RELIABILITY-BASED METHODOLOGY FOR BRIDGE INSPECTION PLANNING

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By

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The undersigned, appointed by the Dean of the Graduate School, have examined the thesis entitled

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ABSTRACT

This research program was created with the goal of improving bridge safety and reliability while also improving the allocation of bridge inspection resources. The research reported herein was completed as a part of a larger project with the objective of developing a recommended bridge inspection practice for bridges within the United States.

Traditionally, bridges in the United States are inspected at fixed time intervals of 24 months, with special programs in place to either extend or lessen this interval, based on certain conditions. This fixed inspection interval results in newer bridges, with little or no damage, being inspected with the same frequency as older, possibly more deteriorated bridges. This creates a situation where bridge inspection resources are allocated evenly across an inventory even though the inspection needs of certain bridges may be greater than others.

Through this research program, a bridge inspection planning methodology has been developed which is based on reliability theory and incorporates the knowledge and expertise of bridge owners to more rationally determine bridge inspection needs. The methodology is based on the determination of the likelihood of failure for specific bridge components based on design, loading, and condition characteristics and the perceived consequence of failure, based on an owner's expertise and experience. By combining these expressions of likelihood and consequence for each component, a maximum inspection interval for the entire bridge can be determined through the use of risk matrices.

1 INTRODUCTION

In the United States, the National Bridge Inspection Standards (NBIS) [1] mandate the frequencies and the methods used for safety inspections of highway bridges. The inspection frequencies specified in the NBIS are calendar-based and generally require routine inspections to be conducted at a maximum interval of 24 months. Special programs have been created to lessen this interval, if necessary, or to extend the interval to 48 months, if certain conditions are met. This calendar-based inspection interval, when applied uniformly across the national 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 fewer years of exposure to a service environment, may be much less likely to develop serious damage over a given time interval than an older bridge which has been exposed to a service environment for many years. As such, inspection needs may be lesser for a newer bridge, and greater for an aging bridge, relative to the uniform interval which is currently applied. Bridges that are in benign, arid operating environments are inspected at the same interval as bridges in aggressive marine environments, where damage from corrosion may develop much more rapidly, resulting in increased inspection needs.

Current practices make it difficult to recognize if the same levels or if improved levels of reliability and safety can be achieved by varying inspection frequencies and methods to meet the needs of a specific bridge, based on its design, structural condition, and operational environment. By accounting for differences in the design, condition and operating environments of bridges, inspection requirements that better meet the needs of individual bridges can be created to improve both bridge and inspection reliability.

A more rational approach to inspection planning would determine the interval and the scope of an inspection according to the specific characteristics of a bridge, and the likelihood that damage would occur over a given period of time, given those characteristics. This would allow for resources to be focused where most needed to ensure the safety and reliability of bridges. 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 their resistance to certain damage modes, their anticipated deterioration mechanisms, their current condition, and their loading to evaluate the reliability of the component and determine appropriate inspection requirements, 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 research project were to develop a reliability-based inspection practice to meet the goals of:

(1) Improving the reliability and safety of bridges and

(2) Optimizing the resources available for bridge inspection.

The objective of this project was to develop a recommended bridge inspection practice for consideration for adoption by AASHTO. The practices developed through the project were based on rational methods to ensure bridge safety, serviceability, and the effective use of resources. This report describes certain portions of the research which contributed to the creation of a bridge inspection planning methodology, based on reliability principles, which was developed during the course of this research project.

2 RESEARCH APPROACH

This report describes the research conducted as a part of the development of the draft "Guidelines for Reliability–Based Bridge Inspection Practices." The Guidelines were developed in consideration of modern industrial practices, and were 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 processes and methodologies focused on the unique needs for highway bridges. This research included identifying an effective strategy regarding development of a reliability-based bridge inspection practices for the U.S. highway transportation system. Through this investigation, a systematic process for determining the frequency and scope of highway bridge inspections based on reliability concepts was developed.

In common industrial terms, inspection practices that use reliability theories for development of inspection and maintenance strategies are sometimes termed 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. Often, 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 risk-based inspection practices was conducted to determine the most applicable methodologies for the inspection of highway bridges.

The best practices and the successful implementation of these inspection practices were reviewed, analyzed and considered by a research team which consisted of researchers from Purdue University and several key consultants. Regular teleconferences were held for discussion of findings in the literature to evaluate the methods and approaches with the most potential to be implemented for highway bridges, and to discuss and review how the findings could be incorporated into the various tasks included in the overall research program.

Information from this literature review process is included in Appendix A. This appendix provides a description of the activities undertaken to conduct the literature review as well as several key results and conclusions which were drawn from the reviewed literature. Several important aspects that were considered during the literature review ranged from the inclusion of bridge reliability in inspection planning, to reliability- and risk-based inspection methods both in civil engineering and in other industries, to the use of risk matrices for inspection planning, and to how consequences and likelihood estimates are best presented to determine inspection needs.

Following the conclusions of the literature review, 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 strategy 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 methodology was developed for reliability-based bridge inspection (RBI). In summary, the methodology developed has its foundation based on risk analysis which includes both the anticipated reliability of bridges (and their elements) and the consequences of damage to a bridge. The methodology is strongly grounded in industrial practice.

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 and loading parameters. 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 thesis. 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 validate.

Additionally, these methods are normally strength-based, and do not address the serviceability requirements that are important in terms of bridge inspection. To illustrate the importance of serviceability, serviceability is best defined as the capacity of a bridge to safely carry traffic, while the strength of a bridge is best defined as the bridge's ability to physically withstand the loads that are placed on it. If a bridge has poor serviceability characteristics, traffic will most likely still be able to cross the bridge, but the driving conditions for the vehicles may be dangerous, leading to increased rates of traffic accidents. Important serviceability requirements that should be considered include deflections (sagging), cracking, and the presence of spalling either on or below the bridge deck.

The methodology described in this thesis and in the project's Guidelines have been developed based on the well-established methods used in other industries for practical inspection planning for facilities for petroleum and power production industries, among others. Such industrial standards, which are discussed in detail in the project interim report [3] and in Appendix A, provide a technical foundation for the methodology suggested. The approach has been customized to provide a practically implementable tool that can be expanded and developed over time. This research resulted in the development of a guidelines document, which presents the tools, methodologies and requirement for RBI practices.

3 OVERVIEW

3.1 Introduction

The Guidelines developed as a part of this project described the methodology for RBI practices for highway bridges. The goal of the methodology was to improve the safety and reliability of bridges by focusing inspection efforts where most needed and optimizing the use of resources. The overall Guidelines provide 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, importance, environment, loading, prior problems, and other characteristics that contribute to the reliability and durability of highway bridges.

Generally, the methodology requires a bridge owner to perform a reliability assessment of the 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. 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 at the owner level. The expert panel, known as a Reliability Assessment Panel (RAP), conducts an engineering assessment of the likelihood of severe 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 effect the durability characteristics of bridge elements. These attributes influence the likelihood that a particular element will fail over of 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 the knowledge of bridge owners regarding the performance of the bridges within their specific operational environments given typical loading patterns, ambient environmental conditions, construction quality, etc.

It is important to note that across the United States, several different types of operating environments can be found. These differences in operation environment relate both to geography, such as temperature fluctuations between seasons, amount of annual snowfall, level of seismic activity, and to bridge population, such as the prevalence of one design type over another, load ratings, construction practices, average structure age, etc. For example, bridge owners in North Dakota may have different considerations for their bridge population than bridge owners in Texas.

Next, the reliability estimate is combined with an evaluation of the potential outcomes or consequences, in terms of safety, of damage progressing to a defined failure state. The data is then used to determine and prioritize inspection needs for specific bridges, or families of bridge with similar design and condition states. This includes determining a suitable inspection interval and the 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 it 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 severe deterioration over a specified time, typically have a longer inspection interval than a bridge with less reliable characteristics, or where 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 with a non-redundant design or other safety risks. 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.

3.2 Overview of Methodology

The RBI process involves an owner establishing a team of experts referred to as a Reliability Assessment Panel (RAP) in order to define and to assess the durability and the reliability characteristics of bridges within their state. The RAP uses engineering rationale, experience and typical deterioration patterns to evaluate the reliability characteristics of bridges and to evaluate the potential outcomes of damage. This evaluation 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, and estimate the likelihood of each damage mode causing severe deterioration in the next 72 months. The design, loading and current condition attributes of each bridge should be considered in order to categorize the likelihood of serious damage occurring into one of four Occurrence Factor categories 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 that given damage modes occur. Categorize the potential consequences into one of four Consequence Factor categories ranging from low (minor effect on serviceability) to severe (i.e. bridge collapse, loss of life).

Step 3: Determine the inspection interval and scope. Use a reliability matrix to prioritize inspection needs and then 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. A 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. This RBI procedure differs from current inspection practices generally since 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. 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 Guidelines is shown in Figure 1. The process begins with the selection of a bridge or a 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 Occurrence Factors, and the appropriate Consequence Factors associated with the potential outcomes or results damage are analyzed.

Based on the assessment of the Occurrence and Consequence Factors for the various elements of the bridge, the inspection procedure is established including the interval and scope (procedures) for the inspection, and criteria for reassessment of the inspection procedure. The criteria for reassessment are typically based on conditions that may change as a result of deterioration or damage, and may affect the Occurrence Factors 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 Occurrence Factors. If such changes exist, a re-assessment of the Occurrence Factors is completed and the inspection practice is modified accordingly.

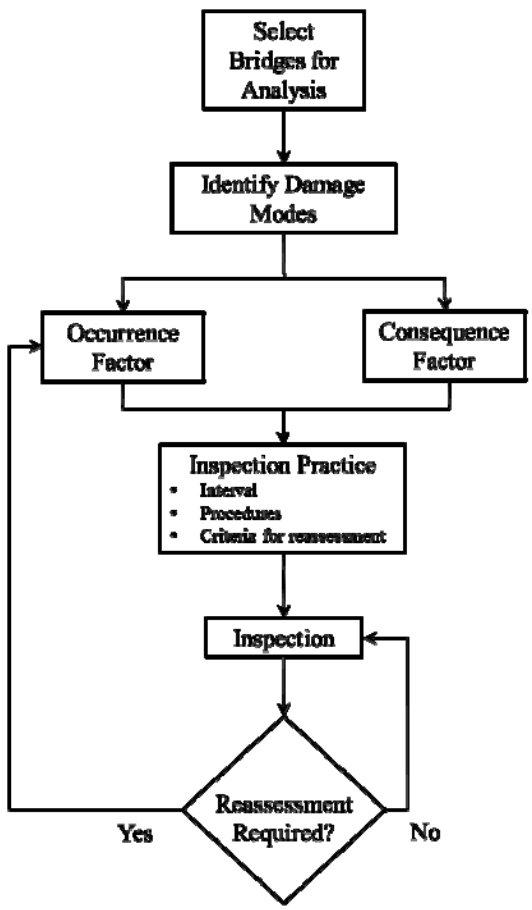

Figure 1. Schematic diagram of the RBI process.

The method of determining the inspection interval using a reliability matrix is shown schematically in Figure 2. The inspection interval is based on the RAP assessment of the Occurrence and the Consequence Factors, then plotted on a two-dimensional reliability matrix as shown in the Figure. Numerical values for the Occurrence and Consequence Factors are used to place typical damage modes in an appropriate location on the matrix. In this figure, the horizontal axis represents the Consequence Factor as determined for a particular damage mode for a given bridge element. The vertical axis represents the outcome of the Occurrence Factor assessment for a given damage mode for a 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 occur, and/or the consequences are low if they did occur, require less frequent inspections. This is simply a rational approach to focusing inspection efforts. Inspections are most beneficial when damage is likely to occur and is 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 of the bridge.

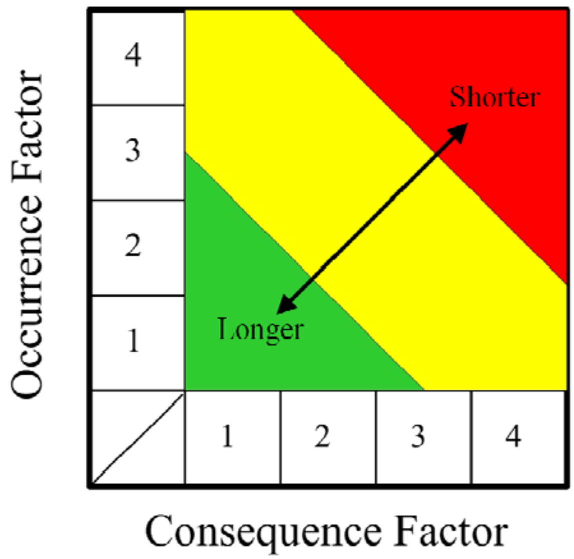

Figure 2. Reliability matrix for determining maximum inspection intervals for bridges.

Through this process, individual bridges, or families of bridges of similar design characteristics, can be assessed to evaluate the inspection from a reliability-based engineering analysis of the likelihood of serious damage occurring and the effect of that damage on the safety 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 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 important theories that underlie the proposed methodology as well as the key elements of the RBI practices for bridge inspection. Section 3.3 provides information on key background data underlying the RBI process including important reliability concepts such as probability of failure (POF), the application of reliability theory to 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 in structural design codes.

Section 3.4 will describe the important parts of the proposed inspection planning methodology. This includes descriptions of what is meant by Occurrence and Consequence Factors, how such factors should be determined, inspection procedures for RBI, how to create a Reliability Assessment Panel and conduct expert elicitation, and information on how data can be incorporated into the RBI process.

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. Reliability is defined here 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 Guidelines 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 probability of failure 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 limits 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, instead they characterize the load carrying capacity of the bridge based on engineering analysis and consider 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 estimated 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 a detailed review is included in the project interim report [3, 9-12].

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 US impractical. Finally, the results of the structural reliability assessments are based on probability of failure 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 which includes an assessment of the consequences associated with failure due to specific damage modes [13-16].

Based on the analysis of the research on reliability methods, a semi-quantitative, reliability-based framework for inspection practices was developed. 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 Guidelines for this research, 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 accumulation during the service life of the bridge. For example, corrosion of steel elements in a bridge that develops over the service life of the bridge which results 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:

$$R(t) = \mathrm{Pr}\ (T \geq t)$$

Where R(t) is the reliability as a function of time, T is the time to failure for the item, and t is the designated period of time for the item's operation, or the service life of the bridge, in this case. 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) which expresses the time to failure of an item (T) as some generic distribution, such as normal, log normal, etc. [17-19]. This distribution can be used to calculate a probability of failure 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 expressed as:

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

In other words, the reliability is the probability that the item will not fail during the time period of interest. The challenge for RBI is 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 material, design and operational environment seen across the bridge inventory.

When a large population of test data of identical or near identical components exposed to the same operational environment is 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 probability of failure (POF) which indicates the number of events (failures) expected during a given time period.

However, such test data is 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 is 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 conditions vary, and performance characteristics are ever changing.

As a result, the 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 design types, 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 where data to adequately characterize anticipated future behavior is limited, or where failure is rare, engineering judgment and experience can be used estimate the expected reliability of a specific bridge within a given operational environment [20-22]. 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 [20-23]. Such engineering judgment and knowledge provides data when quantitative data is missing, incomplete or inadequate. In this 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 period of time 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 which can lead to failure. The damage modes that typically effect bridge elements are well known in most cases, and corrosion is obviously a significant deterioration mechanism in both concrete and steel bridge elements.

The likelihood of failure occurring in some future time interval depends on the attributes of an element such as its materials of construction, local environment, 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 and 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 load and to maintain serviceability. For example, a bridge deck with severe spalling may represent a failed condition for the deck, even though the deck may have an adequate load-carrying capacity, since the ability of the deck to reliably carry traffic is compromised.

Therefore, for the case of reliability assessment for determining bridge inspection needs, it was necessary to adopt a commonly understood definition of failure that addresses the common deterioration patterns in bridges and can effectively be assessed through the inspection process. Additionally, failure must be defined in a commonly understood manner that can be readily assessed using typically available data, 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 three, "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. The subjective condition rating of three is defined within the Recording and Coding Guide [24] 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.

In terms of the AASHTO Bridge Element Inspection Guide, this condition generally aligns with elements in condition state four, "serious' [25].

These condition descriptions are widely understood and there is significant past experience in the conditions warranting a condition rating of three throughout the bridge inventory, for the myriad of different materials and design characteristics that exists.

This condition description provides a practical frame of reference for assessing 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 seven, 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 three 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 four, 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 a 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 with the extent of damage recorded in the inspection results. Spalling, cracking, scaling, sagging etc. are all examples of 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, where the deterioration mechanism is corrosion. If the deck is located in an aggressive environment, the corrosion mechanism may be fast-acting, or 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 ADTT, then the likelihood of the damage mode progressing is lower than if 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 effecting how likely debonding is to occur differ from those that effect 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 the bridge elements is to understand 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 probability of failure, as a function of the time. The bathtub curve shows the initial failure of new components due to defects (infant mortality), the useful life period, and the wear-out period.

For bridges, 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 quality controls 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 will lead to a shorter than expected service life.

Following the period where infant mortality may occur, elements typically have long service lives where 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 to reduce 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.

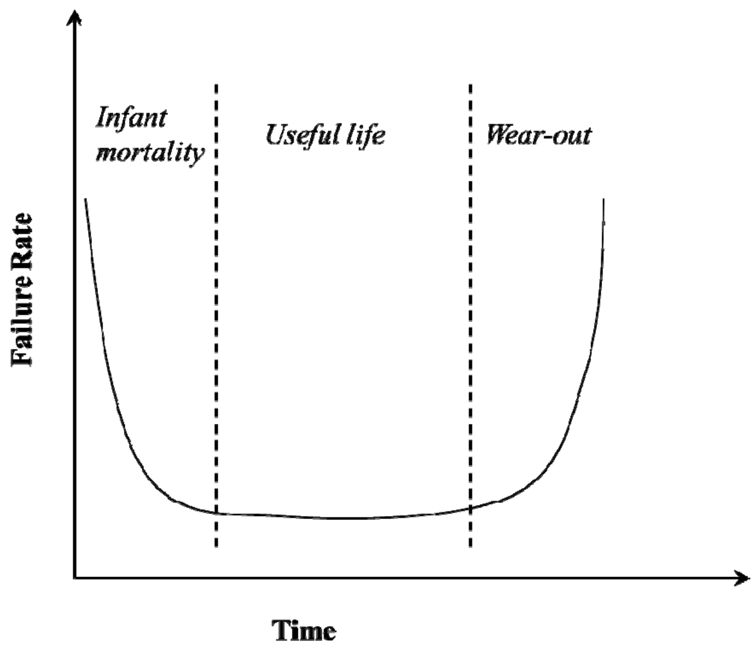

Figure 3. Plot of the "bathtub" probability curve.

Many different methods are available to model failure processes to 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 analysis methods like Markov Chain modeling, which use expert opinions and empirical data to estimate transition probabilities [26].

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 based on these models. 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 indicate the level of data resources that need to be either determined empirically or estimated using statistical tools and probability functions [9, 10, 27, 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 bridge design, operational environments and construction practices. Verification of the assumption requires observation of bridge performance over its service life, and by definition therefore cannot be determined in time to be usefully applied. It’s also notable that among the many variables 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 estimate future performance effectively.

Table 1. Variables use 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

3.4 Key Elements of RBI

This section of the thesis provides an overview of key elements of the RBI process described and detailed in the Guidelines for Reliability-Based Inspection Practices, which were developed through this research program. This includes a description of the Occurrence Factor, the Consequence Factor, inspection procedures for RBI, the Reliability Assessment Panel (RAP), and the incorporation of data into the RBI process.

3.4.1 The Occurrence Factor

Within the RBI process, an estimate of the probability of failure for a given bridge element is expressed as an Occurrence Factor, O. This factor is an estimate of the likelihood of severe damage occurring during the specified time interval, considering the likely damage modes and deterioration mechanisms acting on the element. Key attributes of the element that effect the likelihood should be 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 probability of failure for the component given certain attributes of the component and its operational environment [29]. These empirical equations include factors associated with the attributes of a 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 is combined with industrial modeling tools to estimate the POF for specific components or systems [14, 31-34]. For cases where historical data is scarce, where systems are complex and/or evolving such that relevant historical data is unavailable, expert judgment and expert elicitations are used.

To develop an estimate of the POF over a certain period, several factors need to be considered in including what constitutes a practical definition of failure, as described previously, 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, the ASME guidelines suggest that a first-level qualitative analysis can be achieved using a three level scale shown in Table 2 [20]. An estimate of the annual POF associated with a qualitative ranking is also provided in this table. In this context (for this industry) a high probability of failure 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 two 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.

Table 2. ASME probability of failure 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)

For application to the RBI assessment of 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 the analysis.

3.4.1.1 Assessment Interval

Given the typically long service life of a bridge and the generally slow rate of deterioration mechanism such as corrosion, annual POF estimates such as those described above may have little 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 has failed, if the valve is not able to open when required, it has failed.

However, 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's 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 a failed condition.

For the RBI process for bridge inspection, an Occurrence Factor 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 [18, 35-43]. For example, commonly available corrosion models indicate that significant periods of time transpire between construction of a bridge and initiation of the 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 damage 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 six 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 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, and at a 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, since the likelihood is low that these deterioration mechanism could result is 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 and determining an appropriate inspection interval. 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 due 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 which was currently rated in good condition was likely to progress to a serious state in one year, that estimate would be very low, since deterioration mechanisms are slow acting. However, if the time period of ten years were used, the uncertainty could be very high. The time interval of 72 months was selected in part to provide a suitable balance over which damage progression could be reasonably predicted based on engineering analysis and rationale.

3.4.1.2 Occurrence Factor Categorization

A four category, qualitative scale was determined for the estimate of the Occurrence Factor for RBI practices. The scale ranges from remote, when the likelihood is very small but may still be possible, to high, where the likelihood of the event is quite possible. The categories and associated verbal descriptions are shown below in Table 3.

Table 3. 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 Occurrence Factor is determined by expert judgment considering key characteristics, or attributes, of the important elements of a bridge. These attributes are characteristics of a bridge element which 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 the reliability of the element. These attributes are identified and assessed through the expert elicitation process.

Table 4 includes numerical ranges that could be used to describe the Occurrence Factor scale quantitatively. 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 is scarce, as previously discussed.

The numerical values shown in Table 4 are target values that can be used to map such data or models to the qualitative scales used in the analysis, if available. For example, data from PONTIS deterioration curves or data from probabilistic analysis, or from other deterioration models, could be incorporated directly into the assessment of the Occurrence Factor 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.

Table 4. Occurrence Factor 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	1/100-	1% or less

		occurrence	1/1,000	
4	High	High likelihood of occurrence	>1/100	> 1%

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 an expert were asked 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 Occurrence Factors

Occurrence Factors are determined through an expert elicitation of the RAP which has been assembled by a bridge owner. The RAP provides experience and knowledge on the performance of materials, designs and construction quality and methods within a specific operational environment. This knowledge and experience is used to categorize the Occurrence Factor considering credible damage modes and deterioration mechanisms for bridge elements. Although based on expert judgment and a qualitative ranking or categorization, supporting rationale for the categorization is developed through a systematic analysis of common attributes of the element under consideration.

The analysis in conducted systematically by identifying critical design, loading and condition characteristics, or attributes, that affect the durability and future performance 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 a 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 Occurrence Factor 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 five, which represents a "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 Occurrence Factor rating of moderate or high. Past experience with decks of a similar design and with the characteristics of the specific operating environment, combined with engineering judgment, are used to support the assessment of the specific Occurrence Factor 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 the design, construction and maintenance. Design attributes are frequently intrinsic characteristics of the element that do not change over time, such as the amount of concrete cover or the material of construction (concrete, steel, 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. This may include structural loading, traffic loading (ADT, ADTT), 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 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.

Key attributes for a bridge element are identified by the RAP and used to assess the appropriate Occurrence Factor for the given element and damage mode being considered. The attributes identified are ranked according to the importance of each attribute in assessing the reliability and the durability of the bridge element. For example, attributes that play a primary role in determining the likelihood of damage, a scale of 20 points can be used, 15 points for an attribute that has a moderate role 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 by the RAP. The RAP may also consider the 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 it 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, a simple high, moderate and low ranking can be used. Using a 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 is 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 means of identifying the appropriate Occurrence Factor for a specific element.

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 Occurrence Factor assessment. This is not a "one size fits all" approach, but rather is intended to conform to the varying needs of different operational environments and bridge inventory characteristics.

The commentary section of the Guidelines, reproduced in Appendix C, 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 similar 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.

Several illustrative examples of attribute scoring are also provided in Appendix D as guidance for making these assessments. This includes scoring sheets for tabulating scores for different elements of bridges, and using those scores to determine the Occurrence Factor. However, once the attributes and their rankings have been determined by the RAP, the scoring process may be more readily accomplished by integrating or developing software for scoring the characteristics of the bridge elements. Many of the attributes identified by the RAP may already be stored in existing databases and bridge management systems.

Certain key attributes should be identified as part of criteria for reassessment of bridge inspection requirements, following subsequent RBI 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 Occurrence Factor), reassessment of the inspection requirements is necessary.

Other attributes can be identified as screening criteria, to limit the scope of assessments being conducted by the RAP. Screening attributes can 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. Screening attributes are typically attributes that:

- make the likelihood of serious damage occurring very high
- make the likelihood of serious damage occurring unusually uncertain
- identifies bridges with different anticipated deterioration patterns relative to other bridges in a group

The RAP must identify the appropriate value or condition for the attribute to be used as a screening tool. For example, if considering the likelihood that a steel bridge will suffer corrosion damage which reduces its rating to a three, and the current rating is four, the RAP may consider that such condition indicates that there is a high likelihood of further damage developing over the next 72 month period. 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, since failure is highly likely to occur in the coming time interval.

Another example would be to screen steel beam elements in bridges that have open decking. Since the open decking allows drainage to flow 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 other 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.

3.4.2 The Consequence Factor

The second key factor to be assessed under the RBI process is the Consequence Factor, which is a categorization of the likely outcome of a given damage mode if it were to result in a failure of the element being considered. This assessment of consequence is geared toward assessing and differentiating the relative ranking of elements in relation to each other and to their 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 consequences of 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 Consequence Factor, C, 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 quantitatively, through necessary analysis and testing, or qualitatively, based on past experience with similar failure scenarios. The four level scale used to assign the Consequence Factor is shown in Table 5. The Consequence Factor categorization ranges from low, which is used to describe failure scenarios that are benign and very unlikely to have a significant effect on safety and serviceability, to severe, where the threat to safety and life is significant.

Table 5. Consequence Factors for RBI.

Level	Category	Descriptive definition	Examples*
1	**Low**	Benign, minor serviceability, effect on ride quality, maintenance need, may require monitoring	Spalling on a deck, coating failure, leaking joints, spalling in deck soffit not over roadway, spalling in abutment or pier, etc.
2	**Moderate**	Moderate effect on serviceability, some disruption of service possible, reduction in posted traffic speed, requires increased inspection and monitoring, planned repair/maintenance	Corrosion damage that requires scheduled repairs, fatigue cracks that require repair, corrosion at bearings, sagging, plate bearing failure
3	**High**	Individual/local member failure, major impacts to traveling public, disruption of service, urgent repair need	Fractures in a multi-girder bridge that require lane closures or truck restrictions, impact damage to a steel or concrete girder requiring traffic restrictions, significant section loss, rocker bearing failure in multi-girder bridge
4	**Severe**	Structural collapse / loss of life	Major collapse of a portion or all of the bridge forcing closure, bridge is not safe for use, significant safety concern for traveler

*The examples are not all inclusive and are simply intended to illustrate the relative scales of different scenarios

In assessing the consequences of a given damage mode for a given element, the RAP must establish which outcome characterized by the Consequence Factors in Table 5 is the most likely to occur. 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 Factor is high and that severe consequences are very remote for a multi-girder bridges, based on their 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 Consequence Factor 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 Consequence Factor for a given scenario. For others, analysis may be necessary using suitable models or other methods. A series of criteria are provided in the Guidelines that are utilized during the assessment to determine the appropriate Consequence Factor for a given element failure scenario. These criteria, combined with owner-specific requirements for assessing bridges and bridge redundancy, are used to determine the appropriate Consequence Factor for a given scenario. A sample of the process for determining the Consequence Factor can also be found in Appendix D.

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 Occurrence Factor 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 RAP analysis of the Occurrence Factor, and this assessment provides a 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 are typically 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, it would be necessary to know if fatigue cracks were currently present. To make that assessment, sufficient access to the superstructure of a bridge is required to determine if fatigue cracking is indeed present and the inspection procedure must include reporting the presence or absence of fatigue cracks. In some cases, nondestructive evaluation (NDE) techniques may be required within the inspection procedure to allow for the reliable detection of a 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 Occurrence and Consequence Factors provides a basis for the inspection requirements to be used in the field, by identifying credible damage modes and prioritizing theses damage modes based on their potential effect on safety and serviceability. Based on the assessment of the Occurrence and the Consequence Factors, damage modes for a bridge can be prioritized based on the product of these factors:

$$IPN = O \times C$$

Where IPN = Inspection Priority Number. For example, if fatigue cracking has a moderate likelihood of occurring and the consequence is severe, then the IPN would be 3 x 4 = 12. If fatigue cracking was moderately likely, but the consequence was 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 x 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 which 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 to improve the effectiveness and reliability of the inspection.

3.4.3.1 Reliability of Inspection Methods

For most reliability-based inspection 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. 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 the results of 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 those 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 Guidelines 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, which 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 a reliability-based inspection 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 this 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 Guidelines 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 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 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 effecting key elements of the bridge
- Report the location and extent of the damage
- Report on key damage precursors as developed through the RAP assessment

Precursors identified through the RAP process may include evaluating specific elements on the bridge such as the joints or drainage systems. Specific conditions that are precursors or are 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 [25], in many cases as bridge management elements. The bridge management elements may need to be more fully developed under the RBI process as needs develop for specific inventories.

3.4.4 The Reliability Assessment Panel (RAP)

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 deicing chemical 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, likewise, vary between states. Typical details such as drainage features, and the use of protective coatings or other deterioration inhibitors, for example, sealers for concrete, vary between bridge owners, as do traditional construction practices, construction details and materials of construction. This is because construction and design requirements are ever evolving and changing over time.

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 the 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 is 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 identify and prioritize key attributes and factors that support the rational characterization of the Occurrence and Consequence Factors.

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 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 [20, 21, 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 [20]. For RBI in the context of bridges, expert elicitation is used to:

- Categorize the Occurrence Factor based on expert judgment
    - Determine credible damage modes for bridge elements
    - Identify and prioritize key attributes that contribute to the reliability and durability of bridge elements
- Assess likely consequence scenarios and categorize the Consequence Factor

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.

The processes are intended to be simple and straight forward. 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 questions are 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%, with the sum of all ratings equal to 100%. The expert may note supporting rationale for their estimate.

The individual results from each member on 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-on 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 in some way. Rationale for each attribute is documented, either by using rationale already provided in the Guidelines, or by 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 be used for assessment of the Consequence Factor by providing different potential failure and consequence scenarios, and asking the RAP to assign a relative likelihood to the outcome of the failure according to the Consequence

Factor scale. This is a useful tool for evaluating the appropriate Consequence Factor for situations that are not well-matched to the examples and criteria provided in the Guidelines, 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 their judgment, experience, available data, and given the scenario presented, to determine what the most likely consequence will be 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 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 likely consequence. However, in some cases, the most likely choice will not be clear and there will not be consensus.

4. <u>Identify Consequence Factor:</u> If there is consensus among the panel regarding the appropriate Consequence Factor, then the rationale for making the determination is recorded. This rationale should be consistent with criteria provided in the Guidelines, or the panel should document deviations or changes and the associated rationale.

In cases where consensus is not reached in the initial elicitation, the experts should discuss their rankings and 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 Consequence Factor. 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 where consensus cannot be reached. However, the method must result in the selection of the most appropriate Consequence Factor, based on the Guidelines provided and sound engineering judgment.

A sample expert elicitation which was conducted by the research team to better calibrate the presented methodology is included in Appendix E.

3.5 Data to Support the Analysis

There are a number of resources available or that could be developed to support the RAP assessment of the Occurrence Factor for bridge elements by providing data to support decision making. While none of these sources for data provide perfect solutions, for example, a quantitative calculation of the Occurrence Factor, they can provide data that supports decision-making and rationale developed through the RBI process. This section of the thesis describes a few of these resources, as well as important considerations for utilizing these data for the reliability assessment of bridges.

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 is typically 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) [20, 22, 29].

Quantitative data is data developed through specific probabilistic models, databases of failure rates, or past performance data such as deterioration rate models. This data is typically more in-depth and detailed than qualitative data and can provide valuable insight and uniformity in approach. However, developing such data can be impractical for realistic situations that are too complex to be modeled effectively.

Data on past performance is frequently incomplete or inaccurate, and in some cases can provide ineffective estimates of future performance [49]. 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 is sparse, includes 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 informed by quantitative data when and where it is available, forming a continuum of data as shown in Figure 7 [22].

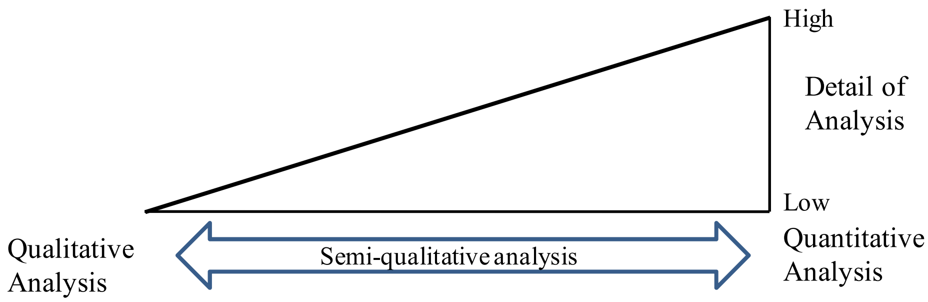

Figure 4. Continuum of data needed for qualitative to quantitative analysis [22].

The accuracy of the results from a qualitative assessment depends on the background and the expertise of the analyst [20]. Quantitative data, such as deterioration rate information measured from NBI or BMS data, can provide supporting rationale for decision-making, if handled appropriately. Estimates of precise numerical values in a 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 is quantitative does not necessarily mean it is more accurate.

Difficulty in effectively representing past experience, expert knowledge and bridge–specific conditions can result in quantitative data that is 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 [20]. Formal methods for eliciting expert opinion for the purpose of risk assessment are included in the Guidelines and in the literature, as previously discussed [50, 51].

3.5.2 Data Needed for Assessment

To perform a reliability-based assessment, the primary data required includes 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 provided some insight. Data on materials and design characteristics is generally available in the bridge files, typical design and detailing practices, and local knowledge of construction practices.

Damage data describes the deterioration which is active or is expected on a structure and estimates 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.

3.5.2.1 Deterioration Rate Data and Previous Failure Histories

Deterioration rate data such as that developed by Agrawal [35] and others [40, 52, 53] can be used to support estimates of future performance of bridge elements. However, there are significant challenges to applying this data to inspection strategies for specific 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 significantly 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 [29]. For elements constructed of concrete and steel, with varying design characteristics and construction quality, and exposed to widely variable environments and loading, this may be very challenging.

However, deterioration curves and probabilistic failure estimates are valuable to the analysis process in several ways. Generic deterioration curves can provide some background and supporting 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. Additionally, 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 it. 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), and 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 effect 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 which provides general information on the deterioration of the structural components over time, based on visual observations. Element-level data is documented for states using PONTIS or other element-level inspection schemes. Obviously, this condition data is 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. This data can also be used to construct deterioration curves to support assessments.

Condition data can also be useful in screening bridge inventories into broad categories according to their anticipated reliability, based on current condition and design characteristics. Element-level systems are emerging more widely, and typically have more specific information on bridge condition. This data can be used to identify criteria from the RAP analysis to key reassessment of inspection needs, as well as provided the means of assessing specific values for attributes that are aligned with bridge or damage elements in an element level system.

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 is 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. This data may include specific, quantitative values 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. This includes 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 is 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 that 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, 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 employed by a RAP during the assessment process, and could be further developed if needed to address specific situations, or utilize 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, particularly in 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 with different design characteristics and located in different geographical regions, the commercial software Life-365 was used to generate benchmark corrosion effects model 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 six 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 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 solid 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 for corrosion initiation in the uncoated rebar. Complete details on the analysis process are available in the literature [36]. An expanded version of this section with more modeling results can be found in Appendix F.

Under our analysis, deck slabs were modeled using a standard concrete mix and uncoated carbon steel reinforcement. This was done to represent worst case scenario for corrosion initiation, given that no corrosion mitigation strategies were employed. Six states across the United States which represented different geographical regions and thus different chloride build-up rates on the surface of the concrete resulting from chlorides in the environment and deicing 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 one inch and three 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 assuming a concrete cover of three inches is presented here.

Figure 8 visually illustrates the difference in the modeled time to corrosion initiation for different geographic regions. As shown in Figure 8, 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 initiation in as little as ~7 years is expected, while in less aggressive environments, such as Arizona, corrosion 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 towards 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 concentration 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 the 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 inspection, however, it is one of the most important and widespread. Data such as that provide through this simple modeling can be used, among other inputs, to provide supporting rationale for categorizing the Occurrence Factor 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 where corrosion mitigation strategies might greatly increase the time to corrosion if they were used, supporting rationale for a lower Occurrence Factor, 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.

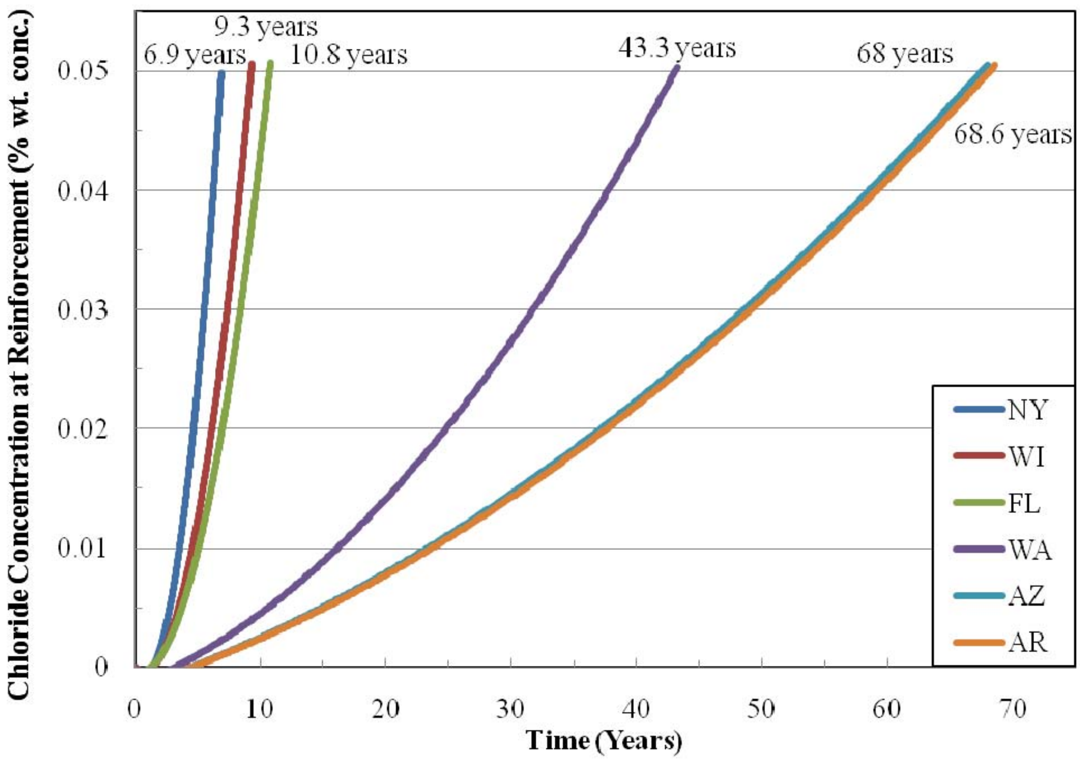

Figure 5. Time to corrosion initiation for different states based on a diffusion model.

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 of protective coatings for corrosion control [37, 44, 45, 54-56]. 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, 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 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.

Due to the relatively slow rate of 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.

4 CONCLUSIONS AND RECOMMENDATIONS

The research detailed in this thesis has 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 “Guideline for Reliability-Based Bridge Inspection Practices,” which has been developed based on the application of reliability theories. This document meets the research project objective of developing a recommended practice for consideration for adoption by AASHTO that 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 guidelines. Background information and the foundation for key elements of this process have been further expanded in this thesis, to provide additional details and perspectives on the research conducted as part of the project. However, the primary conclusion of the study is the comprehensive guidelines developed, which provide 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.

While these guidelines were well developed and comprehensive in nature, their application to an actual inventory or inventories of bridges has yet to be tested. As a result, quantitative data describing the effectiveness of the methodology for improving the safety and reliability of bridges and optimizing resources is unavailable. Implementation challenges have yet to be fully identified, tested, and resolved.

Suggested future research from this project is to develop and implement a pilot study of the RBI practices, such that the effectiveness of the methodologies can be assessed under real-world conditions, and implementation barriers can be further identified and resolved. Such a pilot study is key to the further development of the technology and the advancement of the innovative inspection planning processes developed herein.

To assist in the development of pilot studies, the National Bridge Inventory (NBI) database was analyzed using the criteria developed by this project to identify states which may benefit greatly from this RBI methodology and thus provide excellent pilot studies. The methodology and the results of this NBI data mining can be seen in Appendix G.

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Appendix A: Literature Search

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A.1 INTRODUCTION

This appendix will summarize the findings of a comprehensive literature search which was conducted to determine the current State-of-the-Practice and State-of-the-Art for reliability-based inspection planning. This search included exploring the available literature in a number of areas, including the reliability analysis of bridges in general, the deterioration modeling techniques available for highway bridges, and the current methodologies for assessing vulnerability and risk. The topic of reliability-based inspection planning, in general, is very broad and a wide array of literature has been collected and reviewed. The most relevant literature will be reviewed in detail in the body of this appendix while an attached annotated bibliography will summarize many other sources of information which were reviewed through the research project.

Through this literature search, two challenges with respect to the creation of a reliability-based inspection program were found. First, the definition of what a "reliability-based" inspection methodology is versus a "risk-based" inspection methodology was difficult to define. Although there are literature available on reliability-based inspection methodologies for certain applications, such as for bridges, lock dams etc. [1-3], very similar methodologies are also used in other industries which are frequently termed risk-based, where reliability analysis is used as a tool within the risk-based methodology [4-8].

The second challenge was to identify opportunities in the existing literature to apply the concepts or the methods already developed in other industries for highway bridge analysis. The literature search revealed methodologies which have been developed for dams that seem particularly relevant to the overall concept of the methodologies to be developed for highway bridges, although there are some fundamental differences in the goals of the assessments. The literature search has also documented several well-developed guidelines for the creation of RBI practices from other industries that provide a strong foundation for developing the overall approach for bridges. These include important guidelines from the American Petroleum Institute (API) and the American Society of Mechanical Engineers (ASME), who have developed and applied RBI within their respective industries for many years [10, 27, 29].

Section 2 of this appendix describes the approach taken for conducting this literature search while Section 3 will provide a detailed summary of the more important literature that was discovered. Section 4 will provide several critical conclusions while Section 5 will present the references cited. Section 6 will present an annotated bibliography which will briefly describe other sources of information uncovered during the literature search which may or may not have been presented in detail in Section 3.

A.2 ACTIVITIES IN THE LITERATURE SEARCH

The literature search was conducted utilizing a variety of resources. These include searches through common databases available at University libraries, such as Compendex and Scopus, as well as several industry specific databases such as the American Society of Civil Engineers (ASCE) Research Library and the American Society of Mechanical Engineers (ASME) Digital Library. The literature search has also utilized the Transportation Research Information Services (TRIS), Academic Search Premier, and the Department of Energy Information Bridge. These databases have been searched using criteria and key words that were developed as an ongoing practice throughout the literature search process, to explore avenues and directions where important information could be discovered which may not have been apparent using the original search terms.

In addition to the literature search, contacts in other industries and in other countries that were known to the research team have been polled to determine what, if any, activities were ongoing, specifically in Europe and Japan. It was found that two countries, Germany and Japan, were initiating projects of a similar nature to NCHRP 12-82, but that their efforts were still in the early stages of development.

A.3 RESULTS OF THE LITERATURE SEARCH

The literature search revealed a multitude of articles, standards and practices related to the development of reliability-based inspection methods. The most relevant results of the literature review have been included in the following sections. Section A.3.1 will discuss the relevant research applying to structural reliability approaches in bridge inspection and management. Section A.3.2 will discuss risk-based inspection methods already in place both within the Civil Engineering industry and in other industries. The differences between reliability-based inspection practices and risk-based inspection practices will be highlighted, along with the use of risk matrices for inspection planning. Section A.3.3 will provide information on the differences between qualitative methods and quantitative methods, with attention given to the impact on the categorization of probability and consequence estimates.

A.3.1 Structural Reliability

Several methods and processes have been suggested for the assessment of bridge reliability and for the estimation of inspection requirements based reliability analysis. The spectrum of available literature was reviewed to elicit the most useful aspects of these methodologies for application within the scope of this research project, and a comprehensive listing of papers reviewed has been included in the annotated bibliography.

In general, reliability is defined as the ability of an item to operate under designated operating conditions for a designated period of time or a designated number of cycles, depending on the nature of the item, before the item can no longer perform its intended function. In this sense, the reliability of an item can be designated through a deterministic expression or through a probabilistic expression.

A deterministic approach deals with understanding how and why an item fails and how that item can be designed to prevent such failures from occurring. This may include a deterministic analysis, such as calculating the load capacity, reviewing applicable failure scenarios, understanding the physics of failure, or determining how inspection and testing relate to failure prevention.

A probabilistic approach generally involves calculating the probability of failure for an item within a designated time period. This usually involves developing or identifying probabilistic methods to model future deterioration rates or future loading levels in order to determine the ability of the item to resist the identified loading. As such, probabilistic models are usually a function of the loading and the resistance of an item. In bridge engineering, these models are based on design-notions of bridge failures [2, 11] which attempt to calculate the reserve capacity of a structure in relationship to the anticipated future loads. In general for bridges, these methods consider the bending strength and the shear strength of a bridge in comparison with the applied loading.

There are a number of research papers which describe reliability-based approaches for evaluating highway bridges and other civil structures. Many references were reviewed by the research team, including those related to bridge deterioration, modeling of bridge reliability and optimization of inspection and maintenance based on structural reliability approaches. Applications of structural reliability theories to other civil structures, such as off-shore platforms, were also reviewed in the course of the research [5-8]. The most relevant articles relating to bridge deterioration, safety and inspection are summarized herein.

The research that has been conducted for evaluating in-service highway bridges utilizes the theorems of structural reliability, based generally on methodologies utilized for the development of probabilistic design codes [12]. For example, for a steel girder bridge, a probability-based method for determining inspection intervals was developed by Sommer [13]. This method used an assumed corrosion model to estimate time dependant deterioration. Resistance to failure modes of shear, bending and bearing failure were estimated based on theoretical models of the effect of section loss due to corrosion, and reliability indices were calculated and used to formulate an optimization problem in regard to cost. A probabilistic analysis was used to calculate member capacities based on assumed statistical variations of a number of material properties and assumed corrosion parameters; dead loads and live loads were estimated based on assumed statistical variations and live loading scenarios. A reliability analysis was performed using a software package developed in Norway. The results of this complex analysis indicated that the optimum inspection interval would "probably be an interval of between five and ten years."

More recent applications of structural reliability theory in the development of inspection strategies included an assessment of miter dam gates [1]. In this work, visual inspection results were incorporated into a reliability analysis that considered two failure modes for a miter gate, corrosion and fatigue cracking. Deterioration models for corrosion and fatigue were used. Potential distribution functions for inspection effectiveness were assumed, and a reliability analysis was performed. Other work by the same authors has included the reliability analysis for repair optimization and updating bridge reliability based on visual inspections [3]. The methodologies used are based on structural reliability theories and utilize limit states based on models for the shear and moment capacities of superstructure members, and probabilistic models for the uncertainty of deterministic values. Consideration of multiple limit states has also be investigated [2] which considered serviceability flags in addition to the strength-based limit states typically considered in structural reliability models.

Other significant work in the area of applying structural reliability theory to highway bridges has been conducted by Stewart [14-17]. In these papers, theoretical models are developed to estimate the reliability of components based on corrosion models and various assumed limit states. The results of these reliability analyses are highly dependent on model assumptions. An example presented in one article [15] indicates that calculated reliability indices (β) from these analysis can be highly variable. These reliability indices are typically on the range of 3.5 to 4.5 for design codes, based on a calibration of the codes considering elastic behavior of single structural elements. In one example presented for the reliability analysis of a bridge, the indices varied from 0.24 to 10.4, depending on the modeling assumptions used in the analysis. In another example cited, changing the type of distribution function (normal vs. lognormal) used to model uncertainty in the compressive strength of the concrete changed the calculated probability of failure for a reinforced concrete bridge by six orders of magnitude.

A multitude of similar studies based on structural reliability theory were reviewed in the course of the research project. The conclusion reached, based upon review of this literature was that these approaches were not currently implementable for highway bridge inspection, due to their highly theoretical nature, the complexity of analysis required for even a simple structure, and the uncertainty in the outcome of the analysis in relationship to maintaining the safety of an actual bridge. Because these methodologies were primarily focused on design notions of potential failure, they were most appropriately applied to a narrow set of potential failure modes for which established models exist, for example, for fatigue cracking [18, 19]. However, even in the case of fatigue cracking, the level of analysis required to estimate the structural reliability is such that it would only be practical if a large inventory of structures with very similar design characteristics existed such that a general model could be established. Additionally, the assumptions necessary to complete the analysis, such as the reliability of inspection techniques, are not validated and are sometimes unrealistic.

Significantly, serviceability limit states, such as the spalling of concrete, local deformations or excessive deflection, are not typically considered in the strength-based models which have been developed. As such, a large portion of the conditions important to the long-term maintenance and durability of a bridge, and the operational performance of the bridge (i.e. ability to carry traffic safely), are not addressed. Although it is conceptually possible to develop such models using the general reliability framework, significant research remains that will likely take many years to perform and implement.

Additionally, the outcomes of the reliability model analysis are not easily mapped to inspection decision-making based on terms of safety. The calculated reliability indices, which are related to the probability of failure for the given load and resistance models, are notional and do not necessarily point to specific necessary actions or decisions. The primary reasons for this are that the values are abstract and no target values have been established that could be used to determine if a given value is good or bad, and the values are based on strength models that do not reflect the serviceability limit states (cracking, excessive deformation, spalling) which are important to effective bridge inspection. Further, even if target values were established, there is presently no framework for applying those calculated values to determine inspection intervals. Given the complexity of the modeling and the many assumed values necessary to perform the analysis, the process is impractical for application in the near future and cannot be effectively used for bridge inspection until serviceability considerations can be included.

It is important to note that in industrial applications, probabilistic reliability methods are most commonly used when human interaction with the structures being analyzed is limited. As a result, serviceability limit states are much less important; as long as the structure is carrying loads, it is not important that chunks of concrete are falling off the structure, because the structure is, for example, an offshore platform. This is not the situation for highway bridges, where the human interaction is high and as such, serviceability limit states have tremendous significance. As such, a risk-based analysis which considers both the likelihood of deterioration and the consequences of that deterioration is needed to effectively determine the importance of damage to a bridge, including serviceability considerations, and to identify effective inspection strategies for maintaining highly reliable structures.

A.3.2 Risk-Based Inspection Methods

The concept of risk is something that individuals live with every day. People are constantly making decisions based on risk. Simple decisions such as crossing a busy street or driving to the mall in a car require an assessment and a subsequent acceptance of risk.

In engineering, decisions based on relative risk are made every day in assessing necessary actions such as repairs, or the sufficiency of certain designs. The fundamental idea behind the engineering analysis of risk is to estimate how likely it is for a certain event to occur and to estimate the potential consequences of that event. This analysis is frequently done in an ad-hoc manner in the process of decision making using engineering judgment and experience. In fact, risk assessment is already utilized regularly in bridge inspection and maintenance in decision-making regarding the identification of critical repair and/or maintenance needs.

Interestingly, the existing two year inspection interval was based on engineering risk considerations, as no quantitative engineering assessments were performed. However, the approach taken was very simple and did not attempt to account for any specific engineering aspects. The interval was simply time based; i.e., a one year interval seemed to short, but ten years seemed too long. Hence, two years was settled upon under the assumption that within that interval, events or conditions that could cause failure would likely be identified prior to the event occurring.

Risk-based approaches have been adopted in many industries as a tool for inspection planning to focus attention on the components which would most adversely affect the whole system if they failed. Well-developed recommended practices and standards for implementation of systematic risk analysis for inspection planning exist that can provide a technical foundation for the development of a system customized to address the specific needs of bridge inspection, with consideration of the specific resources, probabilities and consequences associated with the maintenance of the bridge inventory.

For the implementation of a risk-based analysis methodology, risk is defined as the product of the frequency with which an event is anticipated to occur and the consequence of that event's outcome. This could be expressed mathematically as:

$$Risk = Frequency\ x\ Consequence$$

Frequency, in this equation, is the rate at which a certain event occurs, when applied to a specific time interval. This is typically expressed as either a probability of failure (POF) for that time interval or as an estimated likelihood of occurrence for a certain event. Consequence is a measure of the impact of the event occurring, which may be measured in terms of economic, social, or environmental impacts.

Risk assessment is the process of identifying the sources of hazards, estimating the risk and evaluating the results. Risk assessment processes address three fundamental questions to determine risk [20]:

1. What can go wrong?

2. How likely is it?
3. What are the consequences?

Risk can be expressed quantitatively, as a measure of loss per unit of time, or qualitatively. Presenting risk qualitatively is frequently an effective method of illustrating risk. Figure A1 shows a qualitative risk matrix [20]. This matrix shows a good representation of the overall concept and basic principles of risk. A high likelihood combined with a high consequence results in a high risk, located in the upper right corner of the matrix. A low likelihood combined with a low consequence results in low risk, located in the lower left hand corner of the matrix.

High risk and low risk elements typically do not create challenges in decision making. Items that have a high risk are generally not acceptable and changes will be required to lower their risk level either by reducing the likelihood of the event occurring, by mitigating the consequence of the event, or both. Items that are low risk are generally acceptable and no changes will be required. The challenge of decision making is typically focused on the medium risk areas, where questions arise about how much risk is acceptable and how many resources are available to ensure that level of risk across the entire system.

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

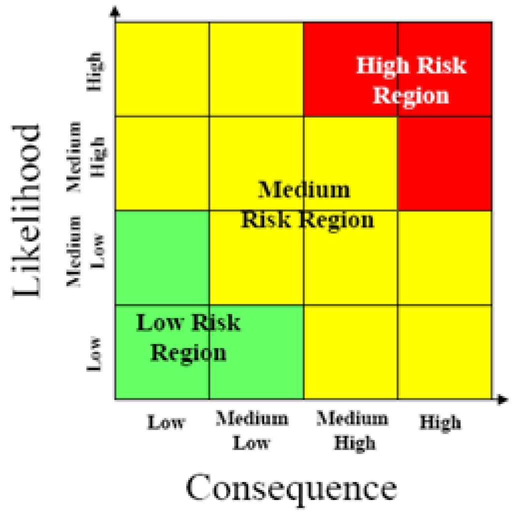

Figure A1. Risk matrix showing high, medium, and low risk values [20].

Risk-Based Inspection (RBI) is an inspection planning process using risk assessment to determine inspection frequencies and scope. The setting of inspection frequencies within RBI is not a rigid process, such as is the case in calendar-based inspection programs. Rather, it is a process that evolves and changes over the life of an asset such that inspection frequencies change as risk increases or decreases. Therefore, the frequency of inspection is aligned with the needs of the asset and the associated risks, focusing attention on the most at-risk assets. The general logic applied to the scope and frequency of the inspection is:

- Higher risk assets generally have a shorter interval between inspections and potentially have more in-depth or thorough inspection requirements.
- Lower risk assets often have extended inspection intervals and may have reduced inspection scopes.

An important aspect of the RBI planning approach is that the process focuses attention not on the items that are most likely to fail, but rather on those items whose failure will most adversely affect the system by considering both the likelihood of failure and the associated consequences. This approach has been widely accepted in many industries which have facilities that can be considered to be analogous to highway bridges; very large, expensive, and complex structural systems that are exposed to similar environmental conditions and mechanical loading. These industries have spent a great amount of resources and time developing RBI procedures that appear to be serving them very well. The groundwork laid by these other industries can be used to build a RBI methodology that is applicable to highway bridges. The following sections describe characteristics of industrial RBI processes which can be used to provide a foundation for the development of risk-based processes for highway bridges.

A.3.2.1 Reliability-Based Inspection vs. Risk-Based Inspection

Reliability is a concept that can be applied generally to measure the ability of a bridge to perform at an acceptable level under certain conditions for a designated period of time. Structural reliability is the application of this concept which assumes that the acceptable level is a function of the load carrying capacity of the structure, based on design concepts such as shear or bending strength. For bridge inspections, the acceptable level includes strength considerations, but also includes a variety of serviceability limits states that, while related to strength, are not direct measures of strength. These serviceability limit states include considerations related to durability, deflection, and local damage such as spalling or cracking. The fundamental problem with applying reliability concepts, which have also been used to develop the LRFD codes [12], is that the envisioned deterioration modes (loss of load carrying capacity), represents only a portion of the required analysis for bridge inspection. Serviceability limit states are an important component of bridge inspection that has not been incorporated in typical reliability-based analysis [2]. These include items like spalling of concrete, which may not present a significant effect on the reliability index of the particular component (i.e. strength), but may have a catastrophic consequence (death of a motorist) if the spalling occurs over a roadway.

Additionally, the catastrophic failure modes of a bridge, such as a collapse due to the applied load exceeding the moment capacity, for example, are relatively rare occurrences and typically only occur when the overall condition of the structure is severely deteriorated. In fact, bridge failure in the field rarely follows a neat and orderly process which might be envisioned while developing a design code and the associated failure modes (bending, shear, etc). Rather, it is stochastic in nature and frequently involves the misalignment of members, localized deterioration at bearings, and other modes of damage that are not easily considered in a structural reliability analysis.

Looking across the spectrum of available literature, it is apparent that such a reliability analysis is most frequently applicable as one component within a risk-based inspection plan [21-23]. For example, in the case of fatigue cracking, load and resistance models based on significant experience and research have been developed. As a result, it is possible to reasonably develop a deterioration model that can be used to express reliability as a some ration of demand and capacity [6, 24, 25]. This reliability analysis then provides input, in the form of a likelihood of occurrence, into a risk-based model for inspection planning. As such, a reliability analysis is one tool among many that can provide input for a risk-based analysis for the purposes of inspection planning. For a risk-based analysis, the consequences of failure must be considered, not just the possibility of failure.

A.3.2.2 Risk-Based Inspection Practices within Civil Engineering

A.3.2.2.1 Risk Assessment for Dams

Risk-based methodologies extend across many industries, including the assessment of dams. The dam community has developed several different prioritization methodologies for their maintenance and safety programs. These methodologies have been created by individuals such as Andersen and Bowles [33, 34] as well as by entities such as the Bureau of Reclamation. The report drafted by Harrald et al for the U.S. Army Corp of Engineers (USACE), entitled "Review of Risk Based Prioritization/Decision Making Methodologies for Dams" provides an overview of the current practices [35]. These methodologies are relevant to bridge inspections because they generally describe risk-assessment procedures for a class of civil structures that are susceptible to similar environmental impacts, have relatively sparse historical data on specific designs, and provide examples of rational methodologies developed for the purpose of prioritization of inspection and maintenance.

A methodology known as Portfolio Risk Assessment (PRA) was developed by Bowles [33, 36]. This approach seeks to apply risk based management to an entire portfolio of dams owned by a given entity. The process developed is usually semi-quantitative and involves both in-house and outside personnel. In this process, each individual dam receives a Failure Modes and Effects Analysis (FMEA) to identify the most likely failure modes. Then, by using both decision trees and historical data, the probability of failure is estimated.

Another methodology reviewed by Harrald et al, is the Risk Based Profiling System (RBPS). This system was developed and implemented by the Bureau of Reclamation (BOR). Its goal is to prioritize dam safety activities and resources and to identify those structures which represented the greatest risk to the public. This system considers hydraulic, seismic, and static loads as the primary factors that lead to dam failure along with operating and maintenance (O &M) issues.

To quantify these risks, RBPS uses the "Failure Index" which is the product of a load and the dam's response. To determine the Failure Index, the evaluator must complete several worksheets that address the range of loading conditions and the physical condition of the dam. The semi-quantitative process utilizes the experience and expertise of the evaluator to estimate the probability of failure or risk.

Andersen and Torrey's method, known as the Condition Indexing Method for Embankment Dams uses a total-systems approach, with a rating system that describes the current conditions in a uniform manner. This includes a ranking system based on expert judgment and a procedure to prioritize operation and maintenance activities. In this procedure each dam is rated based on three factors that contribute to vulnerability:

- I: Intrinsic and time-invariant properties of the dam such as the height, dam type, foundation type, and the storage capacity.
- E: External time-variant properties, such as the age of the dam and the seismicity.

- D: Design properties such as the spillway capacity and the mass movement factor of safety.

The overall vulnerability is calculated as the product of the three factor scores, with each individual factor score being an average value for that factor (I, E or D). A hazard potential for each dam is determined by assigning a score depending on the potential for loss of life, economic losses, environmental damage, and the disruption of lifeline facilities. Here, there are only three qualitative scores available, high, medium and low. The dam's importance is then calculated as the product of the vulnerability score and the hazard potential score.

It is interesting to note here that although some quantitative failure data is available from a national database, subjective expert elicitation is the preferred method of determining the likelihood of a particular failure mode for the dam. This is done because although the national data indicates historical failures, it does not indicate the likelihood of a particular failure mode looking forward. *The actual, conditional probabilities of a certain failure mode for a particular dam are a function of site specific and dam-specific factors that can vary dramatically from national averages* [28]. Rather, the probability of a particular failure mode (of the four considered based on experience) is determined from expert elicitation by posing questions like:

> "Considering a failure of the dam, what is the relative likelihood that each of the four failure modes will be the initiating event?"

Here, it is the responsibility of an experienced evaluator to assign subjective, conditional probabilities, usually with a precision of 10%. Data from previous dam failures is used as a starting point or background information, but is not used explicitly.

Next, the evaluator assigns conditional probabilities to each of the nine physical conditions that may be associated with a given failure mode. These conditions relate to dam attributes such as spillway capacity, crest elevation, and the state (condition) of the embankment. To achieve subjective ratings for the physical conditions, the following question is posed to the expert:

> "Considering the particular failure mode, what is the relative likelihood that each of the associated physical conditions would have precipitated it?"

In other words, the expert is asked that if you know the dam failed by overtopping, for example, rate the relative likelihood that each of the listed physical conditions precipitated that failure. This methodology provides relative probabilities for various deterioration modes for a dam, based on expert elicitation, and uses this data to rank the relative importance of physical conditions at a dam to identify focus areas for inspection and maintenance.

A.3.2.2.2 Risk-Based Assessments for Bridge Engineering

It should be noted that there are existing risk-based processes currently implemented within the bridge inspection program in the U.S. The most readily apparent is the assignment of reduced inspection intervals for certain bridges in the inventory, based on criteria developed by states which consider factors such as age, traffic composition and known deficiencies [37].

Such a process is an ad-hoc system of risk assessment, considering the likelihood of a negative event, such as a member failure, and the consequences of that event, based on the traffic characteristics of a bridge and/or its importance to the overall transportation system. For example, an interstate bridge with a low superstructure condition rating or a specific known deficiency may receive more frequent inspections, while a bridge with the same characteristics, but which is located on a low-volume route, may not receive the same reduced inspection interval. This represents a practical distribution of resources to focus inspections on the higher-traffic, more critical bridges rather than the bridges which are less critical, with lower traffic volumes. This is a form of risk assessment that may be informal, but shares the same considerations as this research project.

Another functioning risk assessment currently implemented within the NBIS is the extension of inspection intervals of up to 48 months [38]. The Federal Highway Administration (FHWA) guidance for extending inspection intervals identifies the following bridge characteristics as not being considered for an inspection interval of greater than 24 months:

- Bridges that have a condition rating of five or less
- Bridges that have an inventory rating less than the state's legal load
- Structures with spans greater than 100 feet in length
- Structures without load path redundancy
- Structures that are very susceptible to vehicular damage, e.g. structures with underclearances less than, 14 feet, narrow thru trusses or pony trusses
- Uncommon or unusual designs where there is little or no performance history

The parameters identified in this policy reflect an informal risk analysis that includes both probability of occurrence for a negative event and a consideration of the consequences for a negative event. For example, the exclusion of structures without load path redundancy expresses the high consequence of a negative event occurring; the exclusion of bridges susceptible to vehicular damage expresses the increased likelihood of damage for bridges with low vertical clearance.

A review of the applications for extended inspection intervals from 15 states revealed that for some cases, an ADT as high as 100,000 vehicles was considered in combination with condition ratings of six or greater. In some cases no age limitation has been used, relying on the condition ratings as reflective of the actual condition of the bridge, regardless of age. A review of the individual state 48 month inspection policies has been summarized in Appendix B.

It is worth noting that the 48 month inspection interval, although implemented unevenly on a national basis, has been implemented to the extent that 44% of routine inspections are done on a 48 month basis in Arizona, and 42 % in Illinois [39]. For Arizona, the policy applies to roughly 3,350 culvert structures which are defined as bridges in the NBIS. The rationale for the policy in Arizona includes the fact that "only about 50 of the 3,350 structures built since 1930 have a condition rating less than six" and that the culverts are generally not susceptible to vehicular damage [40].

In Illinois, the rationale for extended inspection frequency for slab and stringer bridges includes condition ratings of seven or better, load capacity, vertical clearance, ADT and ADTT, structural redundancy and age (less than 50 years) among other parameters [40]. These policies represent ad-hoc risk assessments for portions of the inventory; the subject project proposes a formalized version of these assessment for the bridge population overall.

The following sections present other risk-based processes applied to highway bridges, including analysis of unknown foundations, international activities and vulnerability analysis.

A.3.2.2.2.1 Bridge Scour Analysis

A well-developed, quantitative risk assessment process is described in NCHRP Report 107, Risk-Based Management Guidelines for Scour at Bridges with Unknown Foundations. The aim of this guideline is to provide guidance in conducting a risk analysis for the approximately 60,000 bridges with unknown foundations in the United States. The outcome of the risk assessment is a prioritization of bridges for further foundation investigation, installing scour monitoring devices, installing countermeasures and/or developing a bridge closure plan. This methodology uses the traditional risk-based approach, defining risk according to the following equation:

$$Risk_A = K \cdot P_A \cdot Cost$$

$Risk_A$ = Annual risk of scour failure (dollars per year)

K = risk adjustment factor based on foundation type and type of span based on NBI items and, where available, from more developed databases with foundation information

P_A = annual probability of failure based on NBI items 26, 60, 61, 71 and 113

$Cost$ = total cost of failure (dollars)

The methodology generally uses risk to assign monetary values to failure scenarios, and then uses this value to compare the cost of various mitigations strategies, such as installing scour countermeasures or automated scour monitoring. Costs are determined not just from the cost of bridge replacement, but also from user costs associated with the bridge being out of service.

Of particular interest in the overall risk assessment scheme is the process by which the annual probabilities of failure (POFs) were determined. Probability of failure in this context referred to the likelihood of hazard-induced bridge failure within a specified range of time. Given that reliable data on the actual annual rate of bridge failure is scarce, a series of phone interviews with State DOT personnel were conducted.

Based on these interviews, and limited data available from such sources as the NYSDOT bridge failure database, the average annual probability of failure was estimated to be about 1/5,000 (0.0002), or about 100 bridges annually. A previously developed software program, HYRISK, assumed a series of POF estimates for the purposes of simply ranking scour risk relatively between bridges. These POF estimates indicated approximately 60,000 failures per year when applied to the inventory, so clearly this estimate needed to be revised.

The individual POFs used in the process had been adjusted according to the recorded values for NBI items 60, Substructure Condition, and 61, Channel Protection, which were used to subjectively determine scour vulnerability. These POFs were further adjusted based on NBI items 26, Functional Class, and 71, Waterway Adequacy. This process essentially ranked the POF according to the bridge attributes, such that bridges with good channel protection and good substructure condition had a lower POF relative to bridges with poor channel or substructure conditions. Given that model, adjustments were then made to the annual POF estimates such that when applied to the bridge inventory, the resulting rate of failure was roughly 100 bridges, matching the estimate derived from expert elicitation. These estimates are shown in Figure A2 [41].

Table 12 Annual Probability of Scour Failure

Scour Vulnerability *(from Table 14)*	Overtopping Frequency *(from Table 13)*			
	Remote (R)	*Slight (S)*	*Occasional (O)*	*Frequent (F)*
(0) Failed	1	1	1	1
(1) Imminent failure	0.01	0.01	0.01	0.01
(2) Critical scour	0.005	0.006	0.008	0.009
(3) Serious scour	0.0011	0.0013	0.0016	0.002
(4) Advanced scour	0.0004	0.0005	0.0006	0.0007
(5) Minor scour	0.000007	0.000008	0.00004	0.00007
(6) Minor deterioration	0.00018	0.00025	0.0004	0.0005
(7) Good condition	0.00018	0.00025	0.0004	0.0005
(8) Very good condition	0.000004	0.000005	0.00002	0.00004
(9) Excellent condition	0.0000025	0.000003	0.000004	0.000007

Figure A2. Table of scour failure probabilities from NCHRP 107 [41].

Although using specific POF values, it is clear that these values are, at best, estimates developed through expert elicitation and engineering judgment. In effect, they are qualitative values given specific numerical values. It is interesting to note the use of quantitative risk assessment based on sparse POF data, which is similar to the situation for bridge inspection. This approach was carefully considered by the research team.

Ultimately, it was decided that given the large number of potential variables for the various elements on a bridge, and the potential for specific POFs to not match the appropriate conditional probabilities for a given bridge, that a more qualitative scale and a more detailed process for estimating the POF was required. Additionally, it was felt that the utilization of specific values for the POF suggested a level of precision that simply does not exist at this time. However, in the future, this process could provide guidance to a bridge owner who wants to develop quantitative probabilistic analysis for a specific population of bridges.

A.3.2.2.2.2 International Risk-Based Assessment of Bridge Inspection

The current State of the Practice for risk or reliability-based bridge inspection practices on the international stage was also explored through the literature search. It was found that both Germany and Japan were initiating projects to explore bridge inspection based on reliability concepts. In Japan, there was a project initiating to rationalize inspection frequencies, however, there were no reports available at the time, but it is interesting to review the response of Masahiro Shirato [shirato@pwri.go.jp] of the Public Works Research Administration (PWRI) regarding the status of efforts in Japan:

> *We have just started a project to rationalize inspection frequencies (intervals) and inspection items for road bridges. We are also seeking a way to set different required performance criteria for bridges that may change with the importance and designated function of route such as emergency transportation routes, so that the inspection frequencies and items may also change with them.*
>
> *Although a thorough inspection is needed to predict the deterioration rate and life cycle cost for individual bridge, such a program might not need to be implemented for all bridges but all bridges should be at least checked if there is no expectation for the fatal failure. The frequency and items of inspection should be determined based on scientific data for both needs, respectively.*
>
> *Until 2003, bridge inspection was not mandatory in Japan, and some owners inspected their bridges at a typical inspection interval of once in 10 years. Because bridge stocks constructed during the rapid economic period of 1960s to 1970s will become older than 50 years soon and many instances of damage have already appeared which strongly influence bridge strength, the government issued the directive of bridge inspection in 2003 and an every-five-year visual inspection is now mandatory for the bridges on national essential highways administrated by the government. Accordingly, now we have just completed the inspection of every*

government's bridge for the first time.

NILIM keeps tracking down and analyzing inspection results on the national inspection data in terms of surrounding environment, age, daily traffic, structural features, position of individual structural members, design specifications, construction methods, etc with some consideration of reliability and probabilistic concepts. However, it is well known that there is limit of hands-on visual inspection to find out serious damage that can lead to bridge failure. There is no effective methodology to estimate the remaining strength based on visual observations, because the uncertain loss of sectional areas is usually hidden under the multi paint layers or inside concrete. CAESAR is in the middle of seeking what we can do in research to resolve this issue and have just kicked off new projects of autopsy survey using decommissioned bridges.

We are happy to share our efforts with US bridge administrative or research communities, but so far both NILIM and CAESAR have yet to publish any reports on the topics above.

In Germany, there was also a project initiated to develop risk/reliability-based inspection practices at the University of Munich. However, similar to Japan, no reports were available.

In Denmark, visits with officials by the research team, and personal communication with Arne Henrikson of the Danish Road Directorate had indicated that risk analysis is an implicit part of their bridge inspection system, and that this characteristic was found in several countries during the AASHTO/FHWA scanning tour conducted in 2007 [42]. That is, the frequency of future inspections is determined based on the present condition of the bridge, on damage modes that are expected to occur, and on engineering judgment regarding the need for future inspections.

The text below is from email communication with Mr. Henriksen of Denmark which provides a concise summary of their approach regarding risk analysis within their inspection program. It should be noted in the text, that the routine inspections occur one or two times a year, and are conducted by transportation employees. It is our understanding that these inspections are relatively limited in scope and documentation. The principle inspections are more in-depth, and occur at intervals of one to six years, with an average of 5.5 years, according to Henriksen.

Routine inspection: A quick visual inspection, minor work up to 20.000 US will be reported and carried out later the same year (deposits, suddenly emerging erosion minor concrete repair etc.) The inspector has to report to the principal inspector immediately if he finds some unforeseen on a bridge. He has access to all former inspections. In Denmark and many other countries this might happen once for each 200 to 500 bridges.

Principal inspection: Supplementary to the routine inspection, the inspector is also often responsible for all repair work above 20.000 US. Beside the inspection training, we obtain very skilled personnel that understand both deterioration processes and repair methods. Thus we have skilled personal that might estimate inspection frequencies for more than 95% of our bridges. The remaining, less than 5% ? (We are very close to a major repair or he is not able to evaluate the deterioration or damage) Here he will recommend a special inspection.

Special inspection (as you saw in Copenhagen) will be carried out according to the needs.

- *The final report will contain a*
    - *Recommendation of principal inspection intervals*
    - *Repair proposal, normally more strategies*
    - *Budget*

On top of this we sometimes select one type of bridges or one type of bridge elements in a selected area for special inspection

In this way we feel, that we obtain most value for our money, we use most of our efforts on the most critical / costly bridges

Risk-based analysis of bridge items were also found in England, where the Highways Agency has developed interim advice notes for bridge piers and vehicle parapets based on risk analysis. For the analysis of parapets, a detailed risk-assessment procedure has been developed based on the ALARP principles. ALARP stands for "As Low As Reasonably Practical," and is a philosophical framework for managing risk that originated in the nuclear industry as a method for ranking and prioritizing responses to risk. There is a level of risk considered intolerable to individuals and society, and similarly a level of risk considered to be broadly acceptable.

If the risk falls into the tolerable level between these two levels, it is required that the risk be reduced to a level which is as low as reasonably practicable provided that the cost or effort required to reduce the risk in not grossly disproportionate to the benefits (the ALARP principle). Using this framework, a methodology has been developed for assessing the risk presented by bridge parapets to consider safety and upgrading needs. The methodology focuses on a series of common characteristics of parapets and the roadway designs to provide a tool for engineers.

A number of international academic references relating to the reliability assessment of bridge elements were also found. These included detailed analysis of prestressed and reinforced concrete bridges [14, 15, 17]. Specifically, the references include detailed analysis of certain potential failure modes, such as corrosion of reinforcing steel, to estimate reliabilities and to assess risk based on economic factors. Among the most useful data from such analysis is the prioritization of the parameters, such as concrete cover, rebar coatings, etc. that affect the time-dependant deterioration of concrete structures.

Detailed risk analysis in regard to multi-hazards have also been reported, although this analysis is more focused on the management of systems of bridges [43]. Deterioration modeling for the purposes of assessing corrosion in reinforced concrete is compared with PONTIS modeling by Roelfstra [44].

A.3.2.2.2.3 Vulnerability Analysis

To develop a rational methodology for determining the appropriate inspection intervals for highway bridges, it may be useful to examine various vulnerability analyses that have been developed by highway agencies to prioritize hazards to bridges such as scour. New York State in particular has developed several vulnerability guides to assess the relative vulnerability of bridges. These vulnerability assessments are essentially relative risk assessments based on qualitative descriptions of the likelihood of various damages affecting a structure. Vulnerability assessments for terrorist attacks on bridges have also been developed, and these vulnerability assessments are briefly described below.

A.3.2.2.2.4 NYSDOT Vulnerability Manuals

To address needs for managing their bridge inventory, NYSDOT has developed a series of vulnerability rating manuals that provide qualitative assessments of scour, overload, concrete details, steel details and seismic effects. These manuals are used within their bridge safety assurance (BSA) program to supplement condition-based bridge evaluations (inspections) by assessing and rating the degree of risk that is associated with certain design details and circumstances. The program evaluates a bridge by using current design practices as a reference, rather than evaluating a bridge based on the current condition of elements, as is done during a typical inspection. The process is a qualitative risk assessment process based on engineering knowledge.

The analysis procedure results in a numerical ranking for a structure that is then used to determine the relative vulnerability class for the structure that represents the likelihood of failure (probability of failure). This ranking is qualitative, resulting in a ranking or either high, medium or low. This likelihood score is then combined, in this case by summation, to the ranking score for the consequences of the failure. The consequence in this case is a combination or ratings for the failure type (catastrophic, partial collapse or structural damage) and an exposure score that represents the importance of the structure to the bridge inventory.

The process used in the NYSDOT vulnerability assessment is very similar in some ways to the overall bridge inspection methodology suggested through this research. The method capitalizes on the engineering knowledge and experience of the evaluators, combined with systematic analysis of the attributes for a given bridge to determine the likelihood of failure for the given failure mode. This is then combined with a factor to account for the consequences of that failure, which are a combination of the failure effects (catastrophic failure, etc.) and the importance of the bridge in terms of ADT and the highway system.

A.3.2.2.2.5 Vulnerability Analysis for Terrorist Attacks

A qualitative methodology for assessment of risk from terrorist attack has been developed by Ray [45]. The methodology developed ranks bridge components according to relative potential for a terrorist attack and the associated consequence from the attack. The method is based on factors such as the component's importance to the overall structure stability, the component's exposure to attack, and its resistance.

The methodology, which has been broadly accepted as an effective and practical tool for evaluating terrorism threats and for prioritizing mitigation measures, identifies component-specific risk factors and modifying attributes which adjust those factors. The methodology developed is very similar to that proposed by this research, although more narrowly focused. The method is quantitative in that is assigns specific relative values for risk factors. However, in general these factors are qualitative, assigning values based on qualitative descriptions such as high, medium and low.

A.3.2.3 Risk-Based Inspection Practices in Other Industries

A.3.2.3.1 American Petroleum Institute (API) 580

An important reference for the proposed inspection planning approach is the American Petroleum Institute's API 580, "Risk-Based Inspection Planning: Recommended Practice 580" [27]. The purpose of API 580 is to provide plant operators with the general methodology necessary to develop their own RBI programs. This document represents an accepted industrial recommended practice for the application of risk analysis to inspection planning, in this case for fixed equipment and piping in the hydrocarbon and chemical process industries. The document covers topics ranging from the definition of risk, risk mitigation and RBI planning.

API 580 defines risk as the combination of the probability of some event occurring during a time period of interest, and the consequences associated with the event. Note that likelihood is sometimes used as a synonym for probability. The general equation is written as:

$$Risk = Probability\ x\ Consequence$$

In this equation, probability is the likelihood of a certain event occurring, such as the loss of containment of a hazardous material due to a leaking pipe, and the consequence is the outcome associated with that event occurring. The consequences may be expressed in a variety of ways; for API applications, the consequences are typically based on the cost of the event or on some measure of the environmental impact. The consequence may also be expressed as a measure of the area affected, such as the area over which a certain type of spill progresses. Consequences may also be expressed in terms of the impact of the event on safety.

Risk, probability, and consequence are expressed either in qualitative terms or in quantitative terms. If using a qualitative approach, risk parameters are typically defined in categories such as low, medium and high. The qualitative process depends primarily on the knowledge of experienced personnel and may involve limited data gathering, depending on the scope of the program.

A quantitative approach ranks risk by a discrete, calculated number. This process relies more on extensive data gathering and logic analysis through event trees or fault trees, and may involve probabilistic analysis and/or structural reliability calculations. The recommended practice indicates that only certain components under analysis have the level of data gathered to support a quantitative calculation of the probability of failure. For many components, insufficient historical data exists on which to base such a calculation, and/or design characteristics are unique such that previous history with components of similar design may not exist.

Additionally, the cost and resources required to complete a quantitative analysis may not be justified based on the consequence and likelihood of failure. In these cases, qualitative approaches based on expert opinion are used to estimate the probability of failure for a given component. Expert opinions are typically assembled from expert groups or panels which include plant operators themselves, who have the most in-depth knowledge of the operating conditions at the plant and have past experiences with similar components within the specific plant environment. In most cases, risk analysis is semi-quantitative, a mix of qualitative and quantitative analysis.

A.3.2.3.2 American Petroleum Institute (API) 581

The recommended practice API 580 is supplemented with API 581, "Risk Based Inspection Technology." API Recommended Practice 581, which is an application of API 580, provides quantitative procedures to establish a RBI program for fixed, pressurized equipment. For the API 581 approach, failure is typically defined as a loss of containment resulting in leakage or rupture. The practice extends over 500 double-sided pages of detail on how to evaluate risk for specific components of pressurized equipment, such as pressure relief devices, atmospheric storage tanks, etc.

A key element of this risk-based analysis system is to determine the probability or likelihood of a certain event occurring. This is typically based on some measure of the probability of failure resulting from deterioration modes that may occur. The general model for probability of failure used in API 581 is defined as:

$$P_f = gff \cdot D_f(t) \cdot F_{MS}$$

Where *gff* is a generic failure rate, $D_f(t)$ is a situation dependent damage factor and F_{MS} is a management systems factor. The generic failure frequency, *gff*, is based on the refining and petrochemical industry's failure data, and can be thought of as a rough estimate of the anticipated performance for a certain type of component. Adjustment factors are applied to this generic failure frequency to reflect departures from the industry data to account for damage mechanisms specific to the operating environment and to account for the reliability management practices of the plant.

The management systems factor, F_{MS} , accounts for the efficacy of the plant's management practices. The value of F_{MS} is derived from an evaluation of a facility's risk management system, obtained through questionnaires administered to plant operators. It accounts for the probability that damage will be discovered in time to prevent a loss of containment, based on a general assessment of the management system utilized at the specific plant in which the analysis is being conducted.

Damage factors, $D_f(t)$, are determined based on the applicable damage mechanisms (local and general corrosion, cracking, creep, etc.) relevant to the materials of construction and the process service, the physical condition of the component, and the inspection technique used to quantify the damage. This factor adjusts the generic failure frequency to apply to a specific piece of equipment under a specific set of operating parameters.

In general, the procedures utilize the industrial knowledge base to generalize adjustments to a generic failure frequency based on specific conditions of a specific piece of equipment. To illustrate the characteristics of the damage factors, the damage factor for thinning damage will be used as an example. This refers to wall thinning in a pressurized pipe. The equation for this damage factor is as follows:

$$D_f^{thin} = \frac{D_{fB}^{thin} \cdot F_{IP} \cdot F_{DL} \cdot F_{WD} \cdot F_{AM} \cdot F_{SM}}{F_{OM}}$$

Where the F_{xy} variables shown in the equation are a series of estimates of various attributes of piping systems that are known to affect the corrosion behavior of piping. For example:

D_{fb}^{thin} is the base value damage factor for thinning,

F_{IP} is the damage factor for injection points which ranges from one to three; three if injection points are present, one if special inspection techniques for the injection points are used or there are not injection points,

F_{WD} is the damage factor estimate for welded construction (one for welded construction), otherwise (i.e. riveted construction) ten

Etc.

The details of each factor are not as important here as the overall concept of using a series of attributes or characteristics of the system to estimate the damage factor, as a means of estimating the associated failure probabilities of failure. Of particular note is F_{OM}, the adjustment factor for on-line monitoring, which is a denominator in the equation and ranges in value from one to twenty. In other words, for two identical systems, the damage factor for a system with appropriate on-line monitoring may be 1/20th that of an unmonitored system.

These concepts are relevant to highway bridges because they illustrate how another industry has addressed the question of how to utilize failure data from experience, for example, the collective experience of states with the service life of bridge decks, and utilize that data for a specific set of bridge decks that have a specific set of operating conditions. Starting with generic data based on industry experience, factors are utilized that broadly characterize the effect of various attributes or characteristics on the relative likelihood that a particular component will have a performance that can be expected to be greater than, or less than, some generic value.

The same concept could be adopted for highway bridges, where generic failure frequencies for specific bridge elements such as beams, girders, decks and substructures can easily be estimated, given their typically long lives and relatively slow modes of deterioration (corrosion, primarily). Such generic industry experience with bridge elements needs to be adjusted to account for the specific operating conditions of a specific bridge; the challenge here is to develop a methodology to implement this concept for highway bridges.

API procedures are mature and have been developed over several decades, as such detailed analysis procedures for a variety of components have been developed and documented in API 581. Such procedures allow for more precision in the risk analysis. For the case of highway bridges, where risk analysis for inspection planning is only initiating, developing such a detailed analysis would prove to be a significant barrier due to the high cost and extensive time required for implementation. Starting with a more qualitative process, as proposed by this research, will allow for maturation toward such detailed methodologies while still providing an accurate, practical methodology.

A.3.2.3.3 American Society of Mechanical Engineers

The American Society of Mechanical Engineers (ASME) also has well-developed documentation on the use of risk based inspection methods. An industrial guideline was developed in 1992 on behalf of the Nuclear Regulatory Commission in Washington, D.C. These guidelines present a four part inspection process that is multi-disciplinary in nature. The main parts of the program include system definition, qualitative risk assessment, quantitative risk analysis using Failure Modes, Effects and Criticality Analysis (FMECA), and development of an inspection program using risk analysis. The methodology developed is very similar to that described in API 580.

A standard entitled "Inspection Planning Using Risk-Based Methods" was also published in 2007 [26]. This ASME standard builds on the concepts of the 1992 guidelines, but the standard specifically applies to fixed, pressurized equipment, not nuclear power plants. The standard explicitly states that it is based on API 580, but has been enhanced to be applicable to a broader spectrum of industries. The standard provides a general approach to RBI planning, covering all critical aspects of the process.

A.3.2.3.4 The Offshore Industry

A guide entitled "Surveys Using Risk-Based Inspection for the Offshore Industry" has been developed by the American Bureau of Shipping [20]. This document is essentially similar to those developed by API and ASME, describing the approach and methodologies to be used when developing inspection frequencies and scopes based on risk analysis. In this case, RBI is applied to static pressure retaining equipment and offshore floating and fixed-base platforms. It is clear from these industrial standards that RBI is an accepted and well-developed approach for inspection planning that is sufficiently flexible to be applied across a broad spectrum of industries. In fact, it could be concluded from these standards and guidelines that RBI is increasingly the standard methodology for inspection planning, and that the calendar-based methods applied to highway bridges lags significantly behind other relevant industries.

Each of these standards, or guidelines, recognizes the value of qualitative risk assessments, and indicates that most developed approaches are semi-quantitative, including both qualitative and quantitative components. It is apparent from these documents, and other literature reflecting implementation of these methods, that qualitative methodologies are very common, and purely quantitative assessments are relatively limited to a few situations, such as fatigue cracking, where adequate models exist and their application has been validated over time. For most situations, methods involve somewhat qualitative assessments of the likelihood of events being high, medium of low and the assignment of numerical values to these estimates creates a quantitative framework for systematic calculation on a numerical scale. Fundamentally, these are engineering judgments based on experience and expert knowledge.

A.3.2.4 Risk Matrices

Many risk-based procedures utilize a risk matrix to visualize and analyze risk for the purpose of indentifying the most at-risk components, assessing the risk relative to subjective risk thresholds, and planning inspection and risk mitigation strategies (i.e. repairs or retrofits to reduce risk levels) [26, 27, 29]. Risk thresholds are inherently subjective, because the level of acceptable risk is a combination of societal opinion, cost implication and engineering decision-making. As such, standards and practices such as API 580 develop methods for assessing risk, but require owners to determine their own policies regarding the acceptable level of risk given their industrial and societal norms.

Regardless of how acceptable risk thresholds are developed, an effective way to represent risk is to create a risk matrix. A risk matrix helps to visualize the assigned values for probability of failure and consequences of failure, whether determined through a qualitative assessment or through a quantitative assessment. For example, API has used a five point scale with recommended probability and consequence values to aid in the creation of a risk matrix.

Probability has been arranged into categories ranging from one to five with five being the most severe. Likewise, consequence is arranged into five categories, ranging from A to E with E being the most severe. Figure A3 shows a suggested risk matrix from API 581 [27]. This risk matrix is color-coded to indicate the different levels of risk, and considers the risk levels to be non-symmetric. This is because consequence may be given a greater influence on risk than probability, based on subjective judgment. Items towards the top-right corner represent the greatest risk, and should have the highest priority for inspection; items in the lower left corner are low risk and consequently should have the lowest priority.

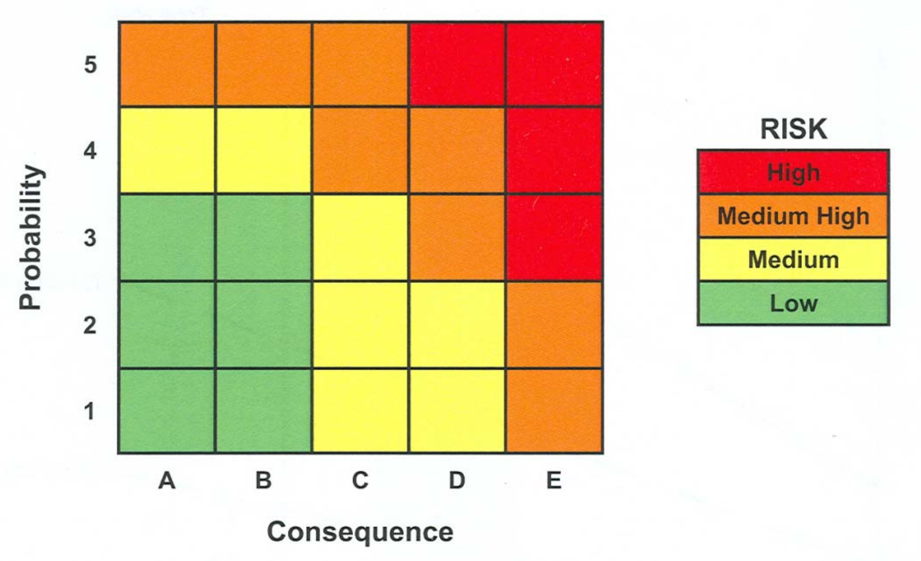

Figure A3. API risk matrix showing color-coded risk acceptance levels [27].

Such a risk matrix can be applied to either qualitative or quantitative risk assessments. For more quantitative assessments, the axes may be continuous rather than delineated into a selected number of steps. The results of a risk analysis can also be plotted onto a two dimensional scale, for cases such as the assessment of nuclear facilities. These axes may have log-normal scales to represent the levels of magnitude of probability of failure and consequence.

An example from the ASME guidelines [10] is shown in Figure A4, which illustrates the plotting of risk calculations along with iso-risk lines for categorization. These iso-risk lines represent equal risk values, assuming consequences and probabilities of failure are given equal weight. For the example illustrated in Figure A4, the high risk areas (A in the figure) are separated from moderate risk (B) and low risk (C). Obviously, the determination of exactly where the divisions between high, moderate and low reside is a subjective decision that depends on the management policies of a specific facility.

An alternative policy may regard the consequence of failure as the primary driver for risk, regardless of the probability of the failure actually occurring. In this case, divisions would be vertical lines on the plot, where every component or machine for which the consequence exceeds a certain level is considered high risk, and receives the focus of attention. In reality for many applications, the acceptable risk values are some combination of these two approaches, with consequence receiving somewhat more weight than probabilities, but still regarding extremely unlikely events as low risk, even if their consequences are high.

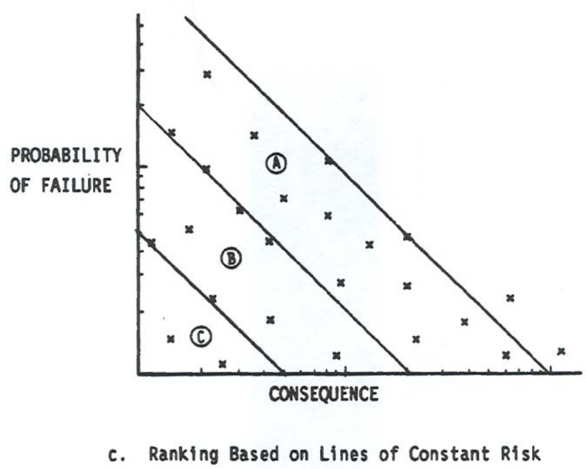

Figure A4. Iso-risk lines applied to a quantitative risk plot [10].

A.3.3 Probability and Consequence Rankings

An important aspect of the system to be developed for highway bridges is the consideration of how to establish probability and consequence rankings. This section will discuss the scaling systems suggested in several industrial standards that were used in developing the proposed ranking system described by this research. The section begins with a discussion of qualitative vs. quantitative data, and then discusses some common qualitative scales that are found across the industrial landscape.

A.3.3.1 Quantitative vs. Qualitative Analysis

Industrial standards for risk assessment recognize both quantitative and qualitative methods for risk assessment. Qualitative data is typically information developed from past experience, expertise and engineering judgment. Inputs are often expressed in data ranges instead of discrete values, and may be given in qualitative terms such as high, medium and low (although numerical values may be associated with these levels) [26, 27, 29].

Quantitative data is data which has been determined through specific numerical models, databases of failure rates, or past performance data such as deterioration rate models. This data is typically more in-depth and detailed than qualitative data. The data may be treated probabilistically to develop a uniform model that provides qualitative and quantitative insight about the level of risk. 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 is frequently incomplete or inaccurate, and in some cases can provide ineffective estimates of future performance [28]. 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 on specific components is sparse. Qualitative data enables the completion of risk assessment in the absence of detailed quantitative data. This qualitative data is informed by quantitative data when and where it is available, forming a continuum of data as shown in Figure A5 [29].

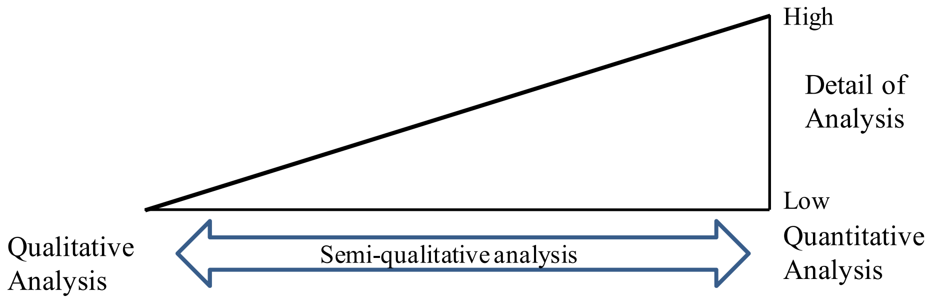

Figure A5. Continuum of RBI approaches from qualitative to quantitative [29].

The accuracy of results from a qualitative assessment depends on background and expertise of the analyst [26]. With nearly 40 years of experience in the inspection and management of highway bridges, the expertise of bridge owners can provide a valuable tool for risk assessment; and in fact, risk assessment has been ongoing in an ad-hoc fashion. On the other hand, certain quantitative data, such as deterioration rate information determined from NBI or BMS data, can provide supporting rationale for decision-making, if handled appropriately.

Risk presented as a precise numeric value (quantitative analysis) implies a higher level of accuracy when compared to qualitative analysis. API 580 states that "The implied linkage of precision and accuracy may not exist because of the elements of uncertainty that is inherent with probabilities and consequences." It is the quality of the data that is most important to support risk-based assessments, and the fact that data is quantitative does not necessarily mean that it is more accurate. It is important to keep in mind that the goal of the proposed methodology is to ensure accuracy, not precision. A very detailed calculation, if based on bad data, can be "precisely wrong".

Difficulty in effectively representing past experience, expert knowledge and bridge–specific conditions can result in quantitative data that is biased and inaccurate, or whose applicability to a specific situation is unknown due to a complex array of assumptions utilized in developing the data. In practice, there can be extraneous factors that could affect the predicted damage and magnitude of failure, which cannot be taken into account in a single fixed model. On the other hand, qualitative data based on the experience of bridge owners regarding the performance of specific bridge designs within specific environments can be an invaluable tool for accurate assessment of risk and the prediction of future performance.

Careful elicitation of expert knowledge from those most familiar with the operating environments, historical performance characteristics within those environments, and the expected future performance, can provide the basis for risk assessment and has been recognized as an effective means of establishing risk [26]. Formal methods for eliciting expert opinion for the purpose of risk assessment are included in the industrial guidelines and in the literature [30, 31].

To develop an effective and implementable system for reliability–based inspection practices, it is necessary to recognize the value of both qualitative and quantitative data. The proposed method will utilize a semi-qualitative approach that applies quantitative data, where available or developed, to create a semi-quantitative risk assessment that accurately determines the risk for specific bridges for the purposes of estimating a required inspection frequency. The procedures for estimating the critical risk factors developed through the research will provide a rational framework for utilizing both qualitative and quantitative data in a technical decision-making process that is structured to ensure uniform and reliable outcomes of the analysis process.

A.3.3.1.1 Qualitative Rank Scales

To institute a RBI system, a key decision to be made is how to establish a qualitative or semi-quantitative scale which can provide realistic estimates of occurrence and consequence. To develop a RBI practice based on semi-quantitative methods, it will be necessary to select a rating scale by which to evaluate the likelihood of deterioration modes and their associated consequences. There are a variety of scales utilized in other industries that provide useful guidance. Several of these scales have been compiled below to provide an overview of how other industries have approached this issue. The tables below show some possible probability of failure estimates for both qualitative and quantitative methodologies.

The ASME guidelines suggest a first-level qualitative analysis can be achieved using a simple three level scale as shown in Table A1 [26]. An estimate of the annual probability of failure rate for the qualitative ranking is also provided. In this context (for this industry) a high probability of failure is intended to represent failure rates on the order of a 0.01 or 1 in 100, while a moderate probability is intended to cover two orders of magnitude from 0.01 to 0.0001, with a low probably being less than 0.0001.

In a purely qualitative analysis, the numerical representation would not be estimated. 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 quantitative to qualitative rankings. ASME also suggests that a six-level scale may be appropriate, as shown in Table A2, which provides slightly finer gradations in the ranking scale.

Again, qualitative ranking are mapped to quantitative values that describe order of magnitude estimates of the probability of failure. In practical applications, even if quantitative methods are used, the estimated probabilities of failure are typically considered to be, at best, order of magnitude estimates, due to the inherent variation and uncertainty in engineered systems.

Table A1. ASME probability of failure rankings using a three level scale [26].

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)

Table A2. AMSE probability of failure ranking using a six level scale [26].

Possible Qualitative Rank	Annual Failure Probability
Remote	<0.00001 (1/100,000)
Very Low	0.00001-0.0001 (1/100,000 – 1/10,000)
Low	0.0001-0.001 (1/10,000 – 1/1000)
Moderate	0.001-0.01 (1/1000 – 1/100)
High	0.01 – 0.1 (1/100-1/10)
Very High	>0.1 (1/10)

Table A3 shows a typical ranking scale utilized by NASA for reliability-centered maintenance activities that are used to assess and prioritize facilities at the NASA centers [32]. These ranking scales, which have been selected based on engineering judgment for the specific application, are tied to both a quantitative estimate of the ranking values, and enhanced with commentary intended to lead the evaluator in selecting the appropriate level for his or her given situation.

This ranking scale is used as a tool, combined with the qualitative rankings of criticality/severity for the consequence of failure as shown in Table A4, to assess and prioritize maintenance for facilities and ancillary equipment based on risk/reliability analysis. These risk scales are implemented within a Failure Modes and Effects Analysis that assesses each system, subsystem and component within the scope of the analysis. This assessment comprises listing the individual component or system, identifying the credible failure modes and resulting failure effects (based on engineering judgment and experience) for that component or system, and assigning the appropriate probability of occurrence and probability of failure rank.

Table A3. Probability of occurrence ranking scale from NASA's reliability-centered maintenance guide [32].

Ranking	Effect	Comment
1	1/10,000	Remote probability of occurrence; unreasonable to expect failure to occur
2	1/5,000	Low failure rate. Similar to past design that has, in the past, had low failure rates for given volume/loads
3	1/2,000	Low failure rate. Similar to past design that has, in the past, had low failure rates for given volume/loads
4	1/1,000	Occasional failure rate. Similar to past design

		that has, in the past, had similar failure rates for given volume/loads
5	1/500	Moderate failure rate. Similar to past design that has, in the past, had moderate failure rates for given volume/loads
6	1/200	Moderate to high failure rate. Similar to past design that has, in the past had moderate failure rates for given volume/loads
7	1/100	High failure rate. Similar to past design that has, in the past, had high failure rates that has caused problems.
8	1/50	High failure rate. Similar to past design that has, in the past, had high failure rates that has caused problems
9	1/20	Very high failure rate. Almost certain to cause problems.
10	1/10+	Very high failure rate. Almost certain to cause problems.

Table A4. Criticality/severity categories from NASA's reliability-centered maintenance guide [32].

Ranking	Effect	Comment
1	None	No reason to expect failure to have any effect on Safety, Health Environment or Mission.
2	Very Low	Minor disruption to facility function. Repair to failure can be accomplished during trouble call
3	Low	Minor disruption to facility function. Repair to failure may be longer than trouble call but does not delay Mission.
4	Low to Moderate	Moderate disruption to facility function. Some portion of Mission may need to be reworked or process delayed.
5	Moderate	Moderate disruption to facility function. 100% of Mission may need to be reworked or mission delayed
6	Moderate to High	Moderate disruption to facility function. Some portion of Mission in lost. Significant delay in

		restoring function.
7	High	High disruption to facility function. Some portion of Mission is lost. Significant delay in restoring function.
8	Very High	High disruption to facility function. Al of Mission is lost. Significant delay in restoring function.
9	Hazard	Potential Safety, Health or Environmental issue. Failure will occur with warning.
10	Hazard	Potential Safety, Health or Environmental issue. Failure will occur without warning..

A.3.3.1.2 Assessing Inspection Effectiveness

One question to be considered is the assessment of inspection effectiveness, and how inspection effectiveness fits within a risk or reliability based program for planning inspections. On a conceptual level, effective inspections have the effect of reducing the probability of failure from a given deterioration mode. This is because effective inspections reduce the uncertainty in the current condition of an element or component, allowing for more accurate estimates of the likelihood of damage in the future.

If the current condition of the element is unknown, the estimate of the likelihood of failure occurring in the next time interval must necessarily make the conservative assumption that deterioration may already be occurring, adopting a “worst case” scenario in many cases. If effective inspections have been conducted, such that the current condition of the element is well known, then the estimate of the likelihood of failure in the next time interval has a lower uncertainty and consequently a smaller estimate of the likelihood of failure occurring (assuming no damage is present) [26]. Additionally, effective inspections increase the likelihood that the deterioration will be detected such that remediation measures can be implemented. This will further reduce the likelihood of failure by repairing the damage which may exist, returning the element to an undeteriorated state. As such, the likelihood of failure across the next time interval is reduced [29].

For the most quantitative of risk assessments, such as those used for assessing cracking in nuclear power plants or oil and gas facilities, the specific inspection effectiveness may be quantified using probability of detection (POD) and reliability analysis of specific inspection techniques. This may be justified based on the significant risk associated with those facilities, including the both the high economic and environmental consequences of various failure modes.

However, for more general assessments of risk, the effectiveness of inspections is more likely used to rank various inspection approaches on a relative scale using engineering judgment. Here, POD studies may still be used to provide data input for the rankings. For example, API has created a five category rating system used for several components described in API 581 [27]. This system ranges from A to E, with A being "highly effective" and E being "ineffective." Typically, a suggested mode of inspection is provided as an example of the highly effective or ineffective inspection technique. For example, for the general thinning of pipes, Table A5 is provided to describe the general categories of inspection effectiveness:

Table A5. API 581 descriptions of inspection effectiveness [27].

Inspection Category	Inspection Effectiveness category	Intrusive Inspection Example	Non-intrusive Inspection Example
A	Highly Effective	50 to 100% examination of the surface (partial internals removed), and accompanied	50 to 100% ultrasonic scanning coverage (automated or manual)

		with thickness measurements	or profile radiography
B	Usually Effective	Nominally 20% examination (no internals removed) and spot external ultrasonic thickness measurements	Nominally 20% ultrasonic scanning coverage (automated or manual), or profile radiography, or external spot thickness (statistically validated)
C	Fairly Effective	Visual examination with thickness measurements	2 to 3 % examination, spot external ultrasonic thickness measurements, and little or no internal visual examination
D	Poorly Effective	Visual examination	Several thickness measurement, and a documented inspection planning system
E	Ineffective	No inspection	Several thickness measurements taken only externally, and a poorly documented inspection planning system

What is important to note in Table A5 is that quantitative values for the exact effectiveness of the various inspection strategies are not provided. Rather, the inspection approaches are generalized based on experience, the general body of industry knowledge, and assumptions regarding the effectiveness of the method for a specific situation. Based on the effectiveness of the inspection, the damage factor may be reduced, which will decrease the calculated risk of an item.

This does not reduce the inherent risk in the piece of equipment. It only provides knowledge of the current state of damage which reduces the uncertainty in the probability of failure. It is also important to note that these are not required inspection approaches, but rather examples. The owner could substitute equivalent methodologies for inspection, if these methods are believed to have equivalent effectiveness.

A.4 CONCLUSIONS FROM THE LITERATURE REVIEW

From the preceding sources, several conclusions have been drawn from the literature search. First, structural reliability-based methods for inspection planning are not sufficiently developed at this time to account for the serviceability requirements of bridge inspection planning. Second, risk-based methods have been well developed in other industries and can be adapted for the use of bridge inspection planning. Finally, the use of qualitative to semi-quantitative data, such as that gathered through engineering judgment and previous experience, are accepted means of estimating both probability of failure and consequence of failure.

Based on these conclusions, this research effort has pursued a path to develop a semi-quantitative, risk-based framework for bridge inspection planning. This risk-based method will incorporate the experience and the knowledge of bridge engineers and inspectors in a framework which would allow, in the future, for detailed reliability analyses to be incorporated within a larger framework that considers both the probability of failure (for both serviceability and strength limit states), and the importance of that failure in terms of bridge safety.

A.5 REFERENCES

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A.6 ANNOTATED BIBLIOGRAPHY

This section will present an annotated bibliography of the documents which were discovered throughout the course of this literature search. This bibliography is intended to show documents which, while critical to this research program, may or may not have been presented in detail in this thesis. The documents presented have been broken down into general categories relating to the document's relevance such as applied risk-based inspection, deterioration, expert elicitation, etc.

Applied Risk-Based Inspection

1. Adey, B., R. Hajdin, et al. (2003). "Risk-based approach to the determination of optimal interventions for bridges affected by multiple hazards." Engineering Structures **25**(7): 903-912.

In this article, the authors present a methodology useful for determining the optimal intervention for a bridge when affected by multiple hazards such as flooding and earthquakes. This is in contrast to conventional bridge management systems which are focused on the structural condition of deteriorating bridges with respect to traffic loads.

2. Antaki, G. A., T. M. Monahon, et al. (2005). "Risk-Based Inspection (RBI) of Steam Systems."ASME Conference Proceedings **2005**(41928): 65-71.

The authors describe the implementation of a risk-based inspection program in a large chemical process facility. They address the development of RBI matrices, likelihood attributes, consequence scores, and overall risk in terms of safety and cost.

The risk ranking is followed by inspection planning, which considers both location and technique.

3. ASME (1992). Risk-Based Inspection - Development of Guidelines v2 p1 Light Water Reactor (LWR) Nuclear Power Plant Components. New York, NY, ASME Technical Publishing Department.

This document is the second of a series developed by the ASME Research Task Force on Risk-Based Inspection Guidelines. This volume in particular is directed at the in-service inspection (ISI) of pressure boundary components of light water reactor nuclear power plants. In particular, pressurized water reactor (PWR) and boiling water reactor (BWR) systems.

4. ASME (1998). Risk-Based Inspection - Development of Guidelines v2 p2 Light Water Reactor(LWR) Nuclear Power Plant Components. New York, NY, ASME Technical Publishing Department.

This document is the fourth installment of the previously mentioned series on Risk-Based Inspection Guidelines. This volume contains several pieces of information such as: a risk-based inspection process for pressurized piping, the results from a pilot application for the Surry-1 and Millstone-3 plants, and the results from a structural reliability/risk analysis (SRRA).

5. Bertolini, M., M. Bevilacqua, et al. (2009). "Development of Risk-Based Inspection and Maintenance procedures for an oil refinery." Journal of Loss Prevention in the Process Industries **22**(2): 244-253.

This paper reports on the application of an RBI&M method to two stages in the maintenance of a refinery. The method has six modules: identification of the scope, functional analysis, risk assessment, risk evaluation, operation selection and planning, J-factor computation, and operational realization. This method incorporates expert elicitation and historical data.

6. Brickstad, B., A. Letzter, et al. (2002). Risk Based Inspection Pilot Study of Ignalina Nuclear Power Plant, Unit 2, American Society of Mechanical Engineers - ASME, New York (United States).

The authors describe the quantitative risk analysis of 1240 stainless steel welds within the Ignalina Nuclear Power Plant. The damage mechanism considered is intergranular stress corrosion cracking (*IGSCC*). Probabilistic fracture mechanics are used to quantify the failure probabilities.

7. Camburn, J. and K. Saunders (2006). Risk-Based Inspection of Relief Valves. 2006 ASME Pressure Vessels and Piping Division Conference, Vancouver, BC, Canada.

This paper discusses the RBI methodology developed for relief valves and the effects of overpressure on common pressurized plant equipment. The authors describe the application of their methodology to two plants, mainly in terms of how the inspection intervals were modified.

8. Chen, X., T. Yang, et al. (2006). "Application of Risk-Based Inspection in Safety Assessment of Pressure Equipment of Chinese Petrochemical Plants." ASME Conference Proceedings **2006**(47527): 541-549.

In this paper, the effect of a quantitative RBI program using Bureau Veritas's RB.eye software and database is described. This RBI program was applied to ten plants in the SINOPEC and PetroChina system. The authors present the highlights of their analysis along with several observed problems regarding implementation.

9. Choi, S.-C., S.-I. Han, et al. (2005). "Development and Application of Risk-Based Inspection for Refinery Unit." ASME Conference Proceedings **2005**(41928): 405-409.

This paper reports the methodology and the results obtained from an application of the KGS-RBI software program. This program was developed by the Korean Gas Safety Corporation in reference to the API and ASME codes.

10. Chien, C.-H., C.-H. Chen, et al. (2009). "A strategy for the risk-based inspection of pressure safety valves." Reliability Engineering & System Safety **94**(4): 810-818.

In this article, the authors present a RBI system based on the statistical analysis of as-received test data of the aging conditions for pressure safety valves (PSV) in lubricant process units. The relationship between aging conditions and the corresponding PSV parameters resulted in a semi-quantitative RBI process.

11. Geary, W. (2002). Risk Based Inspection - A Case Study Evaluation of Onshore Process Plant, Occupational Health & Safety Information Service.

The objective of this report is to examine the differences between seven independently created RBI programs. Each participant explains their methodology and how that methodology is implemented. Also, each participant is given four pieces of equipment and asked to conduct their own RBI assessment, the results of which are also compared.

12. Jovanovic, A. (2003). "Risk-based inspection and maintenance in power and process plants in Europe." Nuclear Engineering and Design **226**(2): 165-182.

This paper provides a review of the current practices in RBI/RBLM, primarily by comparing American and European approaches. The needs of industry and the relationship between RBI/RBLM and other approaches are discussed along with problems yet to be overcome.

13. Kaita, S., T. Shibasaki, et al. (2004). "Global Assessment System for Structural Safety in Large Size Storage Tanks." ASME Conference Proceedings **2004**(46857): 177-185.

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14. Khan, F. I. and M. R. Haddara (2004). "Risk-based maintenance of ethylene oxide production facilities." Journal of Hazardous Materials **108**(3): 147-159.

This report describes a RBM approach which integrates reliability and risk assessment to create an optimum maintenance schedule. A case study involving an ethylene oxide production facility is presented which includes a fault tree and a sensitivity analysis.

15. Krishnasamy, L., F. Khan, et al. (2005). "Development of a risk-based maintenance (RBM) strategy for a power-generating plant." Journal of Loss Prevention in the Process Industries **18**(2): 69-81.

In this paper, the authors describe the methodology behind their RBM program. The methodology has four modules: identification of the scope, risk assessment, risk evaluation, and maintenance planning. A case study from a power-generating plant is included to illustrate the methodology.

16. Lee, S.-M., Y.-S. Chang, et al. (2006). "Application of an Enhanced RBI Method for Petrochemical Equipments." Journal of Pressure Vessel Technology **128**(3): 445-453.

The objective of this report is to propose an enhanced RBI method which resolves the shortcomings of the API RBI approach. The main focus of the enhanced approach is to fully incorporate the characteristics of different materials. The proposed program has qualitative, semi-quantitative, and quantitative modules.

17. Peterson, R. (2004). Developing an efficient, predictive, risk based inspection/maintenance program for recovery and power boilers. American Society of Mechanical Engineers, Pressure Vessels and Piping Division (Publication) PVP, San Diego, CA.

This paper details a step by step approach for the RBI analysis of boilers. The main focus of the analysis is section loss in the metal shell. A risk matrix is provided where each block relates to an inspection interval. The intervals range from six months to four years.

18. Pugh, R., B. F. Gore, et al. (1991). Auxiliary feedwater system risk-based inspection guide for the Ginna Nuclear Power Plant**:** Medium: ED; Size: Pages: (30 p).

This document presents a compilation of auxiliary feedwater system failure data for the purposes of inspection planning. This data was obtained from PRAs and then reviewed to identify the root causes of component failure.

19. Roberts, I. P. (2002). Storage Tank Risk Based Inspection / Management. Aberdeen. AB11 6DJ. Scotland, Tischuk International Ltd.

The author explains the origins and basic approaches to risk-based inspection. He then explains Tischuk International and TR System's approach for the analysis of atmospheric storage tanks.

20. Sweet, L., L. C. Kaley, et al. (2001). "Risk based inspection prioritization applied to an ammonia plant." Ammonia Plant Safety and Related Facilities **41**: 147-156.

This paper presents a case study in the application of a modified API RBI program. The authors describe the RBI methodology which was applied to an ammonia plant. Pertinent results including forecasted risk matrices are included.

21. Topalis, P., G. F. Alajmi, et al. (2006). Implementation of an Integrated Risk Based Inspection (RBI) System in an Onshore Installation. EuroMaintenance 2006 – 3rd World Congress on Maintenance, Basel – Switzerland.

In this document, the authors present the implementation of an integrated RBI system to an onshore installation in the Middle East. The main focus of this system is a data management network which can coordinate inspections, exchange information with other plants, and maintain asset and inspection data.

22. Wong, S. M., J. C. Higgins, et al. (1995). Application of Risk-Based Methods to optimize inspection planning for regulatory activities at nuclear power plants. American Nuclear Society international topical conference on the safety of operating reactors, Seattle, WA.

This document discusses RBI methodologies created by the Brookhaven National Laboratory for use by nuclear power plants. The main focus is to combine information from Probabilistic Risk Assessments and Individual Plant Evaluations with information from operating experience.

23. Zhao, J. P. (2006). Risk-based inspection analysis for high pressure hydrogenation cracking unit. 2006 ASME Pressure Vessels and Piping Division Conference, Vancouver, BC, Canada, ASME.

This paper outlines the quantitative RBI method which was applied to an aromatic hydrocarbon factory. Based on API 581, a risk distribution including 553 items was obtained and used to develop an optimum inspection plan.

Dams & Levees

24. Andersen, G. R., L. E. Chouinard, et al. (1999). "Ranking Procedure on Maintenance Tasks for Monitoring of Embankment Dams." Journal of Geotechnical and Geoenvironmental Engineering **125**(4): 247-259.

In this document, the authors describe a multistep strategy for the prioritization of maintenance and repair funds. Elements are ranked according to their importance and then their condition in measured through field inspections. This process is based on expert elicitation and utilizes an interactive C++ program to assist in implementation.

25. Andersen, G. R., L. E. Chouinard, et al. (2001). "Risk Indexing Tool to Assist in Prioritizing Improvements to Embankment Dam Inventories." Journal of Geotechnical and Geoenvironmental Engineering **127**(4): 325-334.

This document describes a risk indexing tool which is used to prioritize maintenance activities for embankment dams, less than 100 feet tall, and with little or no performance history. The tool considers four main failure types, determined by expert elicitation, and utilizes checklists to guide onsite inspections.

26. Andersen, G. R., C. W. Cox, et al. (2001). "Prioritization of Ten Embankment Dams According to Physical Deficiencies." Journal of Geotechnical and Geoenvironmental Engineering **127**(4): 335-345.

This paper outlines the methodology and the results of a risk prioritization procedure applied to ten embankment dams managed by the Massachusetts Office of Dam Safety. Each dam is assigned a priority ranking which identifies potential deficiencies and then evaluates the overall importance of those deficiencies.

27. Andersen, G. R. and V. H. Torrey Iii (1995). "Function-Based Condition Indexing for Embankment Dams." Journal of Geotechnical Engineering **121**(8): 579-588.

This document presents a total-systems approach for the development of a condition-indexing system. This system defines the physical state of a structure in terms of a numerical index which can then be used to prioritize repair and maintenance activities. An application is given for embankment dams.

28. Bowles, D. S. (1999). Alamo Dam Demonstration Risk Assessment. Australian Committee on Large Dams Annual Meeting, Jindabyne, New South Wales, Australia.

This paper summarizes the results of a demonstration risk assessment conducted on the Alamo Dam as part of a Corp of Engineers research program. The existing dam and 19 risk reduction alternatives were evaluated for flood, earthquake, and normal operating conditions.

29. Bowles, D. S. (2000). Evaluation and Use of Risk Estimates in Dam Safety Decisionmaking. Ninth United Engineering Foundation Conference on Risk-Based Decisionmaking in Water Resources, Santa Barbara, California, USA, ASCE

This report discusses the limitations of simple threshold criteria and of estimates of cost-effectiveness used for ALARP (as low as reasonably practicable) considerations. There is also a discussion on the use of risk estimates to formulate and justify dam safety improvement programs.

30. Bowles, D. S. (2000). Advances in the Practice and Use of Portfolio Risk Assessment. Australian Committee on Large Dams Annual Meeting, Cairns, Australia.

In this paper, the author explores several issues related to the implementation of portfolio risk assessment. These issues include: advances in the state-of-the-practice, pitfalls and limitations, the use of PRA results, and the importance of targeted PRA.

31. Bowles, D. S., L. R. Anderson, et al. (1998). Portfolio Risk Assessment: A Tool for Dam Safety Risk Management. 1998 USCOLD Annual Lecture. Buffalo, New York, Utah State University.

The authors describe an approach to portfolio risk assessment which they have developed and applied to several dam portfolios. This approach is useful for prioritizing risk reduction measures and strengthening other aspects of a dam safety program.

32. Cecilio, C. B. (1989). "Private Sector Risk Analysis: Applied to Dam Safety." Journal of Management in Engineering **5**(4): 379-384.

At the time of this document, risk-analysis was slowly gaining acceptance in the decision process but was still difficult to apply. This paper discusses the method of risk analysis used in the private sector as it pertains to dam safety.

33. Ellis, H. L., C. B. Groves, et al. (2008). Rapid Levee Assessment for Reliability and Risk Analysis. GeoCongress 2008: Geosustainability and Geohazard Mitigation (GSP 178), New Orleans, Louisiana, ASCE.

This paper presents a review of levee failure causes and describes methods that quickly incorporate risk and reliability into the repair decision-making process. The methods include: rapid field assessment of condition, analysis of causes and mechanisms, and ranking by a first-order estimate of reliability.

34. Harrald, J. R., I. Renda-Tanali, et al. (2006). Review of Risk Based Prioritization/Decision Making Methodologies for Dams. U. A. C. o. Engineers, The George Washington University.

A deliverable for two USACE contracts, this document reviews eight methods currently used for dam risk analysis. For each approach, the methodology and how that methodology is implemented are explained. In addition, there is a review of several decision analysis techniques including decision tress and influence diagrams.

35. Hartford, D. N. D. and R. A. Stewart (2001). Dam Risk Management---A Discussion Paper on the Principles. World Water and Environmental Resources Congress 2001, Orlando, Florida, USA, ASCE.

This paper examines the fundamental principles of risk assessment and how they lead to decisions about management which could not be carried out in the past. A framework for managing risks is presented along with guidance for securing permits for its implementation.

36. Huang, Y. and X. Yu (2008). Risk Based Design of Levee System. GeoCongress 2008: Geosustainability and Geohazard Mitigation (GSP 178), New Orleans, Louisiana, ASCE.

In this document, the authors describe the process for the risk-based design of levees. This process requires a system vision, such as treating the levee and the flood plain as integral system components. This effectively turns levee design, construction, and maintenance into an optimization problem regarding economic and societal factors.

Deterioration

37. Agrawal, A., A. Kawaguchi, et al. (2009). Bridge Element Deterioration Rates. N. D. o. Transportation. Albany, New York, The City College of New York, Department of Civil Engineering**:** 105.

This report describes the development of bridge element deterioration rates using the NYSDOT bridge inspection database using Markov chain and Weibull-based approaches. Both approaches have been incorporated into a computer program which can generate deterioration curves based on historical data.

38. Alampalli, S. (2001). Correlation Between Bridge Vibration and Bridge Deck Cracking: A Qualitative Study Albany, New York, New York State Department of Transportation 50.

This document describes an observational study based on the possible relationship between bridge deck cracking and bridge vibrations. The study has documented correlations between the two but recommends further study using quantitative data.

39. Amleh, L., Z. Lounis, et al. (2003). Assessment of Corrosion-Damaged Concrete Bridge Decks A Case Study Investigation, National Research Council Canada**:** 9.

In this paper, the results of a comprehensive condition assessment of a decommissioned RC bridge are presented. The main focus of the assessment is the impact of the concrete mix on corrosion and subsequently on the bond strength of concrete structures. Results show a quasi-linear decrease in bond capacity with reinforcement mass loss.

40. Bulusu, S. and K. Sinha (1997). "Comparison of Methodologies to Predict Bridge Deterioration." Transportation Research Record: Journal of the Transportation Research Board **1597**(1): 34-42.

This article examines two methods for the estimation of bridge condition states. One method is based on Bayesian statistics while the other is based on a binary probit model. An application of these methodologies is demonstrated for substructure elements using the Indiana bridge database.

41. Cardimona, S., B. Willeford, et al. (2001). Bridge Deck Condition Studies in Missouri Utilizing Ground Penetrating Radar. Jefferson City, MO, Missouri Department of Transportation Research, Development and Technology.

In this study, the authors present the results of a comparison between ground penetrating radar (GPR) and manual in-depth inspection. Despite some problems with implementation, the study demonstrates that GPR will yield good estimates of chain drag hollow areas, debonding, half-cell potential, and rebar corrosion.

42. Costa, A. and J. Appleton (2002). "Case studies of concrete deterioration in a marine environment in Portugal." Cement and Concrete Composites **24**(1): 169-179.

This paper describes a series of case studies of different types of concrete structures which suffered extensive deterioration due to chloride-induced corrosion. Several degrees of deterioration associated with different exposure conditions in both reinforced and prestressed concrete structures are illustrated.

43. Ehlen, M. A., M. D. A. Thomas, et al. (2009). "Life-365 Service Life Prediction Model Version 2.0." Concrete International **31**(5).

This article describes the features of the computer program “Life-365”. This program is useful for estimating the life cycle costs of structures exposed to external sources of chlorides such as parking garages, bridge deck, and transportation infrastructure.

44. Fanous, F., H. Wu, et al. (2000). Impact of Deck Cracking on Durability. Ames, IA, Center for Transportation Research and Education, Iowa State University.

The objective of this paper was to determine the impact of deck cracking on durability and estimate the remaining functional service life for bridge decks constructed with epoxy coated rebar (ECR). 81 bridges were sampled and the results show that ECR can significantly extend the service life of bridges, compared to black rebar.

45. Naus, D. J. (2007). Primer on Durability of Nuclear Power Plant Reinforced Concrete Structures - A Review of Pertinent Factors. Oak Ridge, TN, Oak Ridge National Laboratory.

This report contains a discussion on reinforced concrete durability and the relationship between durability and performance. Concrete's history and basic constituents are reviewed, along with information on the environmental factors that can affect performance.

46. Nowak, A. S. and M. M. Szerszen (2003). Life-Cycle Deterioration Models for Concrete Deck Slabs, Lausanne, Switzerland, ASCE.

This document discusses the development of deck performance models as a function of age, type of concrete and traffic volume. Causes of deterioration considered include: poor material properties, inadequate pouring procedures, freeze and thaw cycles, truck loads, fatigue, corrosion, and the application of deicing chemicals.

47. Nowak, A. S., M. M. Szerszen, et al. (2000). Michigan Deck Evaluation Guide. Ann Arbor, MI, University of Michigan.

The objective of this report is to provide a guide for evaluating bridge decks and as a support document for other MNDOT policies. Deck investigation was based on inspection results, data from the Bridge Inventory Database, and finite element modeling.

48. Nowak, A. S., M. M. Szerszen, et al. (2000). Development of the Procedure for Efficient Evaluation of Bridge Decks. Ann Arbor, MI, University of Michigan.

This report provides a background for the Michigan Deck Evaluation Guide. It consists of chapters dealing with material models, field inspection and test results, finite element analysis, and additional information on punching shear failure.

49. Purvis, R. (2003). NCHRP Synthesis 319: Bridge Deck Joint Performance. Washington, D.C., Transportation Research Board**:** 58.

This synthesis report provides state-of-the-art findings about commonly used expansion joint systems. It summarizes performance data for each system type and contains examples of selection criteria and design guidelines.

50. Ramey, G. E. and R. L. Wright (1997). "Bridge Deterioration Rates and Durability/Longevity Performance." Practice Periodical on Structural Design and Construction **2**(3): 98-104.

This paper discusses the importance of bridge design planning with respect to service life. Assessments of durability performance were based on data available in literature along with a subset of the data contained in the historical bridge records of the Alabama DOT.

51. Roelfstra, G., R. Hajdin, et al. (2004). "Condition Evolution in Bridge Management Systems and Corrosion-Induced Deterioration." Journal of Bridge Engineering **9**(3): 268-277.

In this report, the authors present an alternative approach to condition assessment. Their approach, which is applied to chloride-induced corrosion of steel reinforcement, takes into consideration the physical phenomena underlying element deterioration.

52. Russell, H. G. (2004). NCHRP Synthesis 333: Concrete Bridge Deck Performance. Washington, D.C., Transportation Research Board**:** 188.

This report provides information on previous and current design and construction practices which have been used to improve the performance of concrete bridge decks. North American practices for cast-in-place, RC bridge decks on steel beams, concrete I- and T-beams, and concrete box beams are the primary focus.

53. Sanders, J. D. (2002). "Effectively Using Risk-Based Inspection Results to Implement a Corrosion Under Insulation Program." ASME Conference Proceedings **2002**(3591X): 635-649.

This paper discusses how to use RBI results to justify and manage a Corrosion Under Insulation (CUI) program that can be validated by using key performance indicators. A step-by-step methodology based on the experience of 19 plant sites is presented.

54. Sohanghpurwala, A. (2006). NCHRP Report 558: Manual on Service Life of Corrosion-Damaged Reinforced Concrete Bridge Superstructure Elements. Washington, D.C., Transportation Research Board.

This manual was created to assist in the rational decision making of bridge maintenance with respect to corrosion damaged RC superstructure elements. The three main objectives include: assessing condition of these elements, predicting the remaining service life, and quantifying the service life extension expected from alternative maintenance options.

Expert Elicitation

55. Ayyub, B. M. (2000). Uncertainties in Expert-Opinion Elicitation for Risk Studies, Santa Barbara, California, USA, ASCE.

In this paper, the author discusses example techniques for eliciting judgmental probabilities and consequences for dam safety studies. This includes any uncertainties associated with the process of expert elicitation.

56. Hetes, B., H. Richmond, et al. (2009). Expert Elicitation Task Force White Paper. Washington, D.C., EPA Science Policy Council.

The purpose of this white paper, which is a review draft, is to initiate a dialogue within the EPA about the conduct and use of expert elicitation (EE). Topics covered include: what EE is, when to consider using EE, how EE is conducted, and how the results should be presented.

General Risk Based Inspection

57. Anders, G., S. Otal, et al. (2005). Deriving Asset Probabilities of Failure: Effect of Condition and Maintenance Levels, IEEE.

This paper describes how asset probabilities of failure can be derived from life expectancy curves that are based on asset age, failure data and specific asset knowledge. It is shown how asset failure probabilities can then be adjusted for individual assets through measurement of the actual asset condition by the application of health indices.

58. API (2002). "Risk-Based Inspection, API Recommended Practice 580." 45.

This recommended practice is intended to provide guidance on developing a risk-based inspection (RBI) program for fixed equipment and piping in the hydrocarbon and chemical process industries. It covers topics such as what RBI is, what its key elements are, and how to implement an RBI program.

59. API (2008). "Risk-Based Inspection Technology, API Recommended Practice 581."

This recommended practice, which is an application of API 580, provides quantitative procedures to establish a risk based inspection program for fixed, pressurized equipment. This type of equipment includes: pressure vessels, piping, tanks, pressure relief devices, and heat exchanger tube bundles.

60. ASME (1992). Risk-Based Inspection: Development of Guidelines. General Document, American Society of Mechanical Engineers**:** 155.

This document recommends appropriate methods for establishing a risk-based inspection program for a facility or structural system. The process involves four major steps: defining the system, performing a qualitative risk assessment, using this to do a quantitative risk analysis, and developing an inspection program using probabilistic methods.

61. ASME (2007). Inspection Planning Using Risk-Based Methods, The American Society of Mechanical Engineers**:** 92.

This standard provides guidance to owners, operators, and designers of pressure-containing equipment for developing and implementing an inspection program. These guidelines include means for assessing an inspection program and its plan. Safe and reliable operation through cost-effective inspection is emphasized.

62. Ayyub, B. M. (2003). Risk Analysis in Engineering and Economics. Boca Raton, FL, CRC Press.

This textbook covers a range of information on the application of risk analysis to engineering and cost analysis. Some of the topics covered include: risk analysis methods, system definition and structure, reliability assessment, failure consequences and severity, engineering economics and finance, risk control methods, and data storage.

63. Bloch, A., J. D. Sorensen, et al. (2000). Simplified Approach to Inspection Planning. 8th ASCE Specialty Conference on Probabilistic Mechanics and Structural Reliability.

This paper presents a simplified and practically applicable approach for risk based inspection planning of fatigue sensitive structural details. In this approach, fatigue sensitive details are categorized according to their stress intensity factors and their design fatigue life to service life ratio.

64. Cesare, M., J. C. Santamarina, et al. (1993). "Risk-Based Bridge Management." Journal of Transportation Engineering **119**(5): 742-750.

In this article, a methodology for bridge project selection based on reliability methods and optimization procedures is outlined. A Markovian model is used to decay the structural components, then a reliability index is determined for each element using either subjective assessment or first-order reliability methods.

65. Christian, G. A. (2008). Inspection Data for Bridge Asset Management: New York State DOT Approach and Practice. Bridge Management and Preservation Conference, NYSDOT.

In this presentation, the author provides information on NYDOT's bridge management practices. Topics covered include: bridge management background, NYS bridge evaluations, bridge asset management strategies, and bridge management analysis methods.

66. Connor, R. J. and M. J. Parr (2007). A Commentary for A Method for Determining the Interval for Hands-on Inspection of Steel bridges with Fracture Critical Members, Purdue University**:** 54.

This document provides further details on the methodology developed by R. Connor and M.J. Parr, which is discussed in the document below.

67. Connor, R. J. and M. J. Parr (2008). A Method for Determining the Interval for Hands-On Inspection of Steel Bridges with Fracture Critical Members, Purdue University**:** 32.

In this document, an assessment procedure to establish in-depth (hands-on) inspection intervals for steel bridges with fracture critical members is discussed. This method was designed to be simple and provide an alternative approach to the pure calendar based methodology specified by the Code of Federal Regulations.

68. Eckold, G. and K. Adamson (2006). The Need for Risk-Based Inspection Planning, ESR Technology Bentley Systems, Inc.**:** 11.

In this white paper, the authors provide opinion and supporting evidence for why risk-based inspection programs are superior to calendar based inspection programs.

69. Faber, M. H., M. A. Maes, et al. (2006). On the Quantification of Robustness of Structures. 25th International Conference of Offshore Mechanics and Arctic Engineering, Hamburg, Germany.

In this document, a generic framework for risk assessments of engineering systems is presented in which robustness is related to the ability of the system to sustain damages. This framework is then applied to quantify robustness of structural system and to develop a robustness index in order to consistently rank different structures.

70. Flaig, K. and R. Lark (2005). "A risk-based decision-support system for bridge management." Proceedings of the Institute of Civil Engineers: Bridge Engineering **158**(BE3): 101-106.

This article presents a framework for an advanced bridge management system. It has a modular format which includes components for dealing with: data collection and storage, deterioration modeling, structural assessment, economic appraisal through whole-life costing, optimization methodologies, and a number of reporting facilities.

71. Guthrie, W., E. Linford, et al. (2007). "Development of an Index for Concrete Bridge Deck Management in Utah." Transportation Research Record: Journal of the Transportation Research Board **1991**: 35-42.

This document reports on research which was undertaken to develop a new index method for concrete bridge deck management in Utah. 15 concrete bridge decks of varying age were inspected and used to develop the Utah Bridge Deck Index which was then used prioritize maintenance and repair work.

72. Harnly, J. A. (1998). "Risk based prioritization of maintenance repair work." Process Safety Progress **17**(1): 32-38.

This paper describes the development of a risk ranked inspection recommendation procedure that is used by one of Exxon's chemical plants to prioritize repairs that have been identified during equipment inspection.

73. Helmerich, R., E. Niederleithhinger, et al. (2008). "Bridge Inspection and Condition Assessment in Europe." Transportation Research Record: Journal of the Transportation Research Board **2044**.

In this article, the authors present an overview of the latest research projects and integrated bridge management systems in Europe. The potentials of nondestructive testing are presented, with special focus given to RC and post tensioned concrete bridges.

74. Hjelm, H., J. D. Sorensen, et al. (2005). Reduction in Inspection Costs for Dynamic Sensitive Steel Structures by Modal Based Fatigue Monitoring. 24th International Conference on Offshore Mechanics and Arctic Engineering, Halkidiki, Greece.

This document presents a technique that is based on continuously measuring the accelerations of a few points of a structure. The data from these measurements is

analyzed using finite element models and then used to reduce the number of required inspection.

75. Jandu, A. S. (2008). "Inspection and maintenance of highway structures in England." Proceedings of the Institute of Civil Engineers: Bridge Engineering **161**(3): 111-114.

This paper describes the inspection and maintenance of highway structures by the Highways Agency in England. The background of the inspection requirements, the assets managed, and the maintenance activities are provided.

76. Kauer, R., A. Jovanovic, et al. (2004). Plant asset management rimap (risk-based inspection and maintenance for European industries) the European approach. American Society of Mechanical Engineers, Pressure Vessels and Piping Division (Publication) PVP, San Diego, CA.

In this article, an overview of the European project RIMAP (Risk-Based Inspection and Maintenance for European Activities) is presented. This includes the RIMAP description of risk, probability of failure, and consequence of failure and how these factors are determined. A more detailed application for the power industry is also included.

77. Khan, F. I., R. Sadiq, et al. (2004). "Risk-Based Inspection and Maintenance (RBIM): Multi-Attribute Decision-Making with Aggregative Risk Analysis." Process Safety and Environmental Protection **82**(6): 398-411.

This document presents a structured risk-based inspection and maintenance methodology. The method is based on aggregative risk analysis and multi-attribute decision-making. Fuzzy logic is used to estimate the likelihood of occurrence and the consequence.

78. Kowalik, A. (2009). Risk-Based Bridge Evaluations --- A Texas Perspective, Austin, Texas, ASCE.

In this article, the author explains that Texas used to use a risk-based inspection program that set the inspection frequency for fracture critical bridge from 6 months to 60 months. While they no longer use this system, they are still actively showing that risk based inspection is effective.

79. Masada, T., S. Sargand, et al. (2006). "New Inspection and Risk Assessment Methods for Metal Culverts in Ohio." Transportation Research Record: Journal of the Transportation Research Board **1976**.

This article describe Ohio’s new statewide culvert management program, which aimed to reduce the risk of structural failure of culverts that serve major highways. Then, a research program used to validate the effectiveness of the approaches outlined in the new program is described.

80. Modarres, M. (2006). Risk Analysis in Engineering. Boca Raton, FL, CRC Press.

This textbook covers a wide range of information in the field of risk analysis. Some of the topics covered include: an introduction to risk analysis, probabilistic risk assessments, data and modeling for a performance assessment, uncertainty analysis, identifying contributors to risk, decision making techniques, and risk communication.

81. Morcous, G., Z. Lounis, et al. (2003). "Identification of Environmental Categories for Markovian Deterioration Models of Bridge Decks." Journal of Bridge Engineering **8**(6): 353-361.

In this paper, an approach to provide transportation agencies with an effective decision support tool to identify the categories that best define the environmental and operational conditions specific to their bridge structures is presented.

82. Pennsylvania, D. o. T. (2007). Risk Assessment for PENNDOT-Owned Bridges and Structures, PENNDOT Bridge Quality Assurance Division.

This report provides an approach to risk based inspection which has been incorporated into the NCHRP project. In the document, the development of the risk assessment model, data processing, and utilization of the results to establish a plan of action to mitigate risks is discussed.

83. Patidar, V., S. Labi, et al. (2007). "Performance Measures for Enhanced Bridge Management." Journal of the Transportation Research Board **1991**: 43-53.

This article presents research that summarizes the best practices in quantifying a number of performance measures related to bridge management. These is discussion of desirable properties not only of individual performance measures but also of any set of performance measures intended for any particular bridge evaluation problem.

84. Patidar, V., L. Samuel, et al. (2007). NCHRP Report 590: Multi-Objective

Optimization for Bridge Management Systems. Washington, D.C., Transportation Research Board**:** 139.

In this report, network- and bridge-level methodologies that involve multiple performance criteria and also involve selection of investment choices based on optimization is presented. This is expected to aid bridge management decision making and thus enhance the cost-effectiveness of agency spending.

85. Rackwitz, R. and A. Joanni (2009). "Risk acceptance and maintenance optimization of aging civil engineering infrastructures." Structural Safety **31**(3): 251-259.

In this paper, a renewal model for cost-benefit optimization including maintenance is presented. Three types of maintenance strategies are proposed: age-dependent maintenance, block maintenance, and maintenance by inspection and (perfect) repair.

86. Rens, K. L. and T. Kim (2006). Quebec Bridge Inspection Using Common Nondestructive and Destructive Testing Techniques, St. Louis, Missouri, USA, ASCE.

This report discusses the nondestructive evaluation of four pier caps of Denver Colorado's Quebec Street Bridge. The specific details of all testing methodologies used in the study, along with specific results for the northwest pier cap, are presented.

87. RIMAP, C. (2001). Report on Current Practice, RIMAP Consortium**:** 29.

This report, compiled by the Risk Based Inspection and Maintenance Procedures for European Industry (RIMAP) Consortium, discusses the current practices of the members of the organization. The main sectors of industry represented are: power, offshore, and petrochemical.

88. RIMAP, C. (2003). RIMAP Framework, RIMAP Consortium**:** 9.

This document outlines the reporting structure of Risk Based Inspection and Maintenance Procedures for European Industry (RIMAP).

89. Rodriquez, M., S. Labi, et al. (2006). "Enhanced Bridge Replacement Cost Models for Indiana's Bridge Management System." Transportation Research Record: Journal of the Transportation Research Board **1958**: 13-23.

This paper presents details of replacement cost modeling for each bridge component (superstructure, substructure, approach, and other costs) for bridges with concrete slab, concrete beam, or steel superstructures.

90. Shackleton, D. N. (2006). "Reducing failure risk in welded components." Welding in the World **50**(9-10): 92-97.

This paper suggests the use of the requirements of ISO 3834 together with Risk Assessment and Management Appraisal techniques to lessen the possibility of welding failure and reduce costs in manufacturing.

91. Shepard, R. and M. Johnson (2001). "California Bridge Health Index: A Diagnostic

Tool to Maximize Bridge Longevity, Investment." TR News **215**(July/August).

This article discusses the Health Index which is a single-number assessment of a bridge's condition based on the bridge's economic worth, determined from an element-level inspection. The index makes it possible to ascertain the structural quality of a single bridge or a network of bridges and to make objective comparisons with other bridges or networks.

92. Sorensen, J. D. (2006). Safety and Inspection Planning of Older Installations. Aalborg, Denmark.

This document presents a risk based inspection approach with two additional aspects. First, the annual fatigue probability of failure is assessed assuming there are many fatigue critical elements. Second, information obtained from one element can be used to represent and update the information for adjacent elements.

93. Sorensen, J. D., M. H. Faber, et al. (1991). Modeling in Optimal Inspection and Repair. OMAE 91 Conference, Stavanger, Norway.

This paper describes a reliability based optimal inspection and repair strategy. The total expected costs in the lifetime is minimized with the inspection time and efforts, the repair crack size limit, and a design parameter as optimization variables.

94. Speck, J. B. and A. T. M. Iravani (2002). "Industry Survey of Risk-Based Life Management Practices." ASME Conference Proceedings **2002**(46555): 109-116.

In this document, technical and organizational requirements to implement risk-based methods are identified. Additionally, the results of a questionnaire survey used to better understand the plant life management of several companies is presented.

95. Straub, D. (2004). "Generic Approaches to Risk Based Inspection Planning for Steel Structures." 249.

This paper extends the RBI methodology to systems with a stochastic dependency between the individual hot spots. The general decision theoretic problems that arise when dealing with system RBI are discussed. Additionally, the differences to inspection planning for individual hot spots are listed and discussed.

96. Straub, D. and M. H. Faber (2002). "System Effects in Generic Risk Based Inspection Planning." ASME Conference Proceedings **2002**(36126): 391-399.

In this article, a computationally efficient method for the calculation of risk-based inspection plans is presented, which overcomes the problems encountered through the use of a generic approach.

97. Straub, D. and M. H. Faber (2005). "Risk based inspection planning for structural systems." Structural Safety **27**(4): 335-355.

This paper presents an integral approach for the consideration of entire systems in inspection planning, based on new developments in RBI for individual components. Various aspects of dependencies in the systems are presented and discussed, followed by an introduction to the decision problems encountered for structural systems.

98. Straub, D. and M. H. Faber (2006). "Computational Aspects of Risk-Based Inspection Planning." Computer-Aided Civil and Infrastructure Engineering: 179-192.

In this document, the general RBI methodology, a review of the probabilistic deterioration models for fatigue and corrosion of steel structures and the description of inspection performance models is presented. New concepts are introduced for the treatment of corrosion deterioration.

99. Wang, Y.-M., J. Liu, et al. (2008). "An integrated AHP-DEA methodology for bridge risk assessment." Computers & Industrial Engineering **54**(3): 513-525.

In this paper, an integrated analytical hierarchy process (AHP) –DEA methodology to evaluate bridge risks of hundreds or thousands of structures, based upon which the maintenance priorities of the structures can be decided, is presented. AHP is used to determine the weights of criteria in linguistic terms such as high, medium, low, and none.

Miscellaneous

100. AASHTO (2005). Grand Challenges: A Strategic Plan for Bridge Engineering. NCHRP 20-07/Task 199 Final Report**:** 20.

In this report, the Highway Subcommittee on Bridges and Structures of AASHTO has presented a discussion on several aspects of bridge engineering that need further

research. These include: materials, structural systems, bridge management, enhanced specification, computer aided design, and leadership.

101. AASHTO (2008). "The Manual for Bridge Evaluation." 226.

This manual serves as a standard and provides uniformity in the procedures and policies for determining the physical condition, maintenance needs, and load capacity of the nation's highway bridges.

102. Altenbach, T. J. (1995). A Comparison of Risk Assessment Techniques from Qualitative to Quantitative. ASME Pressure and Piping Conference, Honolulu, Hawaii, Lawrence Livermore National Laboratory.

This paper attempts to shed light on the basic issues regarding the demarcation between qualitative and quantitative risk assessment. Risk matrix techniques are surveyed, examining the uses and applicability of each.

103. ASCE and SEI-AASHTO (2009). "White Paper on Bridge Inspection and Rating." Journal of Bridge Engineering: 5.

This study addresses the methods and practices used to ensure the safety of highway bridges across the United States. The gaps, needs, and issues associated with the current practices and policies for the condition assessment of bridges are described. These have been broken down into ten categories.

104. Das, P. (1999). Management of Highway Structures. London, England, Thomas

Telford Publishing.

This book is intended to acknowledge the emerging technology which combines bridge management and financial planning. The latest developments are introduced for engineers, accountants and financial decision makers who are, or may be in the future, be involved in bridge management.

105. Dorgan, D. and K. Western (2008). I-35W Bridge Collapse, Minnesota DOT.

This presentation provides information on the I-35W bridge collapse in Minnesota. The topics covered include: the bridge background, emergency response, the post-collapse investigation, media demands, victim recovery, the political debate, and lessons learned.

106. FHWA (2001). Reliability of Visual Inspection for Highway Bridges, Volume 1: Final Report**:** 486.

This document presents the results of a study on the reliability of visual inspection. The goals of the study included: providing overall measures of the accuracy and reliability of Routine and In-Depth Visual Inspection, studying the influence of several key factors that affect Routine and In-Depth Inspection, and studying the differences between State inspection procedures.

107. FHWA (2005). "Bridge Preservation and Maintenance in Europe and South Africa." 124.

This document presents a scanning study of how highway agencies in Europe and South Africa handle bridge maintenance, management, and preservation. The team focused on bridge management systems, inspection practices, permit load evaluation and routing, and innovative maintenance practices.

108. Gertsbakh, I. (2000). Reliability Theory. New York, NY, Springer-Verlag Berlin Heidelberg.

This book provides a review of reliability theory and its applicability to preventive maintenance programs. Topics covered include: system reliability, preventive maintenance models based on lifetime distributions, best time scale for age replacement, and preventive maintenance with learning.

109. Jardine, A. and A. Tsang (2006). Maintenance, Replacement, and Reliability. Boca Raton, FL, CRC Press.

The purpose of this text is to provide readers with the tools needed for making data-driven physical asset management decisions. It is solidly based on the results of real-world research in physical asset management, including applications of the models presented in the text.

110. Juntunen, D. (2007). Presentation to the Michigan Bridge Conference: MBIS Special Inspections, MBRS Updates, Bridge Deck Preservation.

In this presentation, the author discusses MBIS Special Investigations, MBRS Updates, and bridge deck preservation. Much of the presentation is focused on bridge deck preservation, such as developing performance measures, need indicators, and how to model deterioration.

111. Martin, T., D. Johnson, et al. (2007). Using Historical Repair Data to Create Customized Predictive Failure Curves for Sewer Pipe Risk Modeling. LESAM 2007 - 2nd Leading Edge Conference on Strategic Asset Management, Lisbon, Portugal.

This document presents a sewer pipe failure risk model which was developed by the Seattle Public Utilities. This model is used to calculate the annual aggregate risk cost to the utility of the pipe network and the individual risk cost of failure for each individual pipe segment.

112. NCHRP (2007). NCHRP Synthesis 375: Bridge Inspection Practices**:** 199.

This report is a collection of information on the formal inspection practices of state departments of transportation. These are primarily visual inspections and they provide data to bridge registries and databases.

113. O'Connor, J. S. (2000). Bridge Safety Assurance Measures Taken in New York State. International Bridge Engineering Conference, Tampa, Florida.

This paper provides a description of New York State’s Bridge Safety Assurance Program. Specific examples of interim countermeasures that can be taken to lessen the risk of failure due to hydraulic scour, overload, steel details, collision, concrete details, and earthquakes are provided.

114. Rausand, M. and A. Hoyland (2004). System Reliability Theory. Hoboken, NJ, John Wiley & Sons, Inc.

This book addresses topics in the area of system reliability. Topics covered include: failure models, qualitative system analysis, the Markov process, reliability of maintained systems, life data analysis, accelerated life testing, and Bayesian reliability analysis.

115. Sakai, S. (2006). "Current Status of Developing RBI Guideline in Japan." ASME Conference Proceedings **2006**(47586): 333-337.

This paper describes the outline of Risk Based Maintenance related activities in both the nuclear and non-nuclear fields in Japan. While the Fitness For Service code is now available for many industries, a risk based maintenance standard still needs to be created.

116. Smith, D. (1993). Reliability Maintainability and Risk. Jordan Hill, Oxford, Butterworth-Heinemann Ltd.

This text covers topics in the area of reliability and risk based assessment. Topics covered include: realistic failure rates, interpreting data, risk assessment, design and assurance techniques, factors influencing downtime, project management, safety critical systems, liability and legislation, and finally case studies.

117. Sun, X., Z. Zhang, et al. (2004). Analysis of past national bridge inventory ratings for

predicting bridge system preservation needs. Transportation Research Record: 36-43.

Due to its intensive data requirement, the Louisiana DOT has not fully implemented the Pontis program. This paper presents an innovative approach that uses readily available NBI data in Pontis to evaluate the long-term performance of the bridge system under various bridge management system alternatives.

118. Tamakoshi, T. and T. Nanazawa (2007). State of Bridges and Their Management. 23th US-Japan Bridge Engineering Workshop, Tsukuba Science City, Japan.

In this report, the authors discuss the current state of road bridges, approaches to rational management, and research on measures to counteract specific damages.

119. Thompson, P. and R. Shepard (2000). AASHTO Commonly-Recognized Bridge Elements: Successful Applications and Lessons Learned. National Workshop on Commonly Recognized Measures for Maintenance.

This document discusses the implementation of the CoRe elements system and presents some problems which have been encountered. These problems include: how the parts of a bridge are divided for assessment, how the rating system doesn't detail deterioration processes, and how ratings are vulnerable to subjective interpretation.

120. Zhang, J., A. Gupta, et al. (2007). "Effect of Relative Humidity on the Prediction of Natural Convection Heat Transfer Coefficients." Heat Transfer Engineering **28**(4): 335-342.

This article presents the results of an investigation into the sensitivity of natural convection heat transfer correlation with respect to relative humidity. Several heat transfer correlation were examined and the results show a general trend of an increasing Nusselt number with relative humidity.

Non-Destructive Evaluation

121. ASTM (2007). Standard Test Method for Detecting Delaminations in Bridge Decks Using Infrared Thermography. ASTM. **D 4788 - 03 (2007)**.

This standard presents the method to be used to determine delaminations in Portland-cement concrete bridge decks using infrared thermography. This test method is intended for use on exposed and overlaid concrete bridge decks.

122. Clark, M. R., D. M. McCann, et al. (2003). "Application of infrared thermography to the non-destructive testing of concrete and masonry bridges." NDT and E International **36**: 265-275.

The purpose of this article is to show that even with the low ambient temperatures experienced in Europe, it is still possible to use infrared thermography. The purpose of which is to correctly identify areas of delamination in concrete bridge structures and also to investigate the internal structure of masonry bridges.

Offshore Structures

123. Banon, H. (1994). "Assessing Fitness for Purpose of Offshore Platforms. II: Risk

Management, Maintenance, and Repair." Journal of Structural Engineering **120**(12): 3613-3633.

This article presents a discussion on several major obstacles in the assessment of existing structures as compared to new structures. These obstacles revolve around the need for extensive data gathering, modeling all of the important damage found, structural evaluation, calculation of reliability indices, and the determination of management schemes.

124. Biasotto, P. and A. Rouhan (2004). "Survey and Inspection Management for FPSOs." ASME Conference Proceedings **2004**(37440): 379-388.

The aim of this paper is to present a methodology to establish inspection and survey plans, combining RBI analysis and industry expertise. The several steps to achieve the final inspection campaign are described based on the experience of new-built and converted FPSO's.

125. Chakrabarti, P., I. Abu-Odeh, et al. (2005). "An Overview of the Reassessment Studies of Fixed Offshore Platforms in the Bay of Campeche, Mexico." ASME Conference Proceedings **2005**(41952): 123-134.

This document has been created out of the need to extend the service life of several oil and gas platforms in the Gulf of Mexico. An overview of the reassessment procedure is outlined and pertinent results are presented for more than twenty platforms.

126. El-Reedy, M. A. (2006). "Risk Based Inspection for Prioritizing Repair and

Inspections of Large Numbers of Platforms in Gulf of Suez." ASME Conference Proceedings **2006**(47489): 89-95.

Due to more than a hundred platforms located in the Gulf of Suez, a risk-based process has been developed to effectively implement inspection resources. This process is based upon critical characteristics as well as the results from previous inspections.

127. Faber, M. H., J. D. Sorensen, et al. (2005). "Field Implementation of RBI for Jacket Structures." Journal of Offshore Mechanics and Arctic Engineering **127**(3): 220-226.

This document first provides a brief outline of the simplified and generic approach to reliability and risk based inspection planning. Second, this methodology is applied for the planning of in-service NDT inspections of fixed offshore steel jacket structures in the Danish part of the North-Sea.

128. Faber, M. H., D. Straub, et al. (2005). "Fatigue Analysis and Risk Based Inspection Planning for Life Extension of Fixed Offshore Platforms." ASME Conference Proceedings **2005**(41952): 511-519.

This paper reports on the application of risk based inspection planning as conducted on a project for Pemex Exploracion y Produccion. The engineering assessments required to extend the service life of 23 fixed steel jacket structures in the Bay of Campeche is considered.

129. Faber, M. H., D. Straub, et al. (2003). "Unified Approach to Risk-Based Inspection Planning for Offshore Production Facilities." Journal of Offshore Mechanics and Arctic Engineering **125**(2): 126-131.

Based on recent developments in the field of risk-based inspection planning, the paper presents a unified approach to RBI planning for offshore facilities comprising components and systems of both the structural and the process type.

130. Goyet, J., A. Rouhan, et al. (2004). "Industrial Implementation of Risk Based Inspection Planning Lessons Learnt From Experience: Part 1 --- The Case of FPSOs." ASME Conference Proceedings **2004**(37440): 553-563.

This paper is the first part of a report dealing with industrial implementation of risk base inspection planning methods for offshore structures. The main focus of this document is FPSOs and FSOs. The objective of the report is to convey lessons learned from experience using RBI methods.

131. Heredia-Zavoni, E., F. Silva-Gonzalez, et al. (2008). "Reliability Analysis of Marine Platforms Subject to Fatigue Damage for Risk Based Inspection Planning." Journal of Offshore Mechanics and Arctic Engineering **130**(4): 041001-9.

In this document, an application to risk based inspection planning for the extension of the service life of a platform is given. This method is based on Monte Carlo simulation using limit state functions in which wave, wind, and deck loading are expressed in terms of empirical functions of uncertain maximum wave height.

132. Lee, A. K., C. Serratella, et al. (2006). Flexible approaches to risk-based inspection of FPSOs. Offshore Technology Conference 2006: New Depths. New Horizons, Houston, TX.

Due to an aging FPSO fleet, a multi-level risk-based inspection methodology ranging from simplified deterministic approaches using standard design analysis up to sophisticated probabilistic approaches has been developed. Each level has various levels of usefulness depending on the scope of the inspection program task.

133. Lee, A. K., C. Serratella, et al. (2007). "Multilevel Risk-Based Inspection Scheme for FPSOs." Marine Technology **44**: 118-124.

This document is an earlier version of the previous document.

134. Li, D., W. Tang, et al. (2005). "Cost-Benefit Evaluation of Inspection and Repair Planning for Ship Structures Considering Corrosion Effects." ASME Conference Proceedings **2005**(41960): 69-78.

This paper aims to propose a cost-benefit model of risk based inspection and repair planning for ship structures subjected to corrosion deterioration. The models of expected cost and expected benefit are formulated, and the cost-benefit ratio is adopted to select the optimal inspection and repair plan.

135. Li, D., S. Zhang, et al. (2004). Risk Based Inspection and Repair Optimization of Ship Structures Considering Corrosion Effects. 23rd International Conference on Offshore Mechanics and Arctic Engineering, Vancouver, British Columbia, Canada.

This document outlines a theoretical framework of risk based optimal inspection and repair planning for ship structures subjected to corrosion deterioration. The planning problem is formulated as an optimization problem where the expected lifetime costs are minimized with a constraint on the minimum acceptable reliability index.

136. Onoufriou, T. (1999). "Reliability based inspection planning of offshore structures." Marine Structures **12**(7-8): 521-539.

This paper presents a series of studies on the development and application of reliability based inspection planning techniques for offshore structures including fixed and floating platforms and jack-up drilling rigs. The general methodology is presented and the differences in application between the structure types are highlighted.

137. Rouhan, A., J. Goyet, et al. (2004). "Industrial Implementation of Risk Based Inspection Planning Lessons Learned From Experience: Part 2 --- The Case of Steel Offshore Structures." ASME Conference Proceedings **2004**(37440): 565-572.

This is the second part of a report on the industrial application of risk based inspection, based on fixed steel offshore structures. The challenges related to the full implementation of RBI to an entire set of platforms are discussed.

138. Shipping, A. B. o. (2003). Surveys Using Risk-Based Inspection for the Offshore Industry. Houston, TX, American Bureau of Shipping**:** 62.

This guide contains the technical requirements and criteria employed by the American Bureau of Shipping to consider alternate survey arrangements using risk-based inspection approaches for offshore installations. This approach is applicable to static pressure retaining equipment on structures for offshore floating and fixed-base platforms.

139. Straub, D., J. Goyet, et al. (2006). "Benefits of Risk Based Inspection Planning for Offshore Structures." ASME Conference Proceedings **2006**(47489): 59-68.

In this paper, the economic benefits of applying risk-based inspection planning for offshore structures subject to fatigue are evaluated based on experiences from past industrial projects. The factors influencing the cost of inspection, repair and failure are discussed with realistic values.

140. Wang, G., J. Spencer, et al. (2005). "Assessment of Corrosion Risks to Aging Ships Using an Experience Database." Journal of Offshore Mechanics and Arctic Engineering **127**(2): 167-174.

This article presents a study on the risks of corrosion wastage to aging ships' structural integrity using an experience database. This database is based on over 110,000 thickness measurements collected from 140 trading tankers. In addition to these measurements, information on the hull girder's geometrical and strength properties have also been collected.

Pipelines

141. Chang, M.-K., R.-R. Chang, et al. (2005). "Application of risk based inspection in refinery and processing piping." Journal of Loss Prevention in the Process Industries **18**(4-6): 397-402.

This article discusses a study in which risk based inspection methodology was applied to piping in a refinery and in petrochemical plants in Taiwan. Two actual case studies were corroborated better with quantitative RBI methodology than without the methodology, in terms of risk and cost reductions.

142. Chang, R.-R., J.-J. Jeng, et al. (2001). Risk Management Application on Refinery Pipeline Inspection. Asia Pacific Occupational Safety & Health Organization**:** 12.

In this paper, the authors present the idea of a pipeline corrosive loop which is based on the consideration of risk so that pipeline operators can be provided with optimal inspection planning. The hope is that this will reduce pipeline disasters and still retain an overall economic benefit.

143. Essamin, O., K. El-Sahli, et al. (2005). Risk Management System for Prestressed Concrete Cylinder Pipeline: Practical Results and Experience on the Great Man Made River. The Pipeline Division Specialty Conference, Houston, Texas, USA, ASCE.

This paper is based on an inspection system that uses a deterioration model calibrated from inspection and monitoring data. The practical results of the system after it was used on a large scale application in Libya are discussed with a focus on how the system allows for larger savings on maintenance expenditures.

144. Filho, M. P., J. L. d. F. Freire, et al. (2004). "Application of Risk Based Inspection Methodology of API 581 BRD to Oil Pipelines." ASME Conference Proceedings **2004**(41766): 2621-2629.

This document focuses on the calculation procedure of probability of failure according to the API 581 BRD RBI methodology for equipment under uniform or localized corrosion. This procedure was applied to the data from three existing oil pipelines to better determine the influence of the frequency and the effectiveness of inspection activities.

145. Hopkins, P., R. Fletcher, et al. (1999). A Method for the Monitoring and Management of Pipeline Risk - A Simple Pipeline Risk Audit (SPRA). Advances in Pipeline Technologies & Rehabilitation. Abu Dhabi**:** 24.

This paper presents a summary of risk management methods used for pipelines. The use of surveillance methods for monitoring population encroachment along pipeline routes is covered. Methods for the assessment of risk associated with pipelines are also presented.

146. Muhlbauer, W. K. (2006). Enhanced Pipeline Risk Assessment. Pipeline Risk Management Manual, 4th Ed., Gulf Publishing Co.

This document discusses the recent advancements in the use of risk assessment algorithms. These changes have lead to more robust risk results that better reflect reality and are readily obtained from data used in previous assessments.

147. Singh, M. and T. Markeset (2009). "A methodology for risk-based inspection planning of oil and gas pipes based on fuzzy logic framework." Engineering Failure Analysis **In Press, Corrected Proof**.

This article presents a method to conduct risk based inspection planning using a concept called fuzzy logic. This approach can help reduce the dependence on precise data, allow modeling when a phenomenon is not completely understood, and reduce the difficulties arising from complex computation.

Reliability

148. Akgül, F. and D. M. Frangopol (2004). "Time-dependent interaction between load rating and reliability of deteriorating bridges." Engineering Structures **26**(12): 1751-1765.

In this document, the authors investigate the relationship between reliability-based analysis, representing the future trend in bridge evaluation, and load ratings. The purpose of which is to gain a better understanding of how federal funds will be distributed under a reliability based system which factors live load increases and deterioration into the load rating.

149. Akgül, F. and D. M. Frangopol (2004). "Bridge Rating and Reliability Correlation: Comprehensive Study for Different Bridge Types." Journal of Structural Engineering **130**(7): 1063-1074.

This paper presents the results of a study in which the interaction between rating and reliability of a group of 14 bridges in an existing bridge network was investigated. Rating factors for different bridge groups are identified based on bridge type, and bridge groups are compared based on mean group rating factor.

150. Chalifoux, A. and J. Baird (1999). Reliability Centered Maintenance (RCM) Guide: Operating a More Effective Maintenance Plan, U.S. Army Corps of Engineers.

This manual outlines a comprehensive method of organizing an efficient maintenance program through applying the concepts of reliability centered maintenance. Here, professional intuition has been combined with a rigorous statistical approach.

151. Chung, H. Y. (2004). Fatigue Reliability and Optimal Inspection Strategies for Steel Bridges Civil Engineering. Austin, Texas, University of Texas. **Doctor of Philosophy**.

In this dissertation, two fatigue reliability formulations that can be applied for most details in steel bridges have been developed. The difference between the two is that one is based on Miner's ruler and expressed in terms of stress cycles; the other is based on linear elastic fracture mechanics and expressed in terms of crack size and growth rate.

152. Chung, H. Y., L. Manuel, et al. (2003). Reliability-Based Optimal Inspection for Fracture-Critical Steel Bridge Members. Transportation Research Record. (1845) (pp 39-47), 2003. Date of Publication: 2003.

In this document, a systematic reliability based method for inspection scheduling is proposed to yield the most economical inspection strategy for steel bridges that, at the same time, guarantees an acceptable safety level through the planned service life, is presented.

153. Chung, H. Y., L. Manuel, et al. (2003). Optimal Inspection of Fracture-Critical Steel Trapezoidal Girders. T. D. o. Transportation. Austin, Texas, Center for Transportation Research**:** 32.

This article is a previous version of the work presented in Chung's dissertation, listed previously.

154. Chung, H.-Y., L. Manuel, et al. (2006). "Optimal Inspection Scheduling of Steel Bridges Using Nondestructive Testing Techniques." Journal of Bridge Engineering **11**(3): 305-319.

This paper proposes a probabilistic approach to help select the most suitable nondestructive inspection (NDI) technique and associated optimal inspection schedule for fracture critical member/detail fatigue inspection on a specific steel bridge.

155. Chung, H. Y., L. Manuel, et al. (2006). Simulation-Based Non-Destructive Inspection Scheduling of Steel Bridges. 85th Annual Meeting of the Transportation Research Board, Washington, D.C.

This is a different version of the previous document, which was presented at the 85th Annual Meeting of the TRB.

156. Darmawan, M. S. and M. G. Stewart (2007). "Spatial time-dependent reliability analysis of corroding pretensioned prestressed concrete bridge girders." Structural Safety **29**(1): 16-31.

This document discusses accelerated pitting corrosion tests which were performed to obtain spatial and temporal maximum pit-depth data for prestressing wires. This data is made to represent 7-wire strands and combined with Finite Element Analysis to study the spatial and temporal effects of pitting corrosion on typical pretensioned prestressed concrete bridge girders.

157. Enright, M. P. and D. M. Frangopol (1998). "Service-Life Prediction of Deteriorating Concrete Bridges." Journal of Structural Engineering **124**(3): 309-317.

In this paper, the reliability of RC highway girder bridges under aggressive conditions is investigated using a time-variant system reliability approach. Here, both load and resistance are time dependent and Monte Carlo simulation is used to find the cumulative-time system failure probability.

158. Estes, A. C. and D. M. Frangopol (1999). "Repair Optimization of Highway Bridges Using System Reliability Approach." Journal of Structural Engineering **125**(7): 766-775.

In this article, the authors use an existing Colorado State bridge to demonstrate a system reliability based repair optimization methodology. The bridge is modeled as a series-parallel combination of failure modes and the overall reliability is computed using time-dependent deterioration models and live load models. Development of the optimum lifetime repair strategy and a discussion on that strategy's sensitivity to changes is also discussed.

159. Estes, A. C. and D. M. Frangopol (2001). "Bridge Lifetime System Reliability under Multiple Limit States." Journal of Bridge Engineering **6**(6): 523-528.

This paper proposes the use of serviceability flags as a means to incorporate serviceability concerns into a strength-based reliability analysis. Using highway bridges as an example, available data sources for serviceability flags are considered.

160. Estes, A. C. and D. M. Frangopol (2003). "Updating Bridge Reliability Based on Bridge Management Systems Visual Inspection Results." Journal of Bridge Engineering **8**(6): 374-382.

In this article, it is proposed that the visual inspection data provided from bridge management systems already in place can be used to update the reliability of a bridge. The limitations and necessary modifications to current practice are discussed. An example of a Colorado highway bridge's deteriorating deck due to corrosion is provided.

161. Estes, A. C., D. M. Frangopol, et al. (2004). "Updating Reliability of Steel Miter Gates on Locks and Dams Using Visual Inspection Results." Engineering Structures(26.3): 319-333.

In this document, the authors show how the condition index visual inspection method for locks and dams can be modified to update the reliability analysis of steel miter gates. Two existing miter gates along the Mississippi River are used as examples for corrosion and fatigue deterioration.

162. Frangopol, D. M., J. S. Kong, et al. (2001). "Reliability-Based Life-Cycle Management of Highway Bridges." Journal of Computing in Civil Engineering **15**(1): 27-34.

In this paper, it is attempted to shed some light on the past, present, and future of life-cycle management of highway bridges. It is shown that current bridge management systems have limitations and that these limitations can be overcome by using a reliability-based approach.

163. Frangopol, D. M., K.-Y. Lin, et al. (1997). "Life-Cycle Cost Design of Deteriorating Structures." Journal of Structural Engineering **123**(10): 1390-1401.

In this paper, the authors present a lifetime optimization methodology for planning the inspection and repair of structures that deteriorate over time. The optimization is based on minimizing the expected total life-cycle cost while maintaining allowable lifetime reliability.

164. Guthrie, W. and R. Tuttle (2006). Condition Analysis of Concrete Bridge Decks in Utah, Brigham Young University.

This document reports on the research done to develop a protocol offering guidance as to whether deteriorating bridge decks should be rehabilitated or replaced. Twelve bridges were chosen and inspected using several techniques. The results from each bridge were then compared to threshold values to determine if they should be rehabilitated or repaired.

165. Hong, T.-H., S.-H. Chung, et al. (2006). "Service life estimation of concrete bridge decks." KSCE Journal of Civil Engineering **10**(4): 233-241.

This paper present a new model that predicts the end of service life in concrete bridge decks based on deterioration rate and the condition rating for decommission. Using data from the NBI and an additional questionnaire survey, deck deterioration models for 30 DOTs have been created.

166. Johnson, S. M. (2006). Effect of Superstructure Properties on Concrete Bridge Deck Deterioration. Civil Engineering. Urbana, Illinois, University of Illinois at Urbana-Champaign. **Master of Science:** 112.

In this thesis, the author discusses a project with the objective of correlating superstructure properties, with special emphasis on girder flexibility, to the performance of concrete bridge decks. It has been observed that decks supported on prestressed or reinforced concrete girders demonstrate better performance than those on steel girders.

167. Lounis, Z. (2006). Risk-Based Maintenance Optimization of Aging Highway Bridge Decks. Building Science Insight 2006, NRC Institute for Research in Construction.

This paper presents a practical approach for maintenance optimization of a network of aging highway bridge decks. This approach integrates a stochastic deterioration model based on Bogdanoff's cumulative damage theory with an effective multi-objective optimization approach.

168. Mori, Y. and B. R. Ellingwood (1994). "Maintaining Reliability of Concrete Structures. I: Role of Inspection/Repair." Journal of Structural Engineering **120**(3): 824-845.

This is the first of two papers that describe to role of in-service inspection and repair in maintaining the reliability of concrete structures during a projected service period, taking into account the randomness in existing damage and in damage detection. The degradation in strength due to environmentally aggressive factors is evaluated using growth models based on experimental data.

169. NASA (1997). Flight Assurance Procedure: Performing a Failure Mode and Effect Analysis. NASA.

This procedure establishes guidelines for conducting a Failure Modes and Effects Analysis (FMEA) on GSFC spacecraft and instruments.

170. NASA (2000). Reliability Centered Maintenance Guide for Facilities and Collateral Equipment. N. A. a. S. Administration**:** 348.

This document provides guidance for NASA staff in conducting a reliability centered maintenance program. This process involves identifying actions that, when taken, will reduce the probability of failure and which of these actions are the most effective.

171. NASA (2008). Facilities Maintenance and Operations Management. F. E. a. R. P. Division, NASA.

This document establishes minimum NASA management of facilities maintenance objectives and standards in support of NASA Policy Directive 8831.1

Maintenance and Operations of Institutional and Program Facilities and Related Equipment and 8700.1 NASA Policy for Safety and Mission Success.

172. NASA, L. R. C. (1996). Tools of Reliability Analysis - Introduction and FMEA's. NASA, NASA.

This presentation provides an introduction to Failure Modes and Effects Analysis. Main objectives are: to explain terminology, show the benefits of FMEA, evaluate levels of criticality and redundancy, show how to perform a components level and system level FMEA, and finally show how to apply FMEA results.

173. Nowak, A. S. and M. M. Szerszen (2000). "Structural reliability as applied to highway bridges." Progress in Structural Engineering and Materials **2**(2): 218-224.

This paper presents the application of reliability methods in the development of a load and resistance factor design bridge codes. Structural performance is measured in terms of the reliability index. Load and resistance models are summarized.

174. Nowlan, F. S. and H. F. Heap (1978). Reliability-Centered Maintenance. O. o. A. S. o. Defense, United Airlines.

This book explains basic concepts, principles, definitions, and applications of a logical discipline for the development of an efficient scheduled preventive maintenance program.

175. Onoufriou, T. and D. M. Frangopol (2002). "Reliability-based inspection optimization of complex structures: a brief retrospective." Computers & Structures **80**(12): 1133-1144.

This paper presents a brief retrospective of the development and application of reliability-based techniques for assessment of complex structures with emphasis on inspection optimization of offshore and bridge structures.

176. Poulassichidis, T. (2004). Risk-Based Inspection as a Reliability-Engineering Tool for Fixed Equipment Decisions. 2004 ASME Pressure Vessel and Piping Conference, San Diego, California, Dow Chemical.

This document describes an Olefins plant's gains from applying RBI as a reliability-engineering tool instead of an inspection optimization tool. Fixed-equipment relative risk ranking is established to: set priorities and focus attention, justify capital investment, and address Loss of Primary Containment and Process safety issues.

177. Rashedi, R. and F. Moses (1988). "Identification of Failure Modes in System Reliability." Journal of Structural Engineering **114**(2).

In this report, the reliability of framed structures is studied assuming loads and component strength are random variables. The significant modes are used in a Monte Carlo simulation to obtain accurately a safety index for the system. Applications and numerical examples are presented.

178. Sianipar, P. R. M. and T. M. Adams (1997). "Fault-Tree Model of Bridge Element Deterioration Due to Interaction." Journal of Infrastructure Systems **3**(3): 103-110.

This article discusses the use of a fault tree for qualitative and quantitative evaluation of element interaction phenomena. An example fault-tree model for the accelerated deterioration of concrete bridge decks is presented.

179. Sloth, M. and B. B. Jensen (2006). Probability-based maintenance management plan for corrosion risk - a case study from Faro Bridges. IABMAS'06 Third International Conference on Bridge Maintenance, Safety and Management. Porto, Portugal.

The Faro Bridges were constructed using state-of-the-art technology and procedures, however in the late 1980's samples showed a higher than expected chloride content. This led to a series of investigations, which in 2005, resulted in a probability-based evaluation of residual service life. This paper presents the data basis and how it has been used for maintenance management.

180. Sommer, A. M., A. S. Nowak, et al. (1993). "Probability-Based Bridge Inspection Strategy." Journal of Structural Engineering **119**(12): 3520-3536.

This paper deals with the probability based inspection of corroded steel-girder bridges and presents a procedure for the identification of the critical girders that take the highest load portion.

181. Stewart, M. G. and J. A. Mullard (2007). "Spatial time-dependent reliability analysis of corrosion damage and the timing of first repair for RC structures." Engineering Structures **29**(7): 1457-1464.

This paper presents a spatial time-dependent reliability analysis to predict the likelihood and extent of cracking for RC structures exposed to chloride ion attack. Methods included in the report allow the minimum time to first repair to be estimated which will assist designers in selecting specifications and possible maintenance regimes.

182. Stewart, M. G., D. V. Rosowsky, et al. (2001). "Reliability-based bridge assessment using risk-ranking decision analysis." Structural Safety **23**(4): 397-405.

This paper presents a broad overview of reliability-based assessment methods and then focuses on decision-making applications using updated time-dependent estimates of bridge reliabilities considering a risk-ranking decision analysis.

183. Suo, Q. and M. G. Stewart (2009). "Corrosion cracking prediction updating of deteriorating RC structures using inspection information." Reliability Engineering & System Safety **94**(8): 1340-1348.

In this document, a spatial time-dependent reliability analysis combined with visual inspection data is used to predict the likelihood and extent of RC corrosion-induced cracking. Concrete strength, cover and surface chloride concentrations are modeled as spatial variables.

184. Val, D. V. and R. E. Melchers (1997). "Reliability of Deteriorating RC Slab Bridges." Journal of Structural Engineering **123**(12): 1638-1644.

This paper presents a method for reliability assessment of RC slab bridges with corroded reinforcement. The method includes a traffic load model, a corrosion model,

and a nonlinear finite element structural model. Two types of corrosion – general corrosion, including spalling and debonding between concrete and rebar, and localized corrosion are considered.

185. Vu, K. A. T. and M. G. Stewart (2000). "Structural reliability of concrete bridges including improved chloride-induced corrosion models." Structural Safety **22**(4): 313-333.

In this article, the authors present a structural deterioration reliability model which is used to calculate probabilities of structural failure. New models for reinforced concrete corrosion initiation, corrosion rate and time-variant load models are proposed. Three durability design specifications are considered in a lifetime reliability analysis of a RC slab bridge.

186. Zhang, R. and S. Mahadevan (2000). "Model uncertainty and Bayesian updating in reliability-based inspection." Structural Safety **22**(2): 145-160.

In this paper, a Bayesian procedure is proposed to quantify the modeling uncertainty, including the uncertainty in mechanical and statistical model selection and the uncertainty in distribution parameters.

187. Zhou, J., B. Rothwell, et al. (2009). "Reliability-Based Design and Assessment Standards for Onshore Natural Gas Transmission Pipelines." Journal of Pressure Vessel Technology **131**(3): 031702-6.

This paper briefly reviews the technology development in the reliability-based design and assessment area. Then the progress in the past years in standard development within the ASME and Canadian Standard Association organizations is discussed.

Scour

188. Johnson, P. A. and S. L. Niezgoda (2004). "Risk-Based Method for Selecting Bridge Scour Countermeasures." Journal of Hydraulic Engineering **130**(2): 121-128.

In this paper, a risk-based method for ranking, comparing, and choosing the most appropriate scour countermeasures is presented using failure modes and effects analysis and risk priority numbers (RPN). RPN can provide justification for selecting a specific countermeasure and the compensating actions needed to prevent failure.

189. Stein, S. and K. Sedmera (2006). NCHRP Report 107: Risk-Based Management Guidelines for Scour at Bridges with Unknown Foundations, Transportation Research Board 364.

This report presents a risk-based approach to managing bridges in the absence of foundation information, primarily with respect to scour. The guidelines illustrate how to collect data, estimate risk of failure, and use risk as a structured approach to selecting an appropriate maintenance plan.

State and Government Manuals

190. AASHTO (2002). Commonly Recognized (CoRe) Structural Elements**:** 54.

This guide provides a description of structural elements that are commonly used in highway bridge construction and encountered on bridge safety inspections. These descriptions are meant to provide a uniform basis for data collection for any bridge management system and to enable sharing among the states.

191. Caltrans (2008). Element Level Inspection Guide. C. D. o. Transportation, California Department of Transportation**:** 93.

This manual provides descriptions of all elements used in bridge structures in California. Aside from element numbers and descriptions, there are also guides to assigning condition ratings based on visual description, with corresponding recommended actions.

192. Cambridge Systematics, I. (2004). PONTIS 4.4 User's Manual. Washington, D.C., AASHTO.

This document is the user’s manual for the PONTIS version 4.4 software package. This manual will assist bridge owners in managing their bridge inventory through providing guidance and troubleshooting.

193. Defense, D. o. (1980). Military Standard 1629A: Procedures for Performing a Failure Mode, Effect and Criticality Analysis. D. o. Defense. Washington, D.C., Department of Defense.

This standard establishes requirements and procedures for performing a failure mode, effects, and criticality analysis (FMECA) to systematically evaluate and document the potential impact of functional or hardware failure.

194. FHWA (1995). Recording and Coding Guide for the Structural Inventory and Appraisal of the Nation's Bridges. U. S. D. o. Transportation, U.S. Department of Transportation.

This guide has been prepared to assist States, the Federal Government, and other agencies in the recording and coding of the data elements that will comprise the National Bridge Inventory database.

195. Florida, D. o. T. (2007). Bridge and Other Structures Inspection and Reporting. F. D. o. Transportation, Florida Department of Transportation.

The purpose of this manual is to establish procedures for inspection and reporting requirements for bridges and other structures under the responsibility of the Florida Department of Transportation.

196. Herman, R. Criticality - Mil-Std 1629 Approach.

This PowerPoint presentation outlines the basic approach of Mil-Std 1629. The presentation begins with background information on the differences between quantitative and qualitative approaches. The rest of the presentation is focused on the determination of criticality.

197. NYSDOT (1987). Structural Condition Formula. Albany, NY, New York State Department of Transportation.

This document explains the structural condition formula. This formula assigns weights to each element in proportion to their importance to the bridge. The sum of the weighted condition values for all elements divided by the sum weighed values results in the condition rating for the bridge.

198. NYSDOT (1999). Bridge Inspection Manual. N. Y. S. D. o. Transportation. Albany, New York, New York State Department of Transportation

This manual is a guide to performing bridge inspection as required by the New York State's Uniform Code of Bridge Inspection. It must be used when inspecting any publicly owned, operated or maintained bridge in New York State that is open to vehicular traffic.

199. NYSDOT (2006). Bridge Inventory Manual. N. Y. S. D. o. Transportation. Albany, New York, New York State Department of Transportation.

This manual describes the New York State Data Management System (BDMS); what data is stored, how data is entered and how it is used.

200. TXDOT (2002). Bridge Inspection Manual. T. D. o. Transportation. Austin, Texas, Texas Department of Transportation 147.

This manual provides guidance for bridge inspection personnel and helps ensure consistency in bridge inspection, rating, and evaluation.

Terrorism

201. Burrows, S. K. and S. L. Ernst (2007). Risk Management for Terrorist Threats to Bridges and Tunnels**:** 6.

In this report, the authors present several techniques used to protect vulnerable transportation infrastructure. The first step is to create a response plan which involves local institutions such as the police, the fire department, hospitals, and engineers. From there, bridge owners must focus on the four D's: Deter, Deny, Detect, and Defend.

202. Ray, J. C. (2007). "Risk-Based Prioritization of Terrorist Threat Mitigation Measures on Bridges." Journal of Bridge Engineering **12**(2): 140-146.

This paper describes a risk-based methodology developed to facilitate prioritization of terrorist threat mitigation strategies on individual bridges. This methodology is unique in that it is specifically designed for a single bridge and the risk associated with each of its individual components.

203. Rummel, T., M. Hyzak, et al. (2003). Transportation Security Activities in Texas. T. D. o. Transportation. Austin, Texas**:** 6.

This document explains the five-step approach that Texas has taken to ensure transportation security. These steps include: identifying the most critical bridges, develop options for surveillance and detection, research in cooperation with outside experts, consider military transportation needs, and provide procedure to both internal and external departments.

204. Stewart, M. G., M. D. Netherton, et al. (2006). "Terrorism Risks and Blast Damage to Built Infrastructure." Natural Hazards Review **7**(3): 114-122.

In this paper, a probabilistic risk assessment procedure is developed to predict risk of damage arising from blast damage to built infrastructure. Issues related to risk assessment, “risk transfer”, and comparisons with natural hazards are discussed.

Vulnerability Manuals

205. NYSDOT (1997). Concrete Details Vulnerability Manual. N. Y. S. D. o. Transportation. Albany, New York, Bridge Safety Assurance Unit.

This manual contains a series of assessment procedures that result in a concrete details vulnerability rating for a structure. This rating describes the likelihood and the consequences of a failure. These ratings may be used in conjunction with other ratings in order to establish priorities for corrective action.

206. NYSDOT (2000). Steel Details Vulnerability Manual. N. Y. S. D. o. Transportation. Albany, New York, Bridge Safety Assurance Unit.

This paper discusses a series of assessment and evaluation steps which result in a steel details vulnerability rating. This rating describes the likelihood and the consequences of a failure in terms of the corrective actions which are required and the urgency in which those actions need to be implemented.

207. NYSDOT (2003). Hydraulic Vulnerability Manual. N. Y. S. D. o. Transportation. Albany, New York, Bridge Safety Assurance Unit.

This document sets forth a series of assessment and evaluation steps which result in a hydraulic vulnerability rating for a structure. This rating describes the likelihood and the consequences of a failure in terms of the corrective actions which are required and the urgency in which those actions need to be implemented.

Appendix B: State 48 Month Inspection Policy Summary

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B.1 INTRODUCTION

The National Bridge Inspection Standards mandate that routine bridge inspections take place no less than every two years. However, a special program has been developed which allows states to inspect bridges on an extended frequency of four years, if those bridges meet certain requirements. These requirements are developed by the individual states to meet their own specific needs and then sent to the Federal Highway Administration for approval.

The purpose of this appendix is to present the extended inspection interval criteria which have been developed on the state level to provide guidance for other entities who may wish to create their own extended inspection programs, such as that outlined by this research. It is important to note that the criteria presented in this appendix is used to either exclude or include a structure for a set extended inspection frequency of 48 months. This is in contrast to the methodology presented by this research program where bridges can be placed on varying levels of extended inspection frequencies based on the specific quality of certain attributes. Nevertheless, the considerations presented here are important to the development of risk-based inspection programs and show that risk-inspection programs are already in place, although in an ad-hoc fashion.

Section B.2 of this appendix will present the particular criteria developed by each participating state for a structure to be placed on an extended inspection frequency. A complete listing of the attributes developed by this research program and their associated commentary can be seen in Appendix C.

B.2 STATE 48 MONTH INSPECTION CRITERIA

This section will present criteria which are used by specific states to place their highway structures on an extended inspection frequency of four years. The criteria listed in this section have been copied either from internal state 48 month inspection policies or from personal communications between the research team and state personnel. These criteria are presented based on the general item code they relate to from the FHWA's Recording and Coding Guide [1].

B.2.1 Structure Type

Table B1. State 48 month inspection criteria for structure type.

State	Criteria to receive an extended inspection frequency
Arizona	Culverts
Arkansas	RC box culverts with contract design plans
California	Bridges
Colorado	Slabs, Stringer/Multi beam/Girder, Tee Beams, Box Beams/ Box Girder, Culverts
Connecticut	Bridges, Culverts
Illinois	Several bridge and culvert types
Mississippi	Prestressed concrete, pre-cast concrete, concrete box bridges with load path redundancy
Montana	Prestressed Beams, Prestressed T-Beams
New Mexico	Culverts
North Dakota	Concrete Box Culverts
Oklahoma	Bridges and Culverts
Oregon	N/A
South	Concrete box culverts, continuous concrete slab bridges, continuous

Dakota	prestressed concrete girder bridges, and concrete frame bridges. Each may have further specific criteria.
Texas	Culverts on the state highway system
Washington	Common concrete bridges or steel culverts
West Virginia	Highway Bridges and Culverts

B.2.2 Structure Condition Rating

Table B2. States 48 month inspection criteria for structure condition rating.

State	Criteria to receive an extended inspection frequency
Arizona	≥ 6
Arkansas	≥ 7 (RC box culverts only)
California	≥ 6
Colorado	≥ 6
Connecticut	≥ 6 overall
Illinois	≥ 7
Mississippi	≥ 7
Montana	≥ 6
New Mexico	≥ 5
North Dakota	≥ 6
Oklahoma	≥ 2 for concrete structures ≥ 3 for steel structure
Oregon	N/A
South Dakota	≥ 6
Texas	≥ 5
Washington	≥ 6
West	≥ 6

Virginia	

B.2.3 Scour Rating

Table B3. States 48 month inspection criteria for scour rating.

State	Criteria to receive an extended inspection frequency
Arizona	N/A
Arkansas	N/A
California	≥ 4 (those not evaluated or which have countermeasures are excluded)
Colorado	≥ 5
Connecticut	≥ 7
Illinois	≥ 5
Mississippi	≥ 7
Montana	N/A
New Mexico	≥ 6
North Dakota	N/A
Oklahoma	≥ 6
Oregon	N/A
South Dakota	N/A
Texas	N/A
Washington	N/A
West Virginia	≥ 5

B.2.4 Structure Age

Table B4. States 48 month inspection criteria for structure age.

State	Criteria to receive an extended inspection frequency
Arizona	Built in 1930 or later
Arkansas	N/A
California	N/A
Colorado	≤ 50 years unless reconstructed in the last 30 years
Connecticut	≥ 4 years with no significant problems after an in-depth inspection ≤ 50 years if steel
Illinois	≤ 50 years unless reconstructed in the last 30 years
Mississippi	≤ 25 years
Montana	N/A
New Mexico	1 to 60 years
North Dakota	≥ 5 years (if rehabilitated, needs three inspections)
Oklahoma	≥ 3 years (needs a post construction inspection and a routine inspection)
Oregon	N/A
South Dakota	≥ 4 years
Texas	≥ 2 years but ≤ 50 years
Washington	N/A
West Virginia	≥ 2 years with at least one in-depth periodic condition and inventory inspection ≤ 50 years unless reconstructed in the last 25 years

B.2.5 Inventory Rating

Table B5. State 48 month inspection criteria for inventory rating.

State	Criteria to receive an extended inspection frequency
Arizona	$\geq$ HS20
Arkansas	$\geq$ 15 tons
California	$>$ 32 metric tons (HS20)
Colorado	$\geq$ HS18 for bridges $\geq$ HS15 for culverts under fill
Connecticut	$\geq$ max legal limit
Illinois	$\geq$ HS18 for bridges $\geq$ HS15 for culverts under fill
Mississippi	$\geq$ 36 tons
Montana	$\geq$ HS20
New Mexico	$\geq$ HS17
North Dakota	$\geq$ 36 tons
Oklahoma	$\geq$ State legal loads
Oregon	N/A
South Dakota	$\geq$ State legal loads
Texas	$\geq$ HS20
Washington	$\geq$ HS20
West Virginia	$\geq$ HS30

B.2.6 Span Length

Table B6. State 48 month inspection criteria for span length.

State	Criteria to receive an extended inspection frequency
Arizona	N/A
Arkansas	N/A
California	≤ 100 feet
Colorado	≤ 100 feet
Connecticut	≤ 100 feet (Except for pipes and culverts, max number of spans is two)
Illinois	≤ 100 feet
Mississippi	≤ 100 feet
Montana	≤ 100 feet
New Mexico	Box opening ≤ 15 feet if concrete Box opening ≤ 10 feet if metal ≤ 100 feet max., any structure
North Dakota	N/A
Oklahoma	≤ 100 feet
Oregon	N/A
South Dakota	≤ 100 feet
Texas	Box opening ≤ 12 feet
Washington	≤ 150 feet
West Virginia	≤ 100 feet

B.2.7 Average Daily Traffic

Table B7. State 48 month inspection criteria for average daily traffic.

State	Criteria to receive an extended inspection frequency
Arizona	N/A
Arkansas	$< 50{,}000$ (RC box culverts only)
California	$\leq 20{,}000$ per lane
Colorado	$\leq 30{,}000$
Connecticut	$\leq 50{,}000$ (steel with no fatigue prone details) $\leq 125{,}000$ (concrete) Does not apply to buried structures with more than 3 feet of cover.
Illinois	$\leq 30{,}000$ except culverts under 2 or more feet of fill
Mississippi	$\leq 25{,}000$
Montana	$\leq 50{,}000$
New Mexico	$\leq 50{,}000$
North Dakota	N/A
Oklahoma	$\leq 5{,}000$
Oregon	N/A
South Dakota	$\leq 3{,}000$ except culverts under 2 or more feet of fill
Texas	$\leq 100{,}000$
Washington	$\leq 100{,}000$
West Virginia	$\leq 15{,}000$

B.2.8 Average Daily Truck Traffic

Table B8. State 48 month inspection criteria for average daily tuck traffic.

State	Criteria to receive an extended inspection frequency
Arizona	N/A
Arkansas	N/A
California	≤ 12%
Colorado	≤ 3,000 (does not apply to culverts with more than 2 feet of cover)
Connecticut	≤ 5,000 steel ≤ 12,500 concrete
Illinois	≤ 3,000
Mississippi	≤ 2,500
Montana	≤ 5,000
New Mexico	N/A
North Dakota	N/A
Oklahoma	≤ 500
Oregon	N/A
South Dakota	≤ 600
Texas	≤ 25,000
Washington	≤ 10,000
West Virginia	≤ 2,000

B.2.9 Vertical Clearance

Table B9. State 48 month inspection criteria for vertical clearance.

State	Criteria to receive an extended inspection frequency
Arizona	N/A
Arkansas	N/A
California	≥ 14 feet (all trusses will be ineligible)
Colorado	N/A
Connecticut	≥ 14′3" with no history of repeated impacts
Illinois	≥ 14 feet
Mississippi	Thru-truss and overpass/underpass are not eligible
Montana	≥ 14 feet
New Mexico	N/A
North Dakota	N/A
Oklahoma	≥ 15 feet
Oregon	N/A
South Dakota	≥ 16 feet
Texas	N/A
Washington	≥ 14 feet
West Virginia	≥ 14 feet

B.2.10 Miscellaneous Criteria

Table B10. Miscellaneous state 48 month inspection criteria.

State	Criteria to receive an extended inspection frequency
Arizona	Any culvert with a history of major maintenance in the last 2 years will be inspected every 2 years until it meets the other requirements.
Arkansas	N/A
California	Excluded Structures: open spandrel arches, fracture critical, multiplate, timber, Alkalai Silica Reactivity suspected or identified, no load path redundancy, uncommon designs, new or newly rehabilitated without a clean inventory inspection and clean routine inspection, unknown foundations
Colorado	Excluded Structures: Recently constructed or rehabilitated without a routine inspection more than one year after end of activity, no load redundancy, uncommon design
Connecticut	Excluded Structures: Steel Grider and Floorbeam, Concrete and Steel Box Girders w 3 or less box beams, orthotropic deck, deck truss, thru and pony truss, thru arch, suspension, stayed girder, moveables, segmental box girder, aluminum, wrought iron, cast iron except for aluminum pipes, no load path redundancy, fracture ciritical,
Illinois	Must have one inspection since Jan. 1994. If new or recently received major maintenance, must have at least one regular inspection. Must have load path redundancy. Not part of the Defense Highway Critical Facilities or the primary responsibility of a federal agency.
Mississippi	Any structure with a history of maintenance rehabilitation or reconstruction will not be eligible
Montana	Any new or rehabilitated structure must have a post construction and a two year inspection before extending the frequency.
New Mexico	After major maintenance, culvert will be inspected in 2 years and if applicable will be included.
North	Waterway Adequacy ≥ 4

Dakota	
Oklahoma	Must have load path redundancy. Must not have a history of recurring maintenance problems. No unusual designs.
Oregon	N/A
South Dakota	No scour critical, only repair work may be a deck overlay or bridge rail retrofit or rail replacement. No concrete slab Umbrella-type bridges. Steel girder excluded due to several reasons. Must have deck chloride protection. Post-Rehabilitation procedure is similar to other states. No fracture critical
Texas	No major maintenance in the last 2 years.
Washington	Must have load path redundancy, no uncommon designs. Not scour critical.
West Virginia	No fracture critical, fatigue prone details, no pin and link details, or direct bearing hinge details. No unknown foundations or uncommon designs.

B.3 REFERENCES

1. FHWA, *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges*, 1995, Federal Highway Administration: Washington, D.C.

Appendix C: Attribute Index and Commentary

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LIST OF ATTRIBUTES

Screening Attributes

Design Attributes

C.1 INTRODUCTION

This appendix includes suggested attributes for the reliability assessment of bridges. Users may select attributes from this sample listing but it is recommended that users develop additional attributes that meet the needs of their individual agency. This attribute listing and commentary is organized into four sections relating to attribute type: 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 the group reliability assessment phase. 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 relative to other bridges in a group

Design attributes of a bridge element are those characteristics which describe its design, construction and maintenance. 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 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 (ADT, ADTT), 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 it 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 or refining attributes which 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.

C.2 SCORING SCHEME

Attributes are assigned points based on the importance or the 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 C1. The Ranking Descriptor is intended to provide some verbal description of the weight associated with each score. As shown, three relative levels are presented: Low, Moderate, and High. The Reliability Assessment Panel (RAP) may wish to modify the suggested scoring for a given attribute, based on local conditions, past experience and previous performance within their bridge inventory and operational environment. The scoring scheme should effectively develop sound engineering rationale to support reliability-based inspection practices.

Table C1. Suggested score ranking for attributes.

Ranking Descriptor	Total Points
High	20
Moderate	15
Low	10

C.2.1 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 four 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 a more in-depth analysis to identify their inspection needs. Users could also assign an Occurrence Factor of high without further assessment, since the component is already in poor condition.

Assessment Procedure

This screening attribute is scored based on whether the current condition rating is four or less or greater than four. The current condition rating from the most recent inspection report should be used.

Current condition rating is less than or equal to four	Component is screened from general reliability assessment
Current condition rating is greater than four	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 from a larger group or family of bridges which do not. In such cases, specialized, individual reliability analyses 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 which has reduced the load bearing capacity of the members. In any case, large flexural 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 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 on the bottom flange at mid span or on 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 which 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 towards 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 one year). Primary stresses are those stresses (i.e., stress ranges) which 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 constraint induced fracture (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. 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

C.2.2 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 for a given bridge or a family of bridges. 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. Evaluators 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 depend on the traffic composition of the roadway below, such as the 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 for Items 54 and 26, record the vertical clearance and the functional classification of the route passing under the bridge, and are rated using the model provided in the recording and coding guide [2], which is provided in Table C2.

Table C2. FHWA Coding Guide minimum vertical underclearance provisions [2].

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			

Assessment Procedure

This attribute should be scored based on the 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. Evaluators may wish to consider the functional classification or the typical traffic pattens below the bridge in their assesement. 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 greater than 15 feet to 16 feet	15 points
Vertical clearance is greater than 16 feet to 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 to 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 to 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. Evaluators 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 advances on the most prevalent aging mechanisms in highway bridges; deterioration of concrete associated with corrosion of embedded reinforcement, for concrete structures, 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, relative 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 should not be susceptible to fatigue due to primary or secondary stress ranges.

1994

In 1994, the AASHTO design specifications changed from load factor design (LFD) to 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 the greatest reliability.

2009

In 2008, language was introduced into the AASHTO LRFD Bridge Design Specifications which directly addressed the issue of constraint induced fracture (CIF). The article provided prescriptive guidance on detailing 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 which 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; a user 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 by 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 [3].

An overlay is defined herein as an additional layer of protective material, which is applied on top of the concrete deck and which 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 which 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 [4]. Types of sealers include silanes, siloxanes, silicates, epoxies, resins, and linseed oil.

Surface coatings such as epoxy, coal tar epoxy, polyurethane, or polyurea, acrylic 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 relative 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.

Users may wish to separate certain overlays or membrane systems, based on their past experience. 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 "high performance," 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 high performance concrete 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."

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, evaluators 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.

Stay in place 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 as determined through visual inspection. This attribute is intended to consider the increased level of uncertainly in the deck condition that may exist for decks with overlays.

Assessment Procedure

This attribute is assessed based on the whether 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 [4]. 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 two 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 which 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 require 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 (Skeet et al 1994). 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 C1. In summary, as concrete cover increases, the time to corrosion initiation increases due to the increased depth of which 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 three inches can result in increased cracking, providing pathways for the intrusion of water and chlorides. This may be a consideration in special cases where the concrete cover is unusually large.

Assessment Procedure

This attribute is scored based on the actual, physical clear cover that the specified bridge element operates with. 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.

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

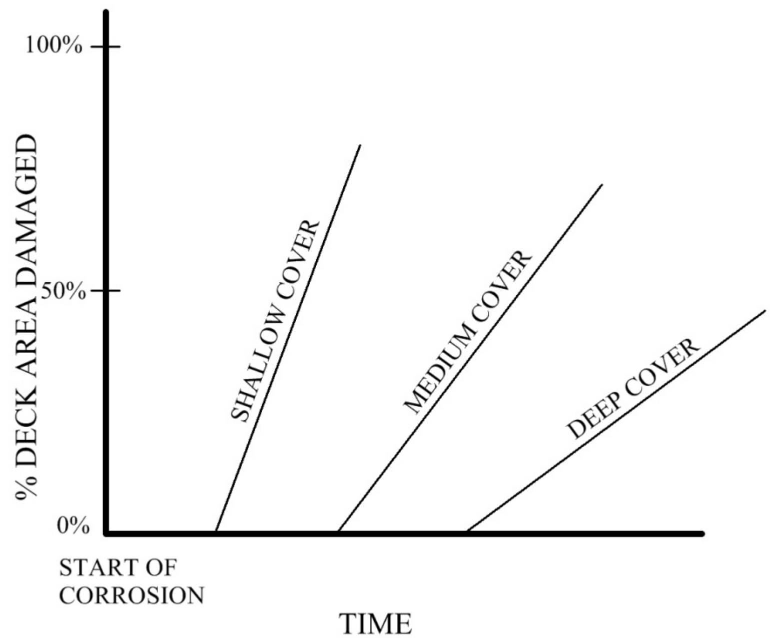

Figure C1. Effect of concrete cover on the time to corrosion initiation and development of damage (Skeet et. al).

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 [5]. This significant increase for when both mats are coated is due to the fact that the electrical resistance increases, 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 five more years to the 10 to 15 years required for corrosion induced stress to manifest in unprotected bridge decks [5].

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 [5].

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 (HPS), generally have a greater fracture toughness than that required by ASTM A709 and of other common bridge steels. By having an improved fracture toughness, HPS are 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 Charpy V-Notch (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 at this time, if the steel just meets the ASTM A709 specification, is questionable, especially for HPS70 and HPS100. This may change as future research becomes available and the minimum required CVN values increase.

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 comprised 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. 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. For welded details that have low fatigue resistance, such as D, E, and E', along with residual stresses and weld toe defects, fatigue cracking is generally of greatest concern.

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 as 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 into 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 fastener, 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 immediate 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 probability of occurrence for 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

C.2.3 Loading Attributes

L.1 Average Daily Truck Traffic (ADTT)

Reason(s) for Attribute

The Average Daily Truck Traffic 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 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 [3]. 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 will have a higher probability of fatigue damage. Of course the converse is 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 D.16 Element Connection Type and D.17 Worst Fatigue Detail Category. 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 Average Daily Truck Traffic. For steel structures, the scoring limits for ADTT were taken from a recent study on fracture critical bridge titled “A Method for Determining the Interval for Hands-On Inspection of Steel Bridges with Fracture Critical Members” [6]. 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 ten. 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.

Evaluators 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 [7].

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 which are already in practice. Marine environments are deemed to be the most severe due to the high levels of ambient chlorides and moisture. A “moderate” environment is intended to characterize those environments where 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 where application of de-icing chemicals is minimal or nonexistent; the environments may be arid, and atmospheric pollutants are 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 chloride ion containing de-icing chemicals 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 [8]. 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 permits 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 chemical are likely to be applied. Evaluators may have other data or information regarding the application of de-icing chemical that can be used to develop rationale 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 which are being picked up and dispersed by travelling 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 lesser 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 ten 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., ten years until replacement) of the element, but is not infinite. Infinite life is the case when 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 which 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

C.2.4 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 five or less to have observable corrosion damage in the form of cracking, delaminations and/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 (six 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 five or less is considered to have a much higher likelihood for accelerated damage than component with higher condition ratings. A condition rating of six is considered to have a lesser likelihood of accelerated damage.

Condition rating is five	20 points
Condition rating is six	5 points
Condition rating is seven 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) are defined that are linked to specific evidence of damage or of deterioration to the subject bridge element. 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 zero points is suggested, for elements where CS 3 is indicated for any portion of the element, a score or 20 points is suggested. Evaluators may wish to utilize appropriate gradations for elements with conditions indicated as CS 2. The severity and the significance of CS 2 varies 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 of 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 which 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.

Evaluators 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 five 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 be leaking in the near future.

Open joints should be expected to allow for the passage of water and debris, and thus should be scored accordingly if this affect is unmitigated. Closed or open joints which are leaking should be given more points due to their failure to function as designed. 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, helps 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 their service lives, 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 their 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 their 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. For concrete bridge elements, impacts can compromise the concrete cover, resulting in the exposure of embedded steel elements. As a result, 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. Unfortunately, some drainage systems function better than others. 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, or section loss in pipes, etc.

High: Deck drains directly onto superstructure or substructure components, or ponding on deck results from poor drainage	20 points
Moderate: 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 as caused by the development of corrosion by-products on the surface of the bar. This expansion leads to cracking of the concrete. 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 (NCHRP, AASHTO et al. 2004). 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 (NCHRP, AASHTO et al. 2004). 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 where 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 level 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, sometimes referred to as patches, are a way to temporary seal the exposed reinforcement resulting from 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 four years to ten years [9], although an FHWA TechBrief indicates that the service life of a patch ranges from four years to seven years [10]. 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 rate of the corrosion process occurs. 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 which has contains repaired areas. Engineering judgment should be exercised.

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 which 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.

Evaluators 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. Evaluators should consider the importance of the spalling in terms of the structural performance of the element under consideration in developing their scoring methodology. For prestressed elements, spalling which leads to the exposure of 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 present 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 which 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) [11]. Those concretes which 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 [11].

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 which has reduced the load bearing capacity of the members. In any case, large flexural 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 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 provides 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 on the bottom flange at mid span or on 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 Precast Prestressed Concrete Institute (PCI) MNL-37-06 Manual for the Evaluation and Repair of Precast Prestressed Concrete Bridge Products

[12]. 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, where any moderate to severe flexural cracking should exclude the bridge from a reliability 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 reliability 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 [13].

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. Use of this attribute in the reliability assessment assumes the cracking in question is minor in nature and has been assessed to determine that 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 in which the coating is in good condition and performing as intended are generally less susceptible to corrosion damage. Elements with significant rusting and corrosion in areas where 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 Average Daily Truck Traffic (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 nonexistence 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/yr (0.010 inches/year). For moderate conditions, rates are typically on the order of 4 mils/yr (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 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, past inspection reports, and bridge inspection experience should be used in determining whether or not the corrosion is active. This attribute may 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 D: Illustrative Scoring Examples

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D.1 INTRODUCTION

This appendix provides three example reliability assessments of typical highway bridges for the purpose of inspection planning. The first is an example of a typical highway bridge built to modern standards and constructed with a superstructure of typical prestressed elements. The second example is an older bridge with a steel superstructure. A third example is of a bridge with a reinforced concrete superstructure.

These bridges are intended to show illustrative examples of implementing a reliability-based inspection analysis. A RAP assembled at the bridge owner level would typically conduct this analysis. The examples also illustrate the process for extending the analysis to other bridges of similar design.

Attribute scoring sheets are shown to illustrate the process of identifying key attributes and applying a numerical scoring process to assess the reliability of bridge elements and develop rationale for determining the appropriate maximum inspection interval. Once attributes and relative scoring methodologies are developed for a given bridge design type or family, the attribute scoring methodology is most readily implemented in software, using data available (or made available) in owner databases or bridge management systems.

In the examples shown, Occurrence Factor categories were determined by simply summing the total possible score for a given damage mode, and calculating the fractional score for the specific bridge for that damage mode on a four-point scale according to 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 $\frac{\sum S_i}{\sum S_o}$ is a value between 0 and 1. Occurrence Factors were then applied such that values of X between 0 and 1 were identified as "Remote," values greater than 1 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.

D.2 EXAMPLE 1: PRESTRESSED CONCRETE BRIDGE

D.2.1 Bridge Profile

This example bridge is constructed of prestressed girders with a composite concrete deck. The bridge has a typical reinforced concrete deck, seven prestressed AASHTO Type IV girders, a reinforced concrete substructure, and 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.

Figure D1. Elevation view of Example Bridge 1.

D.2.1.1 Concrete Bridge Deck

The deck for this structure was cast-in-place and constructed with normal concrete and epoxy-coated reinforcement. From the design plans the concrete cover for the top of the deck is 1 ½ inches. Asphaltic plug joints are present in the deck and have been determined to be in good condition.

Some transverse cracking, spaced two to three feet apart, has 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.

D.2.1.2 Prestressed Girders

The superstructure of this bridge consists of seven AASHTO Type IV prestressed concrete girders. There is at least two 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.

D.2.1.3 Substructure

The substructure was constructed of normal concrete with uncoated carbon steel reinforcement. The minimum design cover was determined to be two 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.

D.2.2 Assessment

This section will show how the methodology is applied to determine the Occurrence Factors, 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 which provides the Inspection Priority Number and the maximum inspection interval.

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 damages 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 considered 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 is:

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

D.2.2.1 Concrete Bridge Deck

The RAP analysis considered that the attributes of a bridge 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 their inventory and other concrete elements. As such, a corrosion profile was developed to rapidly assess the corrosion potential in a concrete bridge deck generally. This profile included typical attributes that were well known to affect the durability of concrete, but did not depend on the current condition of an element. The attributes identified include:

- Poor Deck Drainage
- Years in Service
- Application of Protective Systems

- Concrete Mix Design
- Minimum Concrete Cover
- Reinforcement Type
- Exposure Environment
- Rate of De-Icing Chemical Application
- Maintenance Cycle

Supporting rationale for each of these attributes from the commentary was used. Utilizing this corrosion profile and the suggested rankings in the commentary, the RAP developed a simple scoring sheet to quickly determine the likelihood of developing corrosion related damage for a common concrete bridge deck.

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**

Considering the relevant damage modes, attributes were identified that related to the likelihood of failure for any bare concrete deck. These attributes include the corrosion profile score, plus attributes that relate to the loading and the condition of a particular deck. The RAP made the assumption that the deck under consideration was rated (on the NBIS condition scale) greater than four, and that no fire had occurred without an appropriate follow-up inspection. These attributes were applied as screening criteria. 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 four	Pass
S.2 Fire Damage • No fire damage in the last 12 months	Pass
Corrosion Profile score	60
L.1 Average Daily Truck Traffic (ADTT) • ADTT is moderate (210 vehicles)	10
C.1 Current Condition Rating • Current deck condition rating is seven	0
C.8 Corrosion-Induced Cracking	

• Minor corrosion-induced cracking noted	5
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 observed	5
Corrosion Damage total	**80 out of 290**
Corrosion Damage ranking	**1.1 Low**

This bridge deck, which is still relatively new, has very little damage accumulation. Some minor cracking and efflorescence has been noted but overall the condition of the deck is still good. As a result, the deck scored very low. Based on the attribute scores, the RAP determined that the likelihood of failure for the deck, based on the criteria described, was low, i.e. the Occurrence Factor, O, was Low.

D.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
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 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	Pass

• Superstructure condition rating is greater than four	
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 eight	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
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**

This bridge’s superstructure, which consists of prestressed concrete girders, has a low likelihood of developing serious bearing area damage, based on the key attributes identified and assessed by the RAP. The likelihood of failure due to corrosion between beam ends was determined to be low, the likelihood of shear and flexural cracking was determined to be remote, and the likelihood for strand fracture was also considered to be remote.

D.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 a rotation of one of the substructure elements. However, settlement and rotations were deemed non-credible damage modes because the bridge substructure is founded on rock. To assess the likelihood of the corrosion damage mode, the panel once again used the generalized corrosion profile scoring method to characterize the potential and the likelihood of corrosion damage. The panel then considered appropriate attributes for assessing the likelihood of failure for the corrosion damage modes, 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 eight	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**

The substructure design for this bridge was determined to have a low likelihood of failure due to corrosion damage and few signs of damage are present (rating = 8). For the damage mode of Corrosion Damage, the Occurrence Factor was determined to be Low.

D.2.3 Consequence Assessment

Now that 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 analysis, the most likely failure mode would result in spalling. Since the bridge is over a non-navigable waterway, soffit spalling would have a low consequence. Considering the ADT and the posted speed limit, spalling on the deck surface was determined to have moderate consequence. Hence, considering the damage attribute score and the consequence level, the applicable maximum inspection interval is 72 months for the deck. The rationale for this interval includes that the likelihood of serious damage in the next 72 months is low, and that if it did occur, the consequences be moderate. 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 where 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 they utilized a nearly identical girder spacing and deck configuration.

In both cases, the impact severely damaged at least one of the girders such that the girder's load carrying capacity was effectively reduced to zero. Also, in both cases, the bridge exhibited little or no additional dead load deflection and was 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. Hence, the RAP concluded that the loss of one girder would at most have a moderate consequence. Further, the load rating information for this bridge was reviewed and the bridge possessed a capacity in excess of the required Inventory and Operating ratings. Based on this information, the Consequence category for this superstructure was determined to be Moderate.

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 low, resulting in only a minor disruption of service, since the deck is composite with the superstructure and it's 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, due to damage to the development length of the strands or a loss of section in the shear stirrups. This scenario was considered to be of moderate consequence, based on the same rationale as the other damage modes, that is, a loss of loading bearing capacity for one of the beams. Based on these two scenarios, the Consequence Factor of Moderate 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 panel 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 indicates that serious corrosion damage has a benign immediate effect on the serviceability and safety. Therefore, the consequence category of low was determined.

Based on the assessment and using the acceptable reliability matrix, the maximum inspection interval for this bridge was determined to be 72 months.

D.2.4 Scoring Summary

Table D1 shows a summary of the scoring 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.

Table D1. Reliability assessment scoring summary for Example Bridge 1.

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

D.2.5 Criteria for a Family of Bridges

The RAP assessed that they have many bridges in their 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 their inventory. These criteria described 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 their element-level bridge data.

For example, for the prestressed concrete girders, the panel identified that the individual condition attributes identified by the analysis would be comprised in the element-level inspection data. For a prestressed element, elements that are rated as CS one and CS two would not have the damage characteristics the panel indentified as key to the potential for serious damage to develop. Bridges with CS three would require reanalysis and possibly a reduced inspection interval. Bridges without element ratings of three for the prestressed girder element would not require reanalysis (based on that element). 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 presently represented in their element-level inspection scheme. As a result, the reliability-based inspection practice for bridges in this family needed to include a requirement that the absence of longitudinal cracking be confirmed. This requirement was added to an RBI procedure.

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 bridges in their inventory. These criteria are based on the engineering assessment documented through the RAP analysis. Example criteria to identify the family of bridges include:

- Maximum span length less than 120 feet.
- Four or more AASHTO prestressed girders
- Beam spacing of ten feet or less
- ADTT less than 500 vehicles
- Constructed in 1995 or later

- Condition rating seven or greater for all components
- No structural element with CS three reported
- No joint element with CS three reported
- Load rating meets or exceeds requirements
- Bridge receives RBI-based inspections

The RAP determined that bridges meeting these criteria will be treated uniformly under the RBI methodology. If a particular bridge violates any of these criteria, it is slated for reassessment.

Table D2 summarizes the information from the RAP analysis to be included in the RBI procedure for these bridges. This information includes the identified screening criteria of inspection for cracking attributable to shear and flexure, and to confirm that longitudinal cracking does not exist. Included in the inspection procedure for bridges in this family is the data from the RAP assessment shown in Table below:

Table D2. Table of information to be included in the RBI procedure.

Maximum Inspection Interval: 72 months		
Special Emphasis Items		
S.4 Flexural Cracking S.5 Shear Cracking 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

D.3 EXAMPLE 2: STEEL GIRDER BRIDGE

D.3.1 Bridge Profile

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. All steel reinforcement used in this bridge is regular uncoated mild carbon steel. The observed Average Daily Truck Traffic (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.

Figure D2. Elevation view of Example Bridge 2.

D.3.1.1 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.

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 two to three feet apart, with efflorescence stains. The underside of the approach span at abutment one has heavy efflorescence stains on the left side.

D.3.1.2 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.

D.3.1.3 Substructure

The substructure was constructed of normal grade reinforced concrete with uncoated carbon steel reinforcement. The minimum cover was determined to be 3 3/8 inches. Drainage from the deck is leaking onto the substructure from the deck due to the presence 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 observed 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.

D.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 damages results from the effects of corrosion, these damage modes were group 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
- Fracture Damage

For the substructure, the damage mode considered is:

- 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.

D.3.2.1 Concrete Bridge Deck

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 four	Pass
S.2 Fire Damage • No fire damage in the last 12 months	Pass
Corrosion Profile score	96
L.1 Average Daily Truck Traffic (ADTT) • ADTT is minor (130 vehicles)	5
C.1 Current Condition Rating • Current deck condition rating is six	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
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 likelihood that the deck will deteriorate to a condition meeting the description of failure is low.

D.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 ten years. The RAP agreed that that likelihood of failure of the asphalt overlay was greater than 1% over a 72 month interval, given that the overlay is already in service. The Occurrence Factor for the overlay failure was determined to be High by consensus of the panel.

D.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 attributes is 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	7

• Element is connected by rivets	
D.17 Worst Fatigue Detail Category • Worst fatigue detail category is D	15
L.1 Average Daily Truck Traffic (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 • Bridge constructed in 1954	10
L.1 Average Daily Truck Traffic (ADTT) • ADTT is 130 vehicles	15
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 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 considered indicated that the steel superstructure has a moderate likelihood for fatigue damage, a moderate likelihood of developing corrosion damage with significant corrosion already present, and a low likelihood of developing fracture damage.

D.3.2.4 Substructure

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
Concrete Elements point total	**161 out of 290**
Concrete Elements ranking	**2.22 Moderate**

Based on their analysis, the RAP determined that the likelihood of developing corrosion damage sufficient to meet the failure criteria was moderate. Already, a considerable amount of damage has accumulated in the form of spalling with exposed reinforcement and moderate cracking.

D.3.3 Consequence Assessment

Since the bridge carries a state highway over a non-navigable river, damage to the bridge deck is likely to be in the form of spalling of the bridge deck. The most likely consequence of severe damage to the deck is Moderate because there may be some disruption of service or reduction in posted speed. The bridge is a four girder bridge with typical beam 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 has 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 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

- The bridge is located over a non-navigable river. Thus, the risks to people of 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 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, 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 D3. 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 yrs 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 inspection interval for an RBI is 24 months.

D.3.4 Scoring Summary

The scoring summary for this bridge is shown in Figure D3. Based on the reliability assessment, the maximum inspection interval for this bridge was determined to be 24 months.

Table D3. Reliability assessment scoring summary for Example Bridge 2.

Element	Damage	Occurrence Factor (O)	Consequence Factor (C)	O x C	Interval
Deck	Corrosion Damage	Low (2)	Moderate (2)	4	72 months
Steel Girders					
	Fatigue	Moderate (3)	High (3)	9	24 months
	Corrosion	Moderate (3)	High (3)	9	24 months
	Fracture	Low (2)	High(3)	6	48 months
Substructure					
	Corrosion Damage	Moderate (3)	Low (1)	3	48 months

D.3.5 Inspection Data

Table D4 summarizes the information from the RAP analysis to be included in the RBI procedure to be used for this bridge. This information includes the identified screening criteria of inspection for fatigue cracking and section loss. This table also indicates that for this bridge, the priority items for inspection include fatigue cracking and corrosion damage. Based on the high IPN number for corrosion damage, the RAP recommended utilizing an ultrasonic thickness gauge (UT-T) to assess the areas of section loss to ensure accurate reporting of the remaining section and to mitigate the risks associated with severe section loss. For fatigue cracking, the typical RBI hands-on inspection was considered adequate.

Table D4. Table of information to be included in the RBI procedure.

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

D.4 EXAMPLE 3: REINFORCED CONCRETE BRIDGE

D.4.1 Bridge Profile

This example bridge is a typical, simply-supported three-span reinforced concrete bridge with a bare cast-in-place deck. The bridge owner has more than 100 bridges in their inventory 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 with uncoated reinforcement and highway traffic over a local road. The observed Average Daily Truck Traffic on the bridge is 5,500 vehicles, while the observed ADTT on the local road under the bridge is 18 vehicles. Both the rate of salt application and the surrounding environment are considered to be moderate.

Figure D3. Elevation view of Example Bridge 3.

D.4.1.1 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. As well, hairline diagonal cracks with efflorescence stains have been observed on the soffit of the deck near the abutments. No delaminations are noted on the deck.

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 condition state 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.

D.4.1.2 Reinforced Concrete Girders

The superstructure for this bridge consists of seven reinforced concrete girders which 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 three inches wide by five inches deep due to impact. No exposed reinforcement is noted.

The left exterior girder, girder one, has an eight 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 condition state 5-Fair. From the design plans, the minimum concrete cover is 3 5/8 inches.

D.4.1.3 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 eight inches tall near girder six. Bent cap two, span three, also has an area of cracking and delamination that is 27 inches wide by four inches tall near girder six.

The substructure contains neoprene pad bearings which have curled on the ends but are still in satisfactory condition. Lead plates have been placed under the girders at both abutments and bent one, span one, and at bent two. 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.

D.4.2 Assessment

The primary elements of this bridge are a concrete bridge deck, reinforced concrete girders, deck joints, piers and abutments. For the concrete bridges 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 shear cracking in this type of bridges. 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 a relevant 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 identifies that that likelihood of overload has the greatest effect on the likelihood of flexural cracking progressing; the existence of hairline flexural cracks has a moderate effect, and the fact that bridge is load posted has only a small effect. The load posting was considered to have only a small effect because if the likelihood of overload (above the posting) was small, and there were no existing flexural cracks, there would be little reason for flexural cracks to develop based simply on the fact that it was load posted. As such the RAP assigns 20 pts to L.4, Likelihood of Overload, only 10 pts. to D.2, Load Posting and 15 points to C.14, Flexural Cracking. The attributes are therefore as follows:

- S.4 Flexural Cracking (screening criteria)
- D.2 Load Posting (10, 0 pts)

- L.4 Likelihood of Overload (20, 10, 0 pts)
- C.14 Flexural Cracking (15, 0 pts)

The screening criteria for flexural cracking (S.4) will be utilized to identify bridges with significant flexural cracking, which will receive individual assessment on an as-needed basis. For shear cracking, the relevant attributes to be used are:

- S.5 Shear Cracking (screening criteria)
- D.2 Load Posting (10, 0)
- L.4 Likelihood of Overload (20, 10, 0 pts)
- C.15 Shear Cracking (15, 0 pts)

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

For the substructure, the damage mode considered was:

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

D.4.2.1 Concrete Bridge Deck

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 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 last 12 months	Pass
Corrosion Profile score	86
L.1 Average Daily Truck Traffic (ADTT) • ADTT is high (5,500 vehicles)	20
C.1 Current Condition Rating • Current deck condition rating is six	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
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 likelihood that the deck will deteriorate to a condition meeting the description of failure is low.

D.4.2.2 Reinforced Concrete Girders

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	15

• Reinforcement is uncoated mild steel	
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 contains a simple open joint system	10
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	**121 out of 240**
Bearing Area Damage ranking	**2.02 Moderate**

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.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 275**
Corrosion Between Beam Ends ranking	**2.27 Moderate**

Flexural Cracking, Reinforced Concrete Girder	
Attribute	Score
S.4 Flexural Cracking • Hairline flexural cracking noted, determined to be benign	Pass
D.2 Load Posting • Bridge is not load posted (10, 0)	0
L.4 Likelihood of Overload (20, 10, 0)	10

• Likelihood of overload is moderate	
C.14 Flexural Cracking (15, 0) • Hairline flexural cracking noted	15
Flexural/Shear Cracking point total	**25 out of 45**
Flexural/Shear 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 (10, 0) • Bridge is not load posted	0
L.4 Likelihood of Overload (20, 10, 0) • Likelihood of overload is moderate	10
C.15 Shear Cracking (15, 0) • No shear cracking	0
Flexural/Shear Cracking point total	**10 out of 45**
Flexural/Shear Cracking ranking	**0.88 Remote**

The RAP analysis of key attributes for the damage modes considered indicated that the reinforced concrete superstructure has a moderate likelihood factor due to bearing area damage, a moderate likelihood factor due to corrosion between beam ends, a moderate likelihood factor due to flexural cracking and remote likelihood factor for shear.

D.4.2.3 Substructure

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
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	**156 out of 290**
Concrete Elements ranking	**2.15 Moderate**

Based on their analysis, the RAP determined that the likelihood of failure due corrosion damage was moderate. Already, a considerable amount of damage has accumulated in the form of localized delaminations and spalling resulting in exposed reinforcement.

D.4.3 Consequence

For the concrete bridge deck, based on the damage mode analysis, the most likely failure mode would result in spalling of the concrete. 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, of which 5,500 vehicles are heavy trucks. The roadway below the bridge carries 600 vehicles a day, of which 18 vehicles are heavy trucks. Based on this information, any spalling that takes place on or below the bridge has the potential chance for a piece of spalled concrete to strike a motorist. However, traffic volumes and speed below the bridge are relatively low. Therefore, the consequence scenario for this damage mode is considered to be high.

For the concrete girder superstructure, in order to determine the consequence factor, 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. Regardless of the failure scenario, the bridge carries heavy traffic loading and is over a roadway with low to moderate ADT, such that a local member failure would cause a disruption in service and be an urgent repair need. As a result, the consequences could be high. .

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, resulting in only a minor disruption of service, since the deck is composite with the superstructure and it's 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 was assigned.

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. Considering that this bridge is located over a roadway, there is some chance that falling concrete may strike a passing motorist. Based on this, the consequence scenario for this damage mode is high.

D.4.4 Scoring Summary

Table D5 shows a summary of the scoring for this bridge. Based on the amount of damage accumulation and the presence of a roadway beneath the bridge, the maximum inspection interval was determined to be 24 months.

Table D5. Reliability assessment scoring summary for Example bridge 3.

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

D.4.5 Inspection Data

Table D6 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 cracking due to shear and flexure.

Table D6. Table of information to be included in the RBI procedure.

Maximum Inspection Interval: 24 Months		
Special Emphasis Items		
S.4 Flexural Cracking S.5 Shear Cracking		
RBI Damage Modes		
Element	**Damage Mode**	**IPN**
Deck	Corrosion Damage	6
Reinforced Concrete Girder	Bearing Area Damage	9
	Corrosion Damage between the beam ends	9
	Flexural Cracking	9
	Shear Cracking	3
Substructure	Corrosion Damage	9

Based on the RAP assessment, this particular bridge has high IPN numbers for damage related to corrosion in the soffit area of the bridge (corrosion between the beam ends, bearing area damage on concrete deck), and requires delamination surveys to be conducted to fulfill the requirements of the assessment process (i.e. score the delamination attribute completely). Available technologies to complete the delamination survey for the soffit area of the bridge include hammer sounding, infrared thermography (IR) and Impact Echo (IE). Both hammer sounding and IE require hands-on inspection access. IE is time consuming and provides a level of detail above what is required for general evaluation of the delamination in the soffit. The RAP identifies a delamination survey to be included in the inspection procedures for this and other bridges in this family, by either hammer sounding or IR.

Appendix E: Sample Expert Elicitation Process

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E.1 INTRODUCTION

This appendix provides an example expert elicitation as an illustration of the RAP process. As part of the research for NCHRP 12-82, an expert panel was assembled which consisted 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 risk-based inspection 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 from the district level, through the state-wide programs for inspection, as well as maintenance engineers.

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 weighing bridge element attributes which 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 elicitation process which has been outlined in the Guidelines for conducting the reliability analysis needed as part RBI practices.

This appendix 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 for 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.

E.2 DAMAGE MODES

The first step in the RBI expert elicitation process is to have the RAP identify damage modes for the bridge element under assessment. In this step, the goal is to identify the most likely and credible damage modes for an element and then establish a consensus among the panel regarding the most common damage modes for that element. For the sample workshop, the panel was asked to perform this assessment for a hypothetical steel girder bridge. The following question was posed to the panel “You are told a steel girder is condition state 3, serious condition, according to the current NBI rating scale. Based on your experience, what damage is likely to be present?” The experts were provided a form similar to that shown in Table E1, except that the damage mode likelihood indicators were left blank. Each member of the panel completed the form, identifying which damage modes they thought were most likely by assigning a relative likelihood with a precision of 10%. Table E1 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 fracture 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.

Table E1. 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%

Figure E1 shows the results from each of the panel members for this elicitation exercise. It was the clear 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.

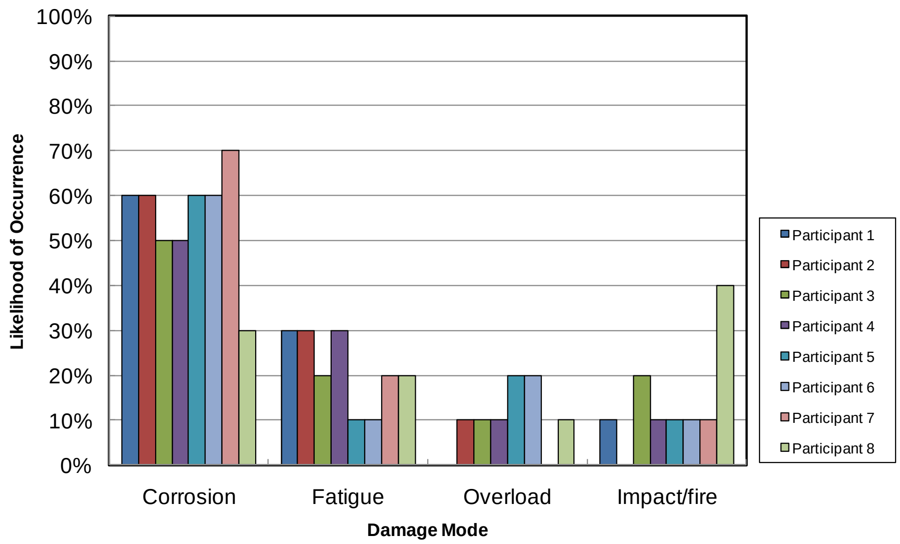

Figure E1. Results of expert elicitation for steel girder damage modes.

The methodology is simple for many bridge elements, where typical damage modes are well known, and establishes the consensus of the panel in regards to the most important 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 their state. The particular state has large areas of arid environment, and hence a different perspective 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 specific bridge inventory are identified through the process since 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 where 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.

E.2.1 Bridge Attributes

Once the primary damage modes were identified, the panel then considered the damage modes individually and determined which element attributes contributed to the reliability of the element. For example, considering the damage mode of corrosion/section lost, 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 determine their relative importance to estimating the future deterioration pattern for the steel girder.

The panel suggested that one of the most important attributes was the maintenance cycle of the bridge, or the maintenance activities that were typically performed as a part of normal operations. This includes such activities as bridge washing, cleaning away of debris that may accumulate, and the 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 a steel bridge elements in terms of corrosion.

The attributes were ranked according to their importance as high (H), moderate (M) or low (L), or if the attribute was potentially a screening criteria (S). Table E2 summarizes the results of the discussion on attributes.

Table E2. 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	M			Debris Accum.	M

As another example of this process, the concept of bridge attributes was further explored by assessing the important attributes for concrete bridge decks. During this discussion, the expert panel agreed that incidence of fire should be considered in the attributes as a screening criteria. It was observed that the effects of fire on a bridge deck typically manifested in the first year after a deck-fire incident. Hence, a given inventory should be screened to provide appropriate inspections following a fire event, even if the other attributes of the deck suggest a longer inspection interval may be appropriate.

The panel also concluded that measuring chloride ion content at the level of the reinforcement was unlikely to be implemented due to the cost, potential damage to the deck from collecting samples, and that the testing does not provide sufficient information beyond the results of a typical inspection to justify its use. There was then a discussion on the effects of girder spacing, superstructure type and deck thickness on the corrosion rates of concrete bridge decks. The panel considered that bridge designs of a particular era were sometimes particularly susceptible to corrosion damage. These factors can sometimes be a characteristic of the specific state in which the analysis was being conducted such that attributes of these types may be needed during an implementation at the state-level.

The panel also discussed the effects of patching on the rate of corrosion in concrete bridge decks. They concluded that patched areas potentially increase the rate of corrosion due to the differing pH levels at the boundaries of the patching. As a result, the existence of a patching on the deck was an important component of assessing the likelihood of future damage developing. Next, the effectiveness of sealers was considered, with some on the panel expressing doubt that sealants were effective. The panel discussed the potential for deck roughness to affect the performance of a deck, with rougher surfaces resulting in increased rates of cracking leading to accelerated corrosion damage. This resulted in a "dynamic damage" attribute being added to the deck attributes for concrete decks. A full listing of the attributes identified for bridge deck can be seen in Table E3.

Table E3. Attributes identified by the expert panel for concrete bridge decks

Design Attributes		Loading Attributes		Condition Attributes	
Attribute	Rank	Attribute	Rank	Attribute	Rank
Cover	H	ADTT	H	Age	L
Rebar coating	H	Salt application	H	Cracking, ASR	H
Mix design - admixtures, HPC, etc.	M	Macro environment - freeze-thaw	H	Delaminations	H
Girder spacing	-	Dynamic loading - rough surface	M	Maintenance cycle	H
Deck thickness	L	Fire damage	S	Presence of spalling	H
Deck form type - SIP, P/C panels	L			Staining/leaching	H
Superstructure type	-			Existing patching	H
Sealers	L			Current condition rating	H, S
Drainage	L			Chloride levels	N/A

It is important to note that the attributes identified by a particular RAP in a specific operational environment may differ from those indicated in Tables E2 and E3, 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 then be developed based on the results and used to categorize the Occurrence Factors based on these attributes.

E.3 CONSEQUENCE SCENARIOS

The expert elicitation process for determining Consequence Factors 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 Consequence Factor within an RBI process.

An exercise was conducted to illustrate and test the use of expert elicitation for evaluating the likelihood of different 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 guidelines for assigning consequence categories, to address situations that may not be sufficiently addressed or unclear, or unique situations for which expert judgment is required. The bridge presented to the panel was a multi-girder steel bridge with an ADT of 2000 and spanning a divided state highway. Photographs of the bridge, as seen in Figure E2, and descriptions of its structural configuration were provided to each member of the panel in written form to aid in assessing different damage modes and their 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 then asked to independently complete the bubble chart for each damage scenario as shown in Table E4. The results were then collected and reduced to summary charts showing the average assigned likelihoods.

Figure E2. Pictures of sample bridge used for evaluating consequence scenarios.

Table E4. Sample table for assessing likelihood for consequence scenarios

Consequence Category	Descriptive definition	Likelihood (%)
Low	Benign, minor serviceability, effect on ride quality, maintenance need, may require monitoring	○○○○○○○○○○
Moderate	Moderate effect on serviceability, some disruption of service possible, reduction in posted traffic speed, requires increased inspection and monitoring, planned repair/maintenance	○○○○○○○○○○
High	Individual/local member failure, major impacts to traveling public, disruption of service, urgent repair need	○○○○○○○○○○
Severe	Structural collapse / loss of life	○○○○○○○○○○

For this phase of the expert elicitation, nine damage scenarios presented to the panel:

Scenario 1. The overlay is debonding; approximately 20% of the deck is spalling.

Scenario 2. The abutment has severe corrosion damage, resulting in rust staining, spalling over 20 % of the surface and some diagonal cracking.

Scenario 3. There are large (6 in.+)out-of-plane distortion cracks found at the diaphragms.

Scenario 4. There is a large fatigue crack found extending 12 in. up the web from the web/flange interface.

Scenario 5. A fracture has occurred in 1 of the steel beams.

Scenario 6. There is severe corrosion damage at the abutment bearing, resulting in > 10 % section loss in the main member.

Scenario 7. The facia girder is damage by an over-height load, causing the flange of the facia girder to be displaced 20 in. from plumb with the top flange.

Scenario 8. Assume that the bearing at the beam-end is a rocker bearing, and it rolls over.

Scenario 9. Prestressed panels have severe corrosion damage at the panel edges, and a 6" x 24" x 1.4" section spalls off of the deck soffit.

The results of this exercise indicated that for certain scenarios there was a strong consensus on the most likely consequence of the indicated damage. For example, consider the results for Scenario 1, as seen in Figure E3. As seen 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, such as if the spalling had reached the level of the reinforcement. Discussion of the assessments quickly yielded that the appropriate Consequence Factor was Moderate.

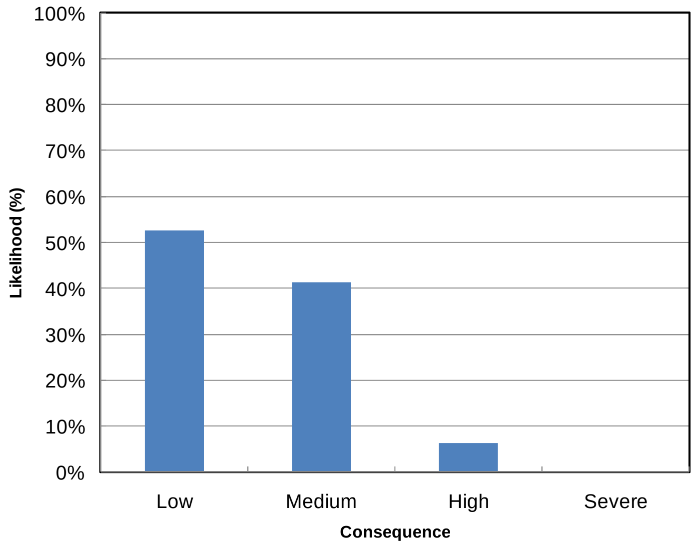

Figure E3. Likely consequences of general deck spalling.

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 (a roughly 50% likelihood), for the latter the most likely consequence was high (greater than 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, with a greater than 60% likelihood that the consequence would be high according to the consequence categories provided. Figure E4 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 be high.

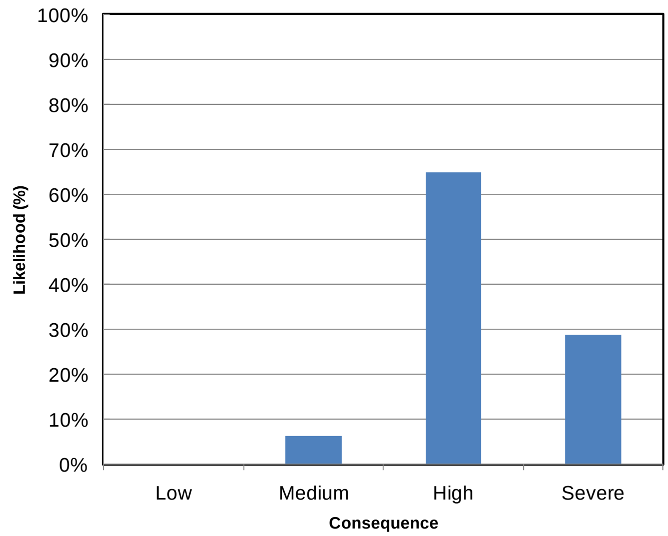

Figure E4. Likely consequences of beam fracture.

For some scenarios, there was a less clear consensus. For example, the experts were asked to determine the most likely consequence for Scenario 9, given that:

"The prestressed panels have severe corrosion damage at the panel edges, and a 6" x 24" x 1.4" section spalls off of the deck soffit,"

A discussion of the results of this scenario, as presented in Figure E5, indicated that the variation in the results of the panel's analysis stemmed in part from whether or not the potential consequences of the concrete falling onto the roadway below was considered. At least one panel member indicated that he based his estimate on the portion of the bridge that was actually over the roadway It was discussed that the location of the damage was not specified in the scenario, and different panel members approached the assessment differently as a result.

In some cases, the panel members did not consider the consequences in terms of traffic below the bridge while in other cases they considered the consequences only in terms of the traffic below the bridge. These results indicated that further guidance was needed in selecting appropriate Consequence Scenarios and as a result, detailed guidance was included in the Guidelines developed by the research team.

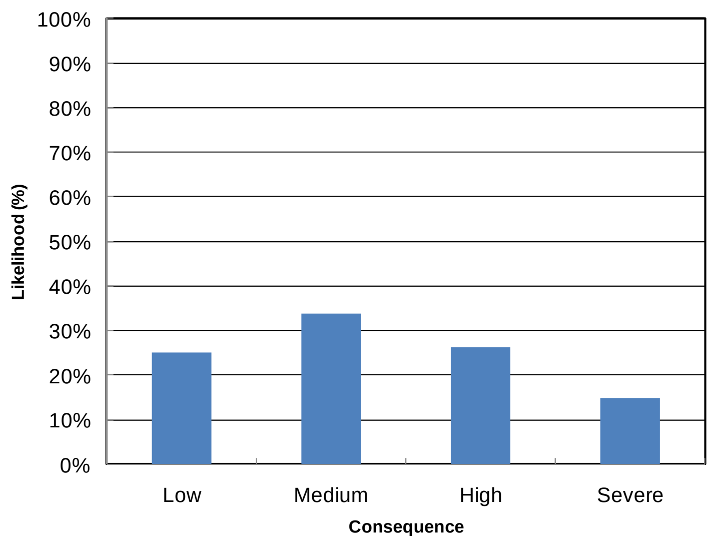

Figure E5. Likely consequences from spalling in deck soffit.

These figures, which illustrate the general 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 Guidelines to assist in this process, and in many cases the Consequence Factors may be governed simply by the criteria in the guidelines or owner policies regarding the treatment of redundancy or other factors. 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 Consequence Factor.

E.4 CONCLUSION

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 is needed for broader implementation of the RBI process, but these data shows successful proof of concept testing and illustration of the process. Further testing and development will be required but was beyond the scope of the current effort, which was focused on the objective of developing the practices included in the Guidelines.

Appendix F: Chloride Diffusion and Embedded Reinforcement Corrosion Analysis

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LIST OF FIGURES

LIST OF TABLES

F.1 INTRODUCTION

In structures with embedded steel reinforcement, metal corrosion can present a substantial risk to structural soundness. The main factors that contribute to metal corrosion are the presence and amount of chloride ions, oxygen, and moisture. In order to better understand how these factors affect structures with varying levels of cover, composition, and location, the commercial software Life-365 has been used to model the chloride diffusion process.

The main objective of this study is to evaluate the potential 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 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 six 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 [1]. Design parameters such as the amount concrete cover, reinforcement 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.

Section F.2 of this appendix will describe the approach taken in illustrating how geographic region affects the time to reach critical chloride levels as well as how Life-365 models the chloride diffusion process. Section F.3 will present the results of the analysis while Section F.4 will present the conclusions drawn from the gathered data.

F.2 MODELING APPROACH

The diffusion modeling program Life-365 uses Fick's second law of diffusion as the governing equation to account not only for changes in location, such as temperature levels and ambient chloride concentrations, but also for changes in concrete composition such as the addition of admixtures, sealers, or different reinforcement types. 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 solid concrete. As time increases, the amount of freely available water within the concrete decreases, which reduces the rate at which chloride ions can diffuse through the 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. Life-365 uses the following relationship to account for time-dependent changes in diffusion:

$$D(t) = D_{ref} * \left(\frac{t_{ref}}{t}\right)^m$$

where $D(t)$ = diffusion coefficient at time t,

D_{ref} = diffusion coefficient at time t_{ref} (28 days in Life-365), and

m = diffusion decay index, a constant

In the above equation, D_{ref} remains a constant value which depends on the water to cementitious material ratio of the concrete mix and the addition of any admixtures, at a concrete age of 28 days. The basic equation to calculate D_{ref}, which is based on a mixture containing only Portland cement, is presented subsequently. Similar to D_{ref}, t_{ref} is also a constant value, which is equal to 28 days.

The diffusion decay index, m, is an expression of the rate of reduction in diffusivity, which depends on the specifics of the concrete mix design. For a mixture with a water to cementitious material ratio of 0.4, using only Portland cement, m is equal to 0.2. If the mixture is composed of 30% slag, m increases to 0.37, and is equal to 0.52 if the mixture contains 40% fly ash, for example.

The following relationship is used to account for temperature-dependent changes in diffusion:

$$D(T) = D_{ref} * exp\left[\frac{U}{R} * \left(\frac{1}{T_{ref}} - \frac{1}{T}\right)\right]$$

where $D(T)$ = diffusion coefficient at time t and temperature T.

D_{ref} = diffusion coefficient at time t_{ref} and temperature T_{ref} (293K).

U = activation energy of the diffusion process (35,000 J/mol)

R = gas constant (8.314 J/K*mol), and

T = absolute temperature

For Life-365, the base concrete mixture assumed by the model contains only Portland cement with no special corrosion protection strategies. For the base case, the following values are assumed:

$D_{28} = 1 \times 10^{(-12.06 + 2.40w/cm)}$ meters-squared per second (m^2/s)

m = 0.20, and

C_t = 0.05 percent (% wt. of concrete)

Where C_t is the critical concentration of chlorides, at the level of the reinforcement, to induce corrosion. While there is some debate within the Civil Engineering community on which concentration is critical to induce corrosion, Life-365's value of 0.05 percent by weight of concrete has been chosen as the benchmark to compare these results.

For the analysis of walls and slabs, the time-to-initiation is estimated deterministically using a one-dimensional Crank-Nicolson finite difference approach, where the future levels of chlorides in the concrete are a function of current chloride levels. The reader is encouraged to consult the Life-365 user's manual for additional information on this and the preceding equations and variables [1].

Under our analysis, the deck slab for each structure type was modeled using a standard concrete mix of 0.42 w/cm, with no additives, uncoated carbon steel reinforcement, and a rebar percent volume of concrete of 1.2%. This was done to represent a fairly worst case scenario for corrosion initiation. By using admixtures, protected reinforcement, or other protection strategies, the time to corrosion initiation and damage propagation may be significantly extended.

Each deck slab was assumed to be eight inches thick with an overall area of 10,000 square feet. Cover depths of one inch and three inches were chosen for analysis to show how changes in reinforcement depth influence the time to corrosion initiation.

To aid in analysis, Life-365 has compiled a database of chloride build up rates which is based on physical measurements and municipality de-icer application rate data. Figure 1, below, is a graphical representation of chloride build-up rates for North America, from the Life-365 manual [1].

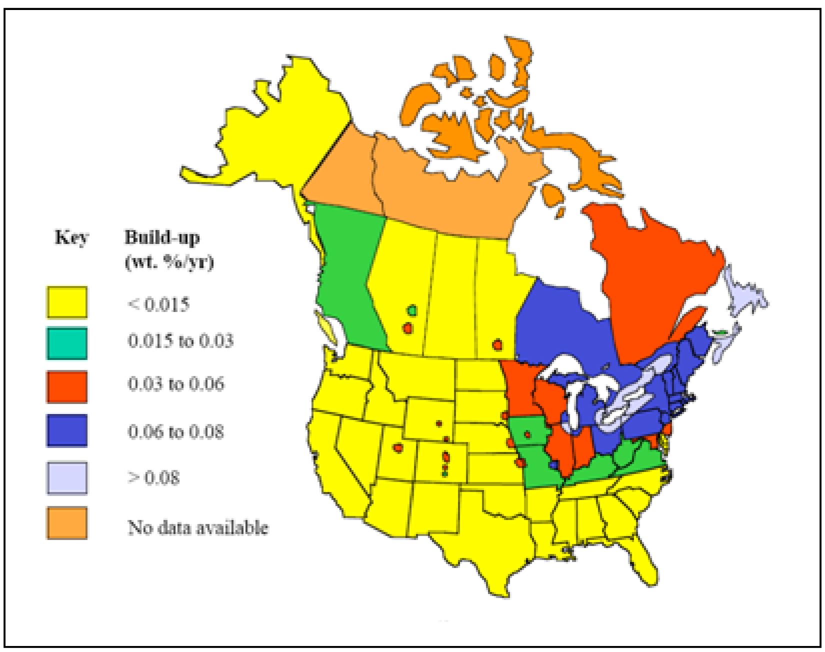

Figure F1. Life-365 chloride build-up rates for North America [1].

As a part of this project, six states across the United States we identified which represented different geographical regions and thus different chloride build-up rates. These states included: Arizona, Arkansas, Florida, New York, Washington, and Wisconsin. For each state, chloride diffusion rates we modeled for rural highway bridges, urban highway bridges, and for marine zones, where applicable.

F.3 RESULTS

F.3.1 By Geographic Region

This section will present the results of the analysis done using Life-365. First, two graphs were constructed to illustrate the difference in the time to corrosion initiation for differing cover depths across the six selected states. Figure F2 presents the time to corrosion initiation for an urban highway bridge deck with one inch of cover while Figure F3 presents the time to corrosion initiation for an urban highway bridge deck with three inches of cover.

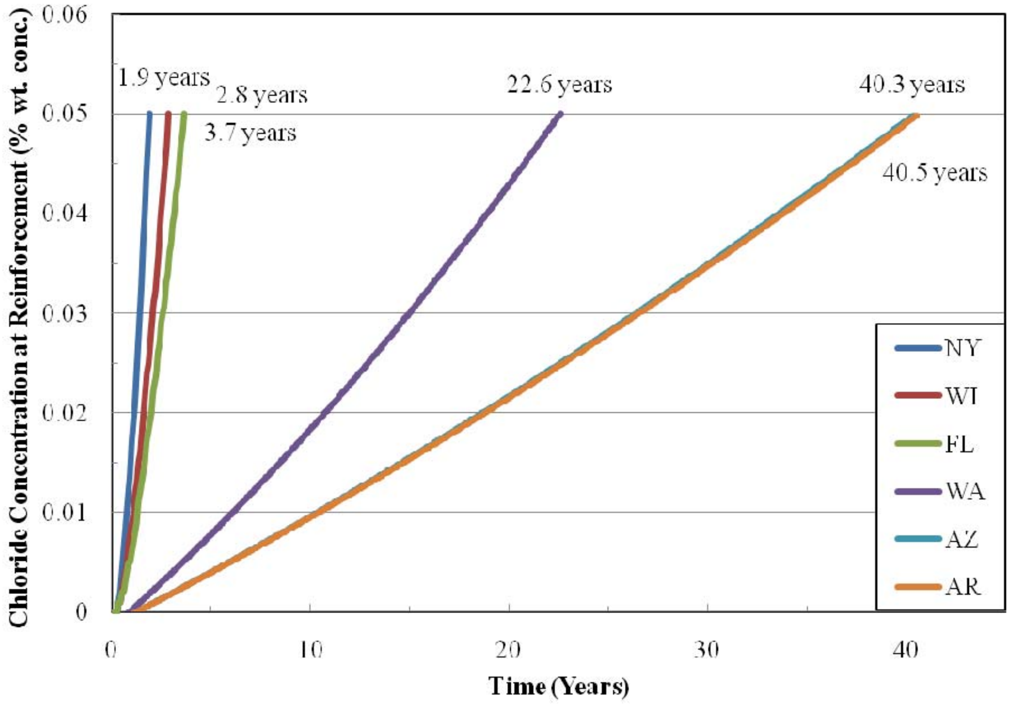

Figure F2. Chloride concentration rates by state: urban highway, one inch cover.

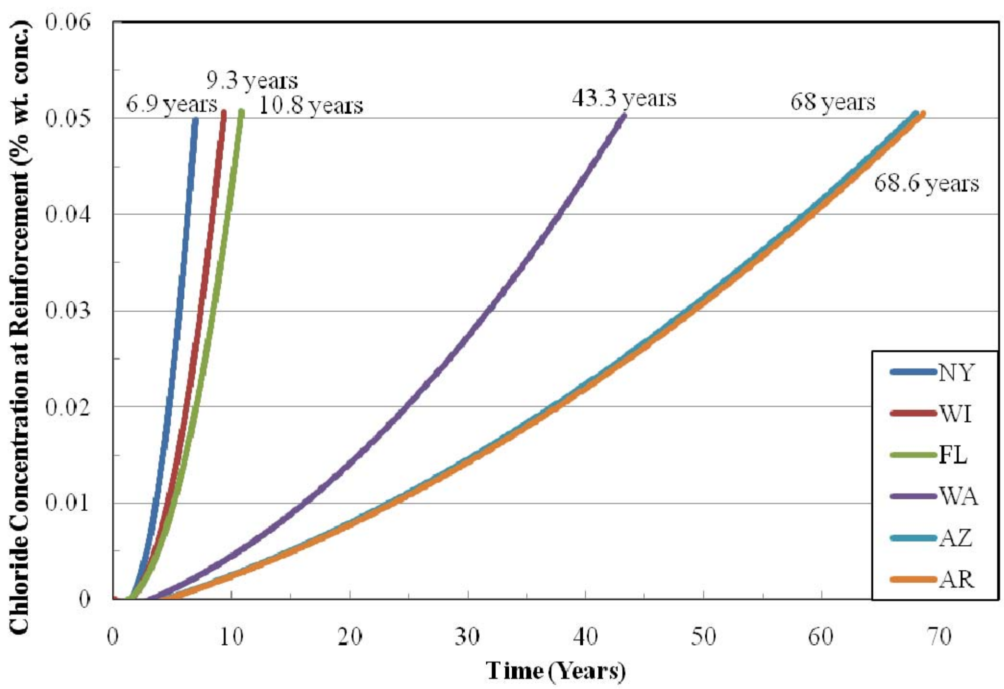

Figure F3. Chloride concentration rates by state: urban highway, three inches cover.

From these graphs, as far as urban highway bridge exposure environments, New York, Wisconsin, and Florida have very similar chloride concentration rates. These environments would fall more towards the severe side of the exposure environment scale. Washington falls within a more moderate exposure environment. Arizona and Arkansas, with the slowest chloride concentration rates, represent more mild environments.

It can also be seen from these graphs that the time to reach a critical chloride concentration can be increased by increasing the depth of concrete cover. For the more severe environments such as New York, an additional two inches of cover correlates to an approximately 263 % increase in the time to corrosion initiation. Due to the already large corrosion initiation times for the more mild environments, this increase is not as large but it is still substantial. For Arkansas the increase in time to corrosion initiation is nearly 70%.

Table F1, below, presents a summary of the time to reach a critical chloride level of 0.05 percent by weight of concrete for each state, structure type, and depth of cover considered in this analysis. The results for the time to corrosion initiation in this table have been rounded to one significant digit in order to better aid the reader and are presented in decreasing order of ambient environmental corrosivity.

Table F1. Comparison of chloride concentration rate data for selected states

Comparison of chloride concentration rate data for selected states						
Structure Type	New York		Wisconsin		Florida	
	One inch cover	Three inches cover	One inch cover	Three inches cover	One inch cover	Three inches cover
	Time (years)	Time (years)	Time (years)	Time (years)	Time (years)	Time (years)
Marine Tidal Zone	0.6	3.3	N/A	N/A	0.6	3.1
Urban Highway	1.9	6.9	2.8	9.3	3.7	10.8
Rural Highway	2.2	7.6	3.3	10.3	4.3	12
Structure Type	Washington		Arizona		Arkansas	
	One inch cover	Three inches cover	One inch cover	Three inches cover	One inch cover	Three inches cover
	Time (years)	Time (years)	Time (years)	Time (years)	Time (years)	Time (years)
Marine Tidal Zone	0.6	3.3	N/A	N/A	N/A	N/A
Urban Highway	22.6	43.3	40.3	68	40.5	68.6
Rural Highway	26.6	49.1	47.7	78.3	47.9	79

F.3.2 By Individual State

The second part of the analysis using Life-365 considered how structure type and cover depth influenced the time to reach critical chloride concentrations by individual state. Here, the results for Arkansas, New York, and Washington will be presented as they each represent the three general chloride concentration rates as seen in Figures F2 and F3.

In the following figures, the build-up rates for three different structure types along with two different cover depths will be presented. The structure types considered were those in marine tidal zones, rural highway bridges, and urban highway bridges. For the case of Arkansas, only rural and urban highway bridges have been analyzed. The different levels of concrete cover analyzed are one inch and three inches. Tables have also been provided which summarize the progression of chloride build-up at the reinforcement for each state.

F.3.2.1 Arkansas

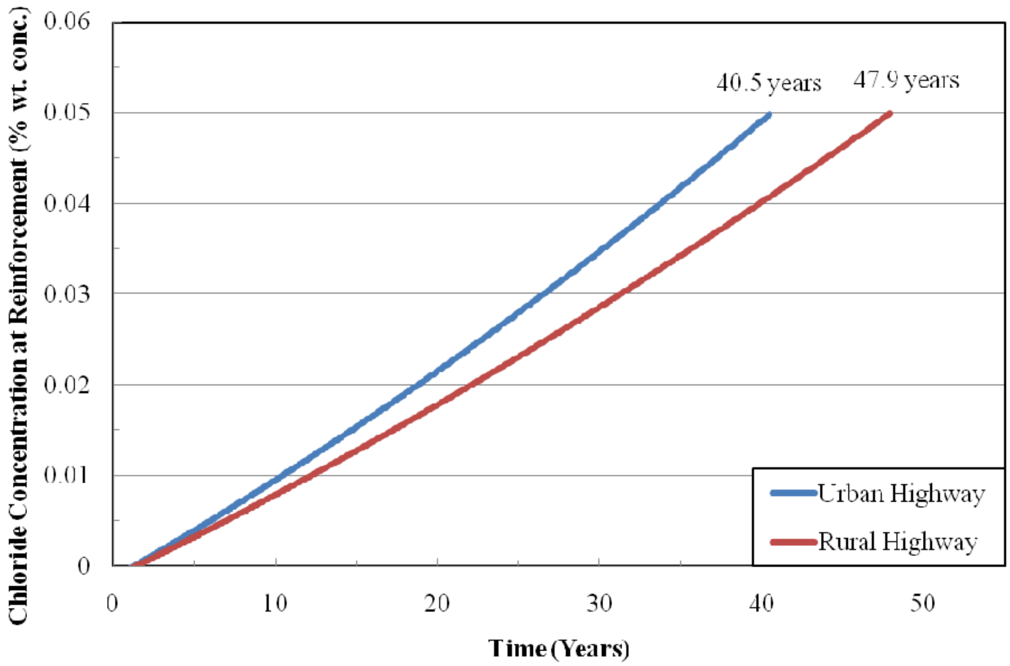

Figure F4. Chloride concentration rates: Fort Smith, AR, one inch cover.

Table F2. Chloride concentration rate data: Fort Smith, AR, one inch cover.

Chloride Concentration Rate Data: Fort Smith, AR, Once Inch Cover			
Urban Highway		**Rural Highway**	
Time (years)	Concentration (% wt. conc.)	Time (years)	Concentration (% wt. conc.)
0	0	0	0
10	0.009	10	0.008
20	0.022	20	0.018
30	0.035	30	0.029
40	0.049	40	0.04
40.5	0.05	40.5	0.041
		47.9	0.05

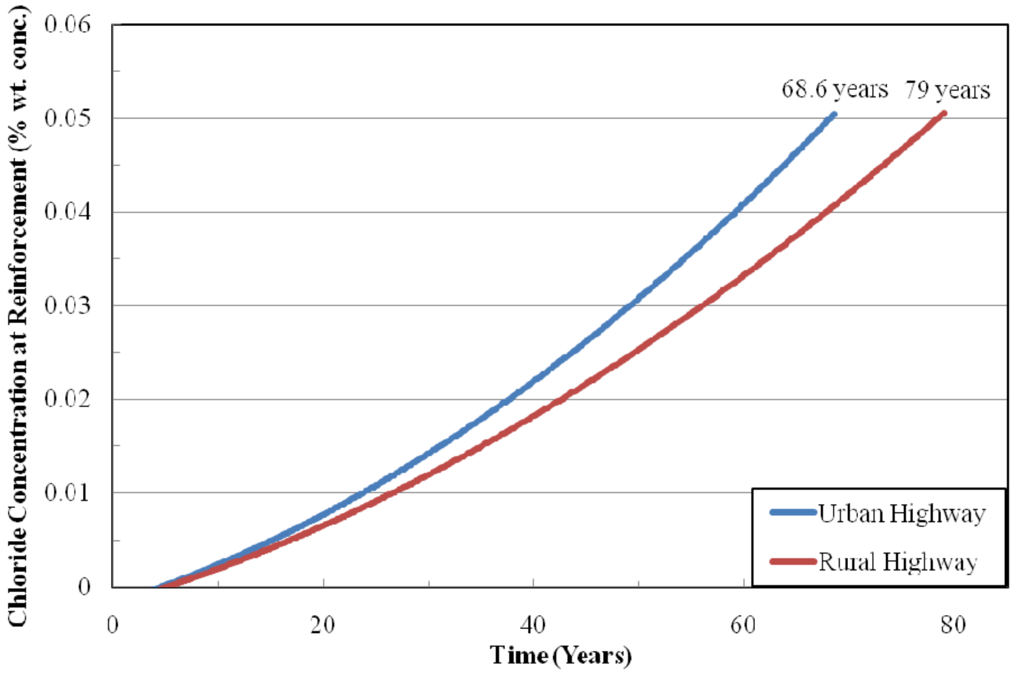

Figure F5. Chloride concentration rates: Fort Smith, AR, three inches cover.

Table F3. Chloride concentration rate data: Fort Smith, AR, three inches cover.

Chloride Concentration Rate Data: Fort Smith, AR, Three Inches Cover			
Urban Highway		**Rural Highway**	
Time (years)	Concentration (% wt. conc.)	Time (years)	Concentration (% wt. conc.)
0	0	0	0
10	0.002	10	0.002
20	0.007	20	0.006
30	0.014	30	0.012
40	0.022	40	0.018
50	0.031	50	0.026
60	0.041	60	0.034
68.6	0.05	68.6	0.041
		70	0.042
		79	0.05

F.3.2.2 New York

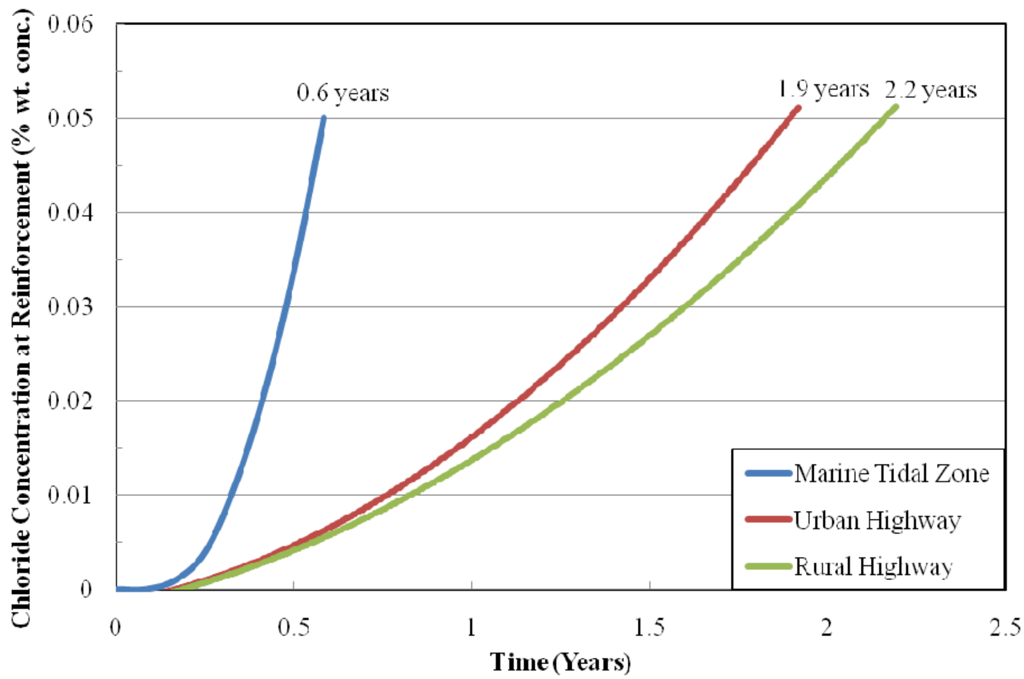

Figure F6. Chloride concentration rates: New York, NY, one inch cover.

Table F4. Chloride concentration rate data: New York, NY, one inch cover.

Chloride Concentration Rate Data: New York, NY, One Inch Cover					
Marine Tidal Zone		**Urban Highway**		**Rural Highway**	
Time (years)	Concentration (% wt. conc.)	Time (years)	Concentration (% wt. conc.)	Time (years)	Concentration (% wt. conc.)
0	0	0	0	0	0
0.5	0.034	0.5	0.004	0.5	0.003
0.6	0.05	0.6	0.006	0.6	0.005
		1	0.017	1	0.014
		1.5	0.033	1.5	0.027
		1.9	0.05	1.9	0.041
				2.2	0.052

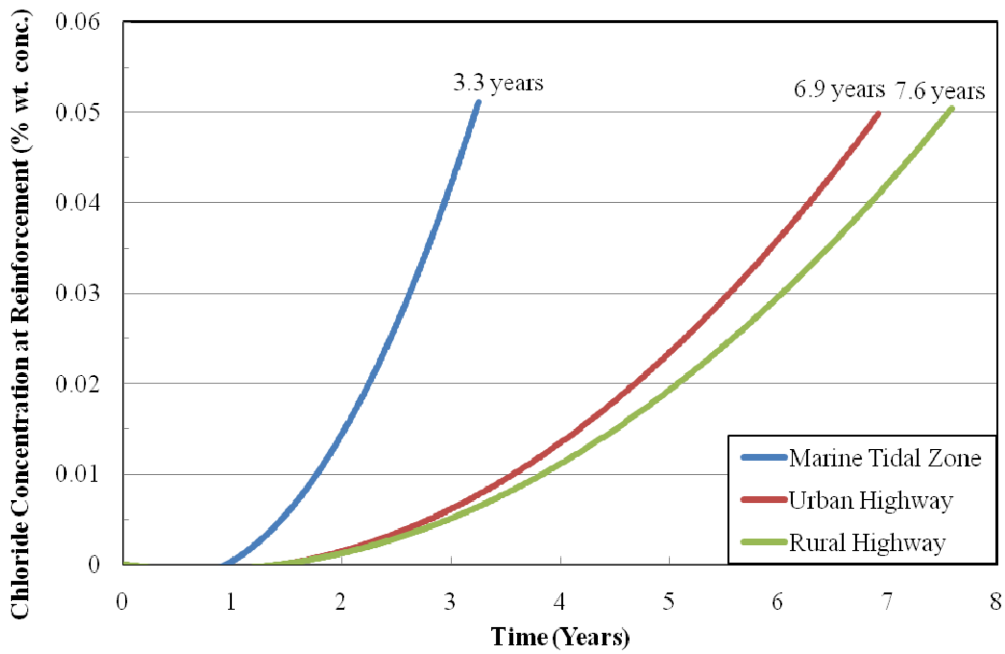

Figure F7. Chloride concentration rates: New York, NY, three inches cover.

Table F5. Chloride concentration rate data: New York, NY, three inches cover.

Chloride Concentration Rate Data: New York, NY, Three Inches Cover					
Marine Tidal Zone		**Urban Highway**		**Rural Highway**	
Time (years)	Concentration (% wt. conc.)	Time (years)	Concentration (% wt. conc.)	Time (years)	Concentration (% wt. conc.)
0	0	0	0	0	0
2	0.014	2	0.002	2	0.001
3.3	0.05	3.3	0.007	3.3	0.006
		4	0.013	4	0.011
		6	0.036	6	0.03
		6.9	0.05	6.9	0.041
				7.6	0.05

F.3.2.3 Washington

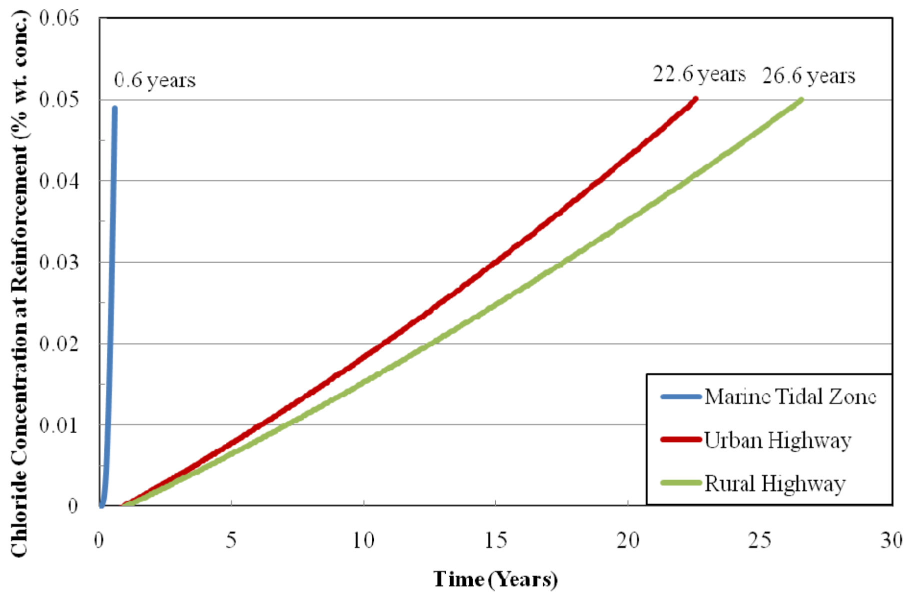

Figure F8. Chloride concentration rates: Seattle, WA, one inch cover.

Table F6. Chloride concentration rate data: Seattle, WA, one inch cover.

Chloride Concentration Rate Data: Seattle, WA, One Inch Cover					
Marine Tidal Zone		**Urban Highway**		**Rural Highway**	
Time (years)	Concentration (% wt. conc.)	Time (years)	Concentration (% wt. conc.)	Time (years)	Concentration (% wt. conc.)
0	0	0	0	0	0
0.6	0.049	0.6	0	0.6	0
		5	0.007	5	0.006
		10	0.018	10	0.015
		15	0.03	15	0.025
		20	0.043	20	0.035
		22.6	0.05	22.6	0.041
				25	0.046
				26.6	0.05

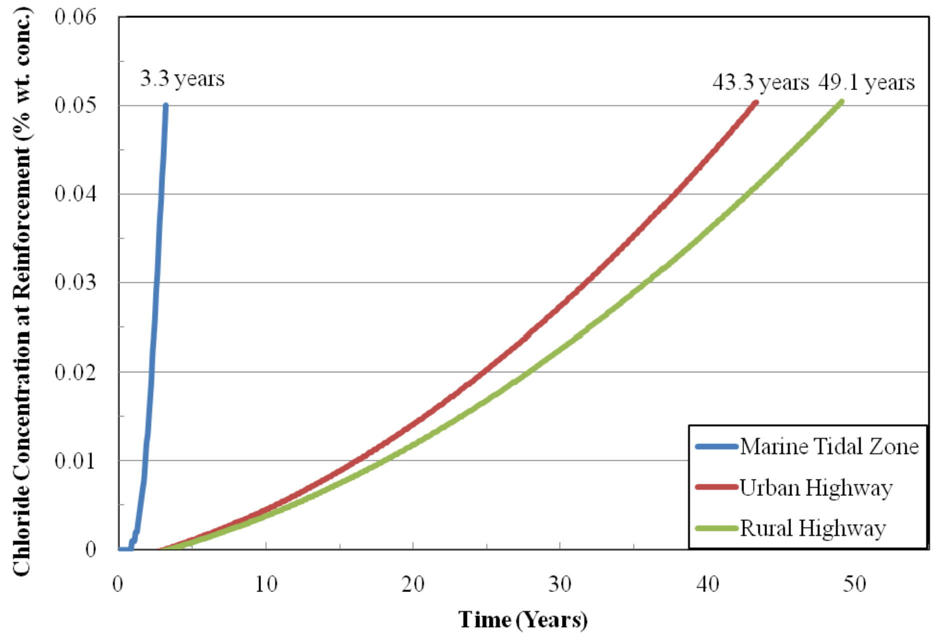

Figure F9. Chloride concentration rates: Seattle, WA, three inches cover.

Table F7. Chloride concentration rate data: Seattle, WA, three inches cover.

Chloride Concentration Rate Data: Seattle, WA, Three Inches Cover					
Marine Tidal Zone		**Urban Highway**		**Rural Highway**	
Time (years)	Concentration (% wt. conc.)	Time (years)	Concentration (% wt. conc.)	Time (years)	Concentration (% wt. conc.)
0	0	0	0	0	0
3.3	0.05	3.3	0	3.3	0
		5	0.001	5	0.001
		10	0.004	10	0.003
		20	0.014	20	0.012
		30	0.028	30	0.023
		40	0.044	40	0.036
		43.3	0.05	43.3	0.041
				49.1	0.05

F.4 CONCLUSION

As seen from the figures and tables, the rate at which chloride concentrates to critical levels at the reinforcement can be influenced by many factors. Increasing levels of concrete cover will increase the time to achieve a critical chloride concentration due to the increased amount of concrete through which chloride ions must diffuse. Geographic location also plays a role in that some locations may have higher levels of ambient chlorides and harsher weather, such as a location on a coast with a moist environment versus a location which is landlocked with an arid environment. Structure type and use also plays a role in that bridges located in urban environments may have higher rates of chloride concentration compared to rural bridges. This is due to the fact that municipalities frequently apply higher levels of de-icing chemicals to bridges with higher amounts of traffic to ensure the safety of the travelling public.

Based on these results, it is clear that given the same structure type and cover depth that corrosion initiation rates vary widely across the United States, depending on geographic region. These differences in corrosion initiation rates should be accounted for in inspection programs as a uniform inspection interval would seem irrational for concrete bridge elements, given these results.

F.5 REFERENCES

1. Ehlen, M.A., M.D.A. Thomas, and E.C. Bentz, *Life-365 Service Life Prediction Model Version 2.0.1 User's Manual*. 2009, Life-365 Consortium II.

Appendix G: National Bridge Inventory Database Results

TABLE OF CONTENTS

LIST OF TABLES

G.1 INTRODUCTION

The results of bridge inspections conducted in the United States are stored in a database known as the National Bridge Inventory (NBI), which is maintained by the Federal Highway Administration (FHWA). This database allows users to view the results of bridge inspections as well as sort these results based on the criteria defined in the FHWA's Recording and Coding Guide [1]. As a part of this research project, the inspection results contained in the NBI have been analyzed to determine the number of bridges which can be easily incorporated into a reliability based inspection program.

To accomplish this, the total number of culverts, prestressed concrete bridges, and steel bridges which are currently in service in the United States have been identified according to their owner. Then, by using criteria which has been developed by this project, the total number of bridges has been reduced to those bridges which are in at least good condition, were built to recent standards, and are of common design. These bridges represent those which are considered to have the lowest level of existing damage and can therefore be expected to remain in a satisfactory condition between inspections if placed on extended inspection frequencies.

Section G.2 will provide a summary of the criteria used to determine the number of culverts, prestressed bridges, and steel bridges in use in the United States, based on data gathered in 2009. General criteria is presented which distinguishes the total number of each structure type, while more advanced criteria is used to distinguish bridges which have certain design types, structural conditions, scour ratings, etc, which are relevant to this research. Section G.3 will provide the results of this phase of the research.

G.2 NATIONAL BRIDGE INVENTORY DATABASE CRITERIA

The National Bridge Inventory is created so that users may view inspection results by assigning values for the inspection items listed in the FHWA's Recording and Coding Guide [1]. As a part of this research, certain criteria have been identified which will distinguish bridges of a certain structural type or condition from the complete listing of inspection results. This section will provide a summary of these criteria, as identified by the FHWA and this research project.

G.2.1 Highway Bridges

To identify bridge structures which carry vehicles and adhere to NBIS definitions, the FHWA recommends the application of the criteria listed in Table G1. These criteria define structures which have applicable service types, where traffic is carried on the structure, and that have lengths which adhere to NBIS standards.

Table G1. NBI highway bridge selection criteria

Item Name	Item Number	Recommended Value(s)	Meaning
Record Type	5A	= “1”	The route described is carried on the structure.
Type of Service	42A	= “1”,“4”,“5”,“6”,“7”,“8”	Type of service is: highway, highway-railroad, waterway, highway-waterway, railroad-waterway, and highway-waterway-railroad, respectively.
Structure Length	49	>=”000061”	Structure length is at least 6.1 meters (~20 feet). 6.1 is 61 in the database.

NBIS Bridge Length	112	= “Y”	The bridge satisfies NBIS bridge length requirements.

G.2.2 Culverts

Along with the highway bridge criteria listed in Table G1, the following criteria have been used to specify the number of culverts in the NBI database. Two scenarios were considered for this analysis. First, the total numbers of culverts were identified by state and then, from that data, the numbers of culverts with a condition rating of seven or greater were identified. Table G2, below, presents the criteria used for the analysis of culverts.

Table G2. NBI culvert selection criteria

Item Name	Item Number	Recommended Value(s)	Meaning
Deck Condition Rating	58	= “N”	Not applicable. Structure is a culvert
Superstructure Condition Rating	59	= “N”	Not applicable. Structure is a culvert
Substructure Condition Rating	60	= “N”	Not applicable. Structure is a culvert
Culvert Condition Rating	62	<> “N”	First scenario. Structure is a culvert of any condition.
Culvert Condition Rating	62	= “7”,”8”,”9”	Second scenario. Structure is a culvert with little to no damage.

Items 58, 59, and 60 relate to the structural condition appraisal of a bridge's deck, superstructure, and substructure, respectively. The value "N" which stands for not applicable is used to distinguish culverts from bridges. Likewise, for the first analysis scenario, Item 62, which is the culvert condition rating, was assigned a value of not "N" to determine the number of culverts of any condition. For the second analysis scenario, Item 62 was assigned values of seven, eight, or nine, to distinguish culverts which are in at least in good condition. For this part of the analysis, no consideration has been given to material type.

G.2.3 Prestressed Bridges

As with culverts, the criteria listed in this section is in addition to the highway bridge criteria described in Table G1. For the analysis of prestressed bridges, general criteria has been developed to distinguish the total numbers of prestressed bridges, as well as more advanced criteria to further refine the results. Four analysis scenarios have also been developed to see how variations in condition rating and scour rating can influence the number of results.

Table G3, below, presents the general criteria used for the analysis of prestressed bridges. This criterion distinguishes bridges which are constructed of prestressed concrete, with typical designs, of any condition type. Table G4 in Section G.2.3.1 will present the four analysis scenarios considered for prestressed bridges and Table G5 in Section G.2.3.2 will present the criteria used for the analysis scenarios.

Table G3. NBI prestressed concrete bridge deck selection criteria.

Item Name	Item Number	Recommended Value(s)	Meaning
Kind of Material	43A	= “5”,”6”	The material type is prestressed concrete or prestressed concrete continuous.
Type of Design	43B	= “02”,”03”,”04”,”05”	The structure type is “Stringer/Multi-beam or girder”, “Girder and Floorbeam System”, “Tee Beam”, “Box Beam or Girders-Multiple”, respectively.
Deck Condition Rating	58	<> “N”	Bridge is not a culvert.
Superstructure Condition Rating	59	<> “N”	Bridge is not a culvert.
Substructure Condition Rating	60	<> “N”	Bridge is not a culvert.
Culvert Condition Rating	62	= “N”	Not applicable. Structure is not a culvert.

G.2.3.1 Analysis Scenarios

For the analysis of prestressed bridges, four scenarios have been developed. These scenarios are intended to illustrate how the selection of structure condition rating and scour condition rating can affect the results of the inventory. The criteria listed in Table G4 are beneficial and seek to describe bridges which are in good condition and can be expected to remain in a good condition between extended inspection frequencies. From left to right, the criteria listed in each scenario become more conservative.

Table G4. Prestressed bridge analysis scenarios.

	Scenario 1	Scenario 2	Scenario 3	Scenario 4
Condition (C)	≥ 6	≥ 6	≥ 7	≥ 7
Scour (S)	≥ 7	≥ 8	≥ 7	≥ 8
Year	≥ 1985	≥ 1985	≥ 1985	≥ 1985
Structure Type	02, 03, 04, and 05	02, 03, 04, and 05	02, 03, 04, and 05	02, 03, 04, and 05

G.2.3.2 Advanced Criteria

This set of criteria, which is used along with the criteria in Table G1, has been identified to distinguish the number of prestressed bridges which are in good condition and of common design. Here, a bridge in good condition is one which is considered to have little or no existing damage and to have a low probability of deteriorating to a poor condition between inspections, if placed on an extended inspection frequency.

The criteria in Table G5 are similar to the criteria in Table G3 except that the condition ratings have been assigned to be in at least satisfactory condition. Additional criteria have also been added to distinguish bridges based on their year of construction, number of spans, bridge posting, and scour rating. For this part of the analysis, bridges with four spans or less have been identified as they represent a more common design. Likewise, bridges which have not been load posted and have a low risk of developing scour damage are also identified as they are perceived to have a low risk of deteriorating to a serious condition between inspection intervals.

Table G5. NBI prestressed concrete bridge advanced selection criteria.

Item Name	Item Number	Recommended Value(s)	Meaning
Year Built	27	>= “1985” or “1990”	Bridges which are 20 to 25 years old.
Structure Open	41	<> “K”	Structure is open to traffic.
Kind of Material	43A	= “5”,”6”	The material type is prestressed concrete or prestressed concrete continuous.
Type of Design	43B	= “02”,”03”,”04”,”05”	The structure type is “Stringer/Multi-beam or girder”, “Girder and Floorbeam System”, “Tee Beam”, “Box Beam or Girders-Multiple”, respectively.
Number of Spans in Main Unit	45	<= “004”	Number of spans in the main unit of the bridge is 4 or less.
Deck Condition	58	>= “6” and/or “7”,”8”,”9”	Last evaluated deck condition rating is at least satisfactory or good.
Superstructure Condition	59	>= “6” and/or “7”,”8”,”9”	Last evaluated superstructure condition rating is at least satisfactory or good.
Substructure Condition	60	>= “6” and/or “7”,”8”,”9”	Last evaluated substructure condition rating is at least satisfactory or good.

Culvert Condition Rating	62	= “N”	Not applicable. Structure is not a culvert.
Bridge Posting	70	= “5”	The bridge is not load posted. This accounts for the different load rating processes among the states.
Scour Critical Bridges	113	= “7” and/or “8” and “9”,“T”,“N”	Bridge has little to no perceived risk of developing scour damage.

G.2.4 Steel Bridges

Similar to the analysis of prestressed concrete bridges, the steel bridges in the NBI have also been analyzed using basic and advanced criteria with four analysis scenarios. Along with the highway bridge criteria, listed in Table G1, the basic criteria listed in Table G6 have been used to identify the total number of steel bridges of common design, with any condition rating. The advanced criteria has been used to further distinguish steel bridges within the inventory which meet standards identified by this research that represent a low risk of the bridge deteriorating to a poor condition between extended inspection intervals. Table G7 in Section G.2.4.1 will present the analysis scenarios used for steel bridges while Table G8 in Section G.2.4.2 will present the advanced criteria.

Table G6. NBI steel bridge basic selection criteria.

Item Name	Item Number	Recommended Value(s)	Meaning
Kind of Material	43A	= “3”,”4”	Material type is steel or steel continuous.
Type of Design	43B	=”02”,”03”	Only “Stringer/Multi-beam or girder” and “Girder and Floorbeam System” were considered for steel beams.
Deck Condition Rating	58	<> “N”	Bridge is not a culvert.
Superstructure Condition Rating	59	<> “N”	Bridge is not a culvert.
Substructure Condition Rating	60	<> “N”	Bridge is not a culvert.
Culvert Condition Rating	62	= “N”	Not applicable. Structure is not a culvert.

G.2.4.1 Analysis Scenarios

As with the analysis of prestressed bridges, four scenarios have also been developed for the analysis of steel bridges. These scenarios are intended to illustrate how the selection of structure condition rating and scour condition rating can affect the results of the inventory. For the analysis of steel bridges, the minimum year of construction is 1985 as that year represents an update to the fatigue detail specifications. Structure type has also been held constant at 02 and 03, as these represent the majority of steel bridge designs.

Table G7. Steel bridge analysis scenarios.

	Scenario 1	Scenario 2	Scenario 3	Scenario 4
Condition (C)	≥ 6	≥ 6	≥ 7	≥ 7
Scour (S)	≥ 7	≥ 8	≥ 7	≥ 8
Year	≥ 1985	≥ 1985	≥ 1985	≥ 1985
Structure Type	02 or 03	02 or 03	02 or 03	02 or 03

G.2.4.2 Advanced Criteria

As with the analysis of culverts and prestressed bridges, the criteria listed in Table G8 has been used to further distinguish the number of steel bridges which are in good condition and of common design. Similar to prestressed bridge, only bridges which have four spans or less have been considered as well as bridges which have not been load posted. An additional criterion has also been added which removes bridges which have fatigue critical details, as such bridges will need further analysis to be included in an extended inspection program.

In accordance with the analysis scenarios presented in Section G.2.4.1, several different values for the condition rating and scour rating have been used to determine how the bridge inventory differs based on these values.

Table G8. NBI steel bridge advanced selection criteria.

Item Name	Item Number	Recommended Value(s)	Meaning
Year Built	27	>= “1985”	Only bridges built after 1985 due to updated fatigue detail specifications.
Structure Open	41	<> “K”	Structure is open to traffic.
Kind of Material	43A	= “3”,”4”	Material type is steel or steel continuous.
Type of Design	43B	=”02”,”03”	Only “Stringer/Multi-beam or girder” and “Girder and Floorbeam System” were considered for steel beams.
Number of Spans in Main Unit	45	<= “004”	Number of spans in the main unit of the bridge is 4 or less.
Deck Condition	58	>= “6” and/or “7”,”8”,”9”	Last evaluated deck condition rating is at least satisfactory or good.
Superstructure Condition	59	>= “6” and/or “7”,”8”,”9”	Last evaluated superstructure condition rating is at least satisfactory or good.
Substructure Condition	60	>= “6” and/or “7”,”8”,”9”	Last evaluated substructure condition rating is at least satisfactory or good.
Culvert Condition Rating	62	= “N”	Not applicable. Structure is not a culvert.
Bridge Posting	70	= “5”	The bridge is not load posted. This accounts for the different load rating processes among the states.
Fracture Critical	92A	= “N” or “N00”	Fracture critical inspection not required.

Scour Critical Bridges	113	= “7” and/or “8” and “9”,“T”,“N”	Bridge has little to no perceived risk of developing scour damage.

G.3 RESULTS

Based on the analysis of the NBI database, select results will be presented in two formats. First, states which have the highest numbers of culverts, prestressed concrete bridges, and steel bridges which meet the beneficial criteria identified by this research will be presented based on AASHTO region. This approach was taken to identify inventories across the United States which have the greatest number of structures which could readily receive a reliability based assessment and may provide excellent pilot studies. A listing of the states in each AASHTO Region can be seen in Table G9 while Table G10 will present the results of this part of the analysis. In Table G10, only the results for Analysis Scenario 1 will be presented for prestressed bridges and steel bridges as well as culverts which have a condition rating of seven or greater.

Second, to provide a greater picture of the bridge inventories in the United States, the results for all 51 reporting agencies will be presented in Tables G11 through G15. Due to the breadth of information generated by this process, the results have been broken into several tables in order to aid the reader. The results for culverts are presented in Table G11, while the results for prestressed concrete bridges are presented in Tables G12 and G13. Tables G14 and G15 will present the results for steel bridges. Across these tables, all analysis scenarios have been presented.

Table G9. States in each AASHTO Region.

AASHTO Region	State Abbreviation
Region I	CT, DE, ME, MD, MA, NH, NJ, NY, PA, RI, VT, DC
Region II	AL, AR, FL, GA, LA, MS, NC, PR, SC, TN, VA, WV
Region III	IL, IN, IA, KS, KY, MI, MN, MO, OH, WI
Region IV	AL, AZ, CA, CO,HI, ID, MN, NB, NV, NM, ND, OK, OR, SD, TX, UT, WA, WY

Table G10. NBI results for top states based on AASHTO Region.

AASHTO Region 1					
Category	State	Steel Scenario 1	P/S Scenario 1	Culverts	Total
		C >= 6 S >= 7	C >= 6 S >= 7	C >= 7	
Top Culverts	**New York**	1,488	1,013	933	3,434
Top Prestressed	**Pennsylvania**	626	1,149	806	2,581
Top Steel	**New York**	1,488	1,013	933	3,434
Top Total	**New York**	1,488	1,013	933	3,434
AASHTO Region 2					
Top Culverts	**Tennessee**	319	1,164	5,470	6,953
Top Prestressed	**Kentucky**	202	2,246	1,314	3,762
Top Steel	**Virginia**	1,202	248	1,540	2,990
Top Total	**Tennessee**	319	1,164	5,470	6,953
AASHTO Region 3					
Top Culverts	**Kansas**	1,202	263	5,633	7,098
Top Prestressed	**Illinois**	884	4,391	3,007	8,282
Top Steel	**Missouri**	3,443	2,264	2,247	7,954
Top Total	**Illinois**	884	4,391	3,007	8,282
AASHTO Region 4					
Top Culverts	**Texas**	746	4,879	12,088	17,713
Top Prestressed	**Texas**	746	4,879	12,088	17,713
Top Steel	**Oklahoma**	1,431	2,164	5,079	8,674
Top Total	**Texas**	746	4,879	12,088	17,713

Table G11. Results of the NBI analysis for culverts.

State	Total NBI Entries	Total Highway Bridges	Total culverts	Culverts C>=7
Total	**723,313**	**612,510**	**129,243**	**84,999**
Alabama	17,378	15,954	6,091	3,432
Alaska	1,272	1,147	66	50
Arizona	8,816	7,495	3,955	3,463
Arkansas	14,223	12,580	2,977	2,312
California	34,900	24,437	3,357	2,979
Colorado	9,667	8,473	1,753	1,181
Connecticut	5,452	4,186	602	335
D.C.	336	246	0	0
Delaware	1,158	862	240	127
Florida	15,759	11,803	2,247	1,517
Georgia	17,153	14,712	5,582	3,207
Hawaii	1,212	1,132	169	106
Idaho	4,548	4,094	113	62
Illinois	30,300	26,255	4,381	3,007
Indiana	20,456	18,546	1,710	1,011
Iowa	26,056	24,799	3,929	2,620
Kansas	27,205	25,552	7,882	5,633
Kentucky	15,832	13,713	2,874	1,314
Louisiana	14,259	13,329	2,495	1,957
Maine	2,837	2,394	323	144
Maryland	6,399	5,177	1,174	541
Massachusetts	5,662	5,035	302	165
Michigan	14,209	10,904	1,504	883
Minnesota	16,064	13,131	5,196	4,032
Mississippi	18,427	17,038	3,464	3,179
Missouri	27,394	24,180	4,853	2,247
Montana	5,880	4,985	205	118
Nebraska	16,244	15,436	3,158	2,804
Nevada	2,202	1,749	738	474
New Hampshire	4,123	2,402	259	140
New Jersey	9,547	6,486	441	286
New Mexico	4,593	3,889	1,690	715
New York	22,661	17,371	1,689	933
North Carolina	24,162	18,004	4,981	2,727
North Dakota	4,706	4,435	902	697
Ohio	33,429	28,122	1,837	1,027
Oklahoma	25,860	23,720	6,883	5,079
Oregon	8,157	7,205	296	179
Pennsylvania	29,335	22,284	1,722	806
Puerto Rico	2,699	2,180	312	127
Rhode Island	1,035	739	29	10
South Carolina	20,616	18,504	1,078	725
South Dakota	6,413	5,920	1,252	900
Tennessee	23,204	19,939	8,611	5,470
Texas	59,445	51,156	18,803	12,088
Utah	3,756	2,904	573	457
Vermont	3,027	2,707	180	116
Virginia	17,905	13,530	3,077	1,540
Washington	8,845	7,657	302	226
West Virginia	8,537	7,049	525	278
Wisconsin	16,610	13,916	2,014	1,243
Wyoming	3,348	3,047	447	330

Table G12. Results of the NBI analysis for prestressed concrete bridges (1 of 2).

State	Total NBI Entries	Total Highway Bridges	P/S Bridges Type=02, 03, 04, 05	P/S Bridges C>=6, S>=7, 1985, 02 03 04 05	P/S Bridges C>=6, S>=8, 1985, 02 03 04 05
Total	**3,616,565**	**3,062,550**	**117,391**	**42,860**	**41,538**
Alabama	17,378	15,954	1,352	512	511
Alaska	1,272	1,147	366	164	164
Arizona	8,816	7,495	1,098	541	535
Arkansas	14,223	12,580	99	13	13
California	34,900	24,437	4,868	1,258	1,252
Colorado	9,667	8,473	1,902	934	930
Connecticut	5,452	4,186	480	101	99
D.C.	336	246	17	0	0
Delaware	1,158	862	151	67	64
Florida	15,759	11,803	3,906	937	937
Georgia	17,153	14,712	1,901	643	643
Hawaii	1,212	1,132	258	20	20
Idaho	4,548	4,094	1,519	263	263
Illinois	30,300	26,255	10,016	4,391	4,360
Indiana	20,456	18,546	6,095	1,586	1,473
Iowa	26,056	24,799	4,445	1,641	1,581
Kansas	27,205	25,552	1,165	263	261
Kentucky	15,832	13,713	4,718	2,246	1,880
Louisiana	14,259	13,329	1,283	183	183
Maine	2,837	2,394	42	33	33
Maryland	6,399	5,177	217	70	67
Massachusetts	5,662	5,035	400	133	119
Michigan	14,209	10,904	3,571	890	845
Minnesota	16,064	13,131	2,824	1,033	1,029
Mississippi	18,427	17,038	4,337	905	900
Missouri	27,394	24,180	3,371	2,264	2,247
Montana	5,880	4,985	1,793	342	341
Nebraska	16,244	15,436	1,315	571	569
Nevada	2,202	1,749	318	197	195
New Hampshire	4,123	2,402	85	40	40
New Jersey	9,547	6,486	1,019	311	284
New Mexico	4,593	3,889	969	219	218
New York	22,661	17,371	2,128	1,013	1,011
North Carolina	24,162	18,004	1,798	964	964
North Dakota	4,706	4,435	195	75	75
Ohio	33,429	28,122	7,171	3,027	2,923
Oklahoma	25,860	23,720	3,847	2,164	2,137
Oregon	8,157	7,205	1,284	358	354
Pennsylvania	29,335	22,284	4,712	1,149	991
Puerto Rico	2,699	2,180	826	335	311
Rhode Island	1,035	739	103	21	19
South Carolina	20,616	18,504	3,014	350	350
South Dakota	6,413	5,920	1,051	269	269
Tennessee	23,204	19,939	3,259	1,164	1,155
Texas	59,445	51,156	12,336	4,879	4,728
Utah	3,756	2,904	969	346	341
Vermont	3,027	2,707	33	8	8
Virginia	17,905	13,530	964	248	224
Washington	8,845	7,657	2,623	738	704
West Virginia	8,537	7,049	1,832	1,500	1,496
Wisconsin	16,610	13,916	3,129	1,428	1,369
Wyoming	3,348	3,047	217	53	53

Table G13. Results of the NBI analysis for prestressed concrete bridges (2 of 2).

State	Total NBI Entries	Total Highway Bridges	P/S Bridges Type=02, 03, 04, 05	P/S Bridges C>=7, S>=7, 1985, 02 03 04 05	P/S Bridges C>=7, S>=8, 1985, 02 03 04 05
Total	**2,169,939**	**1,837,530**	**117,391**	**38,247**	**37,112**
Alabama	17,378	15,954	1,352	486	485
Alaska	1,272	1,147	366	130	130
Arizona	8,816	7,495	1,098	451	447
Arkansas	14,223	12,580	99	13	13
California	34,900	24,437	4,868	788	784
Colorado	9,667	8,473	1,902	862	858
Connecticut	5,452	4,186	480	71	69
D.C.	336	246	17	0	0
Delaware	1,158	862	151	56	54
Florida	15,759	11,803	3,906	923	923
Georgia	17,153	14,712	1,901	596	596
Hawaii	1,212	1,132	258	16	16
Idaho	4,548	4,094	1,519	169	169
Illinois	30,300	26,255	10,016	4,071	4,041
Indiana	20,456	18,546	6,095	1,411	1,310
Iowa	26,056	24,799	4,445	1,541	1,485
Kansas	27,205	25,552	1,165	261	259
Kentucky	15,832	13,713	4,718	1,828	1,511
Louisiana	14,259	13,329	1,283	175	175
Maine	2,837	2,394	42	33	33
Maryland	6,399	5,177	217	63	60
Massachusetts	5,662	5,035	400	115	101
Michigan	14,209	10,904	3,571	761	726
Minnesota	16,064	13,131	2,824	1,006	1,002
Mississippi	18,427	17,038	4,337	899	894
Missouri	27,394	24,180	3,371	2,149	2,132
Montana	5,880	4,985	1,793	302	301
Nebraska	16,244	15,436	1,315	564	562
Nevada	2,202	1,749	318	184	182
New Hampshire	4,123	2,402	85	40	40
New Jersey	9,547	6,486	1,019	288	264
New Mexico	4,593	3,889	969	175	175
New York	22,661	17,371	2,128	842	841
North Carolina	24,162	18,004	1,798	760	760
North Dakota	4,706	4,435	195	72	72
Ohio	33,429	28,122	7,171	2,804	2,708
Oklahoma	25,860	23,720	3,847	1,989	1,978
Oregon	8,157	7,205	1,284	310	307
Pennsylvania	29,335	22,284	4,712	899	780
Puerto Rico	2,699	2,180	826	229	217
Rhode Island	1,035	739	103	17	15
South Carolina	20,616	18,504	3,014	344	344
South Dakota	6,413	5,920	1,051	225	225
Tennessee	23,204	19,939	3,259	1,019	1,011
Texas	59,445	51,156	12,336	4,393	4,257
Utah	3,756	2,904	969	303	298
Vermont	3,027	2,707	33	8	8
Virginia	17,905	13,530	964	203	183
Washington	8,845	7,657	2,623	656	625
West Virginia	8,537	7,049	1,832	1,395	1,391
Wisconsin	16,610	13,916	3,129	1,326	1,269
Wyoming	3,348	3,047	217	26	26

Table G14. Results of the NBI analysis for steel bridges (1 of 2).

State	Total NBI Entries	Total Highway Bridges	Steel Bridges Type=02, 03	Steel Bridges C>=6, S>=7, 1985, 02 03	Steel Bridges C>=6, S>=8, 1985, 02 03
Total	**6,509,817**	**5,512,590**	**157,586**	**23,017**	**22,038**
Alabama	17,378	15,954	2,501	132	128
Alaska	1,272	1,147	388	16	16
Arizona	8,816	7,495	497	25	24
Arkansas	14,223	12,580	3,840	559	552
California	34,900	24,437	2,099	23	22
Colorado	9,667	8,473	2,598	787	776
Connecticut	5,452	4,186	1,999	185	184
D.C.	336	246	149	1	1
Delaware	1,158	862	328	81	79
Florida	15,759	11,803	845	297	295
Georgia	17,153	14,712	3,405	67	66
Hawaii	1,212	1,132	51	2	2
Idaho	4,548	4,094	597	97	95
Illinois	30,300	26,255	6,177	884	874
Indiana	20,456	18,546	4,015	511	497
Iowa	26,056	24,799	6,911	796	744
Kansas	27,205	25,552	7,196	1,202	1,130
Kentucky	15,832	13,713	2,149	202	132
Louisiana	14,259	13,329	939	109	101
Maine	2,837	2,394	1,199	157	154
Maryland	6,399	5,177	2,499	583	573
Massachusetts	5,662	5,035	2,731	173	168
Michigan	14,209	10,904	3,933	142	139
Minnesota	16,064	13,131	2,317	261	255
Mississippi	18,427	17,038	1,692	54	54
Missouri	27,394	24,180	10,800	3,443	3,294
Montana	5,880	4,985	917	79	68
Nebraska	16,244	15,436	6,360	902	863
Nevada	2,202	1,749	188	68	66
New Hampshire	4,123	2,402	1,290	216	211
New Jersey	9,547	6,486	3,285	367	351
New Mexico	4,593	3,889	424	36	36
New York	22,661	17,371	9,487	1,488	1,481
North Carolina	24,162	18,004	7,063	963	960
North Dakota	4,706	4,435	1,023	18	18
Ohio	33,429	28,122	11,094	1,424	1,386
Oklahoma	25,860	23,720	7,644	1,431	1,262
Oregon	8,157	7,205	776	44	41
Pennsylvania	29,335	22,284	6,508	626	576
Puerto Rico	2,699	2,180	304	27	22
Rhode Island	1,035	739	373	25	24
South Carolina	20,616	18,504	2,540	546	546
South Dakota	6,413	5,920	1,458	67	67
Tennessee	23,204	19,939	2,447	319	305
Texas	59,445	51,156	6,451	746	708
Utah	3,756	2,904	697	219	218
Vermont	3,027	2,707	1,567	171	165
Virginia	17,905	13,530	6,147	1,202	1,075
Washington	8,845	7,657	509	65	63
West Virginia	8,537	7,049	2,920	760	755
Wisconsin	16,610	13,916	3,090	174	172
Wyoming	3,348	3,047	1,169	245	244

Table G15. Results of the NBI analysis for steel bridges (2 of 2).

State	Total NBI Entries	Total Highway Bridges	Steel Bridges Type=02, 03	Steel Bridges C>=7, S>=7, 1985, 02 03	Steel Bridges C>=7, S>=8, 1985, 02 03
Total	**5,063,191**	**4,287,570**	**157,586**	**18,440**	**18,120**
Alabama	17,378	15,954	2,501	110	110
Alaska	1,272	1,147	388	10	10
Arizona	8,816	7,495	497	22	21
Arkansas	14,223	12,580	3,840	538	532
California	34,900	24,437	2,099	13	13
Colorado	9,667	8,473	2,598	584	579
Connecticut	5,452	4,186	1,999	166	165
D.C.	336	246	149	1	1
Delaware	1,158	862	328	55	54
Florida	15,759	11,803	845	284	283
Georgia	17,153	14,712	3,405	60	60
Hawaii	1,212	1,132	51	2	2
Idaho	4,548	4,094	597	60	60
Illinois	30,300	26,255	6,177	800	790
Indiana	20,456	18,546	4,015	442	435
Iowa	26,056	24,799	6,911	628	604
Kansas	27,205	25,552	7,196	1,005	998
Kentucky	15,832	13,713	2,149	96	82
Louisiana	14,259	13,329	939	87	82
Maine	2,837	2,394	1,199	133	133
Maryland	6,399	5,177	2,499	485	477
Massachusetts	5,662	5,035	2,731	159	154
Michigan	14,209	10,904	3,933	89	89
Minnesota	16,064	13,131	2,317	231	230
Mississippi	18,427	17,038	1,692	54	54
Missouri	27,394	24,180	10,800	3,087	3,047
Montana	5,880	4,985	917	59	58
Nebraska	16,244	15,436	6,360	839	838
Nevada	2,202	1,749	188	59	57
New Hampshire	4,123	2,402	1,290	201	200
New Jersey	9,547	6,486	3,285	334	324
New Mexico	4,593	3,889	424	23	23
New York	22,661	17,371	9,487	1,267	1,263
North Carolina	24,162	18,004	7,063	794	793
North Dakota	4,706	4,435	1,023	17	17
Ohio	33,429	28,122	11,094	1,064	1,042
Oklahoma	25,860	23,720	7,644	641	634
Oregon	8,157	7,205	776	32	32
Pennsylvania	29,335	22,284	6,508	489	458
Puerto Rico	2,699	2,180	304	15	14
Rhode Island	1,035	739	373	4	4
South Carolina	20,616	18,504	2,540	482	482
South Dakota	6,413	5,920	1,458	23	23
Tennessee	23,204	19,939	2,447	235	235
Texas	59,445	51,156	6,451	531	527
Utah	3,756	2,904	697	182	182
Vermont	3,027	2,707	1,567	156	151
Virginia	17,905	13,530	6,147	910	822
Washington	8,845	7,657	509	54	54
West Virginia	8,537	7,049	2,920	639	635
Wisconsin	16,610	13,916	3,090	131	130
Wyoming	3,348	3,047	1,169	58	57

G.4 REFERENCES

1. FHWA, *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges*, 1995, Federal Highway Administration: Washington, D.C.