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Development of a Formalized Program for In-Service Inspection of Pedestrian Bridges




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SPR-4535 • Report Number: FHWA/IN/JTRP-2024/17 • DOI: 10.5703/1288284317750

RECOMMENDED CITATION

Alharthi, A. A., & Connor, R. J. (2024). *Development of a formalized program for in-service inspection of pedestrian bridges* (Joint Transportation Research Program Publication No. FHWA/IN/JTRP-2024/17). West Lafayette, IN: Purdue University. https://doi.org/10.5703/1288284317750

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NOTICE

TECHNICAL REPORT DOCUMENTATION PAGE

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA/IN/JTRP-2024/17		

4. Title and Subtitle	5. Report Date / 6. Performing Organization Code
Development of a Formalized Program for In-Service Inspection of Pedestrian Bridges	**5. Report Date** March 2024 **6. Performing Organization Code**
7. Author(s) Aedh A. Alharthi and Robert J. Connor	**8. Performing Organization Report No.** FHWA/IN/JTRP-2024/17
9. Performing Organization Name and Address Joint Transportation Research Program Hall for Discovery and Learning Research (DLR), Suite 204 207 S. Martin Jischke Drive West Lafayette, IN 47907	**10. Work Unit No.** **11. Contract or Grant No.** SPR-4535
12. Sponsoring Agency Name and Address Indiana Department of Transportation (SPR) State Office Building 100 North Senate Avenue Indianapolis, IN 46204	**13. Type of Report and Period Covered** Final Report **14. Sponsoring Agency Code**

15. Supplementary Notes

Conducted in cooperation with the U.S. Department of Transportation, Federal Highway Administration.

16. Abstract

Currently, there are no universally accepted procedures or criteria related to the inspection of pedestrian bridges in the U.S. While some states have some criteria in place, the FHWA has no specific recommendations or requirements for such structures. However, pedestrian bridges can be very complex structures in some cases, whether by using non-redundant cable-supported walkways, curved super structures, or other materials not commonly used in highway bridges. Furthermore, many of these bridges are over busy state highways and carry many pedestrians daily. Due to the Indiana climate, these structures are often subjected to deicing chemicals that are applied more frequently than on state highways. Hence, corrosion of reinforcing steel and structural steel is possible. In the absences of any state-wide uniform inspection requirements, the risk associated with such bridges is elevated. This research project developed a proposed risk-based methodology to set inspection intervals and provide guidance on the scope of such inspections within the state of Indiana.

17. Key Words	18. Distribution Statement
bridge inspection, pedestrian bridge	No restrictions. This document is available through the National Technical Information Service, Springfield, VA 22161.

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	93 including appendices	

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

EXECUTIVE SUMMARY

In recent years (circa 2023), the purpose of pedestrian bridges has extended beyond simply providing a safe route for pedestrians to cross an obstacle. Pedestrian bridges are also becoming works of art integrated into the design plan for the whole city. As a result, the pleasant appearance of these bridges often comes at the cost of complex structural analysis and design, unique fabrication requirements, and construction challenges. Furthermore, most of the pedestrian bridges in the U.S. are classified as nonredundant bridges. In the AASHTO LRFD design specification, a non-redundant bridge is defined as a type of bridge in which the failure of one component in the bridge will likely lead to an overall catastrophic failure of the entire bridge (AASHTO, 2018).

Like any in-service bridge, long-term inspection is critical for ensuring safe and reliable operation; therefore, inspecting different types of pedestrian bridges efficiently and adequately is crucial to avoid unexpected failure during their service life. While National Bridge Inspection Standards (NBIS) regulations apply to all publicly owned *highway* bridges longer than 20 ft., there is no standardized inspection criteria applicable for any type of pedestrian bridge (NBIS, 2022). The current criteria, implemented ad-hoc by many owners, is to inspect pedestrian bridges using the traditional calendar-based inspection approach. This method assigns an inspection interval within a time frame (typically 24 months) for all bridges except those receiving special inspections. Although this method may provide an adequate level of safety for some bridges, it does not explicitly account for the current condition and characteristics of the bridge. For example, a bridge in severe condition is inspected at the same interval as a newly constructed bridge that was built according to current design specification.

Furthermore, the current inspection practice of pedestrian bridges considers only the *structural conditions*, while some unique safety and serviceability criteria, such as railing, lighting, and walking surface, should also be considered to maintain an optimum level of safety and serviceability for pedestrians and cyclists.

This project aims to develop an inspection methodology specifically designed for pedestrian bridges that will ensure the best allocation of bridge inspection resources and a high level of safety and serviceability. In its final form, a Risk Based Inspection (RBI) methodology was utilized in conjunction with reliability theory and expert elicitation from the Indiana Department of Transportation Risk Assessment Panel to rationally determine the inspection interval. The proposed methodology was based on the reliability-based inspection procedures presented in NCHRP 782 report (Washer et al., 2014). In this method, the inspection interval was determined based on the risk assessment, which was the product of a combination of occurrence and consequence factors. The occurrence factor was calculated based on design, loading (mechanical and environmental), and condition attributes for each damage mode. The consequence factor measured the outcomes of the occurrence of the damage mode. The consequence factor was evaluated in two stages: (1) an immediate consequence in which outcomes impact the safety of the service on and under the bridge, and (2) a short-term consequence, in which effects influence the service under the bridge.

A new factor, referred to as the inspection effectiveness factor, was also included in the risk assessment. This factor attempted to account for the inspection technique's reliability in identifying and quantifying a specific defect for each element of the bridge. This factor was evaluated based on different inspection techniques, with consideration for their inherit uncertainty, the type of defect, and the accessibility to the component. Finally, the Risk Priority Number (RPN) was obtained by combining the occurrence, consequence, and inspection effectiveness factors. The inspection interval was identified by using risk bar diagram that was based on the RPN (in which the bridge with high RPN number receive a shorter inspection interval).

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1. INTRODUCTION

1.1 Background

The construction of pedestrian bridges has recently evolved; they are no longer solely designed to facilitate pedestrian traffic over obstacles, but also to add aesthetic value to their surroundings. However, this transformation comes with a cost, necessitating complex structural analysis and design, unique fabrication requirements, and, at times, challenging construction obstacles. Consequently, while once considered common and simple structures, modern pedestrian bridges often exhibit complexity surpassing that of major highway bridges.

For highway bridges, inspections should be conducted during three distinct stages: fabrication, construction, and in-service, each intended to collect specific information and meet specific objectives (Ryan et al., 2023). Fabrication and construction inspections represent the initial stages that must be completed once a newly constructed bridge is added to the bridge inventory to establish its structural baseline. The identification of defects during these stages is highly desirable as it allows for corrective action to be taken at a lower cost before deployment into service.

In-service inspections, on the other hand, are performed to assess the physical and functional condition of the bridge and to detect any new changes since the previous inspection. In-service inspection serves as a crucial last-line safeguard for detecting defects and play a key role in ensuring the long-term performance of bridges. In addition, the in-service inspection aims to maintain an optimum level of safety, serviceability, and provide the necessary information for asset management. Consequently, this project provides a guideline for bridge owners to determine a rational in-service inspection interval and evaluate unique safety and serviceability criteria for pedestrian bridges.

1.2 Current Pedestrian Bridge Management

A survey was conducted over the course of the research to explore the current practice of pedestrian bridge management by different agencies in the U.S. and overseas. The survey questions and responses are summarized in Appendix A. The survey consists of ten questions for exploring inspection, management, and design criteria utilized for pedestrian bridges by various owners. Another aspect of the survey was to explore if the agencies maintain a database for pedestrian bridges and what information is stored in the database. The research team sent out the survey to about fifty agencies, specifically targeting state agencies and state DOTs. About forty responses were received from different state DOTs and agencies, three of which are from international cooperatives. In general, the responses suggest that there are fewer requirements for pedestrian bridges compared to highway bridges in terms of design, inspection, and management.

1.2.1 Inspection

Based on the responses received, it appears that there is currently no uniform inspection criteria specifically developed for pedestrian bridges, and most agencies use the same criteria that are used for highway bridges. Out of the agencies that responded, only one response (South Carolina DOT) indicated that they have an in-service inspection manual for pedestrian bridges (SCDOT, 2006). However, upon review, the research team concluded that this manual does not fully address the unique factors associated with pedestrian bridges. For instance, it does not provide guidance for determining inspection intervals or serviceability criteria. Additionally, most state DOTs do not inspect all pedestrian bridges; rather, they only inspect bridges that they directly own or those that cross over state roads to ensure the safety of the service under the bridge.

The survey conducted in this study included a question that inquired about the availability of alternative inspection criteria specifically designed for pedestrian bridges. The results revealed that most respondents reported utilizing the same inspection manual for highway bridges to inspect pedestrian bridges. However, some agencies have adopted inspection intervals ranging from every 4 to 5 years, unless the bridge's condition requires more frequent inspections. Nevertheless, the process of determining the inspection interval is not based on documented criteria, but rather depends on inspector's personal experience. For instance, Texas DOT conducts inspections on all pedestrian bridges every 4 years and may extend the interval by 1 or 2 years based on the bridge's condition (TxDOT, 2022).

1.2.2 Pedestrian Bridge Inventory

The survey included questions exploring whether agencies maintain a database for pedestrian bridges, and what type of information is stored therein. About two-thirds of the responses indicated that pedestrian bridges were not included in their bridge inventory. This is probably because the Federal Highway Administration (FHWA) does not mandate DOTs to report pedestrian bridge data. The remaining third of the responses indicated that data is only collected for pedestrian bridges that cross over or under state-owned roads. When such data is collected, it often includes vertical and horizontal clearances. Recording the vertical clearance is crucial for DOTs to ensure that bridges are not vulnerable to impact loads from trucks.

1.2.3 Pedestrian Bridge Design Specification

Including questions about pedestrian bridge design specifications is important for gaining insight into the unique components and characteristics of pedestrian bridges, ensuring that these elements are considered during routine inspections. Results from the survey revealed that most respondents have adopted the

AASHTO Pedestrian Bridge Design Specifications (AASHTO, 2009). However, a subset of respondents requires additional structural and serviceability requirements in conjunction with AASHTO Design Specifications. For instance, the Colorado Department of Transportation mandates that pedestrian bridges feature non-skid surfaces, such as transverse fiber and broom finishing for concrete (CDOT, 2021).

Some states' design specifications for pedestrian bridges consider the location and significance of the bridge. In Pennsylvania (PenDOT, 2020), for example, footbridge design criteria are categorized into three groups, namely Group I, Group II, and Group III, based on the location of the pedestrian bridge as follows.

- Group I: A pedestrian bridge that crosses the Department Highway.
- Group II: Pedestrian bridge not located over a department highway but crossing a roadway owned by another local agency.
- Group III: Pedestrian bridge not located over a roadway (i.e., a pedestrian bridge crossing a waterway).

Group I include pedestrian bridges that cross the department highway, while Group III includes pedestrian bridges that cross a waterway. The design requirements for each group are specified, including redundancy analysis, inspection requirements, fatigue detail categories, fracture-critical members, and material requirements. Similarly, the California Department of Transportation (Caltrans, 2009) has developed a design memo for steel pedestrian bridges, classifying them into two groups: standard and minor bridges. A bridge is considered standard if it meets specific criteria, such as providing a significant link for public movement to service facilities, such schools, or crossing a highway. The primary objective of such classification is to determine whether the footbridge should be treated as a Fracture Critical Member during fabrication and design.

1.2.4 Pedestrian Bridge Load Rating

Another aspect of the survey aimed to investigate the load rating guidelines specifically applicable to pedestrian bridges. The survey revealed that no load rating guidelines are specifically applicable to pedestrian bridges. Some pedestrian bridges exhibit significant section loss, which may impact their carrying capacity for carrying pedestrian loads in their current condition. Furthermore, in some situations, pedestrian bridges need to have enough capacity to carry emergency vehicles such as ambulances. Therefore, a load rating guideline for pedestrian bridges is crucial, not only to ensure the bridge has enough capacity to support pedestrian loads, but also to carry emergency vehicles if needed.

2. ON-SITE INSPECTION

An on-site inspection was conducted for a selected set of bridges. The primary objective of this inspection was to review and compare the actual condition of the bridges with the documented condition in the inspection reports. Additionally, the on-site inspection enabled the research team to gain further insights into the serviceability and structural criteria that are specific to pedestrian bridges. The selected bridges for the on-site inspection involved a variety of bridge types, materials, and condition ratings, as illustrated in Table 2.1.

2.1 Steel Bridge

The P019-29-08543 pedestrian bridge, constructed in 2001 by the Noblesville Parks Department and prefabricated by the Continental Bridge Company, spans a two-lane road, and is situated adjacent to a railroad bridge. This bridge comprises a prefabricated truss constructed with square hollow structural sections and boasts a maximum span length of 105.6 ft. This portion of the bridge features a concrete deck. Additionally, the bridge includes two approach spans at each end, composed of hot-rolled I-beams and a timber deck, as illustrated in Figure 2.1. Notably, the Indiana Department of Transportation (INDOT) inspects only the truss section of the bridge, as it passes over a state road. The research team also examined another pedestrian bridge owned by the Noblesville Parks Department, which shares similarities with the former bridge, except for a timber deck. The bridge fabricator provided the research team with relevant design, inspection, and maintenance information. The two bridges were designed to carry pedestrian loads as well as 5-ton maintenance vehicles. The fabricator recommends a 24-month inspection interval for both bridges, although it is based on a conservative approach to ensure functionality, rather than a rational risk analysis. Furthermore, the use of de-icing agents is prohibited, and the superstructure should undergo periodic maintenance flushing to remove any debris accumulation. Inspection reports reveal that the bridge underwent its first routine inspection in 2007, and at that time, the National Bridge Inventory (NBI) condition rating was 8 for both the superstructure and the deck.

In 2009, as the bridge was assigned for an arm-length inspection, fractures was noted on of the floor beams and about 40% of the longitudinal attachment to the lower chord's exterior face as shown in Figure 2.2. Moreover, the bridge experienced impact damage in the north lower chord, and the impact data was not reported until it was first observed in 2013. Tubular sections are more susceptible to fracturing because water seeps inside the section and expands when it reaches freezing temperature resulting in fractures of the section.

Hence, based on the research findings, the research team recommended that the tubular sections be properly sealed to prevent water from seeping inside and causing expansion during freezing temperatures, which can lead to fracturing of the sections. Alternatively, drainage holes can be drilled to allow water to drain out from inside the sections. In 2015, the bridge

TABLE 2.1
On-site inspection bridges number, types, parent asset, and location

Bridge Number	Material	Parent Asset	Longitudinal	Latitude
P019-29-08543	Steel-truss	Greenfield	-86.01604	40.04887
47-00128	Timber	Lawrence	-86.66508	38.79704
P(66)460-82-04867	Concrete	Vincennes	-87.55117	37.99939
P930-02-10169	Cable stayed girder bridge	Fort Wayne	-85.11277	41.11361
P000-02-09844	Stayed girder	Fort Wayne	-85.11501	41.11654
P000-02-08612	Stayed girder	Fort Wayne	-85.10449	41.11587

Figure 2.1 View of the (P019-29-08543) steel bridge.

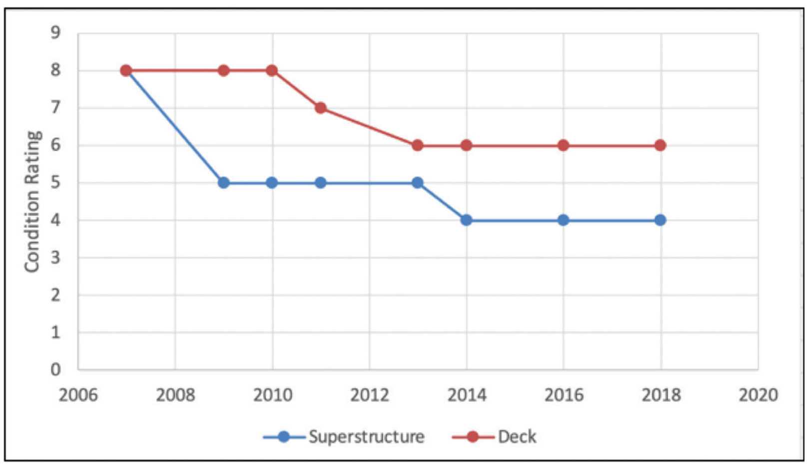

Figure 2.2 NBI condition rating for steel bridge, 2006–2018.

underwent retrofitting, which involved welding the fractures and drilling holes in the bottom of the floor beams. Additionally, the longitudinal and other attachments at the lower chords of the truss were removed. Despite these actions, the bridge continued to exhibit fractures in the floor beams, indicating inadequate execution of the retrofit, as depicted in Figure 2.3.

It was also noticed that the floor beams also experienced significant corrosion, especially at the ends of the floor beams, due to the lack of a drainage system. The rainwater drained away only from the small gap between the concrete deck and the railing. Stringers in the bridge with timber deck was severely corroded because of the direct contact between the stringers and the timber deck. Besides direct drainage onto the superstructure, the timber deck retains moisture for extended periods, continuously exposing the stringers to moisture. As a result, stringers in timber decks are more susceptible to severe corrosion, as shown in Figure 2.4. The research team discussed the rapid corrosion of the stringers in a timber deck with the bridge fabricator. The fabricator mentioned that they usually recommend customers/owners to apply an effective coating on the stringers to prevent such issues. The use of timber decks in pedestrian bridges is ubiquitous, making it crucial to consider its effect on the corrosion development.

The research team also inspected non-structural elements. This inspection aimed to acquire some criteria that may affect the serviceability and pedestrian safety over the bridge. The research team indicated that the quality of the walkway surface was one of the most critical components to inspect to ensure the safety of pedestrians over the bridge. For the portion of the

Figure 2.3 View of fractures in the floor beam and the longitudinal attachment to the bottom chords.

Figure 2.4 View of the stringers and floor beam in timber and a stay-in-place concrete deck.

concrete deck, the surface of the concrete deck did not show any serviceability concerns, such as the skid surface or potholes that may affect pedestrian safety. For the timber deck, the research team mentioned that the timber surface should not have any object harm the users on the bridge, such as projected nails or bolts on the deck surface.

The research team identified several safety criteria that could affect the bridge's serviceability, including the height of the railing, the strength of the bolts attaching the sign, and the vertical clearance. The team found that while the railing height of approximately 5 ft. was adequate for pedestrian safety, it could also increase the risk of objects being thrown from the bridge onto vehicles passing beneath it. Thus, the team recommended installing a fence to protect the service under the bridge. Moreover, the team indicated that the railing elements needed to be checked for any missing or lost elements, as shown in Figure 2.5.

The team also observed that the anchor bolts attaching the sign to the bridge were insufficient to support its self-weight and wind load, and thus suggested using larger anchor bolts to prevent the sign from falling onto passing vehicles. Finally, the vertical clearance of the bridge was 13.5 ft., which could increase impact damage likelihood. Therefore, the team recommended periodic inspections of the bridge to monitor any potential issues that may arise because of the impact.

2.2 Timber Bridge

The 47-00128 bridge, constructed in 1884, holds historical significance as per the National Historic Preservation Act (Erikkson et al., 2000). The bridge consists of two spans, with the maximum one measuring 184 ft. and spanning a waterway, as shown in Figure 2.6. The vertical clearance of the bridge is 16.5 ft. above the ordinary water elevation. The bridge was reconstructed in 1984, repaired in 1999 and 2012. Since the last retrofit, the bridge has been used exclusively by pedestrians.

The deck and superstructure of the bridge were constructed using solid sawn timber and vertical steel hanger rods. The inspection report states that the bridge is rated for a maximum weight of 5 tons for vehicles and can accommodate up to 400 pedestrians. However, the location of the bridge in a rural area is not expected to have a high volume of pedestrian traffic.

The most recent inspection report indicates that the bridge experienced minor damage due to fire and impact in the past. Nonetheless, the overall condition of the bridge was considered satisfactory. Currently, the bridge undergoes a visual inspection every 2 years, along with an underwater inspection conducted by an external consultant. During the rehabilitation project in 2012, the routine inspection of the bridge could not be conducted. Consequently, the inspector assumed that the condition ratings for the deck and superstructure

Figure 2.5 View of missing element and inadequate anchorage of bridge railing.

Figure 2.6 View of the timber bridge.

were excellent (NBI condition rating 8). The first inspection carried out after the retrofit occurred in 2014, revealing condition ratings of 6 and 5 for the deck and superstructure, respectively. Since then, inspection records show that the condition ratings of the superstructure and deck have remained unchanged as shown in Figure 2.7.

Despite the seemingly unnecessary biennial inspection interval for this bridge, it is essential to note that rural bridges, particularly those constructed using timber, are more susceptible to vandalism and arson. Owners of such bridges, therefore, tend to inspect them at shorter intervals to detect and report any acts of vandalism, impact, or arson promptly.

The research team conducted a thorough inspection of both structural and non-structural components of the bridge. According to the inspection report, the structural condition rating of the bridge was evaluated satisfactory, despite the presence of splits and checks in some diagonal members of the superstructure. While enclosures can provide protection against environmental factors, the team observed that ineffective enclosures might accelerate decay development. Particularly in scenarios where water infiltrates structural elements, the consequent lack of airflow can cause the holding of moisture for extended periods, thereby acerating the development of decay. The team also evaluated non-structural elements and recommended that gaps between timber planks do not pose any hazard to pedestrians or cyclists, especially when the planks are placed longitudinally, as shown in Figure 2.8. Thus, the research team recommended that the gaps between timber planks should not exceed ½ inch in any direction to prevent any potential hazards for bridge users.

2.3 Concrete Bridge

The P(66)460-82-04867 bridge is a prestressed box-girder pedestrian bridge comprising two spans that pass over State Road 66 (SR 66). The bridge was constructed in 1963, with a total length of 142 ft., and a vertical clearance of 14 ft. 9 inches, as shown in Figure 2.9. Pedestrian safety is enhanced by metal fences installed on each side of the bridge, which also serve to protect the area beneath the bridge from potential hazards,

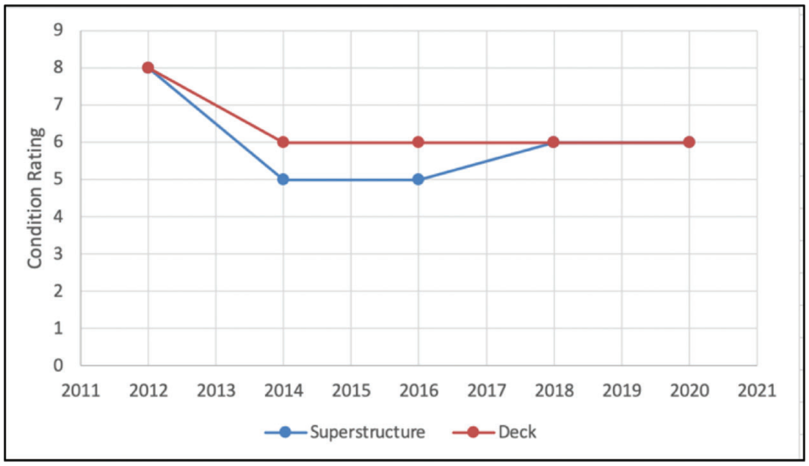

Figure 2.7 NBI condition rating for the timber bridge, 2012–2020.

Figure 2.8 View of the top and the bottom deck of the timber bridge.

such as thrown objects. The bridge features a ramp and stairs on each side to provide easy access. Routine inspections are conducted at 24-month intervals, primarily to ensure the safety of motorists navigating SR 66 below. The average daily traffic on the road under the bridge is recorded at 33,253 vehicles per day, highlighting the importance of regular structural inspections to maintain the safety of traffic below the bridge.

In general, the bridge was evaluated to be in a satisfactory condition according to the inspection report. The condition rating of the superstructure and deck have remained unchanged since 2007, except for spalling areas which appeared to be a result of past collisions. Additionally, the access ramp beams exhibited shallow spalls with exposed stirrup reinforcement and hairline longitudinal cracks near the beam support. Figure 2.10 shows the condition rating of the deck and superstructure over time.

The 24-month inspection interval was designated to identify potential areas of deterioration that could adversely impact the safety of the service under the bridge, despite the possibility of extending the inspection interval. The research team recommended evaluating skid surfaces and potholes as the primary serviceability criteria for concrete decks. However, the general condition of the concrete deck surface for the given bridge showed no signs of skid surfaces and potholes.

2.4 Cable-Stay Bridge

The P930-02-10169 bridge is a cable-stayed structure constructed in 2010 with a maximum span length of 385 ft. and a total length of 465 ft. The bridge comprises two main steel girders connected by cables to two pylons set at both ends, with floor beams and a cast-in-place concrete deck, as shown in Figure 2.11. The design drawings specify that the bridge was designed to withstand live loads of 65 pounds per square foot (psf) and dead loads of 20 psf. Additionally, the bridge was designed to carry an H5 maintenance vehicle, as per the *AASHTO LRFD Specifications for Pedestrian Bridges* (AASHTO, 2009).

Wind loads of 132 psf and a wind velocity of 100 mph were also considered during the design phase, with bent plate wind fairings installed along the longitudinal girders on both sides to minimize wind load effects. The cables comprise assemblies of ASTM A586 structural strand with zinc-filled (spelter) end socket connections, featuring stainless steel connection pins (ASTM, 2018).

Figure 2.9 View of the concrete bridge.

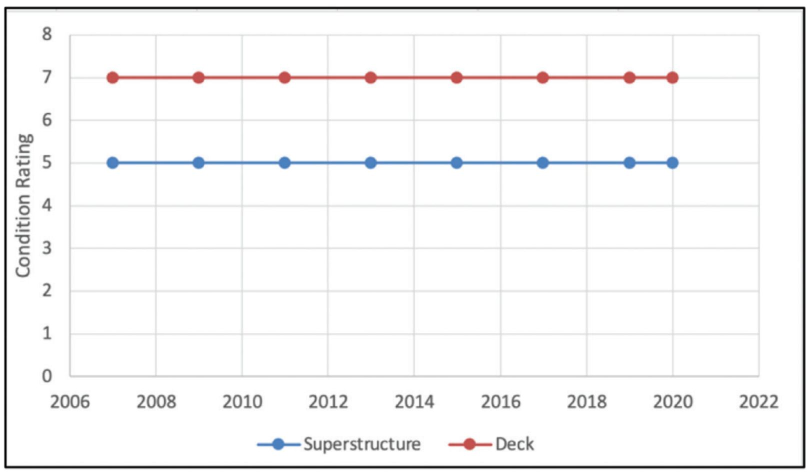

Figure 2.10 NBI condition rating for the concrete bridge, 2007–2020.

Figure 2.11 View of the cable-stay bridge.

Engineering Resources, a private consultant, developed an inspection manual specifically for this bridge to supplement INDOT's methodology of bridge inspection practice. The inspection manual recommends a routine inspection every 2 years and special inspections every 10 years. The routine inspection encompasses all visible components, while the special inspection targets the hidden components, such as lower sockets and main girders, either through a borescope or by removing the cover plate to facilitate inspection. However, these inspection intervals are not based on a documented risk assessment approach, and the rationale behind these intervals is not demonstrated in the manual. Further, the manual does not provide information on what events or conditions might necessitate a shortening these inspection intervals.

The general condition of the bridge was found to be satisfactory for both the superstructure and the deck, as shown in Figure 2.12. However, the deck condition rating decreased to 7 during the 2018 inspection due to transverse hairline cracks across the deck, particularly near the middle of the bridge.

Due to the absence of bollards at the bridge ends, a vehicle attempting to cross the bridge caused severe damage, resulting in a 4-foot section of the decorative railing across the deck being damaged. Thus, the installation of bollards is necessary to protect the bridge from collisions and prevent overloading. Scheduling regular special inspections for hidden components, as shown in Figure 2.13, was a rational decision. However, this schedule may not be entirely reliable if the condition of these components deteriorates significantly, potentially necessitating a shorter inspection interval than the designated 10-year interval. For instance, conducting an in-depth inspection of the cable connection assembly every 10 years may be reasonable for components in good condition. However, a 10-year interval may not be appropriate for elements that have deteriorated extensively, as they tend to deteriorate at a faster rate compared to when they are in a good state. This accelerated deterioration could result in the failure of the component before the next inspection interval occurs. Consequently, the research team emphasized the need to determine the in-depth inspection interval based on a risk assessment that considers the condition and design characteristics of the components.

In the context of serviceability inspection, the research team suggested that certain cable-stayed bridges are susceptible to experiencing excessive vibrations, and it is important for inspectors to judiciously evaluate and document any excessive vibration. Moreover, the research team emphasized the criticality of installing

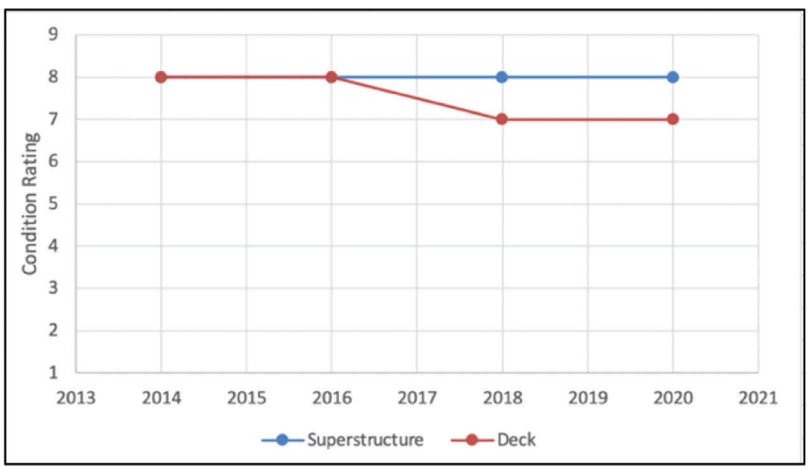

Figure 2.12 NBI condition rating for the bridge cable stay bridge, 2012–2020.

Figure 2.13 View of some hidden components in the bridge.

bollards on pedestrian bridges to avert the entry of unauthorized vehicles onto the bridge, which could result in the bridge becoming overloaded.

Within the assessment of serviceability inspection, the research team pointed out that certain cable-stayed bridges may be subjected to excessive vibrations, highlighting the importance for inspectors to rationally evaluate and document any such occurrences. Furthermore, the team pointed out the critical necessity of installing bollards on pedestrian bridges to prevent unauthorized vehicle access, which could lead to overloading and potential structural compromise of the bridge.

3. OVERVIEW OF THE METHODOLOGY

The National Bridge Inspection Standards (NBIS) mandate that *highway* bridges undergo routine inspections every 24 months, which can be extended to 72 months if specific criteria are met (NBIS, 2022). This approach is commonly referred to as the calendar-based method. Additionally, inspection intervals of less than 2 years may be conducted for certain bridges based on owner-established criteria, such as age and severe condition ratings. However, it is noteworthy that NBIS regulations do not include pedestrian and cyclist bridges. The FHWA has provided the following response to clarify the applicability of NBIS inspection requirements to pedestrian bridges.

> *FHWA Response: "The NBIS is only applicable to highway bridges located on public roads. Bridges that only carry pedestrian and bicycle traffic are not highway bridges and therefore are not subject to the NBIS." (NBIS, 2022*).

Since there are no specific requirements, owners of pedestrian bridges often adopt a calendar-based inspection approach in the absence of specific regulations governing maintenance and assessment of such structures. However, the calendar-based approach overlooked several critical factors that influence the bridge's reliability and safety, such as the current condition, design specifications, operational environment, and the importance of the bridge. The condition of the bridge is a key consideration in inspection planning, as bridges with active defects necessitate greater attention than those with intact elements. Moreover, modern bridges are constructed in compliance with current design specifications which involved enhanced material and durability requirements, extending the bridge's service life.

The operational environment plays a critical role in determining the frequency of inspections, given that certain bridges are exposed to aggressive environments that accelerate the deterioration process and decrease the bridge's lifespan. In addition, some bridges hold greater significance. For instance, when a bridge spans over busy roads with a high volume of daily traffic (ADT), the falling debris from deterioration could pose serious safety risks or hazards to the traffic below. Therefore, it is necessary to consider these factors in inspection planning to optimize resource allocation and enhance the bridge's safety and reliability.

Consequently, the risk-based methodology, developed in conjunction with reliability theory as presented in the NCHRP report 782 (Washer et al., 2014), is considered a practical approach for determining inspection intervals as it comprehensively addresses the aforementioned factors. A fundamental understanding of reliability theory is essential for a comprehensive understanding of this proposed approach. Following is a brief discussion of reliability theory in the context of bridge inspection.

3.1 Reliability Theory

Reliability refers to the capacity of an item to perform its designated function adequately within a specified time frame (Stewart & Rosovsky, 1998). The critical phrase in this definition is that reliability is a function of time, signifying that the reliability decreases as time elapses. This decline in reliability is attributable to deterioration and accumulative damage, which are expected to occur over time. The rate at which component reliability decreases varies depending on factors such as loading conditions and design characteristics of the component. For example, a steel girder located in a marine environment may corrode faster, potentially resulting in significant section loss over a shorter period, compared to the same element located in a benign environment, which would deteriorate more slowly. Consequently, the reliability of the element located in a harsh environment is lower than that of one located in a benign environment over a given time. Conversely, the probability of failure is the likelihood of an element's *inability* to execute its intended function at a given time. Unlike reliability, the probability of failure typically increases with time, as depicted in the expression below (Tsarouhas, 2021).

$$R(t) = P_{Rs}(T \geq t) \qquad \text{(Eq. 3.1)}$$

The reliability of an element at a given operating time t is represented by $R(t)$, while P_r denotes the probability that operating time (t) is less than the time to failure (T). In essence, reliability is the probability that an element's operational period will not surpass its time to failure. The probability of failure, on the other hand, is a measure that reflects the possibility of an element's operational time exceeding its anticipated time to failure. Furthermore, it is noteworthy that reliability can also be expressed in terms of the probability of failure, as demonstrated in the expression below.

$$R(t) = 1 - F(t) \qquad \text{(Eq. 3.2)}$$

The probability of failure is commonly expressed in a probability density function in which the time to failure (T) is expressed in a generic distribution such as Weibull, normal, and lognormal distributions. Such distributions are used to identify the probability of failure function $F(t)$, representing the probability that an element will fail at any time up to time (t).

3.2 Failure Rate Function

An important concept to understand in reliability theory is the failure rate, or hazard function, which is defined as the rate at which an element may fail within a certain time period (Harish Kumar et al., 2021). The failure rate function provides a description of the typical lifetime performance of an element and can take on different forms. The bathtub function can exhibit variability across different bridge elements, influenced by their operational and environmental effects. For instance, the useful life of a precast concrete deck may exceed that of a cast in place concrete deck. However, the bathtub shape function represents a simplified and ideal shape that is often used in the context of bridges as shown in Figure 3.1.

The bathtub curve function for a given element is divided into three primary regions: the infant mortality region, the useful life region, and the wear-out region (Baliga, 2023; Harish Kumar et al., 2021). The infant mortality region represents failures occurring during the initial stages of fabrication and construction. Following this, the failure rate decreases as time progresses, reaching its lowest point at the onset of the useful life region. Risks in this region are more likely associated with accidental incidents unaffected by time, such as collisions, fires, and so forth. In the wear-out region, the failure rate of an element begins to increase rapidly as the element nears the end of its useful life. The failure rate in this region is more likely a result of cumulative damage and aging, leading to a decline in the element's reliability.

Therefore, conducting inspections during these stages is necessary to detect defects at an early stage. For example, fabrication and construction inspections are essential to ensure that the risks associated with the infant mortality region are addressed and mitigated. During the useful life region, in-service inspections are conducted to ensure that the risk to the bridge element remains low and does not enter the wear-out region where the failure rate is high. Thus, the primary objective of in-service inspection is to ensure that the condition of the bridge element remains within the useful life region.

The inspection of bridges during their service life is a crucial phase, as it represents the last line of defense against unpredictable failure. Consequently, this project will primarily focus on in-service inspections of several types of pedestrian bridges. The proposed methodology involves a systematic approach to determining the inspection interval based on the risk associated with each element.

3.3 Risk-Based Concept

The proposed methodology is the risk-based method, which was developed through the NCHRP Report 782 (Washer et al., 2014). The implementation of the risk-based method was established to improve the serviceability and safety of bridges by determining the appropriate inspection interval. The method can be summarized by asking three questions.

1. *What can wrong?* The objective of this question is to identify all possible types of damage that an element may experience during its service.
2. *How likely is it to occur?* The primary purpose of this question is to estimate the likelihood of the occurrence of the damage mode. The response to this question provides an assessment of the occurrence factor, ranging from "remote" if the likelihood of damage is extremely low, to "high" when the damage mode is more likely to occur.
3. *What are the consequences?* The consequence factor should be assessed in terms of safety, serviceability, and economic impacts. Similar categories used in the occurrence factor are used for the consequence factor, ranging from low (i.e., only minor effect on serviceability) to severe (i.e., collapse).

These three questions are formulated to facilitate the risk assessment of a given element within a specified period of time. Generally, risk is simply defined as the

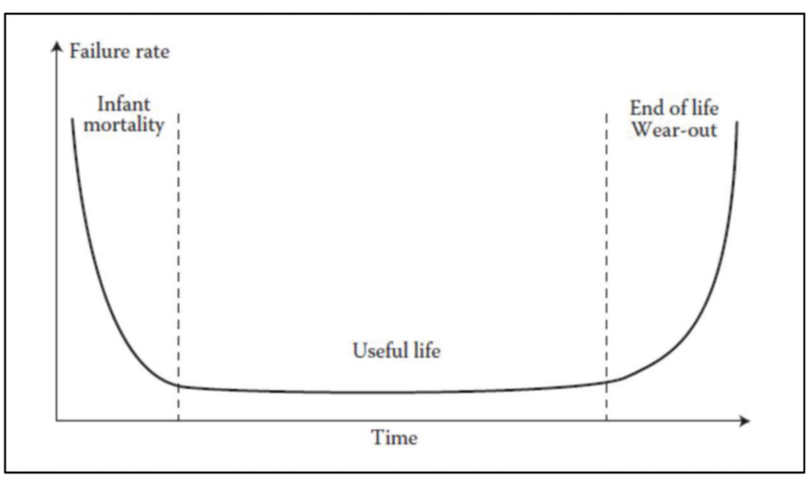

Figure 3.1 Plot of bathtub function.

product of frequency, which represents likelihood, and consequence, as expressed in the equation.

$$Risk = Frequency \times Consequence$$

The frequency in the previous expression represents the occurrence factor or the probability of failure (POF) for a certain damage mode. The likelihood can be evaluated quantitatively if reliable historical data is available; otherwise, it can be estimated qualitatively based on experience or rational engineering judgments. Consequence is a measure of the outcomes of the damage mode occurrence, including, but not limited to, economic, social, safety, or environmental impacts.

The estimation of an appropriate inspection frequency heavily relies on an accurate assessment of both the occurrence and consequence factors. Thus, it is crucial for experts to understand the distinction between identifying the occurrence and the consequence factors. A common point of confusion has been observed when evaluating whether a particular damage mode is associated with high consequence or high occurrence. For instance, there is a misunderstanding that the likelihood of fatigue cracks in two steel girders is higher than in multiple steel girders. However, the truth is that the consequence factor is high in the case of two steel girders, as the resulting damage mode may lead to catastrophic failure. Evaluating the occurrence of fatigue cracks is not based on the number of girders, but rather depends on design specifications, material properties, detailing, ADTT (average daily truck traffic), and other factors. Therefore, the occurrence factor may be low if the girders are designed for finite fatigue resistance and the average daily truck traffic is low.

3.3.1 Damage Mode

It is crucial to distinguish between damage modes and deterioration mechanisms. A deterioration mechanism describes the manner or the process through which an element deteriorates, such as corrosion and fatigue. On the other hand, a damage mode is a result of a deterioration mechanism. For example, the spalling damage mode is a result of rebar corrosion, and the section loss damage mode arises from steel corrosion.

Some damage modes are more critical than others and are referred to as credible damage modes. These damage modes are prioritized due to their higher likelihood of occurrence and significant impact on the element's reliability. For example, corrosion of a steel girder over a waterway may be considered a credible damage mode. Nevertheless, all conceivable damage modes should be considered, regardless of their likelihood of occurrence. The likelihood of a damage mode occurring is subsequently assessed through the occurrence factor.

3.3.2 Occurrence Factor OF

This factor measures the likelihood of severe damage mode development within a specific period. The assessment of the OF in the RBI approach is basically analogous to the evaluation of POF within a given time. Thus, a practical definition of failure and the time frame over which the probability of the damage occurring can be estimated need to be identified to facilitate the estimation of the occurrence factor.

3.3.2.1 Failure definition. The generic definition of failure in engineering typically refers to a catastrophic failure of a structural element. However, this definition may not fully serve the ultimate goals of inspections, which aim to ensure an optimal level of safety, reliability, and serviceability. Moreover, it is worth noting that bridge failures resulting from deterioration are not a very common event. Therefore, an element is considered to be in a state of failure if it fails to meet its intended safety, reliability, or serviceability requirements. For instance, a concrete deck experiencing severe spalling may be considered to be in a state of failure, even if it possesses adequate loading capacity, because the safety of the public on and under the bridge is impacted. To further assist in defining failure, a particular condition rating may be associated with a state of failure. According to the NBIS coding guide, an element in condition rating 3 represents a state where the element is unable to perform its intended serviceability and safety functionality (FHWA, 1995). Although condition ratings are subjective and may vary from one inspector to another, associating a failure condition with a condition rating facilitates the assessment process and provides a comprehensible framework for the risk assessment panel evaluating the risk of an element. Therefore, an element in condition rating 3 is considered to be in a failure condition for the purposes of risk assessment.

3.3.2.2 OF assessment time frame. The assessment of an element's occurrence factor necessitates evaluation over a specified time frame. A 72-month interval is considered for estimating the occurrence factor, based on expert judgment, inspection data, and deterioration models. It was envisioned that experts reasonably predict the bridge's behavior over a 72-month period. Considering a longer duration might introduce greater uncertainty in the expert's assessment.

Inspection data suggest that, in a typical scenario, the condition rating of bridges does not significantly transition from good to severe within this 72-month time frame (Nasrollahi & Washer, 2015). This is because the initiation of some deterioration mechanism is relatively slow as indicated by deterioration models. This is because the initiation of some deterioration mechanisms is relatively slow, as indicated by deterioration models. For instance, models used to estimate the atmospheric corrosion of a steel girder in a highly corrosive environment with a standard coating protection system indicate that corrosion will initiate 15 years after the deterioration of the coating protection system (Kere & Huang, 2019). Therefore, it is unlikely that an element in good condition will transition to a serious condition within 72 months. However, this does not imply that the appropriate inspection interval is 72 months; rather, it is a period over which the reliability of the element can reasonably be estimated. In fact, the inspection interval may be shorter or longer than 72 months based on other considerations such as serviceability and safety consequences. Thus, 72 months is considered an appropriate time frame for evaluating the occurrence factor.

3.3.2.3 OF assessment. Estimating the likelihood of an element to develop a particular damage mode over a specific period necessitates additional factors, including element characteristics, environmental conditions, and loading conditions. Various methods, ranging from fully quantitative to qualitative, can be utilized to estimate the occurrence factor. The quantitative approach involves the use of numerical models, past performance data, and failure rate data. Conducting a quantitative assessment of the OF may be appropriate in some industries where historical data of the component under a specific operating environment is available. For instance, in the American Petroleum Institute Practice (API 581, 2008), the assessment of the OF is conducted using a numerical model based on multiple attributes contributing to damage mode development over time. However, for bridges, such data may either be incomplete or inaccurate, and thus, utilizing this data may not represent realistic situations. Numerical models require many input data, such as the element's characteristics, environmental, and loading variables, which need to be determined empirically or through statistical functions. Additionally, the significant variability of this data across different environments, design, and construction requirements complicates the process. What further complicates this process is that validating the values of these data requires observing the bridges' performance over their service life, and hence such data may not be timely applied. Therefore, rational engineering judgment is considered crucial for determining and ranking the design, condition, and loading attributes to estimate the occurrence factor for a given damage mode.

3.3.2.4 OF categorization. The practical application of identifying the occurrence factor usually presented on the order of magnitude due to the inherent variation and uncertainty in engineering structures. For instance, the American Society of Mechanical Engineers (ASME) suggests evaluating the probability of failure on three scale levels: high, moderate, and low (ASME, 2007). A high probability of failure corresponds to a failure rate of 0.01, a moderate probability of failure corresponds to a failure rate ranging from 0.01 to 0.0001, and a low probability of failure corresponds to a failure rate less than 0.0001. Consequently, the occurrence factors are segmented into four categories, ranging from remote—where the probability of the damage mode occurrence is low minimal—to high, where the damage mode is more likely to occur as shown in Table 3.1. These numerical values of each category could be used to map the quantitative data, if it is available, and facilitate a common language for engineers for OF estimation.

3.3.2.5 Attributes. The occurrence factor for a specific bridge element is evaluated by considering its characteristics and loading conditions, which are also defined as attributes. An attribute is a key feature of the component that contributes to the element's performance, durability, or reliability. These attributes are derived from the design, loading, and conditional characteristics.

3.3.2.5.1 Design attribute. The design attributes refer to specific design features that enhance the performance and durability of a particular element against a specific damage mode. These attributes can be inferred from design specifications and fabrication drawings. For instance, when considering fatigue crack as a damage mode, it is essential to know the design specification that was considered for the bridge's design to determine if the structure has been designed in accordance with the AASHTO/AWS Fracture Control Plan. This information indicates that the steel used must have met the minimum toughness and modern fatigue design requirements. Another way to infer design attributes is by understanding changes in a state's design specifications. For example, if a state mandated the use of coated rebar for all concrete decks during a certain era, it can be deduced that any bridge constructed during that period must have used coated rebar. Overall, design attributes can be inferred from a variety of resources, which can be utilized in the evaluation of an element's performance and durability against particular damage modes.

3.3.2.5.2 Load attribute. Loading attributes refer to the various loading conditions that a structural element experiences over its service life. These conditions

TABLE 3.1
Occurrence factor category description with qualitative and quantitive value (Washer et al., 2014)

Category	Description	Likelihood	Likelihood in Percentage
Remote	Remote probability of occurrence, unreasonable to expect failure to occur	≤1/10,000	≤0.01
Low	Low likelihood of occurrence	1/10,000–1/1,000	≤0.1
Moderate	Moderate likelihood of occurrence	1/1,000–1/100	≤1
High	High likelihood of occurrence	≥1/100	≥1

can include physically applied loads, as well as both macro and micro environmental effects. For instance, bridges in snowbelt regions might be exposed to harsh environmental conditions that can accelerate the deterioration mechanisms for specific damage modes. On the other hand, the environmental impact can be more localized, such as the application rate of de-icing chemicals. Therefore, loading attributes encompass physical, chemical, and environmental factors that significantly influence the acceleration of deterioration mechanisms associated with certain damage modes.

3.3.2.5.3 Condition attribute. The condition attributes refer to the current condition of a bridge element or other related elements that might impact the future reliability of the element under a specific damage mode. For instance, when evaluating the section loss of a steel girder, the condition attributes for the given damage mode not only include the current condition of the steel girder but also the condition of the joint and drainage system. This is because the condition of the joint and drainage system can significantly influence the section loss deterioration process of the steel girder.

Confusion often arises when considering the condition of the element in estimating the occurrence factor for specific damage modes. A common misconception is that if the element has already sustained damage, the occurrence factor is immediately considered high without the need for further evaluation of the occurrence factor. However, the occurrence factor is not only measuring the on the likelihood of an element *initiating* a damage mode. It measures the probability of an element both *initiating* damage and *progressing* from its current state to a more severe condition within the next 72 months. For example, when evaluating the occurrence factor for a steel girder with moderate corrosion, the factor might not be high, even if damage has already developed. In this scenario, the occurrence factor measures the probability of the element deteriorating from its current state, i.e., moderate condition, to a severe one within the next 72 months. Thus, while condition attributes are crucial, they are not the only factors when evaluating the occurrence factor for a specific damage mode; assessment should include design and loading attributes.

3.3.2.6 Attribute ranking. Obviously, certain attributes play a more significant role in the development of specific damage modes than others. To establish a rational approach for estimating the occurrence factor, attributes are ranked through numerical scoring. Each attribute is assessed based on its importance in developing a certain damage mode. Attributes that make a significant contribution to the occurrence factor receive a score of 20 points, 15 points for moderate contributions, and 10 points for low contributions, as illustrated in Table 3.2. The ranking process involves various conditions and scenarios for each attribute. For instance, the presence of a protective system is crucial for preventing corrosion in steel superstructures. If the steel girders have an adequate protective system, a score of 5 points may be given. However, if no protective system is present or if a substandard system is utilized, 20 points may be assigned. It is important to note that the ranking and scoring of attributes vary because bridge inventories possess different operational environments and design characteristics. As such, the evaluation of ranking attributes is conducted by experts familiar with bridge inventories and their operational environments. Ultimately, identifying and ranking key attributes based on their importance provides a systematic and rational method for estimating the occurrence factor for specific damage modes.

3.3.3 Consequence Factor (CF)

This factor measures the outcomes associated with the occurrence of a certain damage mode, assuming the failure of the element being evaluated. The consequence of a specific damage mode includes any significant effects on safety, serviceability, or economic impacts. Although assuming the failure of the element is not an anticipated event when the RBI approach is applied, this assumption is necessary to identify the consequence factor of the component. The outcome of specific damage varies across the bridge elements. For example, a failure of a lateral bracing member may not result in bridge failure, while the failure of the main girder in a nonredundant system may lead to catastrophic failure. Nevertheless, evaluating the consequence factor is not limited to assessing structural integrity but also extends to evaluating serviceability consequences. For instance, the consequence of a certain damage mode might require lane closure under the bridge or even bridge closure for maintenance. It should be noted that serviceability consequences affecting pedestrians and cyclists on the bridge are assessed through the serviceability concerns evaluation and are not considered in the risk assessment. The panel suggested that the serviceability criteria be evaluated in a checkbox

TABLE 3.2
Attribute ranking description

Rank	Description	Score
Remote	Attribute has no or benign contribution in the development of the damage mode	0
Low	Attribute has a low contribution in the development of the damage mode	5
Moderate	Attribute has a moderate contribution in the development of the damage mode	10
High	Attribute has a signification contribution in the damage development	20

TABLE 3.3
Consequence categories descriptions

Category	Consequence on Safety	Consequence on Serviceability	Summary
Low	None	Minor impact on the serviceability of the service under the bridge	Minor impact on the serviceability of the service under the bridge, no impact on the safety
Moderate	Minor	Moderate impact on the serviceability of the service under the bridge	Moderate impact on the serviceability of the service under the bridge, minor impact on safety.
High	Moderate	Major impact on the serviceability of the service under the bridge	Major impact on the serviceability of the service under the bridge, moderate impact on safety
Severe	High	Significant impact on the serviceability of the service under the bridge	Significant impact on the serviceability of the service under the bridge, high impact on safety

format, as discussed in the next chapter. Therefore, the consequence factor should assess structural integrity, public safety, and serviceability of the traffic under the bridge aspects. Based on these considerations, the consequence factor can be divided into four qualitative categories: low, moderate, high, and severe, as shown in Table 3.3. Appendix D provides a detailed descriptions of the consequences for various damage modes and elements.

The procedure for evaluating the consequence factor begins by assuming a failure scenario due to the damage mode of the component under consideration. This failure scenario involves specifying the location, traffic pattern, and structural characteristics of the bridge. Specifying the location of the bridge is necessary as the severity of consequences for bridges located in urban areas is higher compared to those in rural areas because of the pedestrian volume. The traffic pattern is crucial in assessing the consequence factor to ensure public safety. For instance, the consequences of deck soffit spalling are higher for bridges over highways because of the potential safety consequence from falling debris to the traffic below, compared to those over non-navigable waterways where the consequences are lower.

In addition, the structural characteristics of the bridge significantly influence the determination of the consequence factor. For instance, redundant elements are typically considered to have a low to moderate consequence factor, whereas non-redundant elements are evaluated to be severe. Consequently, the assessment of the CF is divided into two aspects: immediate consequences and short-term consequences, accounting for structural integrity and serviceability outcomes, respectively. Rational engineering judgment and experience were considered as practical methods to establish a qualitative assessment of the damage modes' consequences.

3.3.3.1 Immediate consequence. The immediate consequence factor represents the outcomes that affect the safety and integrity of the structure in the event of a failure of a specific element. This includes assessing whether the structure can remain standing after the failure of the element being considered and ensuring the safety of the traveling public. For example, the failure of a load-bearing component in a twin girder system is considered an immediate consequence as it may impact the structural integrity of the bridge. It is worth noting that the immediate consequence is not limited to structural failure but also includes any possible consequences that impact the safety of the service on or under the bridge. For instance, deck spalling is considered an immediate consequence since falling debris can potentially impact the safety of the traffic or the properties under the bridge. Therefore, evaluating the immediate consequence factor requires assessing both the structural integrity and safety outcomes of the element under consideration.

3.3.3.2 Short term consequence. Short-term consequences relate to serviceability outcomes that can affect traffic on the bridge or underneath it, typically arising from maintenance activities. For instance, should a pedestrian bridge require maintenance due to specific damage, the lanes underneath it might be temporarily closed to facilitate the maintenance process. This closure may result in speed reduction and traffic delays of the service underneath the bridge. The location of the bridge is another important factor in evaluating short-term consequences. Bridges linking essential areas, like schools and commercial zones, have more significant consequences than those in rural settings. It should be noted that some pedestrian bridges may have low pedestrian traffic most of the year, while they may experience many pedestrians during annual events. Therefore, when evaluating such bridges, it is important to consider their peak usage. The presence of an alternative route may reduce the impact; however, proper signage directing users to this route is essential. Overall, short-term consequences affect the serviceability for both pedestrians on the bridge and the traffic beneath it.

3.3.4 Inspection Effectiveness

According to NCHRP 782 (Washer et al., 2014), reliable inspection techniques are assumed to be used in conjunction with an appropriate inspection interval to ensure an optimum level of reliability. In other words, applying the risk-based method may only guarantee an optimized level of reliability if an ineffective inspection technique is utilized. Although NCHRP 782 considered uncertainty in some situations, such as the inspection of a deck with a stay-in-place (SIP) form or an overlay, these components still fall in the same risk category as the typical situation where the deck has neither stay-in-place forms nor an overlay. For example, 20 points will be added to the sum of spalling attributes if the bridge deck has SIP form as an attribute that accounts for the uncertainty due to the hidden part of the deck by SIP form. Nevertheless, this attribute does not significantly change the occurrence factor. In fact, the occurrence factor of a deck with SIP form becomes identical to a deck without SIP, and thus, they may have the same inspection interval. Furthermore, some inspection components may not be accessible because of the geometry of the components or decorative enclosures.

The fundamental purpose of the inspection is to determine whether an element is in a reliable condition to perform its intended function and to reduce uncertainty regarding the development of a particular damage mode. Utilizing an unreliable inspection technique raises uncertainty regarding the element's condition, resulting in increased risk associated with the component. Considering the above, the inspection effectiveness factor is incorporated into the risk assessment to enhance the reliability of the RBI approach and assist users in identifying the inspection interval and recognizing the appropriate inspection techniques that can be used for each defect.

3.3.4.1 Overview. Inspection effectiveness is defined herein as the ability of the inspection technique to reliably detect and quantify a particular damage. The inspection effectiveness depends on multiple factors, such as inspector training, skills, the type of defect, accessibility to the component, and the inherent uncertainty of some inspection techniques. The implementation of the inspection effectiveness factor in the RBI approach is not a new concept. For example, in the American Petroleum Institute's (API) Risk-Based Inspection Practice (API 581, 2008), the inspection effectiveness is implemented in calculating the probability of failure. Inspection effectiveness is classified into five categories, A through E, denoting an inspection providing the most effective inspection through a no inspection or not effective inspection technique, respectively, as is demonstrated in Figure 3.2. The graph in shows the relationship between the risk associated with a certain component and the level of inspection effectiveness. As expected, the level of inspection effectiveness clearly influences the risk value.

The main goal of the inspection plan is to ensure that the risk value is less than or equal to the risk target. For example, choosing the inspection effectiveness of level D may not be appropriate as the risk at the planned date exceeds the target risk. Therefore, shortening the inspection interval or choosing a more effective inspection technique may be required to mitigate the risk. A highly effective inspection technique does not explicitly mitigate the physical risk of the component, but it provides prior information about the condition of the element with less uncertainty so that a mitigation plan can be developed in advance. The inspection effectiveness categories mentioned in API are categorized based on the fact that if multiple low effective inspections have been conducted within a designated period, they are equivalent to a single highly effective inspection as shown in Figure 3.2.

In the aerospace industry (Aust & Pons, 2021), inspection effectiveness is incorporated through what is referred to as a contextual factor (cofactor). This factor is a combination of three separate factors, criticality, severity, and detectability factors. Criticality factors are based on the importance of the component, and the severity factor describes the probability of the defect propagating to a severe outcome. The detectability factor accounts for the visibility of the element and the reliability of the inspection technique.

The cofactor factors range from defect always detected to defect visually not detectable, with values ranging from 0.5 to 2.0, respectively. In other words, the cofactor could reduce the risk to half if the defect is always detectable or double the risk if the defect is not visually detectable, as seen in Figure 3.3. As the contextual factor (cofactor) is identified, the risk score (RS) is then calculated by multiplying the consequence, likelihood, and cofactor.

3.3.4.2 Evaluating inspection technique reliability: A review of approaches. As mentioned previously, inspection effectiveness evaluation relies on multiple subjective factors such as skill, training, environment, inherent uncertainty of some inspection techniques, etc.

Table 5.5 – Guidelines for Assigning Inspection Effectiveness – General Thinning

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 by thickness measurements	50 to 100% ultrasonic scanning coverage (automated or manual) 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 measurements, and a documented inspection planning system
E	Ineffective	No inspection	Several thickness measurements taken only externally, and a poorly documented inspection planning system

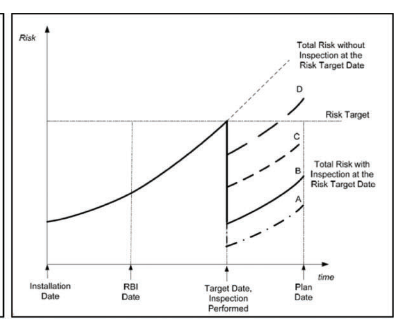

Figure 3.2 Inspection effectiveness category descriptions and relation between risk and inspection effectiveness (API 581, 2008).

Consequence	Score	Likelihood	Score	Contextual Factor	Score
Hazard in control: Defect present but existing barriers prevent progression	1	**Rare:** Theoretically possible but not expected to occur	1	**Minor:** Defect always detected	0.5
Incident without harm: Incident occurs with no harm (system failure)	2	**Unlikely:** Did happen in other industries	2	**Low:** Defect detectable and rarely missed	0.8
Minor harm: Incident occurs and minor harm arises	5	**Possible:** Event does occur in the industry from time to time	3	**Moderate:** Certified inspector should be able to detect this defect	1
Serious harm: Incident results in serious harm	8	**Likely:** Has occurred at least once in the company's history	4	**High:** Defect difficult to detect during visual inspection	1.5
Fatality: Fatalities and possibly catastrophe; recovery systems inadequate	10	**Almost certain:** Annual occurrence in this situation	5	**Extreme:** Defect visually not detectable	2

Risk Score (RS)	Risk Level (RL)	Colour Scheme
$RS < 3$	Minor	Green
$3 \leq RS < 8$	Low	Yellow
$8 \leq RS < 20$	Moderate	Orange
$20 \leq RS < 50$	High	Red
$RS \geq 50$	Extreme	Burgundy

Figure 3.3 Consequence, likelihood and contextual factors description and risk category limits (Aust & Pons, 2021).

Multiple studies have been conducted to study the relationship between these factors and their impact on inspection effectiveness. A study conducted by (Campbell et al., 2020) aimed to establish criteria for evaluating the efficacy of visual inspection in detecting fatigue cracks in steel bridges. The study observed significant variation in inspection results and limited predictive capacity of multiple factors assumed to be associated with inspection performance, including experience, inspection time, and age, among others. FHWA conducted a study to evaluate the reliability of routine and in-depth visual inspections, while also examining the impact of various critical factors that influence the quality of routine and in-depth Inspections (Phares et al., 2001). The study findings show that many in-depth inspections carried out by state bridge inspectors inadequately represent the real condition of the structure. Furthermore, although in-depth inspections are designed to identify specific defects, most of these issues still go undetected.

Non-Destructive Evaluation (NDE) methods may offer more reliability in detecting specific defects that may be undetectable through visual inspection. The reliability of NDE techniques in bridge inspection is highly dependent on several factors, including the expertise and training of inspectors, the material of the components, and the environmental conditions, among others (API 581, 2008). Statistical methods may not be a practical approach for describing uncertainty in NDE due to the limited availability of quantifiable data for the most used NDE methods (Hesse et al., 2015). Furthermore, among various statistical approaches, expert judgment has also been demonstrated as an effective means in assessing NDT uncertainties in pressure vessels (Fong et al., 2018).

Given the limited data, difficulties in correlating parameters with technique reliability, and the proven efficacy of expert judgment in this evaluating the reliability of inspection techniques, expert judgment is a practical approach for evaluating the reliability of inspection techniques. Expert elicitation involves obtaining judgments and opinions from experts with relevant knowledge and experience. This approach can help bridge the gap between the limited available data and the need to make informed decisions in the absence of complete information.

3.3.4.3 Evaluation processes. The process of evaluating inspection effectiveness involves a rigorous approach to identify the type of damage, inspection technique, and component geometry. A hypothetical scenario is presented to experts that illustrate the nature of the damage and the geometry of the structural element under consideration. The identification of the type of defect is crucial to enable experts to determine the most appropriate reliable inspection techniques. The material properties, geometry, and type of the element are necessary elements in this exercise since certain inspection techniques exhibit different reliabilities based on the accessibility and material of the element. Following the identification of the appropriate inspection techniques, experts rank each technique based on its expected effectiveness in detecting the damage mode under scrutiny. Five levels of inspection effectiveness, ranging from highly effective to not effective or component inaccessible, are considered in assessment. Each inspection technique should be ranked in increments of 10% for each level, with the flexibility of using the entire 100% for a specific category as illustrated in Figure 3.4. The sum of the five inspection categories for each technique must be 100%, after which an inspection effectiveness factor is determined for each element and its corresponding damage mechanism. Subsequently, expert elicitation is analyzed to assign an inspection effectiveness factor for each element and damage mode.

3.3.4.4 Inspection effectiveness factor implementation in risk-based assessment. Using a more effective inspection technique enhances the likelihood of early-

Steel Superstructure		Probability of Detection				
Damage Mode	Inspection Technique	HE (%)	ME (%)	FE (%)	LE (%)	NE (%)
Corrosion	Visual Inspection (Hands-on)	80	20	0	0	0
	Visual Inspection (Routine)	70	20	10	0	0
	Ultrasonic Testing (UT)	20	40	40	0	0
	Ultrasonic Thickness Gauge (UT-T)	20	40	40	0	0

Figure 3.4 Example of worksheet used to identify the inspection effectiveness factor for steel superstructure corrosion.

TABLE 3.4
Brief description of the proposed inspection effectiveness categories, approximate probability of detection and their values

Inspection Category	Inspection Effectiveness Category	Description	Approximate Probability of Detection	Inspection Effectiveness Factor
HE	High effective	The defect is always detected	80–100	0.5
ME	Moderately effective	The defect is detected and rarely missed	60–80	0.8
FE	Fairly effective	The defect should be detected by a certified inspector	40–60	1
LE	Less effective	The defect is difficult to be detected through visual inspection	20–40	1.5
NE	Not effective/item is not accessible	The defect cannot be detected through visual inspection/component is not accessible	<20	2

stage defect detection. Identifying the defect at the early stages allows decision-makers to take action to mitigate the risk before the occurrence of undesired consequences. Nevertheless, if the same damage mode is identified after it reaches a severe condition, limited and expensive actions would be taken to address the risk. Therefore, the effectiveness of the inspection technique may decrease the risk by detecting the damage at an early stage, while it may increase the risk when it is detected late. Five levels of inspection effectiveness are proposed, as illustrated in Table 3.4. These levels are categorized such that conducting multiple "ineffective" inspections during a designated inspection interval is approximately equivalent to performing a single inspection of a highly effective inspection.

However, if the component is not accessible or the inspection technique is ineffective in detecting a defect, corrective action should be taken to mitigate the risk. Changing the inspection technique could be an option as well as removing any obstruction to improve accessibility to the element. For example, consider an inspection of a cable connector assembly and the credible damage mechanism is fatigue crack of pin connection. Visual inspection may be a less or ineffective inspection technique for the considered damage mode. As a result, the total risk may increase by 2 or 1.5. As a result, changing the inspection technique is mandatory in this case since the risk is not acceptable, and risk cannot be mitigated by inspection even if multiple inspections are performed. The primary goal of implementing the inspection effectiveness factor is not to conduct multiple inspections in a short period. Instead, the implementing the inspection effectiveness factor provide a guide for users to determine the appropriate inspection interval and the inspection technique or procedure that should be used.

In addition, the implementation of IEF of the highly or moderately effective inspection category (IEF < 1.0) is constrained by the values of CF and OF. IEF can only be applied if the product of the occurrence and consequence factors is less than nine (OF $\times$ CF < 9). Obviously, damage detectability increases as an element reaches advanced stages of deterioration so implementing the IEF may underestimate the risk associated with an element. For instance, the inspection effectiveness factor cannot be applied to steel girders experiencing severe section loss, even though section loss can reliably be detected through visual inspection. In other words, if ultrasonic thickness measurements revealed that corrosion has resulted in 95% section loss of the web, it would not be appropriate to use the IEF to increase the inspection interval simply because a robust and reliable NDT method was used to determine the severity of the damage.

Applying the IEF factor in such a situation may underestimate the risk of the element and may result in component failure before the next inspection. Furthermore, the implementation of IEF is constrained by the CF to mitigate the risk of catastrophic failure in the event of undetected damage. This constraint on the IEF's application by the OF is intended to ensure that the element possesses characteristics that decelerate defect propagation, preventing it from reaching a critical state before the subsequent interval. Concurrently, the

TABLE 3.5
Risk categories and corresponding inspection interval

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

CF's role is to ensure that any undetected defects do not result in catastrophic outcomes. In essence, such restrictions are established to confirm that the IEF factor is not applied in scenarios where both the OF and CF are of moderate or high significance.

3.3.5 Inspection Interval

As the OF, CF, and IEF are evaluated, the optimal inspection interval can be determined using a risk bar,

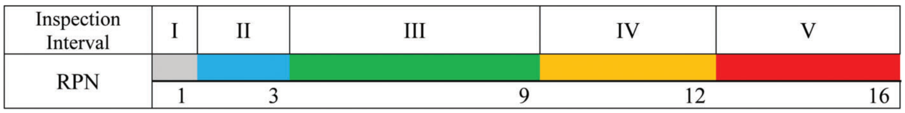

Figure 3.5 Proposed risk bar for determining inspection interval.

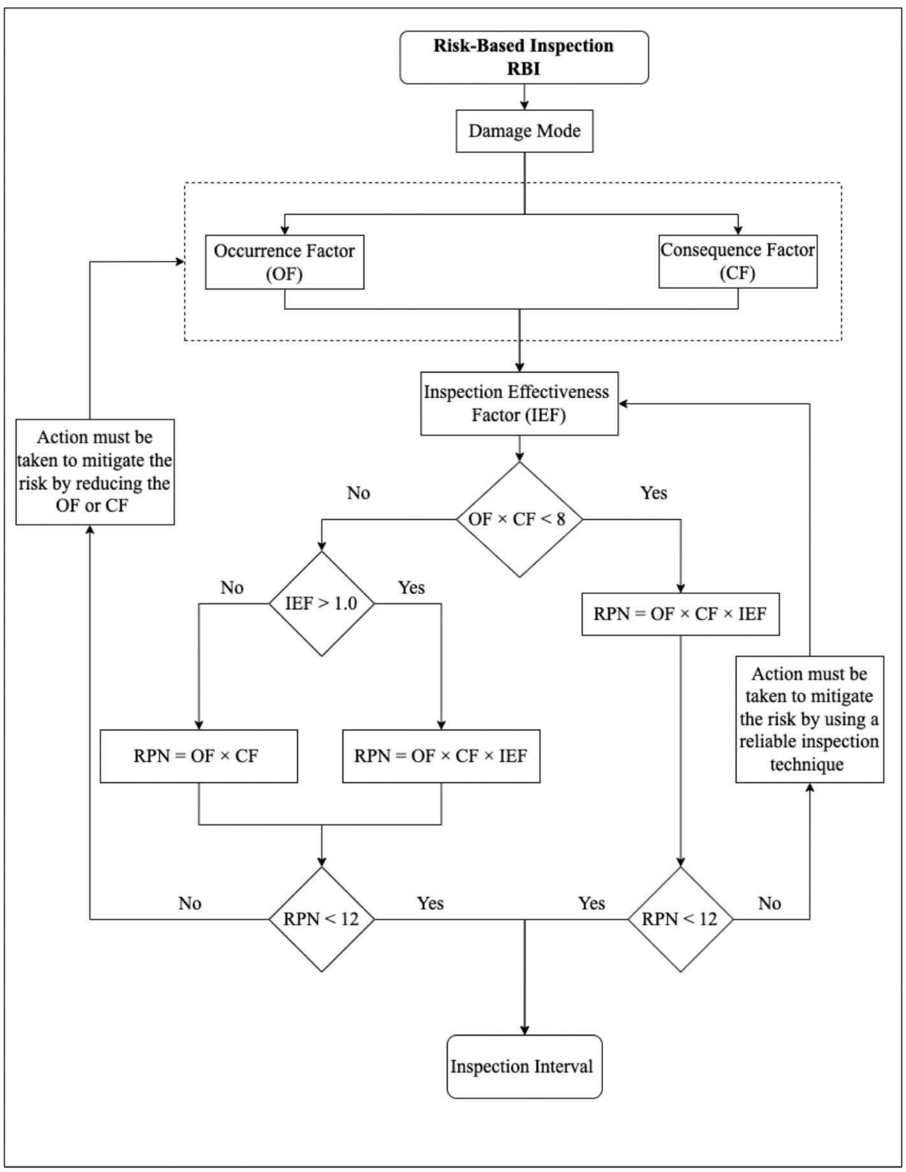

Figure 3.6 Proposed RBI approach flowchart.

as illustrated in Figure 3.5 and in Table 3.5. The risk bar is calibrated to provide an equivalent inspection interval to that of a three-dimensional risk matrix. Furthermore, using the risk bar facilitates the representation of the three dimensions of risk, namely occurrence, consequence, and inspection effectiveness. The risk bar is developed so that damage modes with high probabilities of occurrence, severe consequences, and less effective inspection techniques should be inspected more frequently than damage modes with low occurrence and consequence factors and highly effective inspection techniques.

When the risk value reaches a specified threshold, mere inspection might not adequately address the risk, necessitating further action (ASME, 2007). The acceptable risk level varies among agencies based on the importance of the facility and cost. The acceptable risk limit is typically set at 12 ($RPN < 12$) based on rational experts' judgment and the behavior of bridges in the owner inventory. If the RPN for an element exceeds the risk limit, corrective action should be taken to mitigate the risk. Such actions may include reducing the occurrence, consequence, or utilizing more reliable inspection techniques. Therefore, RPN must be less than 12 to ensure the likelihood and the consequent factor remain low or moderate within the designated inspection interval. Figure 3.6 presents a flowchart that summarize the RBI approach with the implementation of the IEF and risk limits.

4. DEVELOPING RBI CRITERIA FOR PEDESTRIAN BRIDGES

This chapter provides an overview of the risk assessment workshop and presents the development of risk-based criteria for pedestrian bridge components, such as the deck, superstructure, and substructure. The chapter discusses the determination of damage modes, occurrence, and consequence factors for each element. Additionally, the evaluation of the pedestrian bridge's serviceability criteria is discussed.

4.1 Risk Assessment Panel

A Risk Assessment Panel (RAP) is a team of experts and academics assembled by the bridge owner. This team comprises inspection team leaders, structural engineers, material engineers, and inspectors. The primary responsibility of the RAP is to assess the reliability of the bridge using the RBI method under a specific operational environment and within a defined period. Since the RBI method relies on expert elicitation, the careful selection of panel members is crucial. The choice of panel members should be made at the owner level, as bridge performance can vary not only across the United States but also within a state. Such variations in bridge performance can be attributed to differences in the operational environment, design criteria, and maintenance cycle.

The operational environment can significantly influence the rate of deterioration for certain damage modes. For example, in areas with heavy snowfall and extensive salt use, the deck might deteriorate rapidly due to deicing chemicals. Consequently, design criteria, including protective coatings, concrete cover, concrete mixture, and minimum section thickness, may be adjusted to accommodate these environmental impacts. The maintenance cycle is also pivotal, with bridges subjected to regular maintenance often performing better than those with less frequent upkeep. Therefore, RAP members should possess a thorough understanding of the environment, design specifications, and maintenance schedules to assess risk accurately.

4.2 Risk Assessment Workshop

The risk assessment workshop took place at Bowen Laboratory in Purdue West Lafayette, Indiana and was attended by the RAP Panel, consisting of eleven members, including INDOT officials, industrial consultants, and Federal Highway Administration officials as shown in Table 4.1. The workshop was divided into two parts: training and actual assessment. A comprehensive understanding of how to evaluate damage modes, occurrence, consequence, and inspection effectiveness factors is essential for attendees before initiating the assessment. To facilitate this, a real-life example was incorporated to offer a robust understanding of the RBI approach. The assessment of consequence, occurrence, and inspection effectiveness factors for given components was conducted once attendees fully understood the fundamental concepts of these factors. This phase of the workshop was divided into four sessions, each focusing on evaluating a primary factor of the RBI approach. The first and second sessions of the risk assessment centered on evaluating the damage modes and attributes of timber and concrete decks, as well as concrete, steel, and timber superstructures. In the third session, consequences for each component were assessed based on various possible scenarios. The fourth session focused into assessing inspection

TABLE 4.1
Listing of RAP workshop attendees

Name	Affiliation	Title
Participant 1	INDOT	Bridge Inspection Area Engineer
Participant 2	INDOT	Bridge Engineer Supervisor
Participant 3	INDOT	Bridge Area Engineer
Participant 4	FHWA	Division Bridge Engineer
Participant 5	INDOT	Research Section Manager
Participant 6	INDOT	Design Manager
Participant 7	INDOT	Bridge Engineer Designer
Participant 8	INDOT	Load Rating Engineer
Participant 9	INDOT	Bridge Asset Engineer
Participant 10	Collins Engineering	Project Manager/Engineer-Diver
Participant 11	Principal WJE	Senior Project Manager
Participant 12	Ciorba Group	CEO and President

effectiveness factors and the degerming the appropriate inspection interval.

4.3 Risk Assessment Process

The process of expert judgment elicitation is a cornerstone of the RBI approach. This process typically involved four main steps: presenting the problem statement, soliciting expert opinions, comparing results, and documenting the consensus. The problem statement was meticulously crafted to include all necessary details, thereby enabling panel members to provide accurate assessments. Specifically, it included a detailed set of information on the bridge's design details, operational environment, geographic location, and traffic volume. To maintain objectivity and avoid any potential biases in the assessment, special attention was paid to avoiding terms that could skew experts' decisions. For instance, the term *"non-redundant member"* was typically avoided, as some practitioners might argue that such elements are more susceptible to fatigue cracks. However, this is a misconception: the likelihood of crack development is not determined by whether an element is redundant or non-redundant. Instead, it is influenced by other factors such as fatigue detail, ADT (average daily traffic), and fabrication year. The inclusion of such a term could thus introduce an unwarranted bias into the assessment.

4.3.1 Screening Criteria

The screening criteria are necessary to consider validating the applicability of the RBI methodology. These criteria typically highlight cases with a very high likelihood of failure or significant uncertainty. Furthermore, bridges with uncommon deterioration mechanisms, compared to others in the same group, are also subject to these screening criteria. The following list presents the specific screening criteria, emphasizing situations where experts recommend not applying the RBI approach. As a result, bridges with these criteria may require additional in-depth analysis to identify their inspection needs.

- Bridge components with a condition rating of 4 or less.
- Bridges where fire damage has occurred and either no inspection has been carried out 12 months after the fire, or a 12-month post-fire inspection confirms that the bridge's functionality is inadequate.
- Components experiencing fractures due to water freezing inside the section (commonly observed in tubular sections).
- Significant levels of active corrosion and section loss that may impact the component's capacity.

4.3.2 Damage Mode

The assessment of damage modes typically begins by posing the question "What can go wrong?" However, to mitigate the potential for bias and to engage participants more effectively, a real case bridge is presented to the panel members with the following prompt:

> *"You are told a pedestrian steel bridge has a superstructure rating of 3; what damage is most likely to have resulted in this rating, based on your experience and knowledge?"*

Each participant is given a blank sheet to record their responses independently. The responses are then shared and discussed among the experts until a consensus is reached. If a consensus cannot be reached for a particular damage mode of a given element, additional information may be required, or the process may need to be repeated. Table 4.2 provides a summary of the most common damage modes for deck and superstructures of a pedestrian bridge.

TABLE 4.2
Common damage mode for deck and superstructure for different material

Element	Material	Damage Mode
Deck	Concrete	Spalling
		Corrosion
	Timber	Rotting
		Connector deterioration
Superstructure	Steel	Corrosion
	Concrete	Strands/rebar corrosion
	Timber	Rotting
		Connector deterioration

4.3.2.1 Concrete deck. According to RAP members, spalling and cracking are the most common damage modes for a concrete deck. It is important to note that only bottom spalling was considered in the assessment, while the RAP agreed that top spalling would be evaluated in the serviceability concerns checklist. However, if severe spalling occurs on the top surface of the deck and poses a risk of harm or injury to bridge users, appropriate action should be taken to mitigate the risk.

4.3.2.2 Timber deck. There was unanimous agreement among the RAP members that rotting, and connector deterioration are the most credible damage modes for timber decks. Connector deterioration refers to the connectors that link the timber deck to the superstructure, such as bolts, nails, screws, and stressed rods. However, fire damage is not considered in the assessment since it is an unpredictable incident and requires separate evaluation that may involve additional assessments beyond RBI. Table 4.4 summarizes the attributes of decay damage mode for the timber deck.

4.3.2.3 Steel superstructure. For steel pedestrian bridges, consensus was reached that corrosion is the primary damage mode. Some RAP members mentioned that overloading might be a concern for certain pedestrian bridges, particularly those lacking bollards at the ends. However, this damage mode was not considered in the assessment, as it does not occur periodically. Additionally, the RAP members reached a consensus that fatigue is rare in pedestrian bridges due to their relatively low fatigue load. In addition, the panel agreed that fracture of tubular sections was not a major concern, as it is an uncommon defect; only one case was reported among similar prefabricated pedestrian bridges in the owner's inventory.

4.3.2.4 Concrete superstructure. The RAP members concurred that corrosion of rebar strands is the primary damage mode in the concrete superstructure of pedestrian bridges. Flexural/shear cracking was not considered in the assessment since pedestrian bridges typically carry lighter loads than highway bridges. However, in some cases, vehicular bridges may be converted for pedestrian use. Thus, the panel pointed out that if the bridge in question was originally designed for vehicular traffic and experiences flexural/shear cracking, this damage mode should be assessed. Additionally, an analysis should be conducted to ensure that the bridge can safely carry pedestrian traffic in its current condition.

4.3.2.5 Timber superstructure. Rotting and connector deterioration were also considered as significant damage modes for timber superstructures. Connectors, such as screws, nails, bolts, and stressed rods, link the various components of the superstructure. The RAP members agreed that splits and checks are characteristics of rotting damage. Bridges that have been subjected to fire or impact are excluded from the assessment as they require additional analysis to determine their load-carrying capacity.

4.3.2.6 Cable and connection assembly. The RAP members unanimously agreed that the main damage modes for cable and connector assemblies are corrosion and fatigue. For clarity, it is essential to note that the term "cable" pertains to the free length of the cable, while "cable connection assembly" includes all associated connection components, such as sockets, pins, welding/bolts, and connection web plates.

4.3.3 Attribute Assessment

Once the credible damage modes for the deck and superstructure are identified, the occurrence factor for each mode is evaluated. The occurrence factor is evaluated based on associated attributes that contribute to the damage mode development, and these attributes are subsequently ranked by their level of significance. To determine the attributes, RAP members are asked the following questions:

> "*In your expert opinion, how long do you estimate it will take for significant spalling to occur? What information do you need to make that estimate?*"

These questions were crafted to help RAP members determine the relevant attributes for each damage mode. Each member first listed the attributes associated with the given mode individually, after which they were asked to share their input with the panel. Subsequently, the panel collectively ranked these attributes based on their significance, using four categorizations: high, moderate, low, and remote. While many attributes have been well-established in the RBI methodology for highway bridges, RAP members were prompted to discuss potential differences when these attributes are applied to pedestrian bridges. Nevertheless, some attributes were considered to be unique to pedestrian bridges, which will be discussed later. The following is a brief discussion on the key attributes highlighted by RAP members; reasons and assessment procedures for each attribute are discussed in detail in Appendix B.

4.3.3.1 Concrete deck. The RAP consensus was that the attributes of spalling and corrosion that are used in highway bridges could also be applied to pedestrian bridges with only minor adjustments in some attribute limits, such as concrete cover and slab thickness as illustrated in Table 4.3. For instance, after careful consideration, the appropriate concrete cover and slab thickness for a pedestrian bridge deck were determined to be 1.5 inches and 6 inches, respectively. Consequently, the probability of corrosion increases significantly when the concrete cover and slab thickness are less than these values. Additionally, some members of the RAP noted that precast concrete decks are frequently used in pedestrian bridges and are more robust than cast-in-place concrete decks due to the curing process. As a result, the rate of corrosion development in a precast concrete deck is comparatively slow when compared to a cast-in-place deck.

4.3.3.2 Timber deck. The RAP members emphasized that the durability of a timber deck mainly depends on the preservation treatment and the type of timber utilized. In certain circumstances, composite fiber timber decks, particularly prefabricated bridges, are used in pedestrian bridges, and they have shown adequate resistance to decay. Excessive variation in moisture content, such as wet and dry cycles, is the primary cause of decay development. Therefore, the RAP members concluded that enclosing pedestrian bridges can regulate moisture content variation in a timber deck, significantly delaying decay development. Regarding connector deterioration, the consensus was that the quality of timber boards, the type of connector, and the material used all contribute to connector deterioration.

4.3.3.3 Steel superstructure. The RAP panel conducted a thorough analysis of the attributes of the steel superstructure and their respective significance in the development of section loss, as summarized in Table 4.5. While some of these attributes were used in RBI for highway bridges are also applicable to pedestrian

TABLE 4.3
Attributes of spalling damage mode for concrete deck

Similar Attribute in NCHRP 782	Attribute	High	Moderate	Low	Remote
D.4	Poor drainage and ponding		Ponding/ineffective drainage		No problem noted
D.10	Deck overlay		Has overlay		Does not have overly
D.8	Concrete mix design		Poor mix		Good mix
D.11	Concrete cover		<1.5		>1.5
D.12	Reinforcement type		Uncoated carbon steel		Has protective coating
D.20	Deck thickness		<6 in.	6 in. +	
L.5	Exposure environment	North districts	Central districts	Southern districts	
C.5	Maintenance cycle	No routine maintenance	Some limited maintenance		Regular maintenance
C.1	Current condition	5 or below		6	7<
D.20	Type of deck		Cast-in-place		Precast
C.10	Presence of delamination	Significant (>20% by area)	Moderate (5–20 by area)	Minor (<5% by area)	No delamination

TABLE 4.4
Attributes of decay damage mode for timber deck

Attribute Code in NCHRP 782	Attribute	High	Moderate	Low	Remote
	Type of treatment	No/ineffective		Paint	Effective treatment
	Exposure environment	Severe/marine	Moderate/industrial	Benign	
	Overlay		No/ineffective overlay	Overlay without membrane	Overlay with membrane
	Covered bridge		Not/ineffective covered	Fairly covered	Effectively covered
	Existing splits and checks	Significant (>20% by area)	Moderate (5%–20% by area)	Minor (<5% by area)	No splits and checks
	Type of timber		All others		Inherit decay resistance wood species

bridges, the RAP members identified unique attributes of section loss for pedestrian bridges. Specifically, the utilization of a timber deck in a steel superstructure was found to expedite the development of corrosion. This is because of the immediate flow of rainwater onto the superstructure through the gaps between the timber boards, coupled with the inherent propensity of timber decks to retain moisture for prolonged periods and feed it to the superstructure underneath. Another crucial attribute pointed out by the RAP panel was the presence of corrosion-susceptible details in pedestrian bridges. RAP members described these details as any detail that contains a concavity, which can retain water for extended periods, ultimately leading to corrosion development.

Enclosed pedestrian bridges were recognized as a favorable attribute because they protect superstructure elements from natural environmental conditions, thereby delaying corrosion development. The application of de-icing chemicals, another well-known attribute associated with corrosion, is notably used on pedestrian bridges in urban areas. RAP members highlighted that pedestrian bridges in urban settings might be particularly vulnerable to extensive de-icing. Furthermore, overspray from de-icing chemicals, especially when a bridge is near a parallel or overpassing vehicular bridge, can also lead to corrosion.

4.3.3.4 Concrete superstructure. The panel reached a consensus that the attributes utilized for strand corrosion in RBI for highway bridges can be applied to pedestrian bridges with certain modifications as shown in Table 4.6. Some RAP members suggested that the condition of expansion joints be incorporated as an attribute of strand corrosion. In addition, the RAP panel determined that the ideal clear cover for the concrete superstructure of a pedestrian bridge should exceed 1.5 inches.

4.3.3.5 Timber superstructure. The RAP panel also considered the same attributes used for a timber deck when assessing a timber superstructure, as illustrated in Table 4.7. The panel highlighted that decay development is heavily influenced by the type of preservation and wood species. For instance, pressure preservation, which treats both the outer surface and core, was considered to be an effective treatment method. Furthermore, some wood species are naturally resistant to decay

TABLE 4.5
Attributes of corrosion damage mode for steel superstructure

Similar Attribute in NCHRP 782	Attribute	High	Moderate	Low	Remote
D.13	Built-up member		Yes		No
D.15	Constructed of weathering steel		Yes		No
L.3	Exposure environment	Severe/marine	Moderate	Benign	
L.5	Rate of de-icing chemical application	High	Moderate	Low	None
L.6	Steel girders are subjected to overspray	Severe overspray exposure	Moderate overspray exposure	Low overspray exposure	Not over or parallel to a roadway
C.4	Joint condition	Significant amount of leakage at joints	Joints have moderate leakage or debris filled	Joints are present but not leaking	Bridge is jointless
C.7	Quality of deck drainage system	Drains directly onto superstructure	Drainage issues resulting in moderate drainage onto superstructure		Adequate drainage
C.17	Coating condition	Coating system in very poor condition, 3%< rusting	Coating system in poor condition, 1%–3%< rusting	Coating is in fair to good condition, effective for corrosion protection	
C.21	Presence of active corrosion	Significant amount of active corrosion present	Moderate amount of active corrosion present	Minor amount of active corrosion present	No active corrosion present
C.5	Maintenance cycle		No routine maintenance	Some limited maintenance	Regular maintenance
	Deck type	Timber	Precast with open joint		Cast-in-place
	Covered bridge	Not covered		Fairly covered	Effectively covered
	Details and splices	Bridge has bad details		Bridge has good details	

and do not necessitate preservation. Regarding connector deterioration, the RAP members concurred that bolts and stressed rod connectors are superior to nails in preventing such deterioration.

4.3.3.6 Cable and connection assembly. For the cable breakage damage mode, RAP members pointed out that cables are more susceptible to breakage if they are not protected from aerostatic vibration effect. Table 4.8 and Table 4.9 summarizes the attributes of the wire breakage and the connection assembly fatigue crack damage modes, respectively. Cross cables, commonly known as secondary cables, can help mitigate the effects of these vibrations (NCHRP, 2005). Unquestionably, a present broken wire is a significant indicator of the cable breakage damage mode. Some RAP members suggested that a special assessment might be necessary if over 10% of the wires in a cable are broken. Thus, this attribute might be screened; in other words, the RBI approach might not be applicable. The RAP panel highlighted that out-of-plane stress on the connection web, resulting from wind loads, is a primary contributor to fatigue cracks in cable connection assemblies.

Regarding cable corrosion, the panel concurred that some of the superstructure section loss attributes can be apply to both cables and cable connection assemblies. Furthermore, RAP members remarked that, in particular instances, covering the cable connection assembly can slow down the development of corrosion, mirroring the benefits of an enclosed bridge on superstructure corrosion attributes.

4.3.4 Consequence Assessment

In order to assess the consequence factor for each component, separate evaluations for each element were conducted, assuming the failure of the element being analyzed due to a particular damage mode. Although failure is an undesirable outcome, utilizing a failure scenario is essential for evaluating consequences. Each component was evaluated through a given failure scenario, and the RAP members were tasked with evaluating the potential outcomes, assuming the case of a component failure.

4.3.4.1 Deck. For the consequence assessment of a concrete deck, RAP members have suggested that the same consequence factor used for vehicular bridges can be applied to pedestrian bridges as well. In contrast, evaluating the consequence factor for a timber deck

TABLE 4.6
Attributes of spalling damage mode for concrete superstructure

Similar Attribute in NCHRP 782	Attribute	High	Moderate	Low	Remote
D.7	Application of protective systems		Never applied, poor functioning	Penetrating sealer, crack sealer, limited effectiveness	Periodically applied, effective
C.1	Current condition rating	Current rating is 5 or below		Current rating is 6	Current rating is +7
C.6	Previously impacted	Bridge has been previously impacted			Bridge has not been previously impacted
C.8	Corrosion-induced cracking	Significant corrosion-induced cracking	Moderate corrosion-induced cracking		Minor corrosion-induced cracking
C.9	General cracking	Widespread or severe cracking	Moderate cracking present		Minor or no cracking
C.10	Delamination	Significant delamination (>20% by area)	Moderate (5%–20% by area) delamination	Minor, localized (<5% by area) delamination	No delamination present
C.11	Presence of repaired areas	Significant number of repaired areas	Moderate number of repaired areas	Minor number of repaired areas	No repaired areas
C.12	Presence of spalling	Significant spalling (>10% by area with exposed rebar or strands)	Moderate spalling (greater than 1 inch deep or 6 inches diameter exposed reinforcement)	Minor spalling (less than 1 inch deep or 6 inches in diameter	No spalling
C.4	Joint condition	Open	Leaky	New	Jointless
C.13	Efflorescence/staining	Severe efflorescence without rust staining	Moderate efflorescence without rust staining	Minor efflorescence	No efflorescence

TABLE 4.7
Attributes of connector deterioration damage mode for timber superstructure

Similar Attribute in NCHRP 782	Attribute	High	Moderate	Low	Remote
L.5	Rate of de-icing chemical application	High	Moderate	Low	None
	Timber deck treatment	No/ineffective treatment	Paint	Effective treatment	
	Type of connector		Nails/screw	Bolt	
	Existing splits and checks	Significant (>20% by area) splits and checks	Moderate (5%–20% by area) splits and checks	Minor, localized (<5% by area) splits and checks	No splits and checks present
	Connector material type		All others		Galvanized/ corrosion resistance

TABLE 4.8
Attributes of wire breakage damage mode

Attribute Code in NCHRP 782	Attribute	High	Moderate	Low	Remote
	Aeroelastic vibration effect		Not prevented	Moderately prevented	Effectively prevented
	Existing broken wires	Significant (widespread on multiple wire)	Moderate (moderate broken wires)	Minor, (few wires broken)	No broken wires

TABLE 4.9
Connection assembly fatigue damage

Attribute Code in NCHRP 782	Attribute	High	Moderate	Low	Remote
	Aeroelastic vibration effect	—	Not prevented	Moderately prevented	Effectively prevented
	Existing fatigue crack	Yes			No
	Presence of repaired crack	Yes			No

TABLE 4.10
Consequence of timber deck, decay damage mode, and RAP responses

	Low	Moderate	High	Severe
Participant 1	0	10	60	30
Participant 2	0	20	70	10
Participant 3	0	0	20	80
Participant 4	0	20	60	20
Participant 5	10	30	60	0
Participant 6	0	0	30	70
Participant 7	0	30	70	0
Participant 8	0	0	30	70
Average	1.25	13.75	50	35
The consequence of this damage mode is high				

Note: The assumed scenario is 20% of the timber deck in severe

TABLE 4.11
Consequence of timber truss superstructure, decay damage mode, and RAP responses

	Low	Moderate	High	Severe
Participant 1	0	0	0	100
Participant 2[a]	0	0	0	0
Participant 3	0	0	10	90
Participant 4	0	0	20	80
Participant 5	0	0	0	100
Participant 6	0	0	30	70
Participant 7	0	0	0	100
Participant 8	0	0	30	70
Average	0	0	12.86	86.14
The consequence for this damage mode is severe				

[a]The assumed consequence is that the main member lost 100% of its load carrying capacity.

TABLE 4.12
Consequence of two timber beams superstructure, decay damage mode, and RAP responses

	Low	Moderate	High	Severe
Participant 1	0	0	50	50
Participant 2	0	0	10	90
Participant 3	0	10	90	0
Participant 4	0	0	20	80
Participant 5	0	0	0	100
Participant 6	0	0	0	100
Participant 7	0	0	10	90
Participant 8	0	0	10	90
Average	0	1.25	23.75	75
The consequence for this damage mode is severe				

Note: The assumed consequence is that the main member lost 100% of its load carrying capacity.

TABLE 4.13
Consequence of cable/connection assembly, fatigue crack damage mode, RAP responses

	Low	Moderate	High	Severe
Participant 1	20	30	30	20
Participant 2[a]	0	0	0	0
Participant 3	50	50	0	0
Participant 4	30	40	30	0
Participant 5	50	50	0	0
Participant 6	0	0	30	70
Participant 7	20	50	30	0
Participant 8	0	30	50	20
Average	24.3	35.7	24.3	15.7
The consequence for this damage mode is moderate				

[a]The assumed consequence is that the main member lost 100% of its load carrying capacity.

requires consideration of several factors, such as the existence of a secondary deck, the quantity and pattern of damaged timber boards, and other unique characteristics of pedestrian bridges. The secondary deck, which is commonly used in old timber bridges to act as a wearing surface, provides an alternative load-carrying path in case of top timber board failure as shown in Figure 4.1. Another important consideration when evaluating the consequence factor for timber decks is the width of the timber board. Specifically, the failure of boards wider than 6 inches will have a greater consequence than those under 6 inches wide, due to the larger opening from a failed board posing a heightened risk to pedestrians. Obviously, the type of service provided beneath the bridge plays a crucial role in determining the consequence factor. Debris from a damaged timber deck can disrupt traffic below, representing a safety concern for the public. Table 4.10 summarizes the RAP responses of the CF for the timber deck damage mode.

4.3.4.2 Superstructure, cable, and connection assembly. The consequence factor for superstructure is evaluated by assuming that one of the primary members had lost its entire load-bearing capacity. Table 4.11 through Table 4.14 summarizes the RAP responses of the CF for different superstructure systems. Subsequently, the panel members were prompted to share their response and the considerations behind their conclusions. In consensus, it was reached that the consequence factor for superstructure elements could be assessed in a manner similar to that of vehicular bridges with consideration. For example, given that truss and twin-girder bridges are considered as non-redundant systems, the RAP members concurred that the consequence factor could span from severe to high. Nonetheless, it is possible to reduce the consequence factor if structural analysis and documented engin eering experience show that the failure of the component being evaluated does not result in the failure of the entire bridge. Furthermore, there was unanimous agreement among the RAP members that the consequence of the cable or the cable connection assembly would be moderate. This assessment hinges on the design document that states cables can be replaced individually and that other cables are designed to carry the load if one of the cables fails, factoring in the dynamic impact from the pretension inherent in the affected cable. The panel also pointed out that pedestrian volume is an important factor, as the consequence becomes higher when the pedestrian volume is high.

4.3.5 Substructure Assessment

Given the similarities between pedestrian and vehicular substructures, it was unanimously agreed that the same consequence assessment used for highway RBI bridges can also be applied to pedestrian bridges. For concrete substructures, which are commonly used in pedestrian bridges, the corrosion damage mode and the corresponding attributes and consequences used in vehicular bridges can similarly be considered in the pedestrian bridge evaluation process. However, for masonry substructures, it is necessary to exercise engineering judgment. Factors such as the current condition, channel condition, service beneath and on the bridge, and other relevant considerations need to be taken into account to appropriately assess the risk.

TABLE 4.14
Consequence of muti-beams timber superstructure, decay damage mode

	Low	Moderate	High	Severe
Participant 1	10	30	50	10
Participant 2[a]	0	0	0	0
Participant 3	0	50	50	0
Participant 4	0	20	40	40
Participant 5	0	30	60	10
Participant 6	20	80	0	0
Participant 7	0	30	70	0
Participant 8	0	20	40	40
Average	4.3	37.1	44.3	14.3
The consequence for this damage mode is high				

[a]The assumed consequence is that the main member lost 100% of its load carrying capacity.

4.4 Safety and Serviceability Criteria

Direct interaction between pedestrians and a bridge's structure necessitates unique safety and serviceability criteria. While minor issues, such as a missing railing elements or skid surface, might be inconsequential on a highway bridge, they may jeopardize the safety of pedestrians. Therefore, it is essential to consider these criteria to ensure the safety and serviceability of pedestrian bridges. These criteria are classified into two categories: walkway evaluations and miscellaneous evaluations. A checkbox list is utilized to assess the serviceability criteria, and an option provided for reporting any safety concerns.

It is important to note that these criteria are not part of the risk assessment. In other words, these criteria do not affect the inspection interval. The determination of the inspection interval is primarily based on structural criteria, as discussed in previous chapters. However, corrective action must be taken if any of these criteria compromise the safety of pedestrians/cyclists or the service below the bridge.

4.4.1.1 Walkway criteria evaluation. Pedestrian bridge surfaces are among the most crucial elements to evaluate for safety and serviceability. Unlike highway bridges, inspectors should be aware that a small defect on the surface of a pedestrian bridge may result in injuries to pedestrians or cyclists. It is essential to note that the majority of pedestrian bridge liability claims are statistically slip and fall claims. Safety and serviceability criteria for concrete and timber walkway surfaces are

Figure 4.1 Example of timber deck with two layers planks, diagonal and longitudinal planks.

TABLE 4.15
Checklist of the proposed evaluation criteria for pedestrian bridge walkways

Criteria	Assessment	Note
Water Ponding	☐ No sign of ponding ☐ Moderate ponding ☐ Significant ponding	
Skid Surfaces	☐ Nonskid surface ☐ Skid surface	
Uneven Surface	☐ Even surface ☐ Uneven/wavy surface	
Transversal Gaps Between Elements (Applied to Timber and Precast Concrete Panels)	☐ No gaps ☐ Gap < ½ inch ☐ Gap > ½ inch	
Longitudinal Gaps Between Elements (Applied to Timber Deck)	☐ No gaps ☐ Gap < ½ inch ☐ Gap > ½ inch	
Projected Element (Applied to Timber Deck)	☐ No projected nails/bolts ☐ Projected nails/bolts	

discussed, as they are most commonly used in pedestrian bridges. Table 4.15 presents a checklist of the proposed evaluation criteria for pedestrian bridge walkways.

4.4.1.1.1 Concrete deck safety serviceability evaluation. Concrete deck assessment involves several criteria to ensure a safe and functional walking surface for pedestrian and cyclists. Water ponding is often observed due to the lack of an effective drainage system on these bridges, particularly those with a cast-in-place concrete deck. While precast concrete panels offer adequate drainage through transverse gaps between panels, these gaps should not exceed ½ inch to prevent safety issues. Another serviceability criterion for concrete decks is to maintain a non-skid deck surface. The finish of the concrete should be sufficiently rough to mitigate slip-related injuries. It is important to mention that evaluations of the non-skid surface are conducted under standard conditions. Consequently, skid issues caused by external factors like snow or black ice should be addressed at the maintenance level.

4.4.1.1.2 Timber deck serviceability evaluation. Using a timber deck in pedestrian bridges is common due to its lightweight nature and ease of installation. The orientation of timber boards can be configured in either the transverse or longitudinal direction. Regardless of the direction, it is crucial that the spacing between individual timber elements is limited to a maximum of ½ inch to ensure user safety. Additionally, special attention should be given to any projecting elements, such as nails and bolts, to prevent injuries and ensure a proper connection between the timber planks and the superstructure.

4.4.1.2 Miscellaneous criteria evaluation. Pedestrian bridges sometimes feature components not typically present in highway bridges. Table 4.16 presents a checklist for evaluating the unique safety and serviceability criteria of pedestrian bridges. Assessing these components is necessary, both structurally and operationally. For instance, the adequacy of lighting anchorage must be examined in addition to its functionality to ensure proper illumination. Given that bridge inspections usually take place during daylight when lighting systems are off, inspectors need to arrange with owners to activate the lights and spot any malfunctioning units. Some pedestrian bridges come equipped with emergency call devices, necessitating checks to confirm their functionality. Additionally, some pedestrian bridges contain cables and utilities, requiring inspections to ensure these components remain concealed, especially in lit bridge, to prevent electrical shocks to pedestrians and cyclists.

The evaluation of railings and fences is important to ensure the safety of pedestrians and cyclists on bridges. The absence or ineffectiveness of these elements may compromise the safety of individuals using the bridge. Consequently, it is necessary to ensure that gaps between railing and fence elements do not exceed 10 inches for safety purposes. In addition, the anchorage of these elements should be evaluated to ensure they maintain the capacity to withstand lateral loads equivalent to the average human weight. Handrails should be rounded and smooth, devoid of any projecting or sharp features. Bollards on pedestrian bridges are also important, especially when the clear width of the deck exceeds 10 ft. It has been observed that pedestrian bridges lacking bollards at their ends are susceptible to collision damage, as some vehicles attempt to cross. To prevent such incidents and potential overloading, bollards at the bridge ends are essential to prevent vehicular traffic.

Some pedestrian bridges feature a decorative façade that needs to be evaluated. Exterior decorative panels must be assessed to ensure that they are in sound condition and maintain adequate anchorage to prevent such elements from falling onto the traffic below the bridge. Evaluation should also extend to interior

TABLE 4.16
Checklist of the proposed evaluation of unique serviceability and safety criteria for pedestrian bridges

Criteria	Assessment	Note
Lighting	☐ Not present ☐ Effective ☐ Not effective	
Lighting Anchorage	☐ Not present ☐ Effective anchorage ☐ Weak anchorage, no safety concern ☐ Weak anchorage, with safety concern	
Cables and Utilities	☐ Not present ☐ Covered cables ☐ Exposure cables	
Emergency Call Device	☐ Not present ☐ Effective ☐ Not effective	
Railing/Fence	☐ Not present ☐ No missing element ☐ Missing element opening $<$ 10 inch ☐ Missing element opening $>$ 10 inch	
Railing Handrail/Edges Condition	☐ Not present ☐ Sharp edge ☐ Rounded and smooth edges	
Railing/Fence Anchorage	☐ Not present ☐ Effective anchorage ☐ Weak anchorage, no safety concern ☐ Weak anchorage, with safety concern	
Bollards	☐ Not present ☐ Effective ☐ Not effective	
Condition of *Interior* Decorative Panel/Enclosure (Panel/Enclosure Assessment)	☐ Not apply ☐ No damaged panels ☐ Damaged panels, no safety concern ☐ Damaged panels, with safety concern	
Condition of *Exterior* Decorative Panel/Enclosure (Panel/Enclosure Assessment)	☐ Not apply ☐ No damaged panels ☐ Damaged panels, no safety concern ☐ Damaged panels, with safety concern	
Exterior Decorative Elements (Anchorage Assessment)	☐ Not present ☐ Effective anchorage ☐ Weak anchorage, no safety concern ☐ Weak anchorage, with safety concern	
Interior Decorative Elements (Anchorage Assessment)	☐ Not present ☐ Effective anchorage ☐ Weak anchorage, no safety concern ☐ Weak anchorage, with safety concern	
Signs and Attachments Anchorage	☐ Not present ☐ Effective anchorage ☐ Weak anchorage, no safety concern ☐ Weak anchorage, with safety concern	
Drainage System	☐ Not present ☐ Effective ☐ Not effective	
Vibration	☐ No excessive vibration observed ☐ Excessive vibration observed	

decorative panels to ensure they do not pose a risk of injury to pedestrians from falling pieces or sharp edges. Finally, vibration is a common issue in some pedestrian bridges due to their lightweight construction and the resonance between the bridge's fundamental lateral frequency and pedestrian-induced excitation. Inspectors should thoroughly evaluate the bridge's vibration, and if excessive vibrations are observed, appropriate actions should be taken to mitigate the risk.

5. BACK CASTING

The back-casting approach has been employed for various types of bridges to evaluate the effectiveness of the RBI methodology (Washer et al., 2016). Back-casting involves applying the RBI approach to historical bridge data and assessing whether the bridge's conditions would have significantly changed if this approach had been implemented in the past. For example, if a RBI approach using historical inspection data of a bridge in 2006 suggests an inspection interval of 6 years, the inspection reports from 2006 to 2012 can be used to verify that the bridge component being considered has not significantly deteriorated since 2006. By adopting this approach, the efficiency of the RBI methodology can be evaluated, and the reliability of the inspection results can be ensured.

The back casting concept is applied to a sample of fourteen bridges, each varying in their construction materials and structural systems, to demonstrate the practicality of the RBI approach. A comprehensive list and brief summaries of the back casting applications for these bridges are presented in Appendix E. For illustration purposes, the back-casting concept is applied in detail to two bridges. The first bridge, a steel truss structure, was built in 1894 and is located in the southern district. The second bridge, a cable-stayed construction, was built in 2010 in Fort Wayne. Although the back-casting concept should be consistently applied to each subsequent inspection report to update the condition attribute and recognize any changes in the inspection frequency, for the sake of clarity and brevity, the back-casting presented for these two bridges is limited to the initial inspection year. However, back-casting evaluations for subsequent inspection years for these two bridges are also checked and summarized in Table 5.6 and Table 5.17.

5.1 Back-Casting of a Steel Bridge

5.1.1 Overview

This bridge, constructed in 1894, was used for vehicular traffic until 1997. Following some repairs, it was then restricted to pedestrian traffic. The bridge is a steel truss with a timber deck and passes over a non-navigable waterway as shown in Figure 5.1. According to the latest inspection report from 2020, the condition ratings were 7 for the deck, and 6 for both the superstructure and the substructure. Despite these ratings remaining unchanged since 2011, the bridge continues to undergo inspections every 24 months.

5.1.2 Occurrence Factor

Decay is the primary mode of damage for the timber deck, while corrosion is the main concern for the superstructure. It is important to note that fatigue is not typically an issue in pedestrian bridges, as they are not subjected to cyclic loading. The rate of decay development in the timber deck is heavily influenced by the species of timber used and the specific treatment

Figure 5.1 View of the steel bridge.

applied. Considering the design, load, and condition attributes of the timber deck, the occurrence factor is calculated as moderate with an OF value of 2.48 as shown in Table 5.1.

For superstructure corrosion, the key attributes are the type of deck, type of protection system, and application of de-icing chemicals. Notably, the timber deck can contribute to an increase in superstructure corrosion because rainwater drains directly into the superstructure. Additionally, the timber deck retains water for extended periods, slowly feeding it to the superstructure. It is well known that bridges with effective protective systems can delay the development of corrosion. For this bridge, paint is considered to be effective as it appears in adequate condition. De-icing chemicals may be introduced through direct application to the deck or via overspray. However, since this bridge is located in a rural area and does not pass over or run parallel to a road, it is neither exposed to direct de-icing chemicals nor to overspray. Taking into account the design, load, and condition attributes, the occurrence factor of the superstructure corrosion damage mode is evaluated to be moderate with an OF value of 2.29 as shown in Table 5.2.

The occurrence factor of rebar corrosion in the substructure is determined based on design, load, and condition attributes. These attributes and their respective scores are outlined in Table 5.3 and Table 5.4. The presence of the timber deck is found to cause immediate leakage into the substructure. Similarly, the drainage system is seen as contributing to substantial leakage into both the superstructure and the substructure. Consequently, the occurrence factor for rebar corrosion in the substructure has been calculated to be moderate, with an OF value of 2.03.

5.1.3 Consequence Factor

A possible consequence of a failure scenario related to a timber deck is the failure of one of its timber boards. In such a scenario, there would be a significant impact on the safety of pedestrians and cyclists, hence, the immediate consequences are considered high. On the other hand, considering the service under the bridge and the veridical clearance, the overall immediate consequence is moderate. However, when considering the short-term consequences, they are evaluated low for two reasons: (1) there is no vehicular traffic passing underneath the bridge that might be disturbed by maintenance works, and (2) the daily volume of pedestrian traffic is relatively low, reducing the risk of injury. Based on these considerations, the overall consequence factor, which is a measure of the potential negative impacts of such a failure, is rated as moderate for the timber deck.

The evaluation of the consequence factor for the superstructure element is established by assuming that one of the bottom chords lost its entire load bearing capacity. Since the bottom chord is classified as a non-redundant element according to AASHTO specifications (AASHTO, 2018), the immediate consequence of such a failure is evaluated to be severe. However, given the bridge's location over a non-navigable waterway, its low vertical clearance, and minimal pedestrian traffic, the overall consequence factor for the superstructure is classified as high. For substructure, severe spalling is the assumed scenario for the consequence assessment. Such a scenario is unlikely to significantly impact the bridge's overall load-carrying capacity, resulting in a relatively low consequence factor. Thus, the overall consequence factor for the substructure is considered to be moderate.

5.1.4 Inspection Effectiveness Factor

A certified inspector is expected to visually detect and quantify the decay damage mode developed in the timber deck. Superstructure corrosion can visually be detected since the geometry and details of the superstructure elements are completely visible and accessible for inspection. For substructure, however, rebar

TABLE 5.1
Calculation of occurrence factor for timber deck, decay damage mode

Attribute	Maximum Score	Score
D.P.2 Type of Preservations and Treatment: No/Ineffective Treatment	20	20
L.3 Exposure Environment: Benign	20	5
D.P.3 Deck has an Overlay: No Overlay/Ineffective Overlay	15	15
D.P.4 Covered Bridge Component: Not Covered	15	15
C.P.1 Presence of Splits and Checks: Minor, Localized (<5% by Area) Splits and Checks	20	10
D.P.1 Inherent Decay Resistance Wood Species: All Others	15	0
Decay Damage, Deck Total	*105*	*65*
Decay Damage, Deck Ranking		*2.48–Moderate*

TABLE 5.2
Calculation of occurrence factor for steel girders, corrosion damage mode

Attribute	Maximum Score	Score
D.13 Built-up Member: Element is Built-Up Member	15	15
D.15 Constructed of Weathering Steel: Element is NOT Weather Steel	10	10
L.3 Exposure Environment: Moderate/Industrial	20	10
L.5 Rate of De-icing Chemical Application: Low	20	10
L.6 Steel Girders are Subjected to Overspray: Not Over a Roadway	20	0
C.4 Joint Condition: Significant Amount of Leakage at Joints	20	20
C.7 Quality of Deck Drainage System: Deck Drains Directly onto Superstructure or Substructure Components, or Ponding on Deck Results from Poor Drainage	15	15
C.17 Coating Condition: Coating is in Fair to Good Condition, Effective for Corrosion Protection	15	0
C.21 Presence of Active Corrosion: Minor Amount of Active Corrosion Present	20	7
C.5 Maintenance Cycle: Some Limited Maintenance	15	7
D.P.7 Deck Material Type: Timber	20	20
D.P.4 Covered Bridge Component: Not Covered	15	15
D.P.11 Presence of Details Susceptible to Corrosion: Bridge Does Not Have Details Susceptible to Corrosion	20	0
Corrosion Damage, Superstructure Total	*225*	*129*
Corrosion Damage, Superstructure Ranking		*2.29–Moderate*

corrosion may not be adequately detected visually, so visual inspection is considered effective in this case. Therefore, visual inspection is considered highly effective in detecting decay in timber deck and superstructure corrosion and fairly effective for detecting rebar corrosion in the substructure.

5.1.5 Inspection Interval

The inspection interval is determined by calculating the RPN for each element of the bridge. RPN values range from 0 to 16, where a lower RPN indicates lower risk and justifies a longer inspection interval, while a higher RPN necessitates a shorter interval. The shortest inspection interval among the elements is considered as the inspection interval for all the elements of the bridge. For the given bridge, the RPN is calculated based on the inspection data reported in 2011. The RPN for the bridge elements corresponds to a 48-month interval, as illustrated in Table 5.5 and in Figure 5.2. The inspection data from 2011 (when RBI was applied) to 2015 (end of the RBI interval) are reviewed, showing no significant change in the condition of the elements during the proposed interval. Furthermore, the proposed inspection interval would not have led to any maintenance delays if it had been adopted.

This process, however, represents only the back-casting analysis between 2011 to 2015. Thus, the same process is carried out using the 2015 inspection data, and then the inspection data from that year is analyzed up to the period produced by the 2015 analysis. Using the 2015 bridge data, the RPN for each component, aligned also with a 48-month inspection interval. The condition rating has not significantly changed throughout the proposed inspection interval (2015–2019), and no unexpected damage modes were developed. Therefore, if a 48-month inspection interval had been implemented, it would have maintained the bridge's safety and serviceability functions as effectively as the current 24-month interval.

5.2 Back-Casting of a Cable-Stay Bridge

5.2.1 Overview

This cable-stay bridge comprises of stay-in-place concrete deck and twin I-girders connected with four-

TABLE 5.3
Calculation of occurrence factor for substructure, rebar corrosion profile

Attribute	Maximum Score	Score
D.4 Poor Deck Drainage and Ponding: Ponding/Ineffective Drainage	10	10
D.6 Year of Construction Substructures: Pre-1950	10	10
D.7 Application of Protective Systems: Never Applied, Poor Functioning	10	10
D.8 Concrete Mix Design: Not High-Performance Concrete	15	15
D.11 Minimum Concrete Cover: Unknown	15	15
D.12 Reinforcement Type: Uncoated Carbon Steel	15	15
L.3 Exposure Environment: Moderate/Industrial	20	10
L.5 Rate of De-Icing Chemical Application: Low (Southern Districts)	20	10
C.5 Maintenance Cycle: Some Limited Maintenance	15	7
Corrosion Profile Point Total	*130*	*102*

TABLE 5.4
Calculation of occurrence factor for substructure, rebar corrosion damage mode

Attribute	Maximum Score	Score
C.1 Current Condition Rating: Current Rating is 5 or Below	20	0
C.4 Joint Condition: Significant Amount of Leakage at Joints	20	20
C.8 Corrosion-Induced Cracking: No Corrosion-Induced Cracking	20	0
C.9 General Cracking: Moderate Cracking Present	15	10
C.10 Delamination: No Delamination Present	20	0
C.11 Presence of Repaired Areas: No Repaired Areas	10	0
C.12 Presence of Spalling: No Spalling	20	0
C.13 Efflorescence/Staining: Minor Efflorescence	15	5
Corrosion Damage, Substructure Total	*270*	*137*
Corrosion Damage, Substructure Ranking		*2.03 Moderate*

point cable connections at the mid span. The bridge was built in 2003 with a completely covered glass encloser and passes over a highway as shown in Figure 5.3. The latest inspection report indicates that the bridge is generally in a good condition, deck and the superstructure condition ratings are 6, and the substructure condition rating is 7. Currently, a visual inspection is performed regularly every 24 months.

5.2.2 Occurrence Factor

The primary damage modes for the concrete deck and the substructure are rebar corrosion, with section loss being a significant concern for the superstructure girders. Wire breakage and corrosion are the main concerns for the free length of the cable, while corrosion and fatigue cracks are also considered key damage modes. The main factors contributing to rebar corrosion in a concrete deck are the type of reinforcement, the application of de-icing chemicals, and the depth of the concrete cover. According to inspection reports and design drawings, epoxy-coated reinforcement is used in the deck. Furthermore, the design drawings show that there is an adequate concrete cover (more than 1.5 inches). Since the bridge is covered, the application of de-icing chemicals is not anticipated. Generally, the current condition of the deck is good, although there are some transverse hairline cracks. Considering the design, condition, and loading attributes, the likelihood of rebar corrosion is deemed low, with an OF value of 1.27 as illustrated in Table 5.7 and Table 5.8.

As the bridge is adequately covered on all sides, corrosion development is expected to be delayed. Beyond its aesthetic purpose, the glass enclosure serves to protect the superstructure from environmental effects and overspray. Moreover, there is no sign of ponding indicates an adequate drainage system. In addition, since the bridge is jointless and the deck is constructed of cast-in-place concrete, immediate drainage is not anticipated. Thus, the occurrence factor of the superstructure corrosion is remote (OF = 0.75) as shown in Table 5.9.

For the corrosion damage mode concerning the free-length cable and the cable connection assembly, the primary design attributes are the type of protective system applied to the cable and whether the pin used in the cable connection is stainless. The design document indicates that the cable is constructed in accordance with ASTM A586 and anchored to the pylons and bridge using zinc-filled open strand sockets, coupled with 4¾-inch diameter stainless steel headed pins and ½-inch diameter stainless steel cotter pins. Given the design, loading, and condition attributes, the likelihood of corrosion for both the cable and the cable connection assembly is low, having OF values at (OF = 1.46) and (OF = 1.60), respectively, as demonstrated in Table 5.10 and Table 5.11.

Aeroelastic vibration effect and the presence of existing fatigue crack are the primary attributes of the cable breakage and fatigue in connection damage modes. Since there are no neoprene rings, liquid dampers, or cross cables, the aeroelastic effect is not prevented. Furthermore, the inspection report indicates that there is no presence of fatigue crack in the connection nor in the cables. As a result, the occurrence factor for wire breakage and fatigue crack in

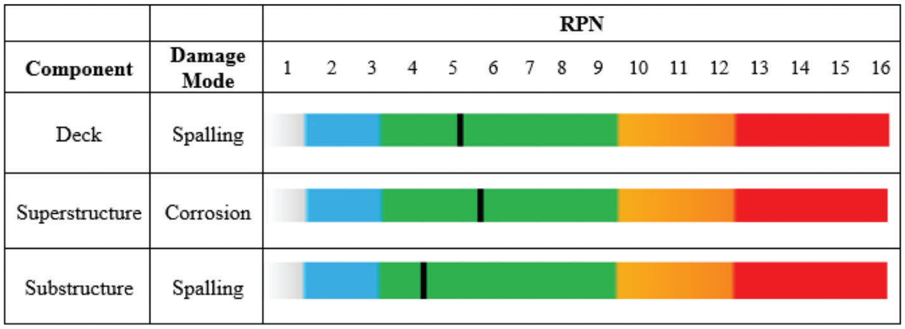

Figure 5.2 Steel bridge RPN for deck, superstructure, and substructure.

TABLE 5.5
Steel bridge Risk-Based Inspection summary and the proposed inspection plan

Component	Damage Mode	Occurrence Factor		Consequence Factor		Inspection Effectiveness Factor		RPN
Deck	Decay	Moderate	2.48	Moderate	2.0	Fairly Effective	1.0	5.0
Superstructure	Corrosion	Moderate	2.29	High	3.0	Moderately Effective	0.8	5.5
Substructure	Corrosion	Moderate	2.03	Moderate	2.0	Fairly Effective	1.0	4.1

Inspection Plan	Interval	Note
Routine Inspection	48 Months	–

TABLE 5.6
Condition rating of the bridge components and appropriate inspection interval

Year	2011	2012	2014	2015	2016	2018	2020	2022
Deck Condition	7	7	7	7	7	7	6	6
Superstructure Condition	6	6	6	6	6	6	6	6
Substructure Condition	6	6	6	6	6	6	6	6
Inspection Interval		48 months				48 months		

connection assembly damage modes is low (OF = 1.71) and (OF = 1.20) as shown in Table 5.12 and Table 5.13, respectively.

Based on the design drawings, the substructure features two abutment structures, each supported by 14-inch steel encased concrete (SEC) piles. These abutments are constructed from normal cast-in-place concrete, not the high-performance variant. Rebar corrosion is identified as the primary damage mode for the substructure elements. The substructure construction follows the guidelines of the *Indiana State Bridge Design Manual*, which mandates a 2.5-inch concrete cover and the incorporation of epoxy-coated rebar. The substructure elements generally are in a good condition, showing no signs of spalling or cracking. Therefore, occurrence factor for these elements is evaluated to be remote (OF = 0.99) as illustrated in Table 5.14 and Table 5.15.

5.2.3 Consequence Factor

The consequence factor of concrete deck spalling is considered to be low because the presence of stay-in-place forms prevents falling debris on the traffic under the bridge. Since the bridge is a twin girder bridge which is considered to be a nonredundant system according to AASHTO standards, the immediate consequence can be considered severe. However, the immediate consequence can be reduced to moderate or high if a structural analysis or documented experience demonstrates the redundancy of the system. The short-term consequence also considers to be high since the bridge passes over a highway and the daily pedestrian volume is relatively high. Therefore, the overall consequence factor of the superstructure is considered to be severe. For the cable and the connection assembly, the consequence is considered to be moderate, assuming the cable and connection assembly are designed to be replaced one at a time and are designed for dynamic impact if a cable loses its entire capacity.

5.2.4 Inspection Effectiveness Factor

The top surface of the deck is the only visible part of the deck that can reliably be inspected visually. Therefore, the effectiveness of visual inspection is considered to be a less effective inspection technique. Again, utilizing a more reliable inspection method such as NDE may extend the inspection interval based on their effectiveness. For the superstructure, the presence

Figure 5.3 View of the cable stay bridge.

TABLE 5.7
Concrete deck corrosion profile calculation

Attribute	Maximum Score	Score
D.4 Poor Deck Drainage and Ponding: No problems noted	0	0
D.8 Concrete Mix Design: Unknown	15	15
D.11 Minimum Concrete Cover: 1.5 in.+	15	0
D.12 Reinforcement Type: Has protective coating or is made from corrosion resistant metal	15	0
L.3 Exposure Environment: Central districts	20	10
L.5 Rate of De-Icing Chemical Application: None	20	0
C.5 Maintenance Cycle: Regular maintenance	20	10
Corrosion Profile Point Total	*115*	*35*

TABLE 5.8
Calculation of occurrence factor for concrete decks, spalling damage mode

Attribute	Maximum Score	Score
C.1 Current Condition Rating: Current rating is 7+	20	0
D.P.12 Deck Construction: Cast in place	15	15
C.10 Presence of Delamination: Unknown	20	20
C.12 Presence of Spalling: No spalling	20	0
D.20 Deck Thickness: 6 in.+	15	5
Spalling Damage, Deck Total	*205*	*65*
Spalling Damage, Deck Ranking		*1.27 Low*

of an enclosure creates a lack of accessibility for inspectors to reliably inspect and detect the superstructure components. Therefore, an action must be taken to improve the effectiveness of the inspection technique. For example, the effectiveness of the inspection technique can be improved by removing some of the enclosure panels at the splices and at the supports. As a result, the inspection effectiveness factor of visual inspection can be considered to be highly effective. It is important to note that whenever the component is completely inaccessible for inspection, an action must be taken to improve the inspection reliability. For cable and connection assembly corrosion visual inspection can be considered to be fairly effective. On the other hand, visual inspection can be considered to be of low effectiveness in detecting fatigue crack in connection assembly. Taking into account the inspection effectiveness, as well as the occurrence and consequence factors, the RPN is computed to determine the inspection interval.

5.2.5 Inspection Interval

Two inspection intervals are assigned to this bridge: one is for routine visual inspection and the other is for in-depth inspection as shown Table 5.16 and in Figure 5.5. The routine visual inspection focuses on maintaining the condition of the visible part of the structure such as deck, cable, connection assembly, and substructure. The in-depth inspection involves removing some

TABLE 5.9
Calculation of occurrence factor for steel girders, corrosion damage mode

Attribute	Maximum Score	Score
D.13 Built Up Member: Element is not a built-up member	15	0
D.15 Constructed of Weathering Steel: Element is NOT weather steel	10	10
L.3 Exposure Environment: Moderate/industrial	20	10
L.5 Rate of De-Icing Chemical Application: None	20	0
L.6 Steel Girders are Subjected to Overspray: Not over a roadway/Not subjected to overspray	20	0
C.4 Joint Condition: Bridge is jointless	20	0
C.7 Quality of Deck Drainage System: Adequate drainage	15	0
C.17 Coating Condition: Coating system is in poor condition, 1%–3% rusting, substantially effective for corrosion protection	15	7
C.21 Presence of Active Corrosion: Minor amount of active corrosion present	20	7
C.5 Maintenance Cycle: Some limited maintenance	15	7
D.P.7 Deck Material Type: All other	20	0
D.P.4 Covered Superstructure Element: Effective cover	15	0
D.P.11 Presence of Bad Details: Bridge has good details	20	0
Corrosion Damage, Superstructure Total	*220*	*41*
Corrosion Damage, Superstructure Ranking		*0.75 Remote*

enclosure panels to ensure the superstructure elements are in an adequate condition. It is important to note that the in-depth inspection is limited to inspect the

TABLE 5.10
Calculation of occurrence factor for free length cable, corrosion damage mode

Attribute	Maximum Score	Score
C.17 Coating Condition: Coating system is in poor condition, 1%–3% rusting, substantially effective for corrosion protection	15	7
D.P.14 Cable Protective Material: Galvanized	15	0
C.21 Presence of Active Corrosion: Minor amount of active corrosion present	20	7
C.5 Maintenance Cycle: Some limited maintenance	15	7
L.6 Cables are Subjected to Overspray: Moderate overspray exposure	20	10
Wire/Cable Corrosion Damage, Steel Superstructure Total	*85*	*31*
Wire/Cable Corrosion Damage, Steel Superstructure Ranking		*1.46 Low*

superstructure elements, so it is not required to inspect the other bridge elements. The scheduling for routine and in-depth inspections is structured so that the interval begins from the date of the last respective inspection, as depicted in Figure 5.4. Specifically, the 48-month interval starts from the date of the last routine inspection, while the 72-month interval begins from the last in-depth inspection. Furthermore, the inspection plan should update after conducting in-depth inspecting to account for any significant change in the condition of the elements.

The back-casting concept has been applied to subsequent inspection reports, demonstrating that the established routine and in-depth inspection intervals are appropriate. This determination stems from the observation that there were no significant changes in condition, no unexpected damage modes emerged within the inspection period, and no maintenance actions were needed earlier than the designated inspection intervals. Consequently, the proposed inspection plan can be implemented to ensure the structural integrity and reliability of the superstructure components.

TABLE 5.11
Calculation of occurrence factor for cable connection assembly, corrosion damage mode

Attribute	Maximum Score	Score
C.17 Coating Condition: Coating system is in poor condition, 1%–3% rusting, substantially effective for corrosion protection	15	7
C.21 Presence of Active Corrosion: Moderate amount of active corrosion present	20	15
C.5 Maintenance Cycle: Some limited maintenance	15	7
L.5 Rate of De-Icing Chemical Application: None	20	0
D.P.9 Pin Material Type: Stainless steel	15	0
D.P.10 Types of Socket Fill Material: Zinc	10	0
D.P.4 Covered Bridge: Effective covered component	15	15
Cable Connection Corrosion Damage, Steel Superstructure Total	*110*	*14*
Cable Connection Corrosion Damage, Steel Superstructure Ranking		*1.60* Low

TABLE 5.12
Calculation of occurrence factor for cable free length, wire breakage damage mode

Attribute	Maximum Score	Score
D.P.8 Aerostatic Vibration Effect: Not prevented	15	15
C.P.2 Presence of Broken Wires in Bridge Cables: No broken wires	20	0
Wire/Cable Breakage Damage, Steel Superstructure Total	*35*	*15*
Wire/Cable Breakage Damage, Steel Superstructure Ranking		*1.71* *Low*

TABLE 5.13
Calculation of occurrence factor for cable connection assembly, fatigue crack damage mode

Attribute	Maximum Score	Score
D.P.8 Aerostatic Vibration Effect: Not prevented	15	15
D.P.15 Existing Fatigue Crack: No	20	0
C.P.4 Presence of Repaired Fatigue Cracks: No repaired fatigue cracks are present	15	0
Connection Assembly Fatigue Damage, Steel Superstructure Total	*50*	*15*
Cable Connection Assembly Fatigue Damage, Steel Superstructure Ranking		*1.20* *Low*

TABLE 5.14
Calculation of corrosion profile for substructure, rebar corrosion damage mode

Attribute	Maximum Score	Score
D.4 Poor Deck Drainage and Ponding: No problems noted	10	0
D.6 Year of Construction Substructures: 1990+	10	0
D.7 Application of Protective Systems: Never applied, poor functioning	10	10
D.8 Concrete Mix Design: Not high-performance concrete	15	15
D.11 Minimum Concrete Cover: 2.5 in. +	15	0
D.12 Reinforcement Type: Has protective coating or is made from corrosion resistant metal	15	0
L.3 Exposure Environment: Moderate/industrial	20	10
L.5 Rate of De-Icing Chemical Application: None	20	0
C.5 Maintenance Cycle: Some limited maintenance	15	7
Corrosion Profile Point Total	*130*	*42*

TABLE 5.15
Calculation of occurrence factor for substructure, rebar corrosion damage mode

Attribute	Maximum Score	Score
C.1 Current Condition Rating: Current rating is 7+	20	0
C.4 Joint Condition: Bridge is jointless	20	0
C.8 Corrosion-Induced Cracking: No corrosion-induced cracking	20	0
C.9 General Cracking: Minor or no cracking	15	0
C.10 Delamination: Unknown	20	20
C.11 Presence of Repaired Areas: No repaired areas	10	0
C.12 Presence of Spalling: No spalling	20	5
C.13 Efflorescence/Staining: Minor efflorescence	15	5
Corrosion Damage, Substructure Total	*270*	*67*
Corrosion Damage/Piers, Abutments, Substructure		*0.99–Remote*

TABLE 5.16
Cable-stay Risk-Based Inspection summary and the proposed inspection plan

Component	Damage Mode	Occurrence Factor		Consequence Factor		Inspection Effectiveness Factor		RPN
Deck	Corrosion	Low	1.46	Low	1	Low Effective	1.5	2.2
Girders	Corrosion	Remote	0.75	Severe	4	Highly Effective	0.5	1.5
Cable	Corrosion	Low	1.46	Moderate	2	Fairly Effective	1.0	2.9
	Cable Breakage	Low	1.71	Moderate	2	Low Effective	1.5	5.1
Connection Assembly	Corrosion	Low	1.60	Moderate	2	Fairly Effective	1.0	3.2
	Fatigue Crack	Low	1.20	Moderate	2	Low Effective	1.5	3.6
Substructure	Spalling	Remote	0.99	Moderate	2	Fairly Effective	1.0	2
Inspection Plan				Interval		Note		
Routine Inspection				48 Months		–		
In-depth Inspection				72 Months		Enclosure panels at supports and splices need to be removed.		

TABLE 5.17
Condition rating of the bridge components and appropriate inspection interval

Year	2007	2009	2011	2013	2015	2017	2019	2021
Deck Condition	8	8	6	6	6	6	6	6
Superstructure Condition	8	8	6	6	6	7	6	7
Substructure Condition	8	8	6	6	6	7	7	7
Inspection Interval				48 months 72 months				

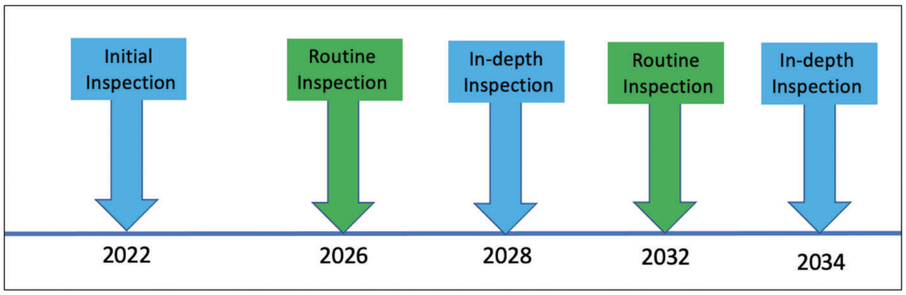

Figure 5.4 Visualization of the inspection plan assuming an initial inspection is conducted in 2022.

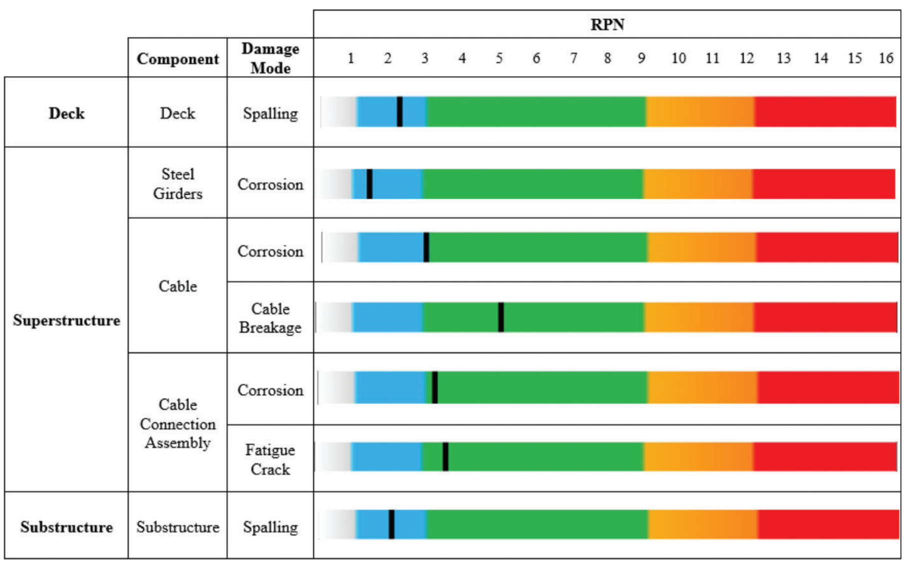

Figure 5.5 Cable-stay RPN for deck, superstructure, and substructure.

6. IMPLEMENTATION AND FUTURE WORK

6.1 Preliminary Training

The successful implementation of the proposed methodology heavily relies on the necessary training and education for individuals who are inspecting pedestrian bridges. As a starting point in the implementation process, a pilot training was conducted for a select group of participants. This training is divided into five sessions, with each session designed to highlight a core aspect of the RBI approach, supplemented with examples. The final session includes a detailed case study on the RBI approach's application to a bridge, followed by an open discussion on challenges in implementation and potential solutions. The primary aim of this training is to gather feedback from participants to enhance the clarity and delivery of the content and training materials. Feedback and comments from participants were collected to further enhance the delivery of the RBI concept and improve the clarity of the criteria.

6.2 RBI Spreadsheet

To further streamline the implementation of the proposed methodology, an intuitive Excel spreadsheet was developed to estimate the inspection intervals for different types of pedestrian bridge systems. This spreadsheet comprises ten sheets. The introductory sheet prompts users to input bridge details into the highlighted yellow cells, as depicted in Figure 6.1. Users can select different deck and superstructure systems via a drop-down list. Once all necessary inputs are provided, users can click on "start assessment." This action will direct users to the relevant sheet based on their specified deck and superstructure types.

Additionally, the spreadsheet incorporates hyperlinks to commentaries, offering a thorough explanation of the consequence and inspection effectiveness factors. The last sheet presents the results, detailing the occurrence, consequence, and inspection effectiveness factors for each damage mode, along with the appropriate inspection interval. This result sheet also features a bar chart plot indicating the RPN value of each element and the corresponding inspection category, as illustrated in Figure 6.2. Furthermore, an alert message is embedded in the results sheet to notify users if the RPN exceeds the risk limit or if inaccessible or ineffective inspection techniques are chosen.

6.3 Future Work

Further tasks are necessary to refine and enhance the implementation of the methodology. These tasks are summarized below.

- Developing comprehensive training for inspectors, equipping them with the necessary knowledge and skills to adequately implement the RBI methodology. This includes developing training sessions and a guideline for inspector on implementing the RBI approach. In addition, the training sessions encompass a demonstration on using the RBI assessment spreadsheet on case studies of real bridges.
- Ensure that the training program and materials are interactive and incorporates feedback and suggestions garnered from the beta version training sessions. This includes revising training materials or methods based on what was learned during the beta training.
- Ensure consistent outputs of the developed RBI assessment spreadsheet among users, regardless of which inspector is using the tool. This can be accomplished by introducing the RBI assessment spreadsheet to different inspectors, analyzing the variation in their outputs, and exploring the reasons for these inconsistencies. If there is a significant variation in the output, additional details and clarification may be added to the RBI approach, or to the training program.
- Refining the existing features of the spreadsheet tool and integrating new features into the spreadsheet tool based

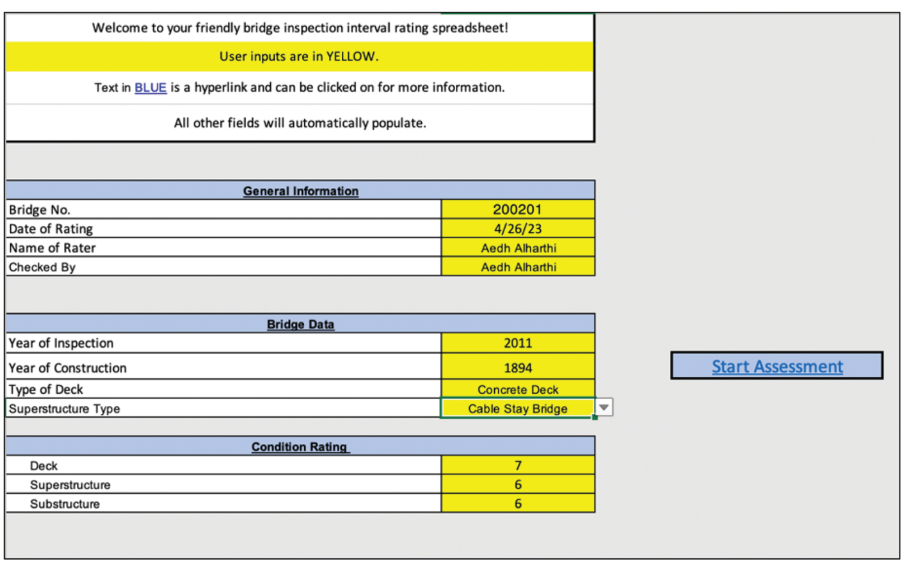

Figure 6.1 Screenshot of the start page of the RBI spreadsheet.

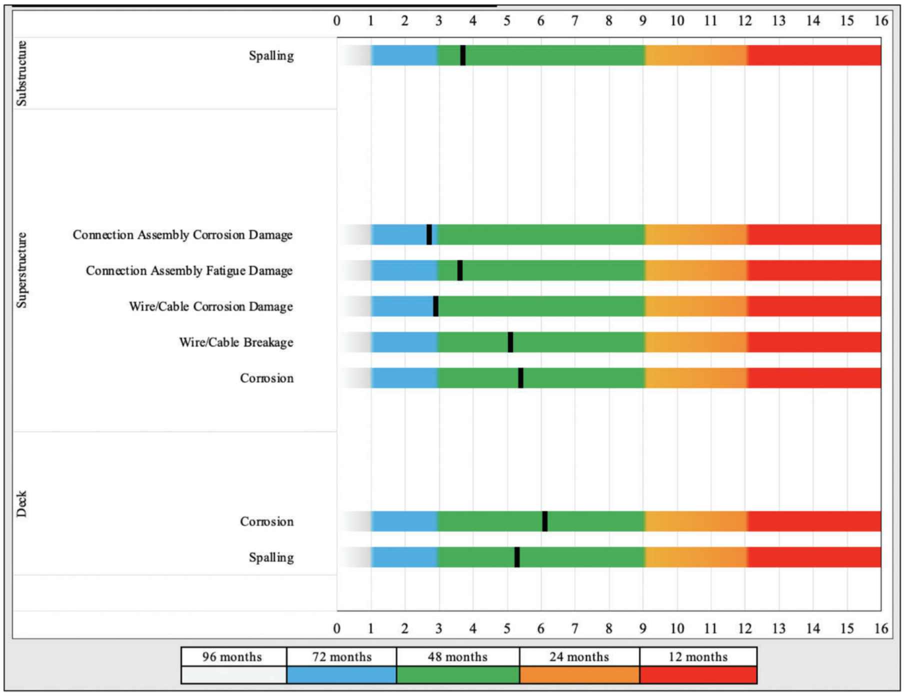

Figure 6.2 Screenshot of the result sheet showing the bar chart plot of RPN and corresponding inspection interval for a cable stay bridge.

on the user's feedback to make them more user-friendly and effective.

- Integrating the unique serviceability criteria of the pedestrian bridges into the bridge inspection database. This ensures that these criteria are handy and intuitive for inspectors, facilitating informed assessments during the inspection process.
- For some pedestrian bridges that are exhibiting severe deterioration affecting their load-bearing capacity, developing a formalized load rating guideline specifically applicable to pedestrian bridges is necessary.
  - The significant variability in pedestrian traffic between rural and urban areas highlights the need for a distinct classification of pedestrian loads. Although the *AASHTO Design Specification for Pedestrian Bridges* suggests a certain pedestrian load that can be used for load rating, it might be overly conservative for some rural pedestrian bridges. On the other hand, this prescribed pedestrian load could potentially underestimate loads for certain rural bridges or those anticipated to host events with unusually high pedestrian volume.
  - Beyond just pedestrian loads, in emergency scenarios, it is essential to confirm that the bridge has the capacity to carry emergency vehicles. Some pedestrian bridges are designed to carry maintenance vehicles like H5 or H10, based on their clear deck width as dictated by the AASHTO Pedestrian Specification. However, it remains ambiguous whether these vehicle specifications accurately represent the actual loads of emergency trucks. Consequently, developing a comprehensive load rating guideline for pedestrian bridges can enhance decision-making, offering users clear guidance on determining the appropriate pedestrian load and the emergency truck load.

6.4 Conclusion

This report concentrates on the development of formalized inspection criteria applicable to pedestrian bridges. The RBI methodology is adopted as a primary basis for determining inspection intervals for different types of pedestrian bridges. In this approach, the inspection interval is determined based on the risk assessment of the main components of the bridge. The risk assessment essentially involves an assessment of the likelihood of a certain damage occurring and the consequences following the occurrence of the damage. Additionally, a new factor, called the inspection effectiveness factor (IEF), is integrated into the RBI approach, which accounts for the reliability of the inspection technique in detecting the specified damage mode. Furthermore, an assessment of unique serviceability criteria of pedestrian bridges is also included to ensure a high level of safety and serviceability for pedestrians and cyclists on the bridge. The effectiveness of RBI approach is verified by conducting the back-casting analysis on various types of pedestrian bridges. The back-casting results indicate that the proposed approach enhances the safety and serviceability of bridge inspections and improves the allocation of inspection resources. However, further implementation tasks are necessary to ensure the effectiveness of the RBI methodology. This includes comprehensive training for inspectors on the RBI concept, utilization of the RBI inspection interval assessment tool, and incorporation of the unique safety and serviceability criteria for pedestrian bridges into the inspection database.

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APPENDICES

Appendix A. Survey Responses

Appendix B. Attribute Reason and Assessment Procedure

Appendix C. Consequence Factor Categories Description

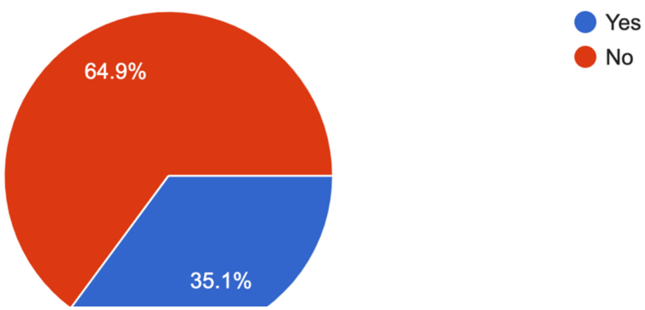
Appendix D. Inspection Effectiveness Factor Categories Description

Appendix E. Back-Casting Analysis

APPENDIX A. SURVEY RESPONSES

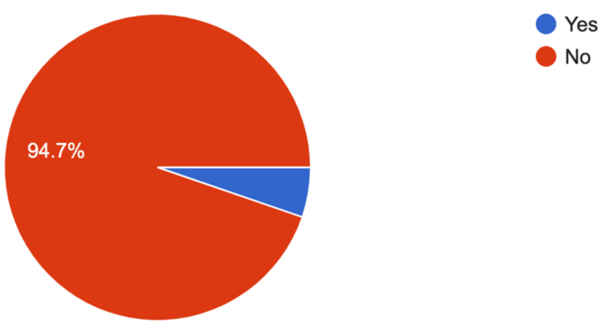
1- Does your Agency maintain a database and an inventory specifically for pedestrian bridges?

37 responses



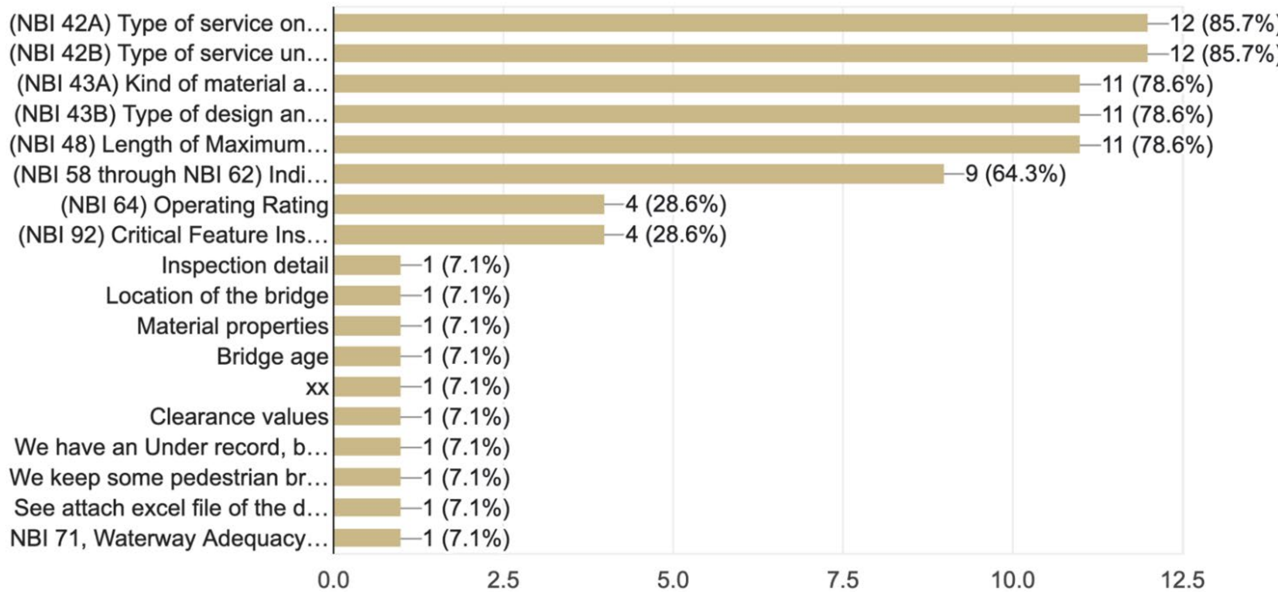
6- Does your Agency have a manual for in-service inspection specifically applicable to pedestrian bridges?

38 responses



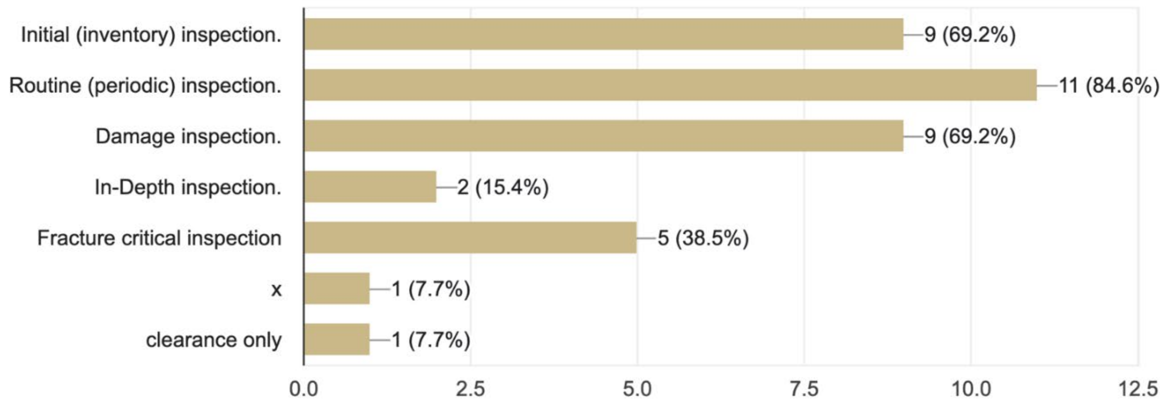
2- Does the database include any of the following (please check all that apply) included in the pedestrian inventory bridge?

14 responses



10- What types of inspection would be the essential to include in an inspection manual for pedestrian bridges?

13 responses



11- What are the alternative criteria or plans that your Agency use for in-service inspection pedestrian bridges, (i.e., using highway criteria for inspecting pedestrian bridges)?

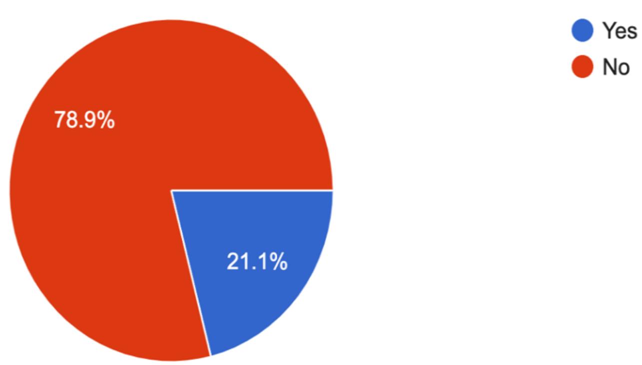
Response 1.	NBIS
Response 2.	Inspection of slip issues, water pooling and lighting come to mind.
Response 3.	They are inspected the same
Response 4.	none
Response 5.	Highway Bridge Inspection criteria
Response 6.	Our agency collects NBI clearance data on pedestrian bridges that cross our ROW. Our inspectors are instructed to also do a cursory inspection of the bridge to see if there are any safety issues that need to be addressed.
Response 7.	use highway criteria
Response 8.	highway criteria with longer inspection frequency permissible
Response 9.	We use regular highway bridge criteria for pedestrian bridges

Response 10.	Check horizontal and vertical clearance only by state. Inspection of structure is placed on local owner using NBIS criteria
Response 11.	We use the NBIS criteria to evaluate pedestrian bridges
Response 12.	There are no state-owned ped bridges in Nebraska. Locally owned ped bridges over state highways are occasionally inspected for clearance only. Local ped bridge owners are encouraged to add their ped structures to the inventory, but it is not required.
Response 13.	We inspect our pedestrian bridges in accordance with the normal NBIS inspection program but do not submit the pedestrian bridge data to FHWA.
Response 14.	We inspect pedestrian bridges as we would any highway bridge structure, following AASHTO MBE, BIRM and internal procedures. The exception is bridge inspection interval, which is typically 72 months, although more frequent intervals are allowed if conditions require. All pedestrian bridge inspection/condition data is retained in our BMS as would any highway bridge.
Response 15.	Not sure I understand your question, but we include all DelDOT-owned ped bridges in our bridge inventory regardless of if the ped bridge spans over water or over a roadway.
Response 16.	Mostly clearance only. Use highway criteria for some.
Response 17.	We do NBI ratings for deck, super, sub and BrM elemental ratings similar to our highway criteria
Response 18.	Unsure. I work in Bridge Design. This survey should be sent to LA DOTD Bridge Maintenance (Section 51) - they are responsible for bridge inspections.
Response 19.	highway criteria
Response 20.	We use the same criteria that we do for highway bridges.
Response 21.	NBI Item 5A inspection
Response 22.	Arizona inspects ped bridges over highways only for vertical clearance considerations. If any structural distress is noted, that is forwarded to the local government agency owner.
Response 23.	Use some highway criteria for inspection.
Response 24.	We use highway criteria for inspecting pedestrian bridges, when applicable.
Response 25.	NDOT uses same procedures for highway and pedestrian bridges.
Response 26.	We use very similar criteria to our highway bridges (see our Pub 238).
Response 27.	We are using highway criteria for inspecting pedestrian bridges.
Response 28.	We use the AASHTO LRFD Guide Spec for ped bridge design.
Response 29.	same as NBIS
Response 30.	We inventory pedestrian bridges for vertical clearance purposes and inspect to make sure no concerns for traffic underneath such as the possibility of falling concrete.
Response 31.	Use the highway criteria according to AASHTO Elements and FHWA Coding Guide.

Response 32.	Pedestrian bridge must cross a highway (or be within Department's R/W) to qualify for an inspection. Department owned pedestrian bridges receive NBI and element inspections for management purposes and others receive a cursory inspection to ensure safety of highway users below. The FHWA Coding Guide and AASHTO MBEI are used for NBI and element inspections respectively.
Response 33.	Pedestrian bridges are inspected to the same standards as a highway bridge per the department's inspection manual. Department's bridge inspection software does not differentiate between the two. We will allow a 6-year interval for 100% "Hands-On" inspection of fatigue prone details and non-redundant members. The wavier is dependent on the bridge's condition.
Response 34.	Use NBI coding and do inspections every 5 years unless condition warrants more frequent inspections
Response 35.	Nonspecific to Ped bridges

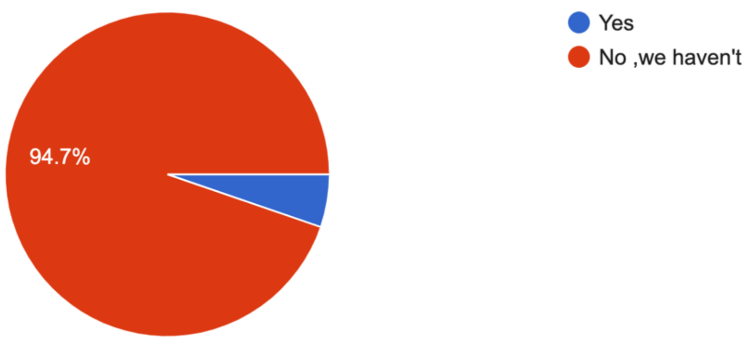
4- Does your Agency have design specifications specifically applicable to pedestrian bridges?

38 responses



14- If your Agency has collected data on the history of a pedestrian bridge maintenance cataloging the deterioration rate and any repairs during the lif...ridge, are you willing to share the data or reports?

38 responses



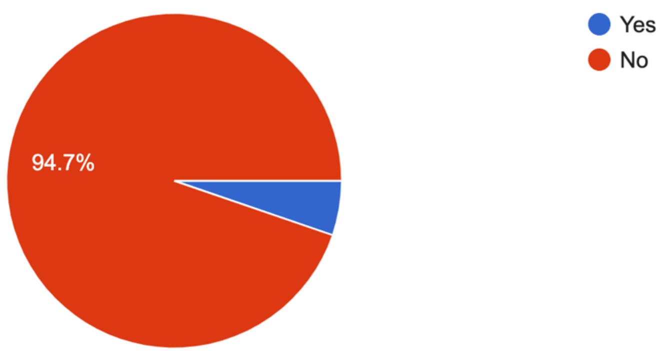
15- Is there any other information or insight that should be considered in our research you would like to share with research team?

Response 1.	Little info specific to pedestrian, they are considered as other bridges, only with smaller service loads (both distributed and concentrated)
Response 2.	For pedestrian bridges located within the state right-of-way, we inventory them along with our bridges and inspect them as such. They are not handled any differently with the exception that they are not submitted to federal highway administration.
Response 3.	SDDOT only owns 1 pedestrian bridge
Response 4.	We treat pedestrian bridges the same as highway bridges in terms of inspecting and maintaining them.
Response 5.	Load rating guidance specific to pedestrian bridges in addition to inspection procedures would be helpful to establish clear and consistent procedures.
Response 6.	If NDOT were to inspect ped bridges, the existing NBI inspection protocol would be sufficient.
Response 7.	We do a routine inspection on pedestrian bridges owned by SCDOT. However, we do not do those inspection on bridges owned by others. We do take clearance values on all pedestrian bridges over public roads for oversize loads to ensure we don't hit them.
Response 8.	Interesting potential topic may be useful for local agency pedestrian bridge owners.

Response 9.	We keep/update these records as many are over the interstate and we need clearance data for our permitting system.
Response 10.	We inspect and maintain bridges with our NBI & State bridge population and seems to be adequate enough.
Response 11.	We would be interested in the results of your research.
Response 12.	Most of the pedestrian bridges are owned by local authorities. This research would be better served by reaching out to them and figuring out what their needs are.
Response 13.	We recommend 48-month inspections for pedestrian bridges, but we do not require them.
Response 14.	Current NDOT policy is to only inspect the portions of pedestrian bridges that are over our right-of-way.
Response 15.	To clarify our response to #1, PennDOT includes pedestrian bridges in our Bridge Management system along with our highway bridges, retaining walls, sign structures, tunnels, etc. We do not maintain a database "specifically" for pedestrian bridges.
Response 16.	Data for pedestrian and foot bridges is in BrM. We do not collect element data for pedestrian and foot bridges.
Response 17.	The vehicular bridge manual would cover everything for pedestrian bridge inspection.

12- Does your Agency have any load rating procedures or guidance specifically applicable to pedestrian bridges?

38 responses



Agency	Inventory	Design Specification	Inspection Manual	Load Ratting	Comment
International Cooperative (Canada)	No	No	No	No	Inspection of slip issues, water pooling and lighting come to mind.
Maryland State Department of Transportation (MDOT)	No	No	No	No	Highway Bridge Inspection criteria
Mississippi Department of Transportation (MDOT)	No	No	No	No	Our agency collects NBI clearance data on pedestrian bridges that cross our ROW. Our inspectors are instructed to also do a cursory inspection of the bridge to see if there are any safety issues that need to be addressed.
South Dakota Department of Transportation (SDDOT)	No	No	No	No	highway criteria with longer inspection frequency permissible
Delaware Department of Transportation (DelDOT)	No	No	No	No	We use regular highway bridge criteria for pedestrian bridges
Georgia Department of Transportation (GDOT)	No	Yes	No	No	Check horizontal and vertical clearance only by state. Inspection of structure is placed on local owner using NBIS criteria
Oregon Department of Transportation (ORDOT)	No	No	No	No	Using NBI criteria
Rhode Island Department of Transportation (RIDOT)	No	No	No	No	Load rating guidance specific to pedestrian bridges in addition to inspection procedures would be helpful to establish clear and consistent procedures.
Nebraska Department of Transportation (NEDOT)	No	Yes	No	No	There are no state-owned ped bridges in Nebraska. Locally owned ped bridges over state highways are occasionally inspected for clearance only. Local ped bridge owners are encouraged to add their ped structures to the inventory, but it is not required.

South Carolina Department of Transportation (SDOT)	Yes	No	Yes	No	We do a routine inspection on pedestrian bridges owned by SCDOT. However, we do not do those inspection on bridges owned by others. We do take clearance values on all pedestrian bridges over public roads for oversize loads to ensure we don't hit them.
West Virginia Department of Transportation (WVDOT)	No	No	No	No	We inspect our pedestrian bridges in accordance with the normal NBIS inspection program but do not submit the pedestrian bridge data to FHWA.
California Department of Transportation (Caltrans)	No	No	No	No	We inspect pedestrian bridges as we would any highway bridge structure, following AASHTO MBE, BIRM and internal procedures. The exception is bridge inspection interval, which is typically 72 months, although more frequent intervals are allowed if conditions require. All pedestrian bridge inspection/condition data is retained in our BMS as would any highway bridge.
New Mexico Department of Transportation (NMDOT)	Yes	Yes	No	No	We keep/update these records as many are over the interstate and we need clearance data for our permitting system.
Arkansas (ARDOT)	Yes	Yes	No	No	We inspect and maintain bridges with our NBI & State bridge population and seems to be adequate enough.
Idaho DOT	No	No	No	No	Mostly clearance only. Use highway criteria for some.
Florida Department of Transportation (FDOT)	No	No	No	No	We use the same criteria that we do for highway bridges.
Kentucky State KYTC	Yes	No	No	No	NBI Item 5A inspection
Arizona Department of Transportation (ADOT)	No	No	No	No	Arizona inspects ped bridges over highways only for vertical clearance considerations. If any structural distress is noted, that is forwarded to the local government agency owner.
State of North Dakota (NDDOT)	Yes	No	No	No	Use some highway criteria for inspection
State of Wisconsin	Yes	Yes	No	No	We use highway criteria for inspecting pedestrian bridges, when applicable.

Nevada Department of Transportation (NDOT)	No	No	No	No	NDOT uses same procedures for highway and pedestrian bridges.
International cooperative (Switzerland)	No	Yes	No	No	Same as highway
State of Kansas Department of Transportation (KDOT)	No	No	No	No	Same criteria as for the inspection of Highway Bridges.
New Hampshire Department of Transportation (NHDOT)	No	No	No	No	Conduct a routine inspection for pedestrian bridges over highways to maintain needed geometric data to inform overweight/ oversize … and to ensure soundness of components both for pedestrian and traveling public below. Typical highway inspection manual is employed.
Texas Department of Transportation (TxDOT)	No	No	No	No	TxDOT's policy for inspecting pedestrian bridges is limited to pedestrian bridges over on-system highways following a lot of the same criteria for highways. As outlined in the TxDOT Bridge Inspection Manual: "Pedestrian-only bridges over On-System routes are to be inventoried, added to the database, and receive a routine safety inspection every 48 months. If conditions warrant, the inspection frequency may be increased to 24 months or 12 months. These inspections are generally considered the same as any other grade-separation inspections and will be included with other routine bridge inspection. work in the district."
International Cooperative (Canada)	Yes	No	No	No	–
Wisconsin Department of Transportation (WisDOT)	Yes	Yes	No	No	We use highway criteria for inspecting pedestrian bridges, when applicable. We recommend 48-month inspections for pedestrian bridges, but we do not require them.

Pennsylvania (PenDOT)	Yes	Yes	No	No	We use very similar criteria to our highway bridges (see our Pub 238). No option to upload pub.
Colorado State (CODOT)	Yes	Yes	No	Yes	We are using highway criteria for inspecting pedestrian bridges.

APPENDIX B. ATTRIBUTE REASON AND ASSESSMENT PROCEDURE

B.1 Inherent Decay Resistance Wood Species

B.1.1 Reason(s) for the Attribute

For most timber structure applications, durability is the primary question of decay resistance. However, based on service records, laboratory tests, and expert opinions, certain wood species have demonstrated adequate inherent decay resistance (AASHTO, 2020; Forest Service & Products Laboratory, 2010). Therefore, timber members constructed from these species are expected to provide sufficient durability and longer service life.

B.1.2 Assessment Procedure

This attribute is assessed based on whether the timber element is constructed from one of the wood species that are known for their inherent decay resistance. Table B.1 lists some examples of species with an inherent decay resistance heartwood (AASHTO 2020). If the wood species is not reported in the design documents, inspector rational judgment should be exercised to identify the wood species.

All other type of wood species	15
Constructed with inherit decay resistance wood specie	0

Table B.1 Species with High Decay Resistance Heartwood (AASHTO, 2020).

Softwoods	Hardwoods
Bald cypress, old growth	Chestnut
Cedar	Locust, black
Juniper	Oak, white
Redwood, old growth	Osage, orange
Yew, Pacific	Walnut, black

B.2 Type of Preservations and Treatment

B.2.1 Reason(s) for the Attribute

Untreated timber is vulnerable to insect attacks such as fungi, parasites, and other types of insects. The primary purpose of treatment is to keep the inner cores of the timber surface dry to prevent the presence of living fungi. Therefore, timber element with an effective treatment provides extended adequate durability against biological deterioration. The effectiveness of treatment and preservation is defined herein as a treatment procedure in which the inner cores of timbers are appropriately treated. Vacuum-pressure treatment, for example, is known as an effective treatment since the internal cores are adequately treated. Paint treatment, however, may not be considered an effective treatment because the treatment is limited to the outer surface of the timber which could allow moisture to seep inside the timber core especially if the surface is experiencing splits and checks. Therefore, an effective treatment delays the formation of biological deterioration.

B.2.2 Assessment Procedure

The evaluation of this attribute can be accomplished through two main aspects: the application, and the effectiveness of treatment. The latter aspect should be evaluated by engineering judgment, considering the definition of effective treatment as mentioned previously.

No/Ineffective treatment	20
Paint	10
Effective Treatment	0

B.3 Deck has Overlay

B.3.1 Reason(s) for the Attribute

This attribute is intended to evaluate the presence and effectiveness of an overlay on timber decks. An overlay is defined as an additional layer applied on top of the deck surface that serves as a riding surface. This includes, but is not limited to, materials like asphalt, concrete, or other suitable materials. An effective overlay, when properly bonded to the deck beneath, provides a crucial water barrier to the superstructure. This can potentially delay decay formation and extend the service life of the timber structure. Conversely, an ineffective overlay can trap moisture, facilitating its penetration into the superstructure over time. This can accelerate the degradation process and potentially compromise the structural integrity of both the deck and the overall bridge. Therefore, the absence of an overlay may sometimes be preferable to having an ineffective one, as it would limit the moisture source to the timber deck alone and prevent the overlay from delaying the drying process or preserving moisture.

B.3.2 Assessment Procedure

The assessment of this attribute can be accomplished through two main aspects: the presence and the effectiveness of the overlay. The presence of an overlay can typically be identified from the bridge's design plans or during a physical inspection. The effectiveness of the overlay, on the other hand, should be evaluated using engineering judgment, considering the definition of an effective overlay as mentioned previously. Factors to consider may include the quality of the bond between the overlay and the deck, the condition of the overlay material, and its ability to prevent moisture penetration.

No/Ineffective overlay	15
Effective overlay	0

B.4 Covered Bridge

B.4.1 Reason(s) for the Attribute

The environment is the primary effect of most of the deterioration mechanisms. For steel superstructures, covered bridges tend to develop corrosion slower than those not covered. Because cover panels protect bridge elements from moisture due to rain or overspray, corrosion development is significantly delayed. In timber bridges, the effective cover also keeps some of the structural elements from moisture which is one of the main factors for fungi presence in timber. An effective cover is defined herein as an adequate cover panel that adequately prevents bridge elements from rainwater and overspray. However, if the bridge is ineffectively covered, moisture penetrates the structural elements, resulting in a delay in the drying process of the component.

B.4.2 Assessment Procedure

This attribute is evaluated based on whether the bridge has an adequate cover or not. The effectiveness of the cover should be assessed based on rational inspector judgment, considering the effective cover definition mentioned previously.

Bride is not/ineffectively covered	15
Bridge is effectively covered	0

B.5 Presence of Splits and Checks

B.5.1 Reason(s) for the Attribute

This attribute is intended to account for the presence of splits and checks on timber bridge elements. Checks are characterized as wood fiber separations parallel to the grain direction. Splits are an advanced form of checks that extend all the way through a timber element. The presence of splits and checks predominately accelerate decay development because they provide an easy path for moisture to reach the inner cores of the timber elements. As a result, the presence of moisture in the inner cores creates an ideal environment for fungi, accelerating the decay and development process (FHWA, 2022).

B.5.2 Assessment Procedure

This attribute is intended to assess the severity of splits and checks based on the inspector's note in the inspection report. Users should pay special attention to evaluating this attribute, especially in cases where significant splits with section loss could lead to a sudden reduction of the element's capacity. As the attribute considers splits and checks by area, the severity level should also be considered in the assessment.

Significant (>20% by area) splits and checks	20
Moderate (5%–20% by area) splits and checks	15
Minor, localized (<5% by area) splits and checks	10
No splits and checks present	0

B.6 Type of Connector

B.6.1 Reason(s) for the Attribute

This attribute is intended to assess the type of connector used to join the timber deck to the superstructure elements or the timber superstructure elements themselves. For instance, timber elements connected with nails or screws tend to decay more rapidly than those with bolted connectors. This rapid decay occurs because moisture can penetrate the connector through the untreated drilled surface between the connector and the timber part. However, in bolted connectors, the surface between the connector and the timber element is drilled prior to the treatment process. As a result, the drilled surface in bolted connectors is considered effectively treated, preventing moisture accumulation between the bolt and the timber surface.

B.6.2 Assessment Procedure

This attribute is evaluated based on the type of connector used to join the timber deck to the superstructure or the timber superstructure elements. Such information can be sourced from the bridge's drawings or through rational engineering judgment.

Nails/Screw	15
Bolt	5

B.7 Connector Material Type

B.7.1 Reason(s) for the Attribute

This attribute is intended to characterize the type of connector's protective coating. The connector described herein is a connector connecting the timber deck to the superstructure, or the superstructure elements to each other. A metallic protective coating, such as a galvanized coating, is well known for its corrosion resistance. Ungalvanized connectors, on the other hand, deteriorate more rapidly than connectors with metallic protective coatings.

B.7.2 Assessment Procedure

This attribute is evaluated based on whether the connector has a metallic protective coating or not. This information can be obtained by reviewing design drawings or exercising rational engineering judgment.

All others	15
Galvanized/corrosion resistance metallic coating	0

B.8 Deck Material Type

B.8.1 Reason(s) for the Attribute

It has been noted that bridges with a timber deck tend to corrode more rapidly than those with a concrete deck. This is because a timber deck preserves rainwater for a long period and constantly provides moisture to the superstructure element underneath it. In some cases, however, a membrane is applied to the top flange of the steel beam to protect the superstructure element from moisture in the timber deck. As a result, the application of a water membrane in the steel superstructure with a timber deck significantly delays corrosion development.

B.8.2 Assessment Procedure

This attribute is assessed based on the type of deck used. If a timber deck is used, users should ensure whether a membrane is applied between the bottom of the deck and the top surface of the steel superstructure. This information may be identified by reviewing design drawings or through rational engineering observations.

Timer deck	15
All other/ timber deck with membrane applied on top surface of steel superstructure.	0

B.9 Aerostatic Vibration Effect

B.9.1 Reason(s) for the Attribute

Vibrations due to wind load, also known as aerostatic vibration, are common in cable-stayed bridges. The lateral movement of the cable due to vibrations creates excessive bending stresses at the two ends of the cable. Neoprene rings, liquid dampers, or cross cables are examples of elements that mitigate the effect of aerostatic vibrations. Engineers should implement rational engineering judgment to examine the effectiveness of the applied damping systems. Neoprene rings, for example, show a significant damping improvement of up to 0.6% compared to a typical cable without neoprene rings (Mehrabi et al., 1998). Some damping systems, such as liquid dampers, outperform those with neoprene rings in terms of damping (Mehrabi et al., 1998). The presence of cross cables also contributes to stabilizing and increasing the damping of the main cables (Lankin et al., 2004). Overall, the effectiveness of the types of damping systems used, can be measured by dynamic analysis or by exercising rational engineering judgment to determine how effectively a system can mitigate the impact of aerostatic vibration.

B.9.2 Assessment Procedure

The assessment of this attribute is based on two main aspects: the presence and the effectiveness of a system that prevents the effect of aerostatic vibration. The latter aspect could be evaluated by rational engineering judgment or by conducting dynamic analysis.

Aerostatic Vibration Effect not prevented/ ineffective damping system.	15
Aerostatic Vibration Effect moderately effective damping system.	10
Aerostatic Vibration Effect moderately effective damping system.	0

B.10 Material Type

B.10.1 Reason(s) for the Attribute

The pin in a cable connection assembly is more susceptible to corrosion due to the crevices and pockets between the connection web plates and the pin, which may retain water, salt, and debris. Consequently, the pin protective system is a primary factor influencing corrosion development in the cable connection assembly. A stainless-steel pin, for example, is commonly used in cable connection assemblies that significantly delays corrosion.

Assessment Procedure

This attribute is evaluated based on whether the pin is stainless or not. This information can be obtained by reviewing a design drawing or by exercising rational engineering judgment.

All other	15
Stainless steel	0

B.11 Types of Socket Fill Material

B.11.1 Reason(s) for the Attribute

In some types of cable connection assembly, the socket is filled with materials such as epoxy or zinc dust (NCHRP 353, 2005). The presence of these filling materials prevents moisture being trapped within the socket and the cable connection assembly. As a result, corrosion development in sockets filled with such materials is expected to be delayed.

B.11.2 Assessment Procedure

This attribute is evaluated based on whether the anchorage socket has a filler material or not. This information can be known by reviewing the design drawings.

All other	10
Zinc/Epoxy	0

B.12 Presence of Details Susceptible to Corrosion

B.12.1 Reason(s) for the Attribute

Some geometries of details are more susceptible to trapping rainwater and debris for an extended period due to a lack of drainage and a delayed evaporation process. These details are common in pedestrian bridges, particularly in aesthetically designed bridges. The presence of such details substantially accelerates the deterioration mechanism of some damage modes, such as corrosion. For example, but not limited to, end plate splices are more susceptible to corrosion if there is a gap between the plates or if they are misaligned. Therefore, users should review the design drawings of the bridge to and recognize the presence of such details.

B.12.2 Assessment Procedure

This attribute is assessed based on whether the bridge has a Details that are more susceptible to corrosion. Such information can be found by reviewing design drawings or by implementing rational engineering judgement.

Bridge does not have bad details	20
Bridge has bad details	0

B.13 Concrete Deck Construction Type

B.13.1 Reason(s) for the Attribute

Precast decks are manufactured in controlled environments, ensuring higher curing quality and consistency in the final product. Therefore, the durability of precast deck is expected to be higher than traditional cast in place deck, resulting in a delay in the delamination process.

B.13.2 Assessment Procedure

This attribute is assessed based on whether the is concrete deck is precast or cast-in-place. This information can be obtained by reviewing design drawings or by applying rational engineering judgement.

Cast-in-place concrete deck	15
Precast concrete deck	0

B.14 Deck Thickness

B.14.1 Reason(s) for Attribute

It was envisioned that a thicker deck would be more resistant to the penetration of water, chloride ions, and other aggressive substances that can degrade the concrete matrix and initiate spalling.

Thus, the development of spalling will be relatively slower in a thicker concrete deck than in a shallower one.

B.14.2 Assessment Procedure

This attribute is evaluated based on the physical thickness of the concrete deck. In some cases, the thickness of the concrete deck can be measured in the field. If the thickness cannot be measured due to lack of accessibility, NDT methods such as Ground Penetrating Radar (GPR) or Impact Echo (IE) can be implemented. Additionally, the thickness of the concrete deck can be obtained from the design drawings. However, users should verify that the actual deck thickness (as-built) matches the thickness in the design drawings.

Concrete deck thickness < 6 in.	15
Concrete deck thickness > 6 in.	0

B.15 Cable Protective Material

B.15.1 Reason(s) for Attribute

This attribute is intended to assess the presence and effectiveness of the protective material, on bridge cables. An effective cable protective material, such as galvanization, can significantly extend the service life of the cable and thus the overall integrity of the bridge structure. Furthermore, well-maintained layers of coating also contribute to delaying corrosion development. It is important, however, to verify the effectiveness of the protective coating based on its current condition and its ability to protect the cable surface from environmental effects. The absence of a protective layer, or the presence of a damaged or degraded coating, exposes the cable to harsh environmental conditions, accelerating corrosion development and potentially leading to cable failure.

B.15.2 Assessment Procedure

This attribute is evaluated based on the presence and quality of the galvanized protective material on bridge cables. Users should ensure that if galvanized protection is applied, it conforms to the ASTM A641/A641M Standard. Information regarding the application and condition of the protective layer can typically be found in the structure's design plans and inspection reports. If this information is unavailable, engineering judgment should be used to evaluate the effectiveness of the protective material used on the cable.

No protective layer present	15
No galvanized coating, but another protective layer is present/galvanized coating present but not fully conforming to ASTM A641/A641M Standard	10
Galvanized coating present and in accordance with ASTM A641/A641M Standard	0

B.16 Presence of Broken Wires in Bridge Cables

B.16.1 Reason(s) for Attribute

This attribute is designed to evaluate the presence and extent of broken wires in bridge cables. Although this attribute can be used as a screening criterion, broken wires, for pedestrian bridges, might not be a major concern unless they comprise more than 10% of the total wires in a cable section, according to expert opinions. Cables designed with recent design specifications are often designed to carry 45% of their maximum strength. However, this does not imply that cables can carry loads even if 50% of the wires are broken because of the dynamic effects that come into play when a wire is broken, generating additional force into the cable. Therefore, users should be familiar with the design specifications and the load factors that were used for designing the cable, as well as ensuring that dynamic effects are accounted for in cable design in considering this attribute. Furthermore, wire being intact at one cross-section does not assure its integrity along the entire length. Therefore, the condition of wires should be evaluated at every cross-section along the full length of the cable.

B.16.2 Assessment Procedure

This attribute should be scored based on the quantity of broken wire present in the cable. Engineering judgment should be used in determining whether the broken wire compromises the cable's capacity. The detection of broken wires can serve as a crucial early warning sign of potential cable issues, and this attribute may also be used as a screening criterion.

Significant presence of broken wires (more than 10% of wires broken any section along their length)	20
Moderate presence of broken wires (between 5% and 10% of wires broken in any section along their length)	15
Minor presence of broken wires (less than 5% of wires broken in any section along their length)	10
No broken wires present in any section along their length	0

B.17 Existing Fatigue Crack

B.17.1 Reason(s) for Attribute

Fatigue cracks may develop in the bridge cable connection assembly due to wind load vibrations. Wind loads can induce aggressive vibrations in the cables, leading to fatigue loads in the connection, especially in the lateral direction. This can result in fatigue cracks in the cable connection's welding, jeopardizing the integrity and overall stability of the connection. While this attribute can be used as a screening criterion, the initiation and growth of fatigue cracks in pedestrian bridges might not be as concerning as in highway bridges. This is because the fatigue load, which is defined by wind load according to AASHTO design specification, neither produces a large number of fatigue cycles nor significant fatigue stress.

B.17.2 Assessment Procedure

The scoring for this attribute is based on the presence or absence of fatigue cracks in the cable connections. These fatigue cracks may be located at the welded cable connections due to lateral stresses induced by wind load vibrations. Thorough inspections and engineering judgments should be applied in the assessment of these cracks.

Fatigue cracks are present and are active/unknown	20
No fatigue cracks are present/ present but have been arrested	0

B.18 Presence of Repaired Fatigue Cracks

B.18.1 Reason(s) for Attribute

This attribute is designed to evaluate the presence of repaired fatigue cracks in bridge cable connections. The presence of such repaired cracks can indicate a history of stress on the components, raising the likelihood of new crack development. Moreover, the quality and effectiveness of the repairs play a significant role in determining the component's future performance. Poorly executed repairs can introduce additional residual stress and imperfections, increasing the potential for new crack initiation.

B.18.2 Assessment Procedure

The scoring for this attribute is based on the presence and condition of repaired fatigue cracks in cable connections. The quality of the repair, the method used, and the current condition of the repaired area should all be considered. Information regarding the repairs can typically be identified from maintenance records and inspection reports. Engineering judgment should be applied when assessing the effectiveness of the repair and its potential impact on the cable's integrity and the overall bridge safety.

Repaired fatigue cracks are present and show signs of deterioration	15
Repaired fatigue cracks are present but show no signs of deterioration	5
No repaired fatigue cracks are present	0

APPENDIX C. CONSEQUENCE FACTOR CATEGORIES DESCRIPTION

Table C.1 Consequence Categories Description for Concrete Deck

<table>
<tr><td colspan="4">Assumed Damage Mode is Spalling</td></tr>
<tr><th colspan="2">Description</th><th>Sample Situations</th><th>Factor to Consider</th></tr>
<tr><td>Low</td><td>Immediate: Damage to the top of the deck does not present a safety concern for the pedestrians/cyclists on the bridge. Falling debris from the bottom of the deck does not affect the safety of the public.
Short-term: Minimal serviceability concerns may require maintenance. Little or no impact to the traveling public.</td><td>• Bridge with a concrete deck over a non-navigable waterway or unused right-of-way land</td><td rowspan="4">• Feature under
• Stay-in-place forms</td></tr>
<tr><td>Moderate</td><td>Immediate: Damage to the top of the deck presents a minimal safety concern for the pedestrians/cyclists on the bridge. Falling debris from the bottom of the deck presents a minor safety concern.
Short-term: Moderate serviceability concerns. Traffic under the bridge is moderately impacted as a result of lane, shoulder, or sidewalk closure.</td><td>• Traffic volume under the bridge is moderate where lane, shoulder, or sidewalk closure would cause minor delays of service under the bridge
• Bridge with stay-in-place forms over roadway where spalls would not reach roadway or waterway</td></tr>
<tr><td>High</td><td>Immediate: Damage to the top of the deck presents a moderate safety concern for the pedestrians/cyclists on the bridge. Falling debris from the bottom of the deck presents a moderate safety concern.
Short-term: Traffic under the bridge is greatly impacted as a result of lane, shoulder, or sidewalk closure.</td><td>• Traffic volume under the bridge is high where lane, shoulder, or sidewalk closure would cause potential delays of service under the bridge.
• Bridge without stay-in-place forms over a heavily traveled waterway or high-volume roadway</td></tr>
<tr><td>Severe</td><td>Immediate: Damage to the top of the deck presents a major safety concern for the pedestrians/cyclists on the bridge. Falling debris presents a major safety concern. Possible loss of life.
Short-term: Significant traffic delays under the bridge.</td><td>• Bridge over a feature where spalling concrete would result in lane closure of the service under the bridge, loss of life, or major traffic delays of the service under the bridge</td></tr>
</table>

Table C.2 Consequence Categories Description for Timber Deck

Assumed Damage Mode is Decay			
Description		**Sample Situations**	**Factor to Consider**
Low	**Immediate:** Damage to the top of the deck does not present a safety concern for the pedestrians/cyclists on the bridge. Falling debris does not affect the safety of the public. **Short-term**: Minimal serviceability concerns may require maintenance. Little or no impact to the traveling public.	• Bridge with low pedestrian volume over a non-navigable waterway or unused right-of-way land.	• Feature under • Pedestrian Volume • The presence of a second layer of time deck
Moderate	**Immediate**: Damage to the top of the deck presents a minimal safety concern for the pedestrians/cyclists on the bridge. Falling debris presents a minor safety concern. **Short-term:** Moderate serviceability concerns. Traffic under the bridge is moderately impacted as a result of lane, shoulder, or sidewalk closure.	• Bridge with more than one layer of timber blanks and the width of the timber board is less than 10 inches over moderately traveled roadway. • Traffic volume under the bridge is moderate where lane, shoulder, or sidewalk closure would cause minor delays of service under the bridge.	
High	**Immediate**: Damage to the top of the deck presents a moderate safety concern to the pedestrians/cyclists on the bridge. Falling debris f presents a moderate safety concern. **Short-term:** Traffic under the bridge is greatly impacted as a result of lane, shoulder, or sidewalk closure.	• Traffic volume under the bridge is high where lane, shoulder, or sidewalk closure would cause potential delays of service under the bridge. • Bridge with one layer of timber planks or the width of timber board is greater than 10 inches over moderately traveled waterway or roadway	
Severe	**Immediate:** Damage to the top of the deck presents a major safety concern for the pedestrians/cyclists on the bridge. Falling debris presents a major safety concern. Possible loss of life. **Short-term:** Significant traffic delays under the bridge.	• Bridge over feature where falling debris would result in lane closure, or major traffic delays for the service under the bridge. • Failure of a single timber board could lead to the loss of a pedestrian life.	

Table C.3 Consequence Categories Description for Steel Superstructure

<table>
<tr><td colspan="4">Assumed Damage Mode is Loss of Primary Load Carrying Member</td></tr>
<tr><th colspan="2">Description</th><th>Sample Situations</th><th>Factor to Consider</th></tr>
<tr><td>Low</td><td>Immediate: Little or no impact on structural capacity is expected based on structural analysis or documented experience. Public safety is unaffected.
Short-term: Minimal serviceability concerns may require maintenance. Little or no impact to the traveling public.</td><td>• Pedestrian bridge over non-navigable waterway or unused right-of-way land
• Pedestrian bridge in rural area with low pedestrian volume</td><td rowspan="4">• Feature under
• Redundancy
• Load carrying capacity
• Pedestrian volume
• Composite construction</td></tr>
<tr><td>Moderate</td><td>Immediate: Structural capacity is expected to remain adequate based on structural analysis or documented experience.
Short-term: Moderate serviceability concerns. Traffic under the bridge is moderately impacted as a result of lane, shoulder, or sidewalk closure.</td><td>• Bridge over multi-use path, railroad or lightly traveled waterway.
• Bridge over moderate volume urban roadway or high-volume rural roadway that would cause moderate delays for service under the bridge.</td></tr>
<tr><td>High</td><td>Immediate: Structural capacity is expected to remain adequate based on structural analysis or documented experience.
Short-term: Major serviceability concerns. Traffic under the bridge is greatly impacted as a result of lane, shoulder, or sidewalk closure.</td><td>• Bridge with alternate load path(s) that has an expectation of adequate remaining structural capacity.
• Lane or shoulder closure under roadway that would cause major delays for drivers.</td></tr>
<tr><td>Severe</td><td>Immediate: Structural collapse. Possible loss of life.
Short-term: Potential for significant traffic delays under the bridge.</td><td>• Bridge with high pedestrian volume over a roadway with high ADT/ADTT that requires closure.</td></tr>
</table>

Table C.4 Consequence Categories Description for Concrete Superstructure

<table>
<tr><td colspan="4">Assumed damage mode is rebar/strand Corrosion</td></tr>
<tr><th colspan="2">Description</th><th>Sample Situations</th><th>Factor to Consider</th></tr>
<tr><td>Low</td><td>Immediate: Little to no impact on structural capacity is expected based on structural analysis or documented experience. Falling debris does not affect the safety of the public.
Short-term: Minimal serviceability concerns may require maintenance. Little or no impact to the traveling public.</td><td>• Bridge over a non-navigable waterway or unused right-of-way land
• Bridge in a rural area with low pedestrian volume</td><td rowspan="4">• Feature under
• Redundancy
• Load carrying capacity
• Pedestrian volume
• Composite construction</td></tr>
<tr><td>Moderate</td><td>Immediate: Structural capacity is expected to remain adequate based on structural analysis or documented experience. Falling debris presents a minimal safety concern to the public.
Short-term: Moderate serviceability concerns. Traffic under the bridge is moderately impacted as a result of lane, shoulder, or sidewalk closure.</td><td>• Bridge over a multi-use path, railroad or lightly traveled waterway
• Bridge over a moderate volume urban roadway or high-volume rural roadway that would cause moderate delays for traffic under the bridge.</td></tr>
<tr><td>High</td><td>Immediate: Structural capacity is expected to remain adequate. Falling debris presents a moderate safety concern to the public.
Short-term: Major serviceability concerns. Traffic under the bridge is greatly impacted as a result of lane, shoulder, or sidewalk closure.</td><td>• Bridge with an alternate load path(s) that has an expectation of adequate remaining structural capacity
• Lane or shoulder closure that would cause major delays for traffic under the bridge.</td></tr>
<tr><td>Severe</td><td>Immediate: Structural collapse. Falling debris presents a significant safety concern to the public. Possible loss of life.
Short-term: Potential for significant traffic delays on or under bridge.</td><td>• Bridge over feature where spalling concrete would result in lane closure, loss of life, or significant traffic delays</td></tr>
</table>

Table C.5 Consequence Categories Description for Timber Superstructure

<table>
<tr><td colspan="4">Assumed damage mode is loss of primary load carrying member</td></tr>
<tr><th colspan="2">Description</th><th>Sample Situations</th><th>Factor to Consider</th></tr>
<tr><td>Low</td><td>Immediate: Little to no impact on structural capacity is expected based on structural analysis or documented experience. Falling debris does not affect the safety of the public.
Short-term: Minimal serviceability concerns may require maintenance. Little or no impact to the traveling public.</td><td>• Bridge over non-navigable waterway or unused right-of-way land
• Bridge in a rural area with low pedestrian volume</td><td rowspan="4">• Feature under
• Redundancy
• Load carrying capacity
• Pedestrian volume</td></tr>
<tr><td>Moderate</td><td>Immediate: Structural capacity is expected to remain adequate based on structural analysis or documented experience. Falling debris presents a minimal safety concern to the public.
Short-term: Moderate serviceability concerns. Traffic under the bridge is moderately impacted as a result of lane, shoulder, or sidewalk closure.</td><td>• Bridge over multi-use path, railroad or lightly traveled waterway
• Bridge over moderate volume urban roadway or high-volume rural roadway that would cause moderate delays for traffic under the bridge.</td></tr>
<tr><td>High</td><td>Immediate: Structural capacity is expected to remain adequate. Falling debris presents a moderate safety concern to the public.
Short-term: Major serviceability concerns. Traffic under the bridge is greatly impacted as a result of lane, shoulder, or sidewalk closure.</td><td>• Bridge with an alternate load path(s) that has an expectation of adequate remaining structural capacity
• Lane or shoulder closure that would cause major delays for traffic under the bridge.</td></tr>
<tr><td>Severe</td><td>Immediate: Structural collapse. Falling debris presents a significant safety concern to the public. Possible loss of life.
Short-term: Potential for significant traffic delays on or under the bridge.</td><td>• Bridge over a feature where spalling concrete would result in lane closure, loss of life, or significant traffic delays</td></tr>
</table>

Table C.6 Consequence Categories Description for Steel Superstructure/Cable Failure

<table>
<tr><td colspan="4">Assumed damage cable/wire failure</td></tr>
<tr><th colspan="2">Description</th><th>Sample Situations</th><th>Factor to Consider</th></tr>
<tr><td>Low</td><td>Immediate: Little or no impact on structural capacity is expected based on structural analysis considering the dynamic effect of cable failure or documented experience. Public safety is unaffected.
Short-term: Minimal serviceability concerns may require maintenance. Little or no impact to the traveling public.</td><td>• Covered pedestrian bridge over non-navigable waterway or unused right-of-way land
• Pedestrian bridge in a rural area with low pedestrian volume</td><td rowspan="4">• Feature under
• Redundancy
• Load carrying capacity
• Cable is designed to be replaced one at time</td></tr>
<tr><td>Moderate</td><td>Immediate: Structural capacity is expected to remain adequate based on structural analysis considering the dynamic effect of cable failure or documented experience.
Short-term: Moderate serviceability concerns. Traffic under the bridge is moderately impacted as a result of lane, shoulder, or sidewalk closure.</td><td>• Bridge over multi-use path, railroad or lightly traveled waterway
• Bridge over moderate volume urban roadway or high-volume rural roadway that would cause moderate delays for drivers.</td></tr>
<tr><td>High</td><td>Immediate: Structural capacity is expected to remain adequate based on structural analysis considering the dynamic effect of cable failure or documented experience.
Short-term: Major serviceability concerns. Traffic under the bridge is greatly impacted as a result of lane, shoulder, or sidewalk closure.</td><td>• Bridge with alternate load path(s) that has an expectation of adequate remaining structural capacity.
• Lane or shoulder closure under roadway that would cause major delays for drivers.</td></tr>
<tr><td>Severe</td><td>Immediate: Structural collapse. Possible loss of life.
Short-term: Potential for significant traffic delays under the bridge.</td><td>• Bridge with high pedestrian volume over a roadway with high ADT/ADTT that requires closure.</td></tr>
</table>

Table C.7 Substructure Consequence Categories Description

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

APPENDIX D. INSPECTION EFFECTIVENESS FACTOR CATEGORIES DESCRIPTION

D.1 Discussion on Inspection Effectiveness Factor (IEF)

Table D.1 through Table D.11 represent the results of the Indiana Risk Assessment Panel (RAP) and NCHRP 782 on the effectiveness of different inspection techniques in detecting certain damage modes for different bridge components. Users can change the rank for each inspection technique if they believe it may not represent the performance level of their inspection program. The first column from the left represents five levels of inspection techniques. Generic descriptions of each inspection technique are shown on the third column with some approximate probability of detection on the fourth column. The value of IEF for each level is given on the fifth column which is then used to calculate the Risk Priority Number (RPN).

D.2 Factor to Consider in Evaluating the IEF

The determination of the IEF depends on two primary aspects: the reliability of the inspection technique and the accessibility and visibility of the component. It is noteworthy that some inspection techniques have inherent uncertainty when detecting specific defects. For instance, considering hands-on inspection to detect fatigue cracks in a pin-hanger assembly may not be as reliable as utilizing ultrasonic testing. In some situations, however, utilizing a reliable inspection technique may not assure sufficient detection reliability unless the component is adequately accessible. For example, using visual inspection to detect corrosion in cable connection assemblies might be less effective if the assembly is covered with a socket. Consequently, the IEF depends on both the reliability of the inspection technique in detecting the considered damage mode and the accessibility of the component.

Table D.1 IEF for concrete deck, spalling damage mode

Inspection Category					
HE	Highly Effective	Defect is Always Detected	80–100	0.5	Hand-On Visual
ME	Moderately Effective	Defect is Detected and Rarely Missed	60–80	0.8	Sounding, IR, GPR, Impact Echo
FE	Fairly Effective	Defect Should be Detected by a Certified Inspector	40–60	1	Routine Visual, Chain Drag
LE	Low Effective	Defect is Difficult to be Detected Through Visual Inspection	20–40	1.5	–
NE	Not Effective/ Item is Not Accessible	Defect Cannot be Detected Through Visual Inspection/ Component is not Accessible	<20	2	–

Table D.2 IEF for concrete deck, corrosion damage mode

Inspection Category	Inspection Effectiveness Category	Description	Approximate Probability of Detection	Inspection Effectiveness Factor	Example of Some Inspection Techniques
HE	Highly Effective	Defect is Always Detected	80–100	0.5	–
ME	Moderately Effective	Defect is Detected and Rarely Missed	60–80	0.8	Sounding, GPR, Impact Echo
FE	Fairly Effective	Defect Should be Detected by a Certified Inspector	40–60	1	IR, chain Drag
LE	Less Effective	Defect is Difficult to be Detected Through Visual Inspection	20–40	1.5	Routine Visual, Hands-On Visual
NE	Not Effective/ Item is Not Accessible	Defect Cannot be Detected Through Visual Inspection/ Component is not Accessible	<20	2	–

Table D.3 IEF for Timber deck, decay damage mode

Inspection Category	Inspection Effectiveness Category	Description	Approximate Probability of Detection	Inspection Effectiveness Factor	Example of Some Inspection Techniques
HE	Highly Effective	Defect is Always Detected	80–100	0.5	–
ME	Moderately Effective	Defect is Detected and Rarely Missed	60–80	0.8	Hands-On Visual
FE	Fairly Effective	Defect Should be Detected by a Certified Inspector	40–60	1	Routine Visual
LE	Less Effective	Defect is Difficult to be Detected Through Visual Inspection	20–40	1.5	–
NE	Not Effective/ Item is Not Accessible	Defect Cannot be Detected Through Visual Inspection/ Component is not Accessible	<20	2	–

Table D.4 IEF for steel superstructure, corrosion damage mode

Inspection Category	Inspection Effectiveness Category	Description	Approximate Probability of Detection	Inspection Effectiveness Factor	Example of Some Inspection Techniques
HE	Highly Effective	Defect is Always Detected	80–100	0.5	UT-T
ME	Moderately Effective	Defect is Detected and Rarely Missed	60–80	0.8	Hand-On Visual
FE	Fairly Effective	Defect Should be Detected by a Certified Inspector	40–60	1	Routine Visual
LE	Less Effective	Defect is Difficult to be Detected Through Visual Inspection	20–40	1.5	–
NE	Not Effective/ Item is Not Accessible	Defect Cannot be Detected Through Visual Inspection/ Component is not Accessible	<20	2	–

Table D.5 IEF for cable/wire, corrosion damage mode

Inspection Category	Inspection Effectiveness Category	Description	Approximate Probability of Detection	Inspection Effectiveness Factor	Example of Some Inspection Techniques
HE	Highly Effective	Defect is Always Detected	80–100	0.5	UT-T
ME	Moderately Effective	Defect is Detected and Rarely Missed	60–80	0.8	Hand-On Visual
FE	Fairly Effective	Defect Should be Detected by a Certified Inspector	40–60	1	Routine Visual
LE	Less Effective	Defect is Difficult to be Detected Through Visual Inspection	20–40	1.5	–
NE	Not Effective/Item is Not Accessible	Defect Cannot be Detected Through Visual Inspection/ Component is not Accessible	<20	2	–

Table D.6 IEF for cable/wire, broken wire mode

Inspection Category	Inspection Effectiveness Category	Description	Approximate Probability of Detection	Inspection Effectiveness Factor	Example of Some Inspection Techniques
HE	Highly Effective	Defect is Always Detected	80–100	0.5	PT, MT
ME	Moderately Effective	Defect is Detected and Rarely Missed	60–80	0.8	Hands-On visual
FE	Fairly Effective	Defect Should be Detected by a Certified Inspector	40–60	1	–
LE	Less Effective	Defect is Difficult to be Detected Through Visual Inspection	20–40	1.5	Routine Visual
NE	Not Effective/Item is Not Accessible	Defect Cannot be Detected Through Visual Inspection/ Component is not Accessible	<20	2	–

Table D.7 IEF for connection assembly, corrosion damage mode

Inspection Category	Inspection Effectiveness Category	Description	Approximate Probability of Detection	Inspection Effectiveness Factor	Example of Some Inspection Techniques
HE	Highly Effective	Defect is Always Detected	80–100	0.5	Hands-On Visual
ME	Moderately Effective	Defect is Detected and Rarely Missed	60–80	0.8	–
FE	Fairly Effective	Defect Should be Detected by a Certified Inspector	40–60	1	Routine Visual, UT-T
LE	Less Effective	Defect is Difficult to be Detected Through Visual Inspection	20–40	1.5	–
NE	Not Effective/Item is Not Accessible	Defect Cannot be Detected Through Visual Inspection/ Component is not Accessible	<20	2	–

Inspection Category	Inspection Effectiveness Category	Description	Approximate Probability of Detection	Inspection Effectiveness Factor	Example of Some Inspection Techniques
HE	Highly Effective	Defect is Always Detected	80–100	0.5	–
ME	Moderately Effective	Defect is Detected and Rarely Missed	60–80	0.8	Hands-On Visual
FE	Fairly Effective	Defect Should be Detected by a Certified Inspector	40–60	1	Routine Visual
LE	Less Effective	Defect is Difficult to be Detected Through Visual Inspection	20–40	1.5	–
NE	Not Effective/Item is Not Accessible	Defect Cannot be Detected Through Visual Inspection/ Component is not Accessible	<20	2	–

Table D.8 IEF for cable connection assembly, fatigue damage mode

Inspection Category	Inspection Effectiveness Category	Description	Approximate Probability of Detection	Inspection Effectiveness Factor	Example of Some Inspection Techniques
HE	Highly Effective	Defect is Always Detected	80–100	0.5	UT
ME	Moderately Effective	Defect is Detected and Rarely Missed	60–80	0.8	–
FE	Fairly Effective	Defect Should be Detected by a Certified Inspector	40–60	1	–
LE	Less Effective	Defect is Difficult to be Detected Through Visual Inspection	20–40	1.5	Hands-On Visual
NE	Not Effective/ Item is Not Accessible	Defect Cannot be Detected Through Visual Inspection/ Component is not Accessible	<20	2	Visual-Routine

Inspection Category	Inspection Effectiveness Category	Description	Approximate Probability of Detection	Inspection Effectiveness Factor	Example of Some Inspection Techniques
HE	Highly Effective	Defect is Always Detected	80–100	0.5	–
ME	Moderately Effective	Defect is Detected and Rarely Missed	60–80	0.8	RT
FE	Fairly Effective	Defect Should be Detected by a Certified Inspector	40–60	1	Hand-On Visual, MFL
LE	Less Effective	Defect is Difficult to be Detected Through Visual Inspection	20–40	1.5	Routine Visual
NE	Not Effective/ Item is Not Accessible	Defect Cannot be Detected Through Visual Inspection/ Component is not Accessible	<20	2	–

Table D.9 IEF for concrete substructure, spalling damage mode

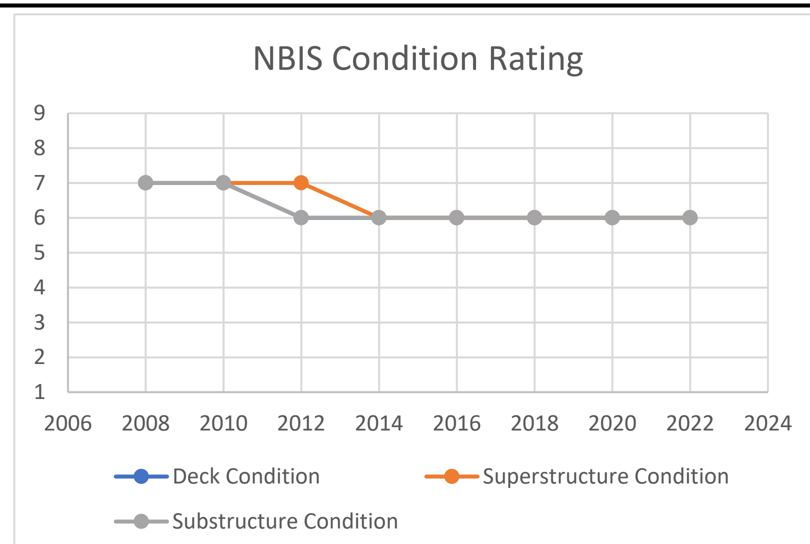
Inspection Category	Inspection Effectiveness Category	Description	Approximate Probability of Detection	Inspection Effectiveness Factor	Example of Some Inspection Techniques
HE	Highly Effective	Defect is Always Detected	80–100	0.5	Hand-On Visual
ME	Moderately Effective	Defect is Detected and Rarely Missed	60–80	0.8	Sounding-IR-GPR-Impact Echo
FE	Fairly Effective	Defect Should be Detected by a Certified Inspector	40–60	1	Routine Visual-Chain Drag
LE	Less Effective	Defect is Difficult to be Detected Through Visual Inspection	20–40	1.5	–
NE	Not Effective/ Item is Not Accessible	Defect Cannot be Detected Through Visual Inspection/ Component is not Accessible	<20	2	–

APPENDIX E. BACK-CASTING ANALYSIS

Bridge #	PI65-116-05940 A
Deck Type	Concrete
Superstructure Type	two girders
Year of Construction	1969
Year of Reconstruction	1996
Maximum Span Length (ft)	142.3
Service Under the Bridge	Highway
Key Damage Mode	Corrosion

Year	2008	2010	2012
Deck Condition	7	7	6
Superstructure Condition	7	7	7
Substructure Condition	7	7	6
Inspection Interval	48 months		

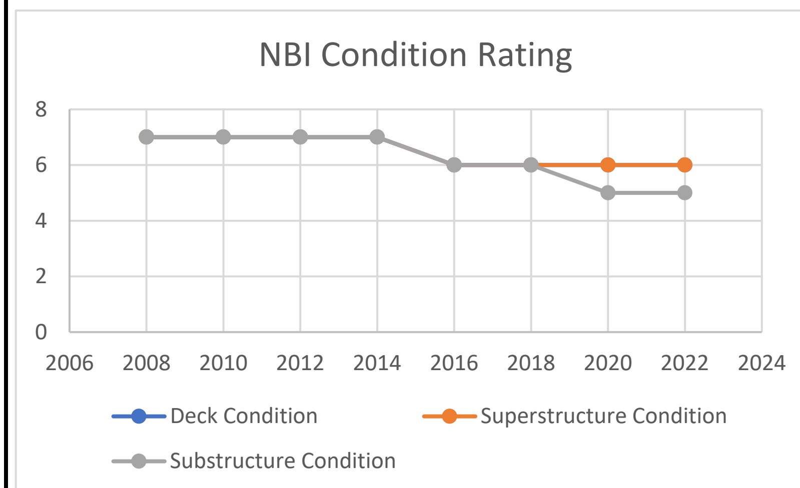
Year	2014	2016	2018	2020
Deck Condition	6	6	6	6
Superstructure Condition	6	6	6	6
Substructure Condition	6	6	6	6
Inspection Interval	48 months			



Bridge #	P066-82-07440
Deck Type	Concrete
Superstructure Type	Two girders
Year of Construction	1988
Maximum Span Length (ft)	117
Service Under the Bridge	Highway
Key Damage Mode	Corrosion

Year	2008	2010	2012
Deck Condition	7	7	7
Superstructure Condition	7	7	7
Substructure Condition	7	7	7
Inspection Interval	48 months		

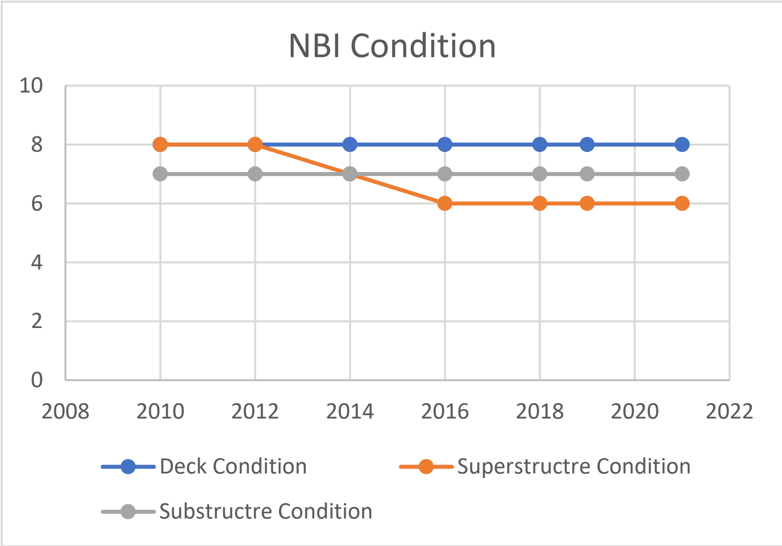
Year	2014	2016	2018	2020
Deck Condition	7	6	6	6
Superstructure Condition	7	6	6	6
Substructure Condition	7	6	6	5
Inspection Interval	48 months			



Bridge #	16-0115H
Deck Type	Concrete
Superstructure Type	Truss
Year of Construction	1915
Year of Reconstruction	2008
Maximum Span Length (ft)	119.4
Service Under the Bridge	Waterway
Key Damage Mode	Corrosion

Year	2010	2012	2014
Deck Condition	8	8	8
Superstructure Condition	8	8	7
Substructure Condition	7	7	7
Inspection Interval	48 months		

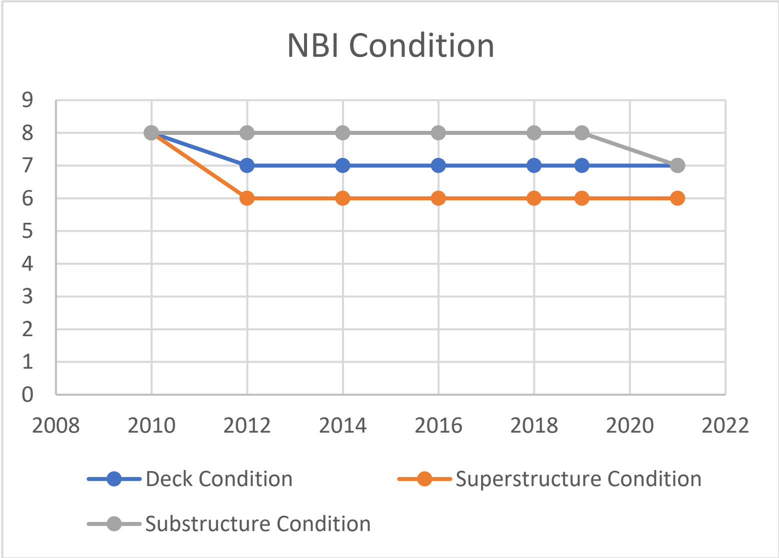
Year	2016	2018	2019	2021
Deck Condition	8	8	8	8
Superstructure Condition	6	6	6	6
Substructure Condition	7	7	7	7
Inspection Interval	48 months			



Bridge #	16-0140H
Deck Type	Concrete
Superstructure Type	Truss
Year of Construction	1915
Year of Reconstruction	2006
Maximum Span Length (ft)	177
Service Under the Bridge	Waterway
Key Damage Mode	Corrosion

Year	2010	2012	2014
Deck Condition	8	7	7
Superstructure Condition	8	6	6
Substructure Condition	8	8	8
Inspection Interval	48 months		

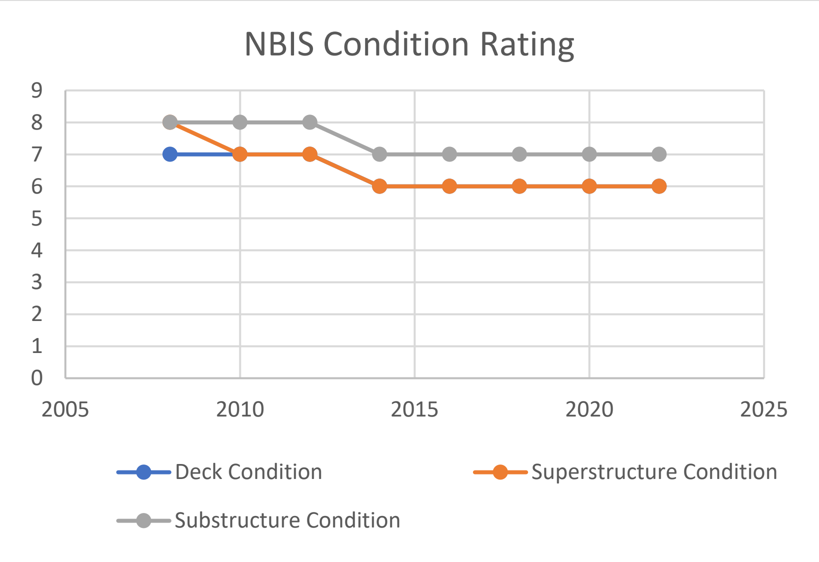
Year	2016	2018	2019	2021
Deck Condition	7	7	7	7
Superstructure Condition	6	6	6	6
Substructure Condition	8	8	8	7
Inspection Interval	48 months			



Bridge #	P066-82-06989
Deck Type	Concrete
Superstructure Type	Truss
Year of Construction	1987
Year of reconstruction	2006
Maximum Span Length (ft)	176
Service Under the Bridge	Highway
Key Damage Mode	Corrosion

Year	2008	2010	2012
Deck Condition	7	7	7
Superstructure Condition	8	7	7
Substructure Condition	8	8	8
Inspection Interval	48 months		

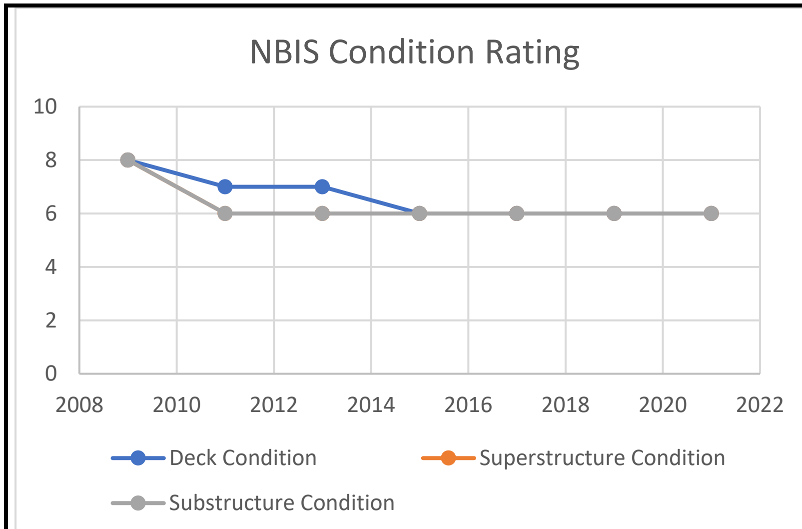
Year	2014	2016	2018	2020
Deck Condition	6	6	6	6
Superstructure Condition	6	6	6	6
Substructure Condition	7	7	7	7
Inspection Interval	48 months			



Bridge #	15-00095
Deck Type	Concrete
Superstructure Type	Truss
Year of Construction	1878
Year of Reconstruction	2008
Maximum Span Length (ft)	297.5
Service Under the Bridge	Waterway
Key Damage Mode	Corrosion

Year	2009	2011	2013
Deck Condition	8	7	7
Superstructure Condition	8	6	6
Substructure Condition	8	6	6
Inspection Interval	48 months		

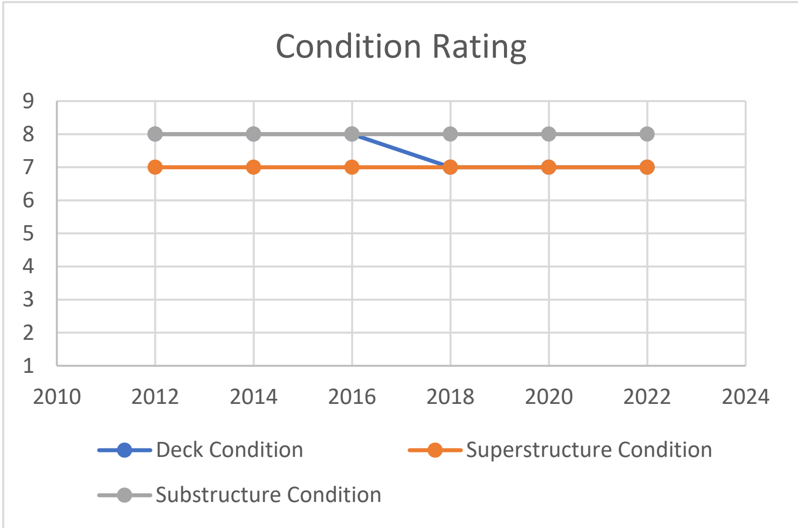
Year	2015	2017	2019	2021
Deck Condition	6	6	6	6
Superstructure Condition	6	6	6	6
Substructure Condition	6	6	6	6
Inspection Interval	48 months			



Bridge #	15-00095
Deck Type	Concrete
Superstructure Type	Truss
Year of Construction	1878
Year of Reconstruction	2012
Maximum Span Length (ft)	159.7
Service Under the Bridge	Waterway
Key Damage Mode	Corrosion

Year	2012	2014	2016
Deck Condition	8	8	8
Superstructure Condition	7	7	7
Substructure Condition	8	8	8
Inspection Interval	48 months		

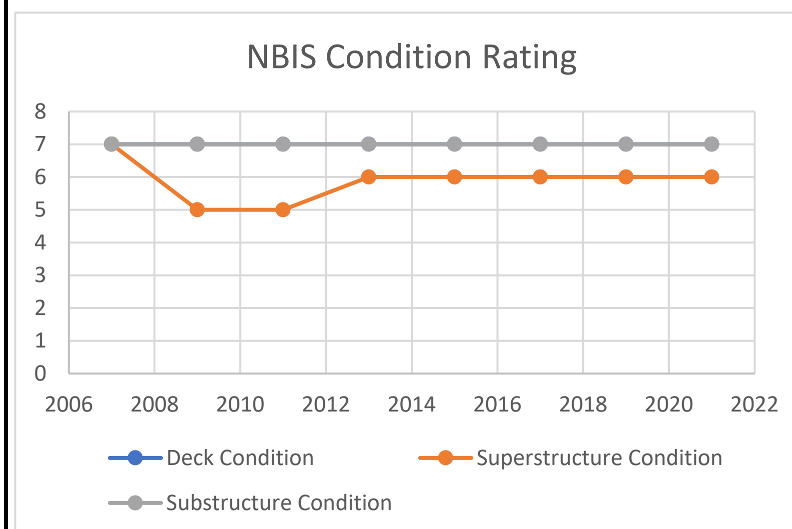
Year	2018	2020	2022
Deck Condition	7	7	7
Superstructure Condition	7	7	7
Substructure Condition	8	8	8
Inspection Interval	48 months		



Bridge #	P027-89-07710 SBL
Deck Type	Concrete
Superstructure Type	Concrete
Year of Construction	1976
Maximum Span Length (ft)	74
Service Under the Bridge	roadway
Key Damage Mode	Spalling, strands corrosion

Year	2007	2009	2011	2013	2015
Deck Condition	7	7	7	7	7
Superstructure Condition	7	5	5	6	6
Substructure Condition	7	7	7	7	7
Inspection Interval	48 months				

Year	2017	2019	2021
Deck Condition	7	7	7
Superstructure Condition	6	6	6
Substructure Condition	7	7	7
Inspection Interval	48 months		



About the Joint Transportation Research Program (JTRP)

On March 11, 1937, the Indiana Legislature passed an act which authorized the Indiana State Highway Commission to cooperate with and assist Purdue University in developing the best methods of improving and maintaining the highways of the state and the respective counties thereof. That collaborative effort was called the Joint Highway Research Project (JHRP). In 1997 the collaborative venture was renamed as the Joint Transportation Research Program (JTRP) to reflect the state and national efforts to integrate the management and operation of various transportation modes.

The first studies of JHRP were concerned with Test Road No. 1—evaluation of the weathering characteristics of stabilized materials. After World War II, the JHRP program grew substantially and was regularly producing technical reports. Over 1,600 technical reports are now available, published as part of the JHRP and subsequently JTRP collaborative venture between Purdue University and what is now the Indiana Department of Transportation.

Free online access to all reports is provided through a unique collaboration between JTRP and Purdue Libraries. These are available at http://docs.lib.purdue.edu/jtrp.

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An open access version of this publication is available online. See the URL in the citation below.

Alharthi, A. A., & Connor, R. J. (2024). *Development of a formalized program for in-service inspection of pedestrian bridges* (Joint Transportation Research Program Publication No. FHWA/IN/JTRP-2024/17). West Lafayette, IN: Purdue University. https://doi.org/10.5703/1288284317750