach. It is critical that the underlying approach and methodologies used in the planning process are understood by the inspectors implementing the practices to ensure adequate inspection in the field to support the overall process. Training of this type was not addressed during the course of the research.

Significant resources for inspector training already exist, and are generally implemented through the National Highway Institute (NHI). Existing training modules will need to be adjusted to accommodate the focus on damage and damage precursors that are a part of the RBI process. Inspector training modules like the 2-week inspector training course and supporting *Bridge Inspectors Reference Manual* will need to be modified to be more focused to incorporate the perspective of the RBI process and its approach to ensuring the safety and reliability of bridges.

Entirely new training modules could be developed to support the RBI Guideline. However, while the approach to inspection planning and the required reporting and inspection results differ for RBI, the damage modes that affect bridges are typically well-covered in the existing training modules. Developing entirely new training for RBI including all of the information and examples already included in the existing modules would be a duplication of effort that is likely unnecessary. However, there are certain subjects not currently included in the existing training modules that will be required to effectively implement RBI. Table 26 describes two training modules that may be necessary for implementing RBI. This training for inspectors describes the underlying concepts and methodologies for RBI. It is intended that the training modules be presented at the appropriate technical level to develop sufficient background knowledge for an inspector that will implement the RBI process. Ultimately, these specific modules may be added to the existing curriculum to provide training continuity and avoid duplication.

The training modules included in Table 26 are intended to include enhanced training in specific inspection methodologies required for implementing RBI, including those identified in the RBI Guideline. For example, increased training for visual inspection to detect fatigue cracking, appropriate lighting and distance requirements, and thoroughness of inspection should be addressed through the training. Implementation of the other basic techniques, such as sounding or concrete, should also be included in the training.

More advanced technologies, such as advanced NDE techniques that may be specified for certain damage modes, will also require training. For example, if the RAP identifies the use of infrared thermography as a means of assessing delaminations in concrete, then specialized training in applying this technology will be needed. Training in advanced NDE technologies is typically advanced and focused, and utilization of the technology is specialized in nature. Specific training in these technologies should be developed as needed to meet specific owner needs.

Table 26. Outline of training for inspectors.

Topics	Notes
Module I Background	
Deterioration mechanism for bridges	Overview of typical deterioration patterns
Fundamentals of reliability theory and application to inspection	Background overview of the underlying theories for RBI, reliability matrices, and likelihood
Reliability assessments for RBI	RAP process and basis for inspection procedures
Module II Practices	
Understanding the IPN	Required thoroughness of inspection and prioritization of damage modes
Inspection needs, criteria, and reporting	Focus and scope of inspections for RBI, access requirements, reassessment criteria, documentation, and reporting requirements
Enhanced inspection methods for RBI	Technologies and methods for detecting identified damage modes, enhanced methods for RBI, sounding and crack detection

4.2.1.5 Task 4. Develop a Communications Strategy

As discussed, the proposed method is a significant change in paradigm for the bridge inspection community, and as such developing an effective communications strategy will be a key element of overall success. Education of policy makers and DOT administrators as to the benefits of the proposed methodologies will be needed. Although it is anticipated that owners (i.e., state bridge engineers) will likely embrace the proposed methodologies, it will require the buy-in of policy makers to actually implement any changes to the bridge inspection program. This group includes DOT administrators as well as appointed or elected officials. Since the current inspection program is covered by the Code of Federal Regulations (CFRs), lack of approval of policy makers could restrict any proposed methods from being implemented. Constant communication with the FHWA regarding the methodology and its progress will be very important to providing decision makers the information necessary to support future changes that may be needed. Additional interactions with state and local government rules, and potential conflicts with other specifications will also need to be assessed.

There also exists the challenge that policy makers may have difficulty separating *gross numbers* of inspections from *quality and effectiveness of inspection.* Numbers of inspections are frequently equated to safety, even though these two factors may be unrelated, particularly when the method of inspection is ineffective for a particular damage mode or deterioration mechanism. There will be a need for clear explanation of the approach to achieve buy-in from the policy makers, and even then challenges should be expected.

There also exist the potential that the cost reallocations will be misinterpreted as a reduction in inspection requirements to save money, rather than a reallocation of resources to be most effective in ensuring bridge safety. If viewed as a cost saving measure, the practice could lead to reduction in available resources for inspection, which is undesirable. Care needs to be taken to illustrate the enhanced reliability realized through allocating resources more effectively, and the benefits in terms of supporting the inspection and repair needs of an aging bridge inventory. Public and political acceptance of a system that may result in fewer inspections will rely on clearly communicating the benefits (more in-depth and focused inspections), not any cost saving.

The communication strategy developed should include the development of non-technical publications that describe the RBI approach and highlight the benefits such as increased resources to focus where most needed and reductions in the risks and costs associated with unnecessary inspections. The improved reliability and safety of bridges that can be realized through improved inspection practices should be described as well as the improved management and responsible utilization of public funds highlighted in these publications.

Technology transfer to the broader engineering community should also be developed as part of the communications strategy.

4.2.1.6 Task 5. Assessment of the Economic and Safety Impacts of RBI

A key element in pursuing widespread implementation and acceptance of the RBI technology will be a critical assessment of the economic and safety impacts of converting from a uniform, calendar-based system to the RBI methodology. Such a study will likely be a required component of gaining the support of AASHTO and policy makers, who would naturally question the cost and safety impact of such a transition.

Because inspection resources are reallocated and optimized under the RBI process, an organized and systematic assessment of the effect of the process on bridge safety will be required. This study should examine both the benefits of increased inspection thoroughness and assess any real or potential diminishment of safety or safety effects on a given bridge inventory associated with varying inspection intervals to match the needs for bridges. The study should also examine the safety effects of continuing the current status quo, addressing both the cost and safety implications of the "do-nothing" approach, including the effects of decreasing available resources for inspection. Data from the case studies can be used to assist in this assessment process for this study.

Study of the economic impact of transitioning to RBI is also needed. Because the methodology requires the investment of increased resources for the planning of inspections, bridge owners may need to restructure traditional responsibilities and staffing to address the needs of full implementation of the technology. Increased engineering efforts are required to complete RAP analysis, particularly in contrast to uniform, calendar-based approaches. Inspections under RBI Guidelines typically have increased scope and increased access requirements relative to traditional routine inspections, and as such are likely to have increased costs. On the other hand, the inspection may be less frequent, such that the overall costs may be unaffected. The economic implications for transitioning to RBI obviously will vary according to the current inspection practice currently used in a state, and additional information on the actual or estimated economic impacts will be needed.

This study of the economic and safety impacts of transitioning to RBI practices will likely be a key tool to the eventual political and policy acceptance of the new technology. This implementation activity is high priority as a means of addressing the issues associated with achieving acceptance of the new technology among the public, with policymakers, and with stakeholders.

4.2.1.7 Task 6. Software Development and Integration

An important element of widespread implementation and acceptance of the new technology will be the development of software tailored to meet the needs of the process, and intended to integrate current or future data collection and storage approaches used by bridge owners. The processes for assessing the OFs, such as identifying and scoring key attributes of bridges, can be repetitive once established, and therefore lend themselves to software implementations. Many of the attributes identified through the analysis process may already be stored in existing databases and bridge management systems. Condition attributes and screening criteria may be implemented through existing software developed for bridge inspection and storing bridge inspection data, or in new software developed with RBI in mind. Such software is widespread in other industries and used for risk assessment and condition-based maintenance of facilities and components. The process of implementing a RBI practice can be simplified by the development of software to more rapidly utilize the methodology. Integration with existing software and databases that store relevant information will be beneficial for efficiency in implementation. The case studies conducted as part of the research reported herein developed some basic software tools for these purposes; these tools will need to be integrated into existing software. Developing software to assist in the RBI process will be necessary for implementation efforts to be successful.

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Abbreviations

ADT	Average Daily Traffic
ADTT	Average Daily Truck Traffic
BME	Bridge Management Element
BMS	Bridge Management Software
CF	Consequence Factor
CFR	Code of Federal Regulation
CIF	Constraint-Induced Fracture
CS	Condition State
CVN	Charpy V-Notch
DOT	Department of Transportation
EMAT	Electromagnetic-Acoustic Transducer
GPR	Ground Penetrating Radar
HPC	High Performance Concrete
HPS	High Performance Steel
HS	High Strength
IE	Impact Echo
IPN	Inspection Priority Number
IR	Infrared Thermography
LFD	Load Factor Design
LIBS	Laser-Induced Breakdown Spectroscopy
MT	Magnetic Particle Testing
NBE	National Bridge Elements
NBI	National Bridge Inventory
NBIS	National Bridge Inspection Standards
NDE	Nondestructive Evaluation
NHI	National Highway Institute
OF	Occurrence Factor
PCI	Precast/Prestressed Concrete Institute
PDF	Probability Density Function
POD	Probability of Detection
POF	Probability of Failure
PT	Dye Penetrant Testing
QA	Quality Assurance
QC	Quality Control
RAP	Reliability Assessment Panel
RBI	Risk-Based Inspection
SIP	Stay-in-Place
SI&A	Structural Inventory and Appraisal
TRL	Technical Readiness Level
UPV	Ultrasonic Pulse Velocity
UT-T	Ultrasonic Thickness Gauge

APPENDIX A

Developing Reliability-Based Inspection Practices: Oregon Pre-Stressed Bridges

Bridge/Deck/Spalling

Similar Items in Guideline	Attributes	Type of Attributes	High	Medium	Low	Remote	Screening	Degree of Severity	Max Score	Source of data
C.9 and C.12	Cracking/Spalling	Condition	#358 CS 4 Or #359 CS 4 or CS 5	#358 CS 3 Or #359 CS 3	#358 CS 2 Or #359 CS 2	#358 CS 1 Or #359 CS 1		M	15	#358 Deck Cracking Smart Flag #359 Soffit Cracking Smart Flag (Oregon Coding Guide Pages 79 and 80)
C.10 and C.11	Delamination/Patch	Condition	>25% CS 4 or CS 5	11%-24% CS 3	<10% CS 2	CS 1		H	20	Concrete Decks and Slabs without an Overlay : #12 - #26 -#27 -#38 - #52 -#53 Concrete Decks or Slabs with a Thin or Rigid Overlay: #18 - #22- #44 - #48.
L.1	ADTT	Loading	>5000	501-4999	<500			M	15	Item 29 NBI
L.3 (Exposure Environment)	Location /Environment	Loading	Coastal and Mountain	Valley (general environment)	Desert			H	20	Bridge File
D.6 (Year Built)	Age	Design	>50 years	10-49 years	<10 years			H	20	Item 27 NBI (Year Built)
L.2	Dynamic Loading	Loading	>40 mph +CS 3	<40mph + CS 2 or CS 3 + <40mph	CS 2 + <40mph	CS 1		H	20	# 325 (Oregon Coding Guide Page 22)
C.21 and C.13	Rebar Corrosion	Condition	Rust/Black/Low Cover		No stains, Epoxy/high cover			H	20	Concrete Elements(Oregon Coding Guide Page 38-41)
L.5	De-icing	Loading	High (Regions like Portland, Bend, Salem, La Grand)		Low (All Other Regions)			M	15	Items 3, 4, and 5 NBI, or Geographical map

Bridge/Deck/Rutting

Similar Items in Guideline	Attributes	Type of Attributes	High	Medium	Low	Remote	Screening	Degree of Severity	Max Score	Source of data
L.1	ADT	Loading	>15000 vpd		1000-14999 vpd	<1000vpd		H	20	Item 29 NBI
D.10	Wearing Surface Type	Design	-	AC	Bare Concrete/STR overlay/Epoxy	Open Grid		M	15	Item 108A NBI (Also page 120 Oregon Coding Guide)
L.3 (potential to be exposed to high ADT with studded tires)	Location	Loading	-	I-5 highway Portland to Salem and I-84 Portland	All other locations	-		H	20	Items 3, 4, and 5 NBI, or Geographical map
C.2	Current Condition (amount of rutting)	Condition	-		Present (>0.5")	None (<0.5")		4 H & 4 M (M) +	15	(Oregon Coding Guide Pages 22 & 23)

Bridge/Deck/Cracking (Non-Corrosion Induced)

Similar Items in Guideline	Attributes	Type of Attributes	High	Medium	Low	Remote	Screening	Degree of Severity	Max Score	Source of data
C.9	Cracking	Condition	Unsealed cracks exist in the deck that are of severe size (>0.060 in. wide) and/or density (<3' apart)	Unsealed cracks exist in the deck that are of moderate size (0.025 to 0.060 in. wide) and density (3' to 10' apart).	Unsealed cracks exist in the deck that are of moderate size (0.025 to 0.060 in. wide) or density (3' to 10' apart).	The surface of the deck is cracked, but the cracks are either filled/sealed or insignificant in size and density to warrant repair activities.		H	20	#358 Deck Cracking Smart Flag (Oregon Coding Guide Page 79)
D.18	Skew	Design	>30 ^{o}		<30^{o}			M	15	Item 34 NBI
L.1	ADTT	Loading	>5000	501-4999	<500			H	20	Item 109 NBI
D.20	Thickness	Design	<7"		>7"			H	20	Bridge File
L.2	Profile/ Dynamic Loading	Loading	>40 mph + CS 3	<40mph + CS 2 or CS 3 + <40mph	CS 2 + <40mph	CS 1		H	20	Item 325 (Oregon Coding Guide Page 22)
S.10	Span Type	Screening					Continuous or Non Continuous			

Bridge/Superstructure/Cracking (Shear)

Similar Items in Guideline	Attributes	Type of Attributes	High	Medium	Low	Remote	Screening	Degree of Severity	Max Score	Source of data
L.4 and D.2	Overload	Loading	If it has already posted for less than legal load or exposed to overload			Other		H	20	Item 41 NBI (See also Oregon Coding Guide on page 95)
D.18	Skew	Design		>30	<30			L	10	Item 34 NBI
D.6 (Year of Construction)	Age	Design		<2000	>2000			L	10	Item 27 NBI (Year Built)
D.20	AASHTO Shear Design	Screening	AASHTO requirements were not considered in design			AASHTO requirements were considered in design				

Bridge/Superstructure/Strand Corrosion

Similar Items in Guideline	Attributes	Type of Attributes	High	Medium	Low	Remote	Screening	Degree of Severity	Max Score	Source of data
L.3 (Exposure Environment)	ENV	Loading	Coastal and Mountain	Valley (general environment)	Desert			H	20	Geographical Map
C.8 (Corrosion Induced Cracking)	Existing Damage	Condition	CS 4	CS 3	CS 2	CS 1		H	20	Prestressed/Post Tensioned Concrete Elements (Oregon Coding Guide Page 40)
C.1	Current Condition	Condition	5 and less	6	7 or greater			H	20	Item 59 NBI (See also page 42 and 104 Oregon Coding Guide)
D.11 (Minimum Concrete Cover)	Cover	Design	1.5" or Less, Unknown	between 1.5" and 2.5"	Greater than or equal 2.5"			H	20	Bridge File
D.12 (Reinforcement Type)	Strand Type	Design	Uncoated			Epoxy coated		L	10	Bridge File
D.20 and S.10	Bad End Detail	Design	Has Strand Exposure to outside environment	Unknown	Do not have Exposure to outside environment			L	10	Bridge File

Bridge/Superstructure/Fire Damage

Reason(s) for Attribute

Incidences of fire on or below a highway bridge are not uncommon. This type of damage is most frequently caused by vehicular accidents that result in fire, but secondary causes such as vandalism, terrorism, or other damage initiators should not be discounted. If fire does occur on or below a bridge, an appropriate follow-up assessment should be conducted to determine how the fire has affected the load carrying capacity and the durability characteristics of the main structural members and the deck. This assessment is typically performed during a damage inspection immediately following the incident.

Damage to bridge components resulting from a fire is either immediately apparent during the damage inspection, or may manifest within the first 12-to-24 month interval following the fire. Based on this observation, bridges that have experienced a fire may be screened from the reliability assessment until an inspection, which has been conducted approximately 12 months or more after the fire, confirms that the fire has not affected the typical durability characteristics of the bridge components. The purpose of this screening is to ensure that damage from the fire has not manifested after the damage inspection.

Assessment Procedure

This attribute is scored based only on the occurrence of a fire on or below the structure being assessed. It is assumed that an appropriate assessment immediately following the fire incident (i.e., damage inspection) has been performed.

Fire incident has occurred and an inspection 12 months after the fire has not occurred	Bridge is not eligible for reliability assessment until inspection confirms that the bridge is undamaged
There have been no incidences of fire on or below the bridge, or inspections conducted approximately 12 months or more after the fire have confirmed that the bridge is undamaged	Continue with procedure

Bridge/Superstructure/Impact

Similar Items in Guideline	Attributes	Type of Attributes	High	Medium	Low	Remote	Screening	Degree of Severity	Max Score	Source of data
D.3	Clearance	Design	<15'	15-16	>17'			L	10	Item 10 NBI (Minimum vertical under clearance)
L.8	High Water	Screening					Look at item 71 in NBI database- if the code is 3 the chance of over top is occasional			Item 71 in NBI database (See also page 117 Oregon Coding Guide)

Bridge/Superstructure/Rebar Corrosion within the Span

Similar Items in Guideline	Attributes	Type of Attributes	High	Medium	Low	Remote	Screening	Degree of Severity	Max Score	Source of data
L.3 (Exposure Environment)	ENV	Loading	Coastal and Mountain	Valley (general environme nt)	Desert			H	20	Items 3, 4, and 5 NBI, or Geographical map
C.6 and C.21 Previously Impacted Active Corrosion	Existing Damage	Condition	#362 CS 2 Prestressed/ Post Tensioned Concrete Elements CS 4	Prestressed /Post Tensioned Concrete Elements CS 3	#362 CS 1 Prestressed /Post Tensioned Concrete Elements CS 2	Prestressed /Post Tensioned Concrete Elements CS 1	CS 3	H	20	#362-Traffic Impact Smart Flag (page 83 Oregon Coding Guide) Prestressed/Post Tensioned Concrete Elements (Oregon Coding Guide Page 40)
D.11 (Minimum Concrete Cover)	Cover	Design	1.5" or Less, Unknown	Between 1.5" and 2.5"	Greater than or equal 2.5"			H	20	Bridge File or Cover meter
D.12 (Reinforcement Type)	Strand Type	Design	Uncoated			Epoxy Coated		H	20	Bridge File

Bridge/Superstructure/Bearing Seat Problem(s)

Similar Items in Guideline	Attributes	Type of Attributes	High	Medium	Low	Remote	Screening	Degree of Severity	Max Score	Source of data
C.21	Corrosion	Condition	CS 4	CS 3	CS 2	CS 1		H+	20	Prestressed/Post Tensioned Concrete Elements (Oregon Coding Guide Page 40)
D.18	Skew	Design	>30 ^{o}		<30 ^{o}			L	10	Item 34 NBI
C.22	Debris	Condition	-	Flood region Debris INS.RPT	Not Susceptible			L	10	Item 113 NBI (See also Oregon Coding Guide on page 121)
L.4	Overload	Loading	If it has already posted for less than legal load or exposed to overload		other			L+	10	Item 41 NBI (See also Oregon Coding Guide on page 95)
S.10	Design Details	Design	Simple Support	Continuous Support	Integral Abutments			M	15	Bridge File
C.4	Failed Joints	Condition	CS 3	CS 2	CS 1	Joint-less		H	20	Deck Joints—Oregon Coding Guide Page 54-60
C.2	Existing Damage	Condition	CS 3	CS 2	CS 1			H	20	Bridge Bearing Elements—Oregon Coding Guide Page 61-66

Bridge/Substructure/Settlement

Similar Items in Guideline	Attributes	Type of Attributes	High	Medium	Low	Remo te	Screening	Degree of Severity	Max Score	Source of data
D.21	Footing Type	Design	Spread FTG on soil/unknown Foundation	-	Drill Shaft friction Pile /ETC		If foundation was based on Rock/Piles we do not need to deal with other following attributes	H	20	Bridge File
D.22	Subsurface Condition	Condition	Slide zone, clay, silt, shale, gravel	Limestone	solid, Rock			H	20	Bridge File
C.3	Existing Settlement	Condition	Active (No monitor data)	Occurred but arrested	None			H	20	Item #360 on page 81 Oregon Coding Guide
S.10	Scour Rating	Screening	4-6 (Oregon Scour Code)	-	>7		<3			Item 113 NBI (See also Oregon Coding Guide on page 124)

Bridge/Substructure/Corrosion Damages (Spalling/Delamination/Cracking/Rust)

Similar Items in Guideline	Attributes	Type of Attributes	High	Medium	Low	Remote	Screening	Degree of Severity	Max Score	Source of data
L.3 (Exposure Environment)	ENV	Loading	Coastal and Mountain	Valley (general environment)	Desert			H	20	Geographical Map
C.8 (Corrosion Induced Cracking)	Existing Damage	Condition	CS 4	CS 3	CS 2	CS 1		H	20	Prestressed/Post Tensioned Concrete Elements (Oregon Coding Guide Page 40)
C.1	Current Condition	Condition	5 and less	6	7 or greater			H	20	Item 59 NBI (See also page 42 and 104 Oregon Coding Guide)
D.11 (Minimum Concrete Cover)	Cover	Design	1.5" or Less, Unknown	Between 1.5" and 2.5"	Greater than or equal 2.5"			H	20	Bridge File
D.12 (Reinforcement Type)	Rebar Type	Design	Uncoated			Epoxy coated		L	10	Bridge File
C.4	Failed Joints	Condition	CS 3	CS 2	CS 1	Joint less		H	20	Deck Joints—Oregon Coding Guide Page 54-60
L.5	De-icing	Loading	High (Regions like Portland, Bend, Salem, La Grand)		Low (All Other Regions)			M	15	Items 3, 4, and 5 NBI, or Geographical map

APPENDIX B

Texas Steel Bridge Attributes Summary

Bridge/Deck/Spalling

Similar Items in Guideline	Attributes	Type of Attributes	High	Medium	Low	Remote	Screening	Degree of Severity	Max Score	Source of data
D.11	Clear Cover	Design	<1"	1"-2"	>2"			H	20	Bridge file or Covermeter
D.10	Overlay	Design	Yes		No			L	10	Item 108A NBI (See also pages 9 and 10 Texas Coding Guide)
C.10	Delamination	Condition	Yes		No			H	20	Pages 5 and 8 Texas Coding Guide
D.8	Mixed design (Water)	Design	Poor Mix/Poor H2O		All Else			M	15	Bridge file
L.1	ADTT	Loading	>5000		<5000			L	10	Item 29 & 109 NBI
D.20	Thickness	Design	<7"	7"-8"	>8"			M	15	Bridge file
D.19	Cold Joints	Design	Yes		No			M	15	Bridge file (or observation)
C.9	Cracking (map dense)	Condition	Yes		No			M	15	Pages 30 and 31 Texas Coding Guide
L.3	Environment	Loading	Above I-20		All Else			H	20	Bridge file
D.6	Age Years of Services	Condition	50+		Other			M	15	Item 27 NBI (Year Built)

Bridge/Deck/Punch Through

Similar Items in Guideline	Attributes	Type of Attributes	High	Medium	Low	Remote	Screening	Degree of Severity	Max Score	Source of data
D.20	Thickness	Design	<7"	7"-8"	>8"			H	20	Bridge file
C.9	Map Cracking	Condition	Yes		No			H	20	Pages 30 Texas Coding Guide (Deck Cracking)
C.10 and C.12	Delamination / spall to rebar	Condition or Screening if more than 10%	Delamination and spalling >6%	Delamination and spalling 2%-5%	Delamination and spalling <1%			M	15	Pages 5 and 8 Texas Coding Guide
D.8	Poor Concrete Mix (Poor Water)	Screening						M	15	Bridge file
L.1	ADTT	Loading	>5000		<5000			H	20	Item 29 NBI
L.3	Environment	Loading	Above I-20		All Else			L	10	Bridge file (PONTIS Report)
	Previous Punch outs /rep	Screening/Yes or No								Pages 5 and 8 Texas Coding Guide

Bridge/Deck/Cracking

Similar Items in Guideline	Attributes	Type of Attributes	High	Medium	Low	Remote	Screening	Degree of Severity	Max Score	Source of data
C.9	Existing Cracking	Condition	Yes		No			H	20	Page 30 Texas Coding Guide (Deck Cracking)
D.20	Construction Tech/Spec	Design	Bad		All Other			M	15	Bridge file
L.3	Environment	Loading	Above I-20		All Else			H	20	Bridge file
D.18 and D.19	Design Details (Cold Joints, Skew)	Design	Yes		None			H	20	Bridge file
D.11	Cover	Design	<1"	1"-2"	>2"			H	20	Bridge file or covermeter

Bridge/Deck/Delamination

Similar Items in Guideline	Attributes	Type of Attributes	High	Medium	Low	Remote	Screening	Degree of Severity	Max Score	Source of data
D.11	Clear Cover	Design	<1"	1"-2"	>2"			H	20	Bridge file or covermeter
D.10	Overlay	Design	Yes		No			L	10	Item 108A NBI (See also pages 9 and 10 Texas Coding Guide)
C.12	Spalling	Condition	>6%	2%-5%	<1%			H	20	Pages 5 and 8 Texas Coding Guide
C.10	Delamination	Condition	Yes		No		If more than 10%	H	20	Pages 5 and 8 Texas Coding Guide
D.8	Mixed design (Water)	Design	Poor Mix/Poor H2O		All Else			M	15	Bridge file
L.1	ADTT	Loading	>5000		<5000			L	10	Item 29 NBI
D.20	Thickness	Design	<7"	7"-8"	>8"			M	15	Bridge file
D.19	Cold Joints	Design	Yes		No			M	15	Bridge file or Observation
C.9	Cracking (map dense)	Condition	Yes		No			M	15	Page 30 Texas Coding Guide (Deck Cracking)
L.3	Environment	Loading	Above I-20		All Else			H	20	Bridge file
D.6	Age Years of Services	Condition	50+		Other			M	15	Item 27 NBI (Year Built)

Bridge/Superstructures/Sectionless

Similar Items in Guideline	Attributes	Type of Attributes	High	Medium	Low	Remote	Screening	Degree of Severity	Max Score	Source of data
L.3	Environment	Loading	Coast	North of I-20	All Else			H	20	Bridge file
S.9	Existing Section Loss	Condition	Yes		No			H	20	Pages 10, 11, and 15 Texas Coding Guide
D.18	Deck Drainage onto Superstructure	Design	Yes		No			L	10	Bridge file
C.22	Debris	Condition	Yes		No			L	10	Pages 23, 25, and 30 Texas Coding Guide (Pack Rust)
C.4	Joint Leakage	Condition	Yes		No			L	10	Pages 23 and 24 Texas Coding Guide
D.13	Built-Up Riveted	Design	Yes		No			H	20	Bridge file
D.19	Deck Cold Joints	Design	Yes		No			M	15	Bridge file or Observation
D.6	Age Exposure	Design	50+		Other			L	10	Item 27 NBI (Year Built)
C.21	Corrosion	Condition	CR 3 or Greater/No Coating or Weather Steel		Else			L	10	Pages 10, 11, and 15 Texas Coding Guide

Bridge/Superstructures/Impact

Similar Items in Guideline	Attributes	Type of Attributes	High	Medium	Low	Remote	Screening	Degree of Severity	Max Score	Source of data
C.6	Existing Impacts	Condition	Yes		No			H	20	Pages 33 and 34 Texas Coding Guide
D.3	Codes For Under clear (Vehicle)	Design	<=15'-6"	17'-6"<H<=15'-6"	17'-6"< Or No Highway under the bridge			H	20	Item 54B NBI (Minimum Vertical Under Clearance)

Bridge/Superstructures/Fatigue Cracking

Similar Items in Guideline	Attributes	Type of Attributes	High	Medium	Low	Remote	Screening	Degree of Severity	Max Score	Source of data
D.17	Detail Category	Design	E0 or E'	D or Unknown	C or Better			M	15	Bridge file or Observation
C.18	History of Previous Cracking that was repaired	Condition	Yes		No			M	15	Page 30 Texas Coding Guide (Steel Fatigue)
D.6	Year built	Design	Before 1975 or Unknown	1976-1984	After 1985			H	20	Item 27 NBI (Year Built)
D.18	Skew Angle	Design	>30		<30			L	10	Item 34 NBI
L.1	ADTT	Loading	>5000		<5000			H	20	Item 29 NBI
S.7, C.19, and C.20	Active or unmitigated cracking due to any cause	Screening		Repair Must be shown to be working						Pages 30 Texas Coding Guide Or Observation

Bridge/Superstructures/Fire Damage

Reason(s) for Attribute

Incidences of fire on or below a highway bridge are not uncommon. This type of damage is most frequently caused by vehicular accidents that result in fire, but secondary causes such as vandalism, terrorism, or other damage initiators should not be discounted. If fire does occur on or below a bridge, an appropriate follow-up assessment should be conducted to determine how the fire has affected the load carrying capacity and the durability characteristics of the main structural members and the deck. This assessment is typically performed during a damage inspection immediately following the incident.

Damage to bridge components resulting from a fire is either immediately apparent during the damage inspection, or may manifest within the first 12- to 24-month interval following the fire. Based on this observation, bridges that have experienced a fire may be screened from the reliability assessment until an inspection, which has been conducted approximately 12 months or more after the fire, confirms that the fire has not affected the typical durability characteristics of the bridge components. The purpose of this screening is to ensure that damage from the fire has not manifested after the damage inspection.

Assessment Procedure

This attribute is scored based only on the occurrence of a fire on or below the structure being assessed. It is assumed that an appropriate assessment immediately following the fire incident (i.e., damage inspection) has been performed.

Fire incident has occurred and an inspection 12 months after the fire has not occurred	Bridge is not eligible for reliability assessment until inspection confirms that the bridge is undamaged
There have been no incidences of fire on or below the bridge, or inspections conducted approximately 12 months or more after the fire have confirmed that the bridge is undamaged	Continue with procedure

Bridge/Superstructures/Deflection Overload

Similar Items in Guideline	Attributes	Type of Attributes	High	Medium	Low	Remote	Screening	Degree of Severity	Max Score	Source of data
D.2	Load Posting	Condition	Cond Posting	Des Post	None			H	20	Item 41 NBI
--	Previous* Overload Damage	Condition	Yes		No			H	20	Bridge file
--	Highway Ownership	Condition	Local		State			M	15	Item 22 NBI

*Overload damages manifest in forms of settlement, rotation, and cracks.

Bridge/Substructures/Corrosion Damages (Spalling/Delamination/Cracking/Rust)

Similar Items in Guideline	Attributes	Type of Attributes	High	Medium	Low	Remote	Screening	Degree of Severity	Max Score	Source of data
L.3	ENV	Loading	Above I-20		All else			H	20	Geographical Map
C.8 (Corrosion Induced Cracking)	Existing Damage	Condition	CS 4	CS 3	CS 2	CS 1	CS 4	H	20	(Texas Coding Guide Page 16–20)
C.1	Current Condition	Condition	5 or less	6	7 or greater			H	20	Item 60 NBI
D.11	Cover	Design	1.5" or Less, Unknown	between 1.5" and 2.5"	Greater than or equal 2.5"			H	20	Bridge File
D.12	Rebar Type	Design	Uncoated			Epoxy coated		L	10	Bridge File
C.4	Joints Condition	Condition	5 or less	6	7 or greater	Joint less		H	20	Joints Condition – Item 58 NBI details in bridge file or items #300 to #304 –Texas Coding Guide pages 23-24

Bridge/Substructures/Settlement

Similar Items in Guideline	Attributes	Type of Attributes	High	Medium	Low	Remote	Screening	Degree of Severity	Max Score	Source of data
D.21	Footing Type	Design	Spread FTG on soil/unknown foundation	-	Drill shaft friction pile /etc		If foundation was based on Rock/Piles we do not need to deal with other following attributes	H	20	Bridge File
D.22	Subsurface Condition	Condition	Slide zone, clay, silt, shale, gravel	Limestone	Solid, rock			H	20	Bridge File
C.3	Existing Settlement	Condition	Active (no monitor data)	Occurred but arrested	None			H	20	Item #405 Texas Coding Guide on page 31
S.10	Scour Rating	Screening	4-6	-	>7 Or "N"		<3			Item 113 NBI (See also item #407 on Texas Coding Guide on page 32)

APPENDIX C

Controlling Damage Modes for Sample Bridges

211 Table C1. Controlling damage modes for RBI analysis of bridges in Oregon (CF Case 4).

212 Table C2. Controlling damage modes for RBI analysis of bridges in Texas (CF Case 3).

Table C1. Controlling damage modes for RBI analysis of bridges in Oregon (CF Case 4).

Bridge ID	Inspection Interval Based on Case 4	Controlling Damage Mode
02376B	48	Rutting in deck and corrosion in substructure
07801A	24	Shear cracking, strand corrosion, and rebar corrosion within the span in superstructure
01741B	24	Corrosion related damage modes in superstructure and substructure
07935A	48	Rutting in deck, shear cracking, strand corrosion, and bearing seat problems in superstructure
07935B	48	Rutting in deck, shear cracking, strand corrosion, and bearing seat problems in superstructure
17451	24	Spalling in deck
16454	24	Rutting in deck, shear cracking, strand corrosion, and rebar corrosion within the span in superstructure
16453	48	Strand corrosion
9546	24	Strand corrosion and rebar corrosion within the span
00988A	48	Shear cracking in superstructure and settlement and corrosion in substructure
01056A	24	Most corrosion related damage modes
9358	24	Strand corrosion and rebar corrosion within the span
16873	72	All damage modes equal
18175	48	Most damage modes in superstructure
01895A	48	Rebar corrosion within the span for superstructure and settlement in substructure
9915	24	Strand corrosion and rebar corrosion within the span
8994	24	Rebar corrosion within the span
8896	48	Cracking in deck
20666	72	All damage modes equal
19739	48	Rutting in deck, corrosion related damage modes in superstructure, and settlement in substructure
19738	48	Rutting and spalling in deck, corrosion related damage modes in superstructure, and settlement in substructure
19284	48	Settlement in substructure

Table C2. Controlling damage modes for RBI analysis of bridges in Texas (CF Case 3).

Bridge ID	Inspection Interval Based on Case 3	Controlling Damage Mode
01-139-0-0769-01-007	24	Corrosion in substructure
02-127-0-0014-03-194	24	Fatigue cracking
02-127-0-0094-04-057	72	All damage modes equal
02-220-0-1068-02-058	24	Cracking in deck—section loss, impact
05-152-0-0067-11-188	24	Fatigue cracking
08-030-0-AA01-31-001	48	Deck damages and substructure
12-085-0-1911-01-003	48	Section loss and corrosion in substructure
12-102-0-0027-13-195	48	All damage modes equal
12-102-0-0500-03-320	48	All damage modes equal
15-015-0-0025-02-162	48	All damage modes equal
15-015-0-B064-55-001	72	All damage modes equal
18-057-0-0092-14-210	24	All damage modes equal (screen because of pin and hanger connection)
18-061-0-0196-01-133	24	Punch through and cracking in deck, corrosion in substructure
19-019-0-0610-06-162	24	Impact
23-141-0-0251-05-020	48	Corrosion in substructure
23-215-0-0011-07-056	48	All damage modes equal
24-072-0-0167-01-059	24	All damage modes equal

Abbreviations and acronyms used without definitions in TRB publications:

A4A	Airlines for America
AAAE	American Association of Airport Executives
AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ACI–NA	Airports Council International–North America
ACRP	Airport Cooperative Research Program
ADA	Americans with Disabilities Act
APTA	American Public Transportation Association
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
ATA	American Trucking Associations
CTAA	Community Transportation Association of America
CTBSSP	Commercial Truck and Bus Safety Synthesis Program
DHS	Department of Homeland Security
DOE	Department of Energy
EPA	Environmental Protection Agency
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FMCSA	Federal Motor Carrier Safety Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
HMCRP	Hazardous Materials Cooperative Research Program
IEEE	Institute of Electrical and Electronics Engineers
ISTEA	Intermodal Surface Transportation Efficiency Act of 1991
ITE	Institute of Transportation Engineers
MAP-21	Moving Ahead for Progress in the 21st Century Act (2012)
NASA	National Aeronautics and Space Administration
NASAO	National Association of State Aviation Officials
NCFRP	National Cooperative Freight Research Program
NCHRP	National Cooperative Highway Research Program
NHTSA	National Highway Traffic Safety Administration
NTSB	National Transportation Safety Board
PHMSA	Pipeline and Hazardous Materials Safety Administration
RITA	Research and Innovative Technology Administration
SAE	Society of Automotive Engineers
SAFETEA-LU	Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (2005)
TCRP	Transit Cooperative Research Program
TEA-21	Transportation Equity Act for the 21st Century (1998)
TRB	Transportation Research Board
TSA	Transportation Security Administration
U.S.DOT	United States Department of Transportation