TRANSPORTATION RESEARCH

CIRCULAR

Number E-C224 November 2017

Eleventh International Bridge and Structures Management Conference

April 26–27, 2017
Mesa, Arizona

The National Academies of
SCIENCES · ENGINEERING · MEDICINE
TRB
TRANSPORTATION RESEARCH BOARD

TRANSPORTATION RESEARCH CIRCULAR E-C224

Eleventh International Bridge and Structures Management Conference

April 26–27, 2017
Mesa, Arizona

Organized by
Transportation Research Board
Standing Committee on Bridge Management

Sponsored by
Transportation Research Board

Cosponsored by
Federal Highway Administration
Oregon Department of Transportation
Arizona Departments of Transportation
AASHTOWare

October 2017

Transportation Research Board
500 Fifth Street, NW
Washington, D.C.
www.TRB.org

TRANSPORTATION RESEARCH CIRCULAR E-C224
ISSN 0097-8515

Transportation Research Board
500 Fifth Street, NW
Washington, D.C.
www.TRB.org

Jennifer Correro, Proofreading and Layout; Lea Camarda, Production Editor

Preface

The Standing Committee on Bridge Management (AHD35) of the Transportation Research Board (TRB) sponsored the 11th International Bridge and Structures Management Conference at the Sheraton Mesa Hotel in Mesa, Arizona, April 26–27, 2017. Also sponsored by FHWA, Oregon Department of Transportation (DOT), Arizona DOT, and AASHTOWare, the conference provided a 2-day program that focused on the impacts of the new Federal Transportation Asset Management Rule and the proposed Federal Performance Measures. Four workshops highlighted the federal rules, implementation of AASHTOWare Bridge Management (BrM) software to help meet those rules and incorporate multiobjective analysis in bridge program planning, and how to use the *NCHRP Guide on Return on Investment from Use of Management Systems*. The conference also highlighted the advancement of asset management in Europe and South America.

The 11th International Bridge and Structures Management Conference included presentations describing papers that were prepared for the conference as well as presentations that were not documented with written papers. All of the papers prepared for the conference are included in this e-circular. For easy reference, the following paragraphs provide a summary of the two previous International Bridge and Structures Conferences, in which papers also were prepared and published. The papers from previous conferences are published as e-circulars and are titled for the conference at which they were presented.

The 10th International Bridge and Structures Management Conference (*Transportation Research E-Circular 128: 10th International Bridge Management Conference*) was held October 20–22, 2008, in Buffalo, New York. This conference was conducted by the TRB Bridge Management committee and the Structures Maintenance committees in cooperation with FHWA, New York State DOT, New York State Thruway Authority, New York State Bridge Authority, Multidisciplinary Center for Earthquake Engineering Research, and the State University of New York at Buffalo.

The 9th conference was held in Orlando, Florida, in 2003. Presentations from the 9th Conference were published in *Transportation Research E-Circular 049: 9th International Bridge Management Conference*. The 9th conference included papers and presentations on future directions and challenges in structures management; design and implementation of bridge management systems; application of bridge management in transportation agencies; bridge preservation, maintenance, and deterioration rates; application of prioritization and optimization routines; structural performance, monitoring, and remaining life; bridge modeling and National Bridge Inventory translator; structure vulnerability and weigh-in-motion; bridge inspection; local, frequency, and thermal imaging; bridge decks and stay cable; and accelerated construction, fiber-reinforced polymers, and corrosion evaluation. The conference was developed to help bridge practitioners, managers, and researchers take advantage of the characteristics of existing systems from around the world, and identifying new and anticipated enhancements.

Contents

Implementation of Bridge and Structure Management Programs and Processes

The Evolution of Structure Asset Management in Wisconsin
Practice and Research

JOSHUA DIETSCHE
Wisconsin Department of Transportation

Beginning in the early 2000s, the Wisconsin Department of Transportation (DOT) Bureau of Structures began a concerted effort to develop processes and tools to help manage the Wisconsin structures inventory. The first major step was the development of a data management tool, the Highway Structures Information System. This application provides Wisconsin DOT with a means to collect, store, and manage structure inventory, design, rating, and inspection data.

A second step was aimed at documenting and standardizing bridge preservation practices across the state. Organizationally, Wisconsin DOT divides the state into five regions. Each has their own maintenance, planning, and scoping staff, with oversight from Wisconsin DOT central office. To promote consistency amongst the regions, Wisconsin DOT created the Bridge Preservation Policy Guide. This guide provides an inventory of preservation actions and also addresses goals, objectives, and performance measures. The aim is to lay the groundwork for more consistent bridge work activities (maintenance, rehabilitation, replacement) around the state.

Most recently, Wisconsin DOT has focused on developing a tool to provide recommendations for current and future bridge work actions. The result of this work is the Wisconsin Structures Asset Management System (WiSAMS). WiSAMS relies heavily on the inventory data and inspection data stored in HSIS and uses a set of rules and deterioration modeling to determine current and future optimal work. The WiSAMS rules are a logical extension of policy in the Bridge Preservation Policy Guide.

As a whole, WiSAMS and Wisconsin DOT's structures asset management program represent a substantial step forward into better management of the Wisconsin structures inventory.

The Moving Ahead for Progress in the 21st Century (MAP-21) legislation contains the following definition for asset management (*1*):

> Asset Management is a strategic and systematic process of operating, maintaining, and improving physical assets, with a focus on both engineering and economic analysis based upon quality information, to identify a structured sequence of maintenance, preservation, repair, rehabilitation, and replacement actions that will achieve and sustain a desired state of good repair over the life-cycle of the assets at minimum practicable cost.

The Wisconsin Department of Transportation (DOT) has developed and is implementing a structures asset management program that meets FHWA's definition. At its most basic level, structures asset management is practiced as shown in Figure 1. The process is continually refreshing itself and is dependent on quality data.

Inventory and inspection data are the foundation necessary to implement a systematic approach to maximizing the life of a given structure. Wisconsin DOT has developed tools to assist with the collection, storage, and manipulation of data. The next step is the derivation of data-driven recommendations for structures work that will optimize the amount of usable life for a given structure. Wisconsin DOT accomplishes this using recently developed software tools. Next is the implementation of those work recommendations by Wisconsin DOT regional planning, scoping, and project development personnel. When these projects are complete, the structure is opened (or reopened) to traffic and must be inspected at regular intervals per FHWA

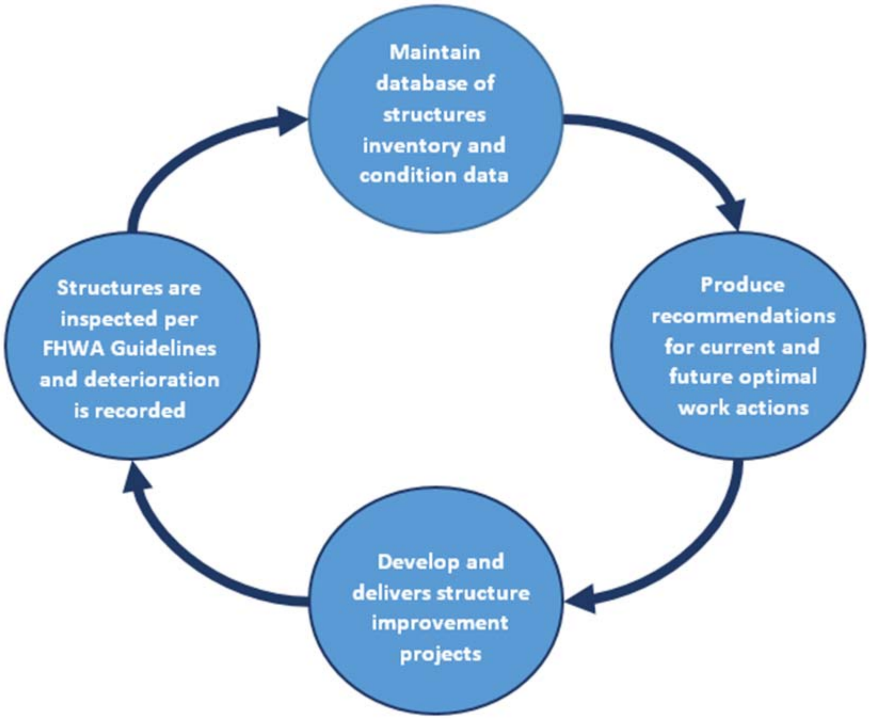

FIGURE 1 Basic management cycle.

guidelines. These inspections record the current condition of the structure, which is collected, stored, and used to help produce recommendations for future structure work—starting the whole process again. This paper documents the process, procedures, and tools that Wisconsin DOT has developed in order to implement a modern structures asset management program.

WISCONSIN BRIDGE INVENTORY

To set the context for Wisconsin DOT asset management practice, it is helpful to understand the nature of current Wisconsin bridge inventory. Per the 2016 Wisconsin DOT Bureau of Structures Annual Bridge Report (*2*), there are 14,116 bridges in the state. Roughly 1/3 of the inventory is state owned, 2/3 local owned, with the specific breakdown shown in Figure 2. Also shown is a breakdown of state-owned bridges by superstructure type. Prestressed girder bridges are the dominant superstructure type in Wisconsin. Most new bridges constructed in Wisconsin are either prestressed I-girder or haunched concrete slab structures, depending on span length.

In general, the Wisconsin bridge inventory performs very well as measured by MAP-21 and FHWA metrics. As seen in Figure 3, Wisconsin has a very small percentage of bridges and bridge decks rated "poor," which is defined as National Bridge Inventory (NBI) condition rating of four or less. A bridge is rated poor if either the superstructure or substructure is in the poor range. With limited monetary resources and substantial needs, the challenge for Wisconsin DOT's structures asset management program in coming years is to maintain the performance displayed below.

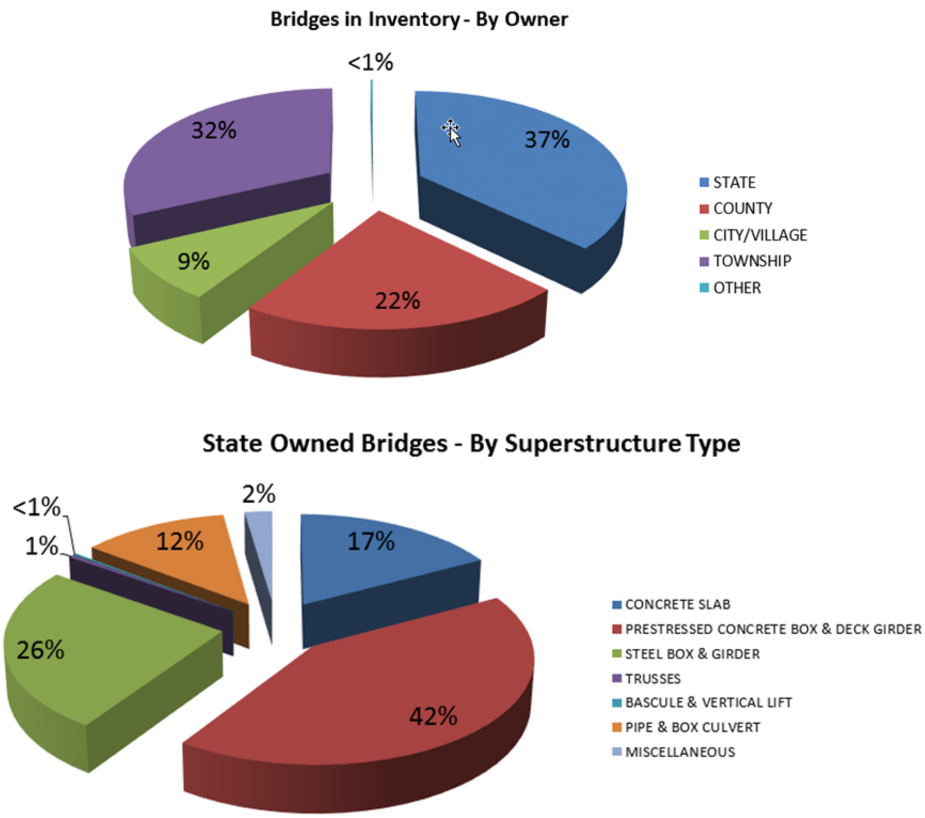

FIGURE 2 Wisconsin bridge inventory: owner and superstructure type (*2*).

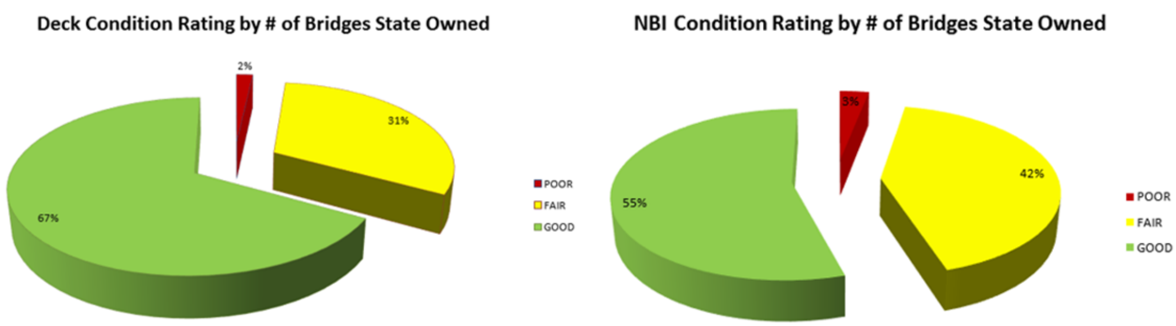

FIGURE 3 NBI condition rating of state-owned Wisconsin bridges and bridge decks by number of bridges (*2*).

Bridge age and bridge deck age are not parameters that speak directly to condition. However, in an environment like Wisconsin, with hot, humid summers, and cold, icy winters, deterioration of the infrastructure is not a question of "if," but "when" and "how fast." Asset management strategies are aimed at maintaining bridges and bridge decks in serviceable condition as long as possible. Figures 4 and 5 show the current age statistics of the Wisconsin bridge inventory.

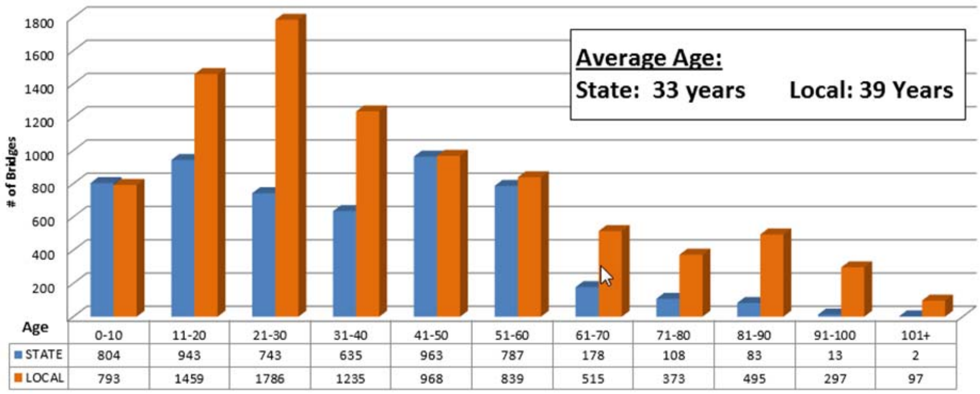

FIGURE 4 Average bridge age, Wisconsin bridge inventory (*2*).

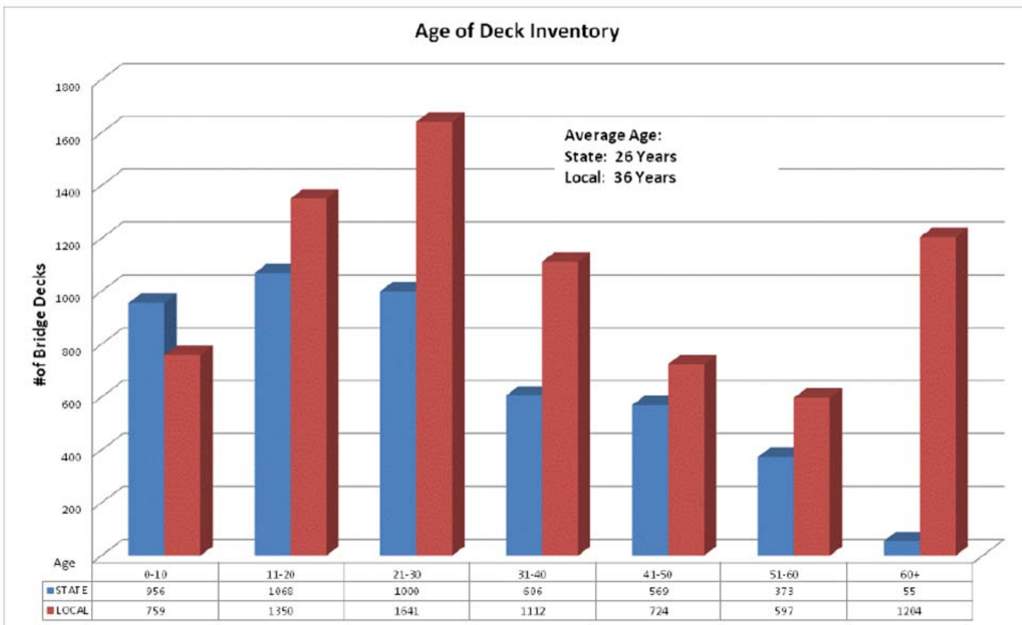

FIGURE 5 Average bridge deck age, Wisconsin bridge inventory (*2*).

WISCONSIN DOT ORGANIZATIONAL STRUCTURE

In order to understand the challenges facing Wisconsin DOT and the implementation of a networkwide structures asset management program, it is necessary to understand the organizational structure. Wisconsin DOT is divided into a central office with statewide responsibilities and regional offices that focus on their portion of the state. Wisconsin DOT divides the state into five regions, as seen in Figure 6. A brief description of the primary contributors in the Wisconsin DOT structures asset management process follows.

FIGURE 6 Wisconsin DOT regions.

Department of Transportation Investment Management

Wisconsin DOT Investment Management (DTIM) is the financial arm of Wisconsin DOT, working with the state transportation budget and determining the allocation of funds for structures improvement projects. DTIM creates the policy on how funds are allocated to the various Wisconsin DOT funding programs, as well as individual major projects. DTIM uses recommendations from the Bureau of Structures on structures repair, rehabilitation, and replacement needs, as well as input from regional planning and scoping to help guide their decisions on how best to allocate funds.

Bureau of Structures

Bureau of Structures (BOS) houses the structural engineering expertise for Wisconsin DOT. The BOS design section is responsible for performing structures design and consultant design oversight. The BOS maintenance section provides oversight for the Wisconsin DOT inspection and fabrication programs. The BOS development section houses a number of activities: software development, research, load rating, curating design policy, and updating the Wisconsin DOT Bridge Manual. The development section also contains the bridge management unit, which provides the technical support for structures asset management. This is where inspection and inventory data is collected, stored, and analyzed to produce recommendations for structures improvement projects.

Regional Planning and Scoping Units

Planning and scoping units from each region are responsible for deciding how allocated funds will be spent. The process takes on added complexity for major projects, but in general, regional and scoping units work to most effectively address structure inventory needs with available dollars. Their work is supported by consultation with DTIM and coordination with BOS, but final responsibility for planning and scoping structures improvement projects lies within the individual regions.

Regional Project Development Sections

Regional Project Development Sections (PDS) are the staff responsible for taking a project that has been planned and scoped and guiding it through letting and construction to completion. They prepare schedules, negotiate contracts, provide construction oversight, and work to ensure that the project is constructed per plans and specifications. Regional PDS consults with BOS for structural expertise as necessary.

Regional Operations

Regional operations contain the maintenance units that house the state bridge inspectors and are responsible for inspecting every state-owned bridge at a regular intervals, logging condition information, and offering their insight on structures improvement needs. They work with local bridge owners to ensure compliance with bridge inspection standards. As time allows, regional maintenance also assists with small repair projects and performs some routine maintenance work.

BUILDING A FOUNDATION FOR MODERN ASSET MANAGEMENT

Data Management Tools

In order to implement a modern structures asset management program, improvements were needed in the tools used to collect, store, and manipulate the bridge data. Idealized bridge management is not performed bridge-by-bridge, but rather at a network level. Wisconsin has more than 5,000 state-owned bridges, a large amount of data to store and manipulate manually. As computer technology evolved, Wisconsin DOT moved from paper to digital files, utilizing a mainframe database to store bridge data. Limited by the available technology, the system was accessible only in the Wisconsin DOT central office, so data entry and extraction was difficult and time-consuming. Collection of data was a manual, time-intensive process and subject to error. The data management system as a whole was not conducive to network-level structures asset management.

With the advent of the Internet, a more-connected accessible database became a possibility. Wisconsin DOT took advantage of this technological advance by developing a new database to collect and store structure inventory and inspection data. The Highway Structures Information System (HSIS) was initially developed in the early 2000s by collaboration of a computer software programmer and Wisconsin DOT bridge management engineers. Continuing this relationship, HSIS has continued to be enhanced to meet Wisconsin DOT needs. Some notable features of HSIS include:

- A web-based interface in order to be widely accessible for parties both internal and external to Wisconsin DOT;
- Live updates as new inventory information is input or new inspections are uploaded, immediately available to access;

- The ability for bridge inspectors to upload inspections directly to the HSIS site;
- Compatibility with multiple Internet browsers and mobile devices (tablets and smartphones);
- The ability for any user to select parameters from dropdown menus and query information from the database;
- The ability to easily create and make available customized reports to meet various business area needs;
- A portal to access bridge plans, shop drawings, and other design documents; and
- Compatibility with other applications used by Wisconsin DOT for easy transfer of data.

The creation of HSIS was a necessary and critical step on the path toward structures asset management. HSIS give Wisconsin DOT the capability to collect, store, and manipulate all the data necessary for structures asset management activities.

Bridge Preservation Policy

From MAP-21 legislation, asset management aims to "…identify a structured sequence of maintenance, preservation, repair, rehabilitation, and replacement actions that will achieve and sustain a desired state of good repair over the life-cycle of the assets at minimum practicable cost" (*1*). Implicit in this definition is the development of policy to document standard practice for the actions noted: maintenance, preservation, repair, rehabilitation, and replacement. Wisconsin DOT addressed this with the creation of the BOS Bridge Preservation Policy Guide (BPPG) (*3*). Completed in the summer of 2015, the BPPG serves as the basis for optimal treatment decisions regarding state-owned bridges. It offers a statewide baseline for planning and scoping bridge projects over the life cycle of the structure. The BPPG represents a collaborative effort among members of the BOS development and maintenance sections, as well as regional bridge inspection program managers. In addition, BOS elicited input from a consultant subject matter expert and coordinated the final version of the document with Wisconsin DOT's FHWA liaison. In creating the BPPG, Wisconsin DOT aimed to address the following:

- Establish goals and strategies for bridge preservation;
- Create bridge preservation-specific objectives and related performance measures; and
- Identify a set of bridge preservation activities to meet program goals and establish eligibility criteria for each.

Goals and Strategies for Bridge Preservation

As stated in the BPPG, "the main goal of a bridge preservation program is to maximize the useful life of bridges in a cost effective way. To meet this goal, many of the strategies are aimed at applying the appropriate bridge preservation activities at the proper time, resulting in longer service life at an optimal life-cycle cost." More specifically, the BPPG documents the following goals:

- Maintain bridges in a "state of good repair" using cost-effective strategies.
- Implement timely preservation treatments on structurally sound bridges to promote optimal life-cycle costs, extend overall service life, and extend the time between major rehabilitation and replacement activities.

- Limit adverse impacts to traffic operations and various affected stakeholders.
- Promote and support budgeting of preventive maintenance activities.
- Establish and monitor progress of performance goals related to preservation of bridges.
- Optimize the benefits and effectiveness of long-term maintenance investments in achieving a state of good repair for Wisconsin DOT's bridge inventory.

The BPPG also documents strategies to meet the stated goals. In general, the strategies are aimed at using data-driven methods to maximize the efficiency and effectiveness of the program. Some of these strategies include

- Regularly analyzing bridge inventory data to establish conditions and trends related to performance;
- Developing estimates of needed financial resources at the project and program level; and
- Prioritizing, planning, and performing preservation treatments.

The strategies noted above indicate the need for a systematic method to analyze data on a networkwide basis. This need led Wisconsin DOT to develop an asset management application, which is discussed in more detail later in this document.

Bridge Preservation Objectives and Related Performance Measures

In order to evaluate the effectiveness of the stated goals and strategies, the BPPG defines a set of specific objectives and performance measures. Although it is at the early stage of implementation, it is Wisconsin DOT's intent to use these performance measures as one tool to evaluate the effectiveness of the bridge preservation policy and its implementation. The objectives and performance measures identified are shown in Table 1.

Bridge Preservation Activities and Eligibility Criteria

Using the experience of Wisconsin DOT bridge maintenance personnel combined with a literature review and a consultant subject matter expert, the BPPG establishes parameters for the consideration of bridge preservation activities. Covering deck, superstructure, and substructure, the identified activities are considered cost-effective, provided they are applied to the right bridge at the appropriate time. The Wisconsin DOT bridge preservation activities are shown in Table 2.

In order to provide guidance to maintenance, programming, and scoping personnel on when each activity should be considered, the BPPG contains eligibility criteria. Based on a combination of NBI and element-level condition data, the criteria aims to provide a window for when each activity is appropriate, when it will provide a cost-effective way to extend the life of the structure. Guidance provided is intended to be used with sound engineering judgment and any other available data, such as ground-penetrating radar surveys, infrared surveys, chloride test results, etc. Eligibility criteria are shown in Tables 3 and 4.

TABLE 1 Bridge Preservation Objectives and Performance Measures (*3*)

Objective	Target/Goals	Performance Measure
Maintain bridges in good or fair condition	95% of bridges	Percentage of bridge in good or fair condition(NBI rating 5 or higher)
Maintain bridge decks in good or fair condition	95% of bridge decks	Percentage of bridge decks in good or fair condition (NBI Rating 5 or higher)
Maintain expansion joints in condition state 2 or better	90% of the overall length of expansion joints	Percentage of strip seal joints (based on overall length) in condition state 2 or better
Maintain coated steel surfaces in condition state 2 or better	90% of coated steel surfaces	Percentage of coated steel surfaces in condition state 2 or better
Maintain bearings in condition state 2 or better	95 % of bearings in condition state 2 or better	Percentage of bearings in condition state 2 or better
Seal eligible concrete decks (NBI rating 6 or higher) with sealant every 4 years	Seal 25% eligible concrete decks	Number of decks sealed (sq. ft of deck area) each year during a 4 year period

TABLE 2 Bridge Preservation Activities (*3*)

Bridge Component	Bridge Preservation Type	Activity Description	Preventive Maintenance Type	Action Frequency (years)
All	Preventive Maintenance	Sweeping, power washing, cleaning	Cyclical	1-2
Deck	Preventive Maintenance	Deck washing	Cyclical	1
		Deck Sweeping		1
		Deck Sealing/Crack Sealing		4-5
		Thin polymer (Epoxy) overlays		10
		Drainage cleaning/repair		As needed
		Joint cleaning		
		Deck Patching	Condition Based	1- 2
		Chloride extraction		1 -2
		Asphalt overlay with membrane		12-15
		Polymer modified Asphalt overlay		6-12
		Joint seal replacement		10
		Drainage cleaning/repair		1
	Repair or Rehab Element	Rigid concrete overlays	Condition Based	As needed
		Structural Reinforced concrete overlay		
		Deck joint replacement		
		Eliminate joints		
Super	Preventive Maintenance	Bridge approach restoration	Cyclical	2
		Seat and beam ends washing		2
	Repair or Rehab Element	Bridge rail restoration	Condition Based	As needed
		Retrofit rail		
		Painting		
		Bearing restoration (replacement, cleaning, resetting)		
		Superstructure restoration		
		Pin and hanger replacement		
		Retrofit fracture critical members		
Sub	Preventive Maintenance	Substructure Restoration	Condition Based	As needed
		Scour Counter Measure		
		Channel Restoration		

TABLE 3 Concrete Deck–Slab Eligibility Matrix (*3*)

	NBI Item 58	Deck Element Distress Area (%) ①	Preservation Activity	Benefit to Deck from action	Application Frequency (in years)
Concrete Deck/Slab	≥7		Deck Sweeping/Washing	Extend Service Life	1 to 2
			Crack Sealing	Extend Service Life	3 to 5
			Deck Sealing	Service life extended	3 to 5
			Polymer Modified Asphalt Overlay	Service life extended	12 to 15
			Polymer Overlay	Service life extended	8 to 12
	=6		Deck Sweeping/Washing	Extend Service Life	1 to 2
		<20%	Crack Sealing	Extend Service Life	3 to 5
		<20%	Deck Sealing	Service life extended	3 to 5
		<5% ②	Deck Patching	Service life maintained	As needed
		<5%	Deck Patching, Cathodic Protection	Extend Service Life	As needed
		<10%	HMA w/ membrane	Improve NBI (58) ≥ 7	8 to 12
		<20%	Polymer Modified Asphalt Overlay	Improve NBI (58) ≥ 7	12 to 15
		<20%	Concrete Overlay	Improve NBI (58) ≥ 7	12 to 30
	=5	<20% ②	Deck Patching	Service life maintained	As needed
		<20% ②	Deck Patching, Cathodic Protection	Extend Service Life	As needed
		20 to 25% ③	Concrete Overlay	Improve NBI (58) ≥ 7	12 to 30
		20 to 25% ③	Structural Concrete Overlay ④	Improve NBI (58) ≥ 7	12 to 30
	≤ 4	<40%	Deck Replacement ⑤	Improve NBI (58) = 9	25 to 50

① **Use NBI and deck distress area together to determine the repair action.**

② Refers to deck defects of delaminations and spall and refer to defect 1080.

③ The maximum area of deck delamination is 25 %. When WisDOT fully transitions to elements, this will refer to defect 1080.

④ Consult BOS - not for deck girder bridges.

⑤ Consider remaining bridge conditions to determine if activity is desirable and cost effective.

With this policy document complete, data collection and storage policies established, and the HSIS application reaching a fully developed stage, the foundation was established for implementing a modern structures asset management program. As noted previously, in order to efficiently implement bridge preservation strategies, tools are required to be able to analyze inventory and condition data on a networkwide basis.

TABLE 4 Other Bridge Elements Eligibility Matrix (*3*)

NBI Item	Element	NBI Criteria	Defect	Element Defect Condition State Criteria	Repair Action	Potential Benefits to NBI or CS	Anticipated Service Life Years
Deck	Joints	Item 58 ≥ 5	2350	CS2, CS3, or CS4	Joint Cleaning	CS1or CS2	
			2310	CS2, CS3, or CS4	Joint Seal Replacement/Restoration ⑦	CS1	5 to 8
			2310 or 2360	CS3 + CS4 ≥ 10%	Joint Replacement ④	CS1	10 to 20
				All Condition State	Joint Elimination ④	Elimination	15 to 25
	Railing	Item 58 ≥ 5		CS3 or CS4	Railing Restoration	CS1 or CS2	3 to 10
				CS3 or CS4	Railing Replacement/Retrofit ⑧	CS1	10 to 20
Super	Steel Elements	Item 59 ≥ 5		N/A	Superstructure Washing/Cleaning	NA	1 to 2
			3440	CS2 + CS3 Area> 5% ⑥	Painting - Spot	CS1	1 to 5
				CS3 Area ≤ 25% ⑥	Painting - Zone	CS1 ①	5 to 7
				CS3 Area ≥ 25% ⑥	Painting - Complete	CS1 ②	15 to 20
		Item 59 ≥ 4		CS2, CS3, or CS4	Superstructure Restoration ③	NBI ≥ 7	5 to 20
	Bearings	Item 59 ≥ 5		CS3 or CS4	Bearing Reset/Repair	CS1 or CS2	1 to 5
				CS2 or CS3	Bearing Cleaning/Painting	CS1 or CS2	5 to 7
				CS3 or CS4	Bearing Replacement	CS1or CS2	10 to 15
Sub		Item 60 ≥ 5		N/A	Substructure Washing/Cleaning	NA	1 to 2
			3440	CS2+CS3+CS4 Area > 5% ⑥	Painting - Spot	CS1	1 to 5
			3440	CS3 Area > 25% ⑥	Painting - Complete	CS1 ②	10 to 20
				CS2 or CS3 or CS4	Substructure Restoration ⑤	NBI ≥ 7	5 to 20
			9290	CS1 or CS2	Pier Protection ⑨	NBI ≥ 7	5 to 20
				CS3 or CS4	Scour Counter Measure ⑩	NBI ≥ 7	5 to 20

① Increase NBI only if combine with structural steel repairs.
② Complete painting only if combined with structural steel repairs to improve the component NBI ≥ 7.
③ Superstructure restoration includes all work related to the superstructure including but not limited to strengthening, pin and hanger replacement, retrofit FC member, etc.
④ Combined with deck overlay or replacement project.
⑤ Substructure restoration includes all work related to the substructure including but not limited to fiber wrapping, strengthening, crack injection, encapsulation, etc.—regardless of material type.
⑥ Element condition state for steel protective coating.
⑦ Includes but is not limited to end block/paving block replacement.
⑧ Must bring railing to current standards or have an approved exception to standards.
⑨ Examples are pier protection dolphins and fender systems.
⑩ Provide scour countermeasures after repairing any other substructure defects.

Asset Management Tools

Even before the development of HSIS and the BPPG, BOS understood the need for structures asset management; the need to provide guidance on future bridge rehabilitation or replacement needs to Wisconsin DOT planning and programming engineers. In the late 1990s, BOS partnered with DTIM to provide 6-year work projections for every state-owned bridge in Wisconsin. There were obvious limitations; condition data was not as detailed as modern element-based inspection data, the equations used to extrapolate future deterioration were somewhat crude, and work recommendations were limited to only activities available at the time in Wisconsin DOT's

Financial Integrated Improvement Programming System. Meaningful interaction between BOS and DTIM or BOS and regional planning–scoping personnel was limited. Based on feedback from regional personnel, the value of these early asset management recommendations was minimal. The development of HSIS and the move to element-based bridge inspections presented an opportunity to implement a more-effective structures asset management program.

Using the BPPG as a reference, the next step in the process was the creation of an asset management application to provide recommendations for future work based on current data. This application represents Wisconsin DOT's efforts to address the first portion of the MAP-21 definition of asset management—to develop a "strategic and systematic process of operations, maintaining, and improving physical assets, with a focus on both engineering and economic analysis based upon quality in information…". The application took shape in the form of the Wisconsin Structures Asset Management System or WiSAMS.

Wisconsin Structures Asset Management System

The WiSAMS application was developed in-house, using a software engineer in the BOS development section for the programming. Subject matter direction came from BOS bridge management engineers. The application utilizes inventory and condition data stored in the HSIS and applies the policies established in the BPPG. In order to forecast needs, future bridge conditions are extrapolated using present-day condition data and applying deterioration curves. This was the initial scope of WiSAMS.

WiSAMS Background Logic

The first and primary issue in creating WiSAMS was to determine how to translate preservation policy into a logic that could be applied systematically to a set of data. This was accomplished by establishing a set of rules. These rules take the form of "if–then" statements. An evaluation of a given condition parameter is performed. If the evaluation criteria are met, then a specific work action is assigned. For illustration, WiSAMS rules 1 and 10 are

- **WiSAMS Rule 1**
  – If the all of the following criteria are met…
    - The current NBI rating for substructure is less than or equal to 3 and
    - The structure is scour critical;
  – …then the recommended work action is "REPLACE STRUCTURE."
- **WiSAMS Rule 10**
  – If the all of the following criteria are met…
    - The current NBI rating for superstructure is less than or equal to 3,
    - The structure is > 50 years old, and
    - The superstructure is fracture critical;
  – …then the recommended work action is "REPLACE STRUCTURE."

The rules shown above are relatively simple in nature and rely largely on NBI condition data. As Wisconsin DOT compiles a history of element-based inspection data, WiSAMS rules will transition to the more detailed, element-based condition data. Some current rules are more complex and take into account element-based condition data, as seen below.

- **WiSAMS Rule 32:**
  - If the all of the following criteria are met…
    - The number of previous overlays (concrete or asphalt) is less than 4,
    - The current NBI rating for deck is greater than or equal to 6,
    - The total quantity of deck area in CS-2, CS-3, and CS-4 for defect 1080 (delaminations, spalls, and patches) is less than 5% of the total deck area,
    - The total quantity of deck area in CS-2, CS-3, and CS-4 for defect 3210 (debonding, spalls, patched area, pothole-wearing surface) is greater than 20% of the total deck area, or
    - The total quantity of deck area in CS-2, CS-3, and CS-4 for defect 3220 (crack –wearing surface) is greater than 50% of the total deck area, or
    - The total quantity of deck area in CS-3 and CS-4 for defect 8911 (abrasion, wear, rutting, or loss of friction-wearing surface) is greater than 20% of the total deck area;
  - …then the recommended work action is "CONCRETE OVERLAY."

For a given bridge, the WiSAMS application will pull the relevant condition and inventory data from HSIS. That data is then used to evaluate each of the rules, in order. When the criteria for a given rule are met, the process stops and the associated work action for that rule is reported as the optimal work action. It should be noted that for a bridge in good condition, there may be no recommended work. It is also important to note that the WiSAMS rules, including those shown above, are a representation of the policy defined in the BPPG. Recommended work actions that are produced from the WiSAMS application are an extension of the policy set forth in the guide.

Forecasting Future Work Actions

As described above, the process for identifying work actions depends on structure condition data. Condition data is updated based on inspections, which typically occur on a 2-year cycle. Present-day condition data is based on the most recent inspection and thus provides an accurate account of the current condition of the structure. In order to project future work actions, there is a need to project future condition data. This is accomplished through the use of deterioration curves. The curves used in WiSAMS were derived using historic Wisconsin NBI condition data as well as some national element-based condition data. These curves are applied to current condition data and used to derive predicted condition data in future years. WiSAMS uses this predicted future condition to produce recommended future work actions. This information is a critical piece of the structures asset management puzzle. With recommended future work actions, regional planning and scoping engineers now have better information to help apply allocated funds to most effectively maintain their bridge inventory in a state of good repair.

Other WiSAMS Features

WiSAMS is intended to be a tool used to provide information to DTIM and region personnel with the best information available in order to most effectively program and scope bridge preservation activities. Working toward this goal, WiSAMS output provides information beyond just current and future recommended work actions. Some of this information includes:

- Cost estimates. For recommended work actions, WiSAMS uses cost data from projects recently let by Wisconsin DOT.
- Condition Assessment Index (CAI). While the CAI is still being developed and refined, the intent is to provide a single parameter to capture the overall condition of the structure. Individual element condition will contribute to this measure, with each being weighted based on criticality. When complete, the CAI will provide a quick-glance measure of structure condition. The CAI will also display the effects of work actions performed on the structure.
- Priority Index (PI). Similar to the CAI, the PI is still being developed and refined. The intent of this parameter is to provide a standard objective measure to assist with the prioritization of work actions. For example, average daily traffic is one factor that will contribute to this measure. A bridge that sees a lot of traffic is more critical than one with low traffic counts. Similarly, bridges that are currently load posted may take priority over those that are not. Each factor contributing to the PI will be weighted and summed to provide an overall priority index.

The intent of all WiSAMS output is to aid in better programming and scoping of bridge work. WiSAMS output can be modified or enhanced as necessary to help meet this goal.

WiSAMS AND THE WISCONSIN DOT STRUCTURES ASSET MANAGEMENT PROGRAM

As noted above, BOS houses the structural expertise for Wisconsin DOT, but the regional offices and DTIM are the primary entities responsible for funding allocations, project selection, planning, scoping, and delivery. Success of this initiative depends on BOS effectively communicating, coordinating, and collaborating with these entities. Figure 7 is a high-level representation of the asset management process at Wisconsin DOT.

WiSAMS is an important tool and a key component of Wisconsin DOT's structures asset management program. The primary WiSAMS output takes the form of projected future work actions and associated costs. Having this information allows BOS to supply more accurate, more refined information to DTIM for purposes of allocating funding; a method to better identify funding needs and the timing for that funding. BOS will also supply reports directly to regional planning and scoping personnel, formatted to best serve their business needs. Here the collaborative process will be key. BOS depends on the feedback loop from the regions to help assess the quality of the recommendations produced by WiSAMS. Constructive feedback on the output help BOS staff identify refinements that may be needed in the WiSAMS logic to produce recommendations that are more in line with actual observed bridge condition and deterioration. The refinements may take the form of modified deterioration curves, additional deterioration curves, or modifying the WiSAMS logic (rules) used to derive work recommendations.

Implementing WiSAMS and its output represents a major step forward from past Wisconsin DOT practice, but it must be noted that WiSAMS is a tool to be used as part of a larger asset management effort. To that end, BOS has shifted resources to be able to better support asset management efforts. This support takes many forms, including

- Answering questions on WiSAMS recommendations and supplying data to regional planning and scoping staff as necessary;
- Working with regional planning and scoping staff to analyze various scenarios based on variable funding, corridor–interchange coordination, coordination with roadway projects, or other scenarios;
- Attending regional planning and scoping meetings to offer a structural perspective on potential bridge projects;
- Coordinating with DTIM personnel to offer perspectives on how various funding levels or distribution of funds may impact the overall condition of the Wisconsin bridge inventory;
- Creating and providing asset management-related materials—reports, tables, charts, graphs, maps, or other visuals—to the various stakeholders; and
- Creating and delivering presentations and talking points to promote bridge preservation and asset management strategies throughout Wisconsin DOT.

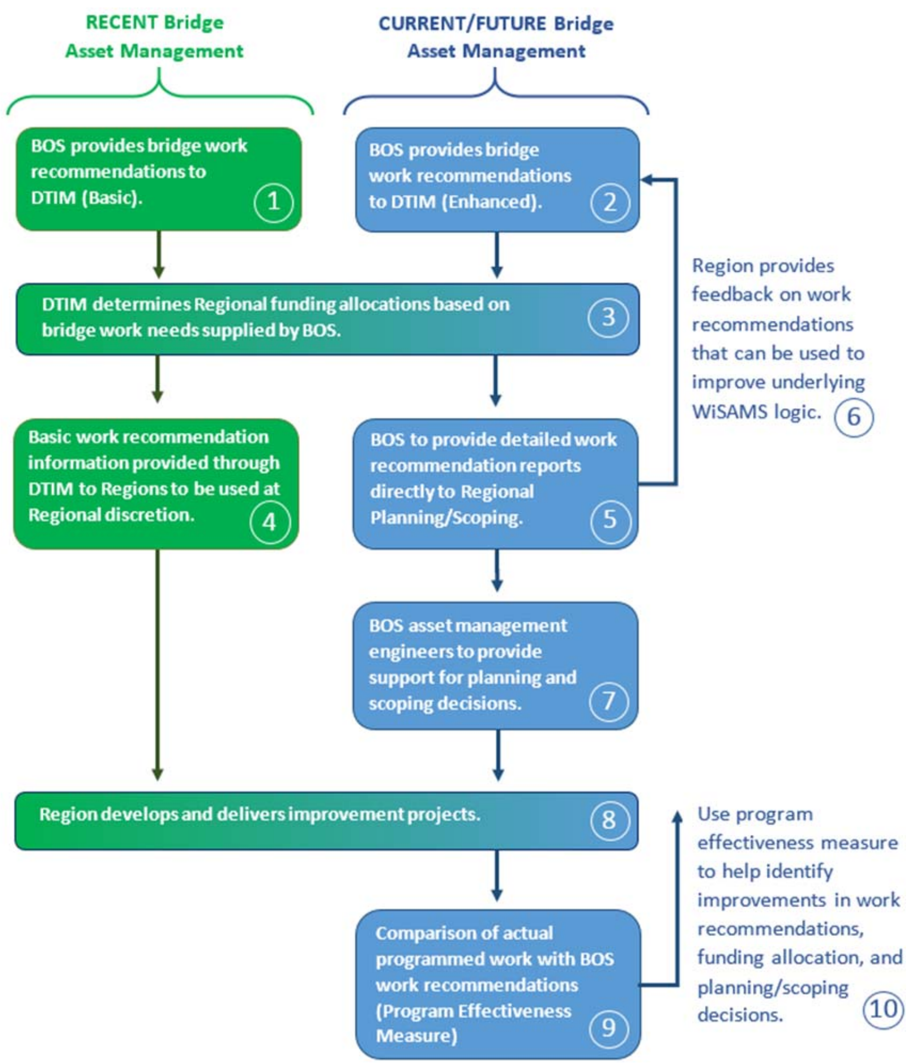

FIGURE 7 Wisconsin DOT structures asset management overview.

BOS is at the front-end of fully implementing WiSAMS and structures asset management. As WiSAMS was being developed, BOS engineers took advantage of a number of existing forums to give presentations and raise awareness on WiSAMS, bridge preservation, and structures asset management. As a working version of WiSAMS became available, BOS engineers held interactive workshops in the several regional offices around the state in order to further educate planning and scoping staff. Current implementation activities are focused on the regular distribution of WiSAMS reports and interacting with regional planning and scoping staff to implement WiSAMS recommendations, as described in the bullet points above.

The final phase of implementation will be establishing a program effectiveness measure. The measure will incorporate the bridge preservation objectives enumerated in the BOS Bridge Preservation Policy Guide, but also aim to evaluate how closely actual programmed work matches WiSAMS recommendations. As WiSAMS recommendations are intentionally idealized, 100% correlation is not expected or required. Rather, the measure will aim to identify projects that successfully and cost-effectively implement bridge preservation strategies and learn from those successes. There is also value in studying those projects that do not meet program effectiveness measures and identify why that was the case so that modifications may be made for future planning, if necessary.

The work noted above has put Wisconsin DOT in the position to implement a modern, data-driven asset management program with the aim to most effectively spend taxpayer dollars to keep the Wisconsin infrastructure safe and serving the travelling public.

CONCLUSION

The Wisconsin DOT is tasked with building and maintaining an effective, efficient transportation infrastructure to serve the citizens of the state. Wisconsin DOT has a responsibility to use allotted taxpayer funds wisely. As described above, recent advances in data collection, management, and analysis have allowed Wisconsin DOT to implement a new structures asset management program grounded in data and analytics. But even with this step forward, Wisconsin DOT is still very much just scratching the surface of what is possible. Every day Wisconsin DOT asset management engineers are working on tasks such as improving how we project deterioration, how we prioritize improvement projects, how we quantify risk, and more. Wisconsin DOT is a large organization with a lot of moving parts. Implementing change on a large scale is not easy, but Wisconsin DOT is committed to continual improvement in the management of transportation assets today and into the future.

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Bridge Model Validation at Indiana Department of Transportation

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The Indiana Department of Transportation (DOT) has had a bridge management system (BMS) since 1982. This system has undergone several enhancements since its inception, with the most recent major one in 2008.

Recent changes to bridge inspection standards in the United States as well diminished confidence in the BMS results precipitated Indiana DOT management to re-evaluate some facets of the BMS, such as the deterioration models, to ensure that the results are still dependable. In 2019, Indiana DOT began a project to validate the current bridge models used by the BMS. The results of this project and the framework used to validate the deterioration models will be discussed.

Deterioration models used by a management system should be validated on a recurring basis. A continuous validation process ensures that results produced by the models remain accurate and reliable as dependent factors change over time: inspections methods, treatment technologies, maintenance policies, traffic volumes, and composition. The model validation method established an historical analysis baseline. Results were then generated based on the actual bridge rehabilitation and maintenance work performed by Indiana DOT and compared to the present day bridge condition. Variances between predicted and actual conditions were evaluated and modifications to the bridge models were addressed.

This paper will present the method Indiana DOT used in a manner that can be adopted by other agencies who wish to validate their own deterioration models.

The Indiana Department of Transportation (DOT) has had a bridge management system (BMS) since 1982. This system has undergone several enhancements over the years, with the most recent major one in 2008 and it is still in use today.

Recent changes to the bridge inspection standards in the United States as well diminished confidence in the BMS results precipitated Indiana DOT management to re-evaluate some facets of the BMS, such as the deterioration models, to ensure that the results are still dependable.

In 2016, Indiana DOT began two separate but complementary projects: the first was to commission a research project to develop new deterioration models for several main bridge components, and the second was to develop the next generation of the BMS, which included new treatments and new deterioration models.

A critical aspect for a BMS is having a high degree of confidence and reliability in the generated results. In order to achieve this, the results must be validated against real-world outcomes. The BMS results are generated by predicting bridge condition into the future, along with defining the agency’s business practices regarding treatment interventions. To have confidence in the results, one must have confidence in the prediction models. This paper discusses a subsequent project undertaken by the author’s firm for Indiana DOT to validate the current bridge models used by the BMS.

PROBLEM STATEMENT

In 2016, Indiana DOT received the results of a research project undertaken by Purdue University to develop deterioration models for the state's bridges for the deck, superstructures, and substructure components. In 2016, Indiana DOT contracted Deighton Associates Limited to develop their next-generation BMS. One aspect of this project was to use Indiana DOT's BMS to validate the predictive accuracy of the models and quantify any deviation of actual measurements of condition from the predicted baseline. This validation, along with establishing a procedure that can be used by Indiana DOT to validate deterioration models into the future is required, were the two primary objectives of this project.

DETERIORATION MODEL DEVELOPMENT

In 2015–2016, Indiana DOT, in cooperation with FHWA and U.S. DOT, commissioned Purdue University to conduct a Joint Transportation Research Program to develop new bridge deterioration models in support of Indiana DOT's BMS.

Deterioration models establish the current and future deterioration patterns of bridge elements over time. A BMS that is equipped with reliable deterioration models can assist bridge engineers with the task associated with the long-term programming, planning, and needs assessment at both the project and network levels. At the project level, bridge engineers can use these models to track the physical condition of the bridge deck, superstructure, and substructure, and thereby provide guidance in prediction the year at which a component's condition reaches agency-specified thresholds for rehabilitation or replacement. At the network level, bridge engineers use these deterioration models to measure the accumulated repair needs of the individual bridge components that—when combined with activity cost models—can determine the systemwide financial needs over a specified future time horizon. Deterioration models also play key roles in other agency business processes, such as highway cost allocation and asset valuation. These functions are facilitated when the bridge manager is capable of reliably predicting the physical condition of each bridge component at any future date.

The bridge deterioration models currently used in Indiana BMS were first developed more than two decades ago. Since then, there have been significant changes in construction techniques and technologies, materials, condition inspection methods, and loading patterns. The past few decades have also seen advancements in statistical techniques for data analysis and model building. In addition, there has been a surge in data resources in terms of the volume and variety of data types and items, and data integrity and reliability. For example, data on truck volumes and climatic conditions are more readily available, making it possible to develop models that account for these deterioration factors. These challenges and opportunities combined indicate that now is an opportune time to develop new models to address the current modeling needs of Indiana DOT bridge managers.

Deterioration models are often developed separately for many state DOTs for the wearing course, deck, superstructure, and substructure. For the wearing course, Indiana DOT recently developed deterioration curves; however, the decades-old models continue to be used for the remaining components. Therefore, Indiana DOT commissioned the research study to update the deterioration models for the remaining components (*1*).

Study Objectives and Scope

The two main objectives of the Purdue research project were to develop a set of bridge condition deterioration curves on the basis of the physical and operational characteristics, climate, and truck traffic and identify the factors that influence bridge component deterioration and measure the direction and strength of the influence of each factor (*1*).

As stated earlier, one of the objectives of the Deighton project was to validate the predictive accuracy of the deterioration models that resulted from the research project.

The Purdue research study was directed to address only the bridges located on the state highway system (Interstates, U.S. roads, and state roads). These bridges were placed into "families" based on their material type, functional class, and administrative–climatic region, and were calibrated for each family. Bridges on local routes were excluded (*1*).

Sample Outcome of Deterioration Model Development

Six deterioration models were built for bridge decks, six for substructure, and 42 for superstructure. It was found that the best models were either exponential or polynomial of the second or third order. The influential variables were found to be as follows:

- Deck age in years (AGE);
- Interstate location (1 if located on Interstate, 0 otherwise) (INT);
- Angle of skew (SKEW);
- Bridge length (LENGTH);
- Type of service under bridge (SERVUNDER);
- Number of spans in main unit (SPANNO);
- Freeze index in 1,000s of degree-days (FRZINDX);
- Average annual number of freeze–thaw cycles (NRFTC);
- Average annual daily truck traffic in 1000s (ADTT); and
- Deck protection (1 with protective system, 0 otherwise), (DECKPROT).

For the purposes of this paper, two curves will be isolated further and validated against actual inspections performed by Indiana DOT: deck condition for National Highway System (NHS) pavements in northern Indiana (Deck 1) and substructure condition for NHS pavements in central Indiana (Sub 2) (*1*). These two curves (Deck 1 and Sub 2) are shown in Equations 1 and 2 and graphically shown in Figure 1 and Figure 2, respectively. (Note: DCR is deck condition rating and SUBCR is substructure condition rating):

$$\begin{aligned} DCR = {} & 8.55637 - 0.24129 \bullet AGE + 0.0096 \bullet AGE2 - 0.0001667 \bullet AGE3 \\ & - 0.04301 \bullet SERVUNDER - 0.01218 \bullet SPANNO + 0.51375 \bullet DECKPROT \\ & - 0.05182 \bullet FRZINDX - 0.01872 \bullet ADTT \end{aligned} \tag{1}$$

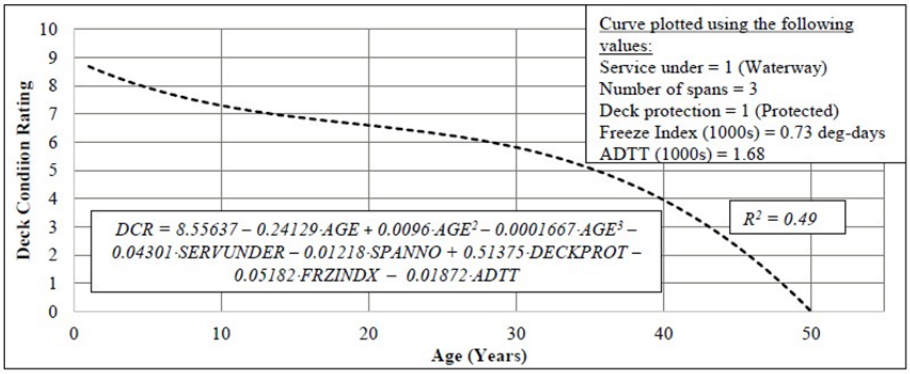


FIGURE 1 Example plot of the bridge deck deterioration model, Northern Districts, NHS.

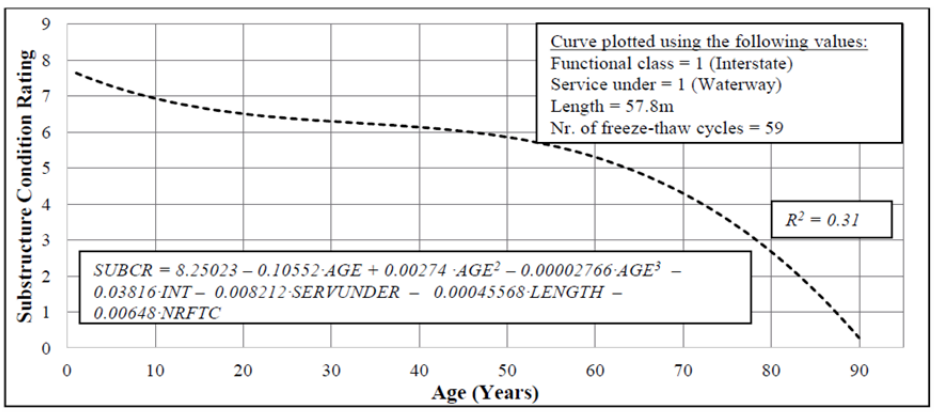


FIGURE 2 Example plot of the substructure deterioration model, Central Districts, NHS.

$$SUBCR = 8.25023 - 0.10552 \cdot AGE + 0.00274 \cdot AGE2 - 0.00002766 \cdot AGE3 - 0.03816 \cdot INT - 0.008212 \cdot SERVUNDER - 0.00045568 \cdot LENGTH - 0.00648 \cdot NRFTC \quad (2)$$

DETERIORATION MODEL VALIDATION

The process of validating the deterioration models is crucial to the ultimate credibility of the results of the BMS, since the results are directly based and attributable to the accuracy of the models. This section details the methodology used in the Deighton project for validating the models for Indiana DOT.

Approach

The approach consists of six basic steps. The BMS plays a critical role in the validation, as it is through the use of the BMS that the automated analysis can take place. The steps used in this approach are (Figure 3):

1. Use the BMS to go back in time and capture the condition of the bridge network for a specific point in time.
2. Capture the actual work done by Indiana DOT in the BMS from that historical point in time to current time.
3. Define the deterioration models that are to be validated in the BMS.
4. Run an analysis using the BMS from that historical point in time to current time.
5. Review the results of the historical analysis and compare to the actual, current bridge condition.
6. Quantify any variances between predicted and actual. Refine the deterioration models as required and redefine the models in the BMS.

The BMS used by Indiana DOT is dTIMS by Deighton Associates Limited. A BMS is critical to the model validation process because it provides an agency with the ability to run automated analyses and compare results quickly. It is essential that a BMS should allow for

- Flexibility of setting a start date for the analysis;
- Multiple key performance indicators to be analyzed;
- Capture of actual work done by the agency;
- Running an analysis from a historical point in time to current time; and
- Comparison of predicted condition values to actual values.

Turning Back the Clock

The first step in the validation process is to choose a historical point in time that is far enough in the past to allow for ample deterioration of the major bridge components, but not too far so that capturing the actual work done between the historical point and current time is an overly onerous process. For the purpose of this exercise, the historical point in time selected was 2010.

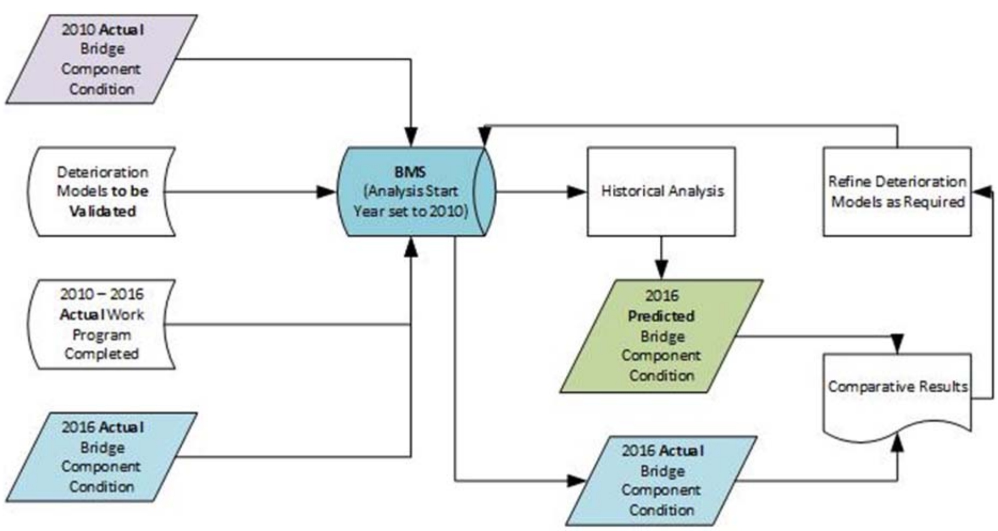


FIGURE 3 Overview of model validation process.

The objective is to make the "current" time in BMS to be 2010, and then capture the actual bridge component conditions as they were in 2010. In essence, you turn back the clock in the BMS to 2010.

For the Deighton project, Indiana DOT provided the bridge conditions for deck, superstructure, substructure, and wearing surface as they were in 2010. This data was loaded into the BMS and the start time for the analysis was set to 2010.

Capturing Actual Work Done

Next, the actual bridge projects that Indiana DOT performed between 2010 and 2016 were loaded into the BMS. These are recorded as committed projects, since the work was actually done. The premise is to start the analysis beginning in 2010 and commit or force the BMS to select the actual work done between 2010 and 2016 but no additional projects. In this way, the BMS is replicating the history that has taken place between 2010 and 2016, and also capturing the actual improvements in bridge condition that resulted from the historical work performed by Indiana DOT.

Defining Deterioration Models in the BMS

The deterioration models that are to be validated are defined in the BMS for each of the components. That is, the equations presented earlier (Equations 1 and 2) plus the equations for all other components and families of bridges are defined within the BMS. In this way, the condition projections made by the BMS will be based on the deterioration models that are to be validated.

A Historical Analysis

At this point in the validation process, the basic building blocks are in place in the BMS and it can now be used to predict the bridge component condition into the future and select bridge rehabilitation projects. The projections will follow the "to be validated" deterioration curves and the bridge projects that are selected are the actual projects performed by Indiana DOT. When the BMS selects a bridge project, the bridge components are improved in condition based on the treatment resets programmed in to the BMS. So, when a deck replacement project is selected by the BMS for example, the deck condition rating and the wearing surface condition are improved, whereas the substructure and superstructure ratings are unaffected.

It is important to note that the only projects the BMS is selecting are the ones that were actually performed by Indiana DOT and no others.

The premise of this analysis is that for every bridge in the network, its predicted condition in the BMS in 2016 is based on the "to be validated" deterioration models, and the actual work that has been performed from 2010 to 2016. We call this predicted condition. This condition is one of the two important parameters required to validate the deterioration models.

The second parameter is the actual bridge component condition. The 2016 actual bridge condition data is loaded into the BMS. This is called the actual condition since it is based on the actual bridge inspections that have taken place. Once both the predicted and the actual bridge conditions are derived or captured in the BMS, the comparison can occur. The model validation process is summarized in Figure 3.

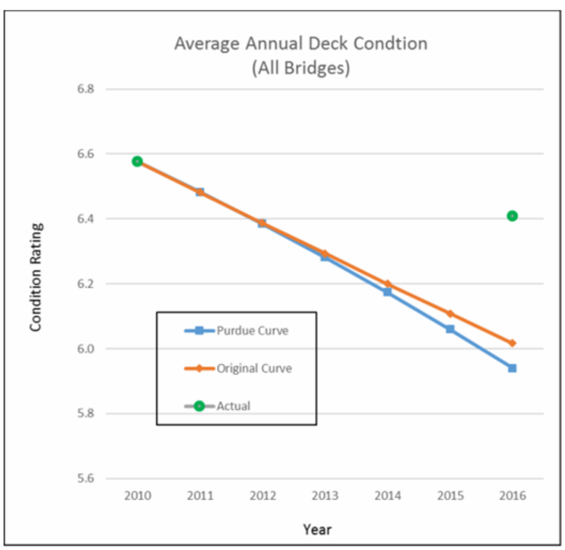
Reviewing the Results

To review the results, one must compare the predicted bridge condition with the actual bridge condition. The comparison can be performed for individual bridges or for a group of bridges. An additional benefit of this comparison with an individual bridge is that you can review each specific bridge component condition and determine the variance between the predicted and the actual values.

The results were reviewed for bridges where work was done between 2010 and 2016, and for bridges where no work at all was done. In the first case, an additional outcome is that you can also validate the treatment resets in the BMS against what actually occurred for the bridge. Both comparisons are valid and each provides a different perspective on the results.

Comparisons for groups of bridges are also useful for quantifying variances and are sometimes more useful for this purpose than for individual bridges since the nuances of each bridge does not affect the comparison. The following results are based on the models presented in Equations 1 and 2 and the validation exercise. These figures focused initially on those structures that did not have work done to them during the study period. This allowed the comparisons between predicted and actual to focus only on actual deterioration and not on the effects of a work program. Additional comparisons were made using bridges that also had work performed but those comparisons are outside the scope of this paper.

Figure 4 shows the 2010 and 2016 actual inspections along with the original deterioration model used in the BMS and the new Purdue model for both deck and substructure components for all Indiana DOT bridges. In both cases, the actual value is greater than predicted. This difference is exacerbated because the inspection ratings are always whole numbers.



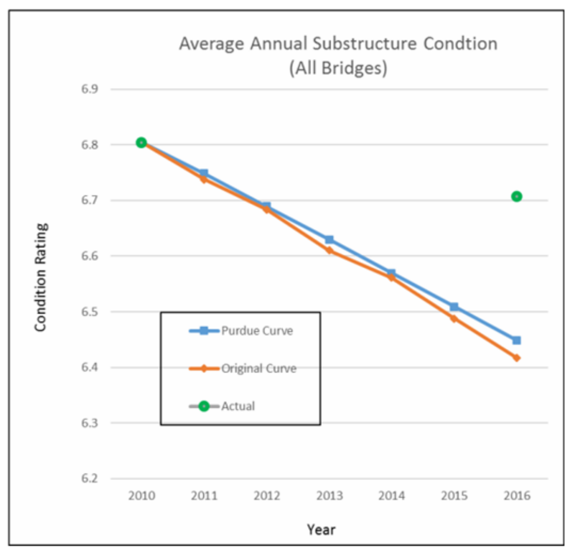


FIGURE 4 Predicted deck and substructure deterioration versus actual inspections.

Figures 5 and 6 show the actual ratings versus the predicted ratings for 2016 for substructure (central Indiana, NHS) and for deck (northern Indiana, NHS) respectively. It is evident that there is a moderate degree of scatter in both plots. The blue dotted line represents the trend line of the points. In Figure 5, the trend line is on top of the 45° line indicating a very good overall correlation between actual and predicted. However, Figure 6 shows the majority of the points below the 45° line, indicating a more aggressive rate of deterioration with the curve than the actual inspections. This is corroborated with the trend line being below the 45° line.

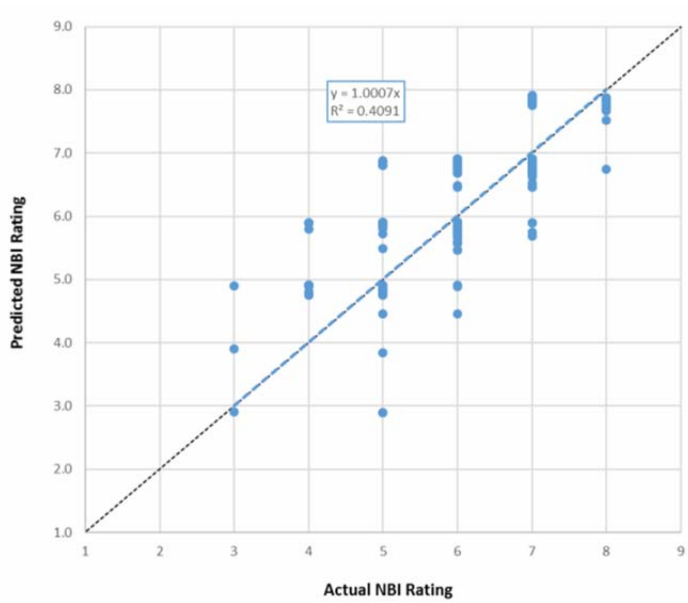


FIGURE 5 Actual versus predicted substructure deterioration for Central Districts, NHS.

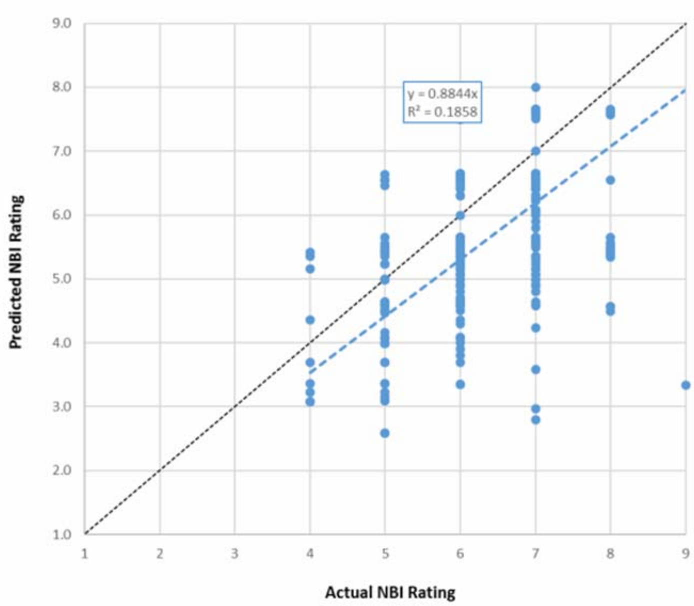


FIGURE 6 Actual versus predicted deck deterioration for Northern Districts, NHS.

RECOMMENDATIONS

The authors of this paper recommend that an agency maintain historical condition records and an accurate history of actual work completed, implement a BMS, conduct a model deterioration validation exercise every 5 years or when new models are developed or there are changes in condition data collection protocols, and revise deterioration models accordingly based on the results of the validation exercise.

CONCLUSION

This paper has presented a framework and methodology that was used at Indiana DOT to validate bridge deterioration models. The main conclusion of this exercise is not to comment on the accuracy of the bridge models that were developed for Indiana DOT, but rather that bridge deterioration models must be validated so that the results from the BMS can be validated and hence provide the consumers of the results with a higher degree of confidence. This framework can be adopted by other agencies that have a BMS or any asset management system so they can validate their own asset deterioration models. The process presented is repeatable and defendable and hence can withstand a high degree of scrutiny.

Any agency that is using an asset management system and has not put their own deterioration models through a similar validation exercise runs the risk of not being able to defend the results of the management system with a high degree of confidence, and therefore may be in danger of tarnishing their credibility along with the credibility of the asset management system.

ACKNOWLEDGMENT

The authors of this paper acknowledge the valuable support, guidance, and overall administrative support provided by the members of the Indiana DOT Bridge Group: Jaffar Golkhajeh, Anne Rearick, Manoj Sutaria, Steven Dilk, and John Weaver.

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AASHTO *Manual for Bridge Evaluation* Update to Chapter 3: Bridge Management Systems
A Practical Tour

DAVE JUNTUNEN
Michigan Department of Transportation

There have been many developments in bridge management systems (BMSs) over the past 10 years. The American Association of State Highway Transportation Officials Subcommittee on Bridges and Structures is updating the *Manual for Bridge Evaluation* Section 3: Bridge Management Systems. This paper features some of the changes to the section, including examples how the Michigan Department of Transportation is implementing their BMS.

There have been many developments in bridge management systems (BMSs) over the past 10 years. The American Association of State Highway Transportation Officials (AASHTO) Subcommittee on Bridges and Structures (SCOBS) is updating the *Manual for Bridge Evaluation* Section 3: Bridge Management Systems (referred to in this paper as MBE Section 3). The updated section will be balloted at the 2017 AASHTO SCOBS annual meeting. Figure 1 shows the table of content for the section. This paper features some of the proposed updates to the section, including examples how the Michigan Department of Transportation (DOT) is implementing their BMS.

MBE Section 3 references the AASHTO Standing Committee on Highways, Planning Subcommittee on Asset Management:

> Transportation asset management is a strategic and systematic process of operating, maintaining, upgrading, and expanding physical assets effectively throughout their lifecycle. It focuses on business and engineering practices for resource allocation and utilization, with the objective of better decision making based upon quality information and well defined objectives (*1*).

MBE Section 3 discusses the purpose of a BMS:

> The section describes how a bridge management system fits into overall transportation asset management as follows. A BMS is a tool or collection of tools integrated through a process whose goal is to assist an agency to meet strategic objectives by connecting inventory management and project selection to agency strategic goals through a data driven process. A BMS should meet the needs of both upper management, where it is a strategic planning tool, and technical decision makers, where it is an engineering tool. BMS helps engineers and decision-makers determine the best fiscally constrained action to take on maintenance programs and short, medium, and long-term capital improvement programs. Its purpose is to determine the optimum use of funding by enabling decision-makers to understand the consequences of their actions and strategies. A BMS assists the bridge owner in expending the appropriate level of resources to maintain the inventory in an acceptable state of good repair. It also provides essential information to help transportation agencies enhance safety, perform risk assessments, extend the service life of bridges, and serve commerce and the motoring public (*2*).

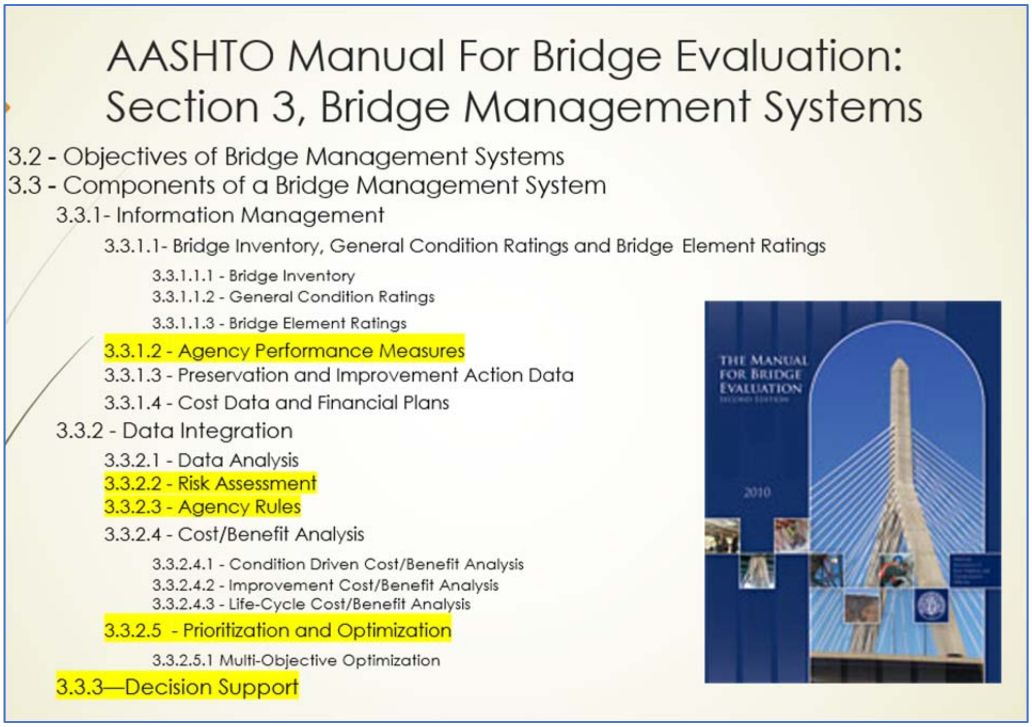

FIGURE 1 Table of contents for the draft MBE Section 3.

The MBE Section 3 discusses BMS data base requirements:

> A BMS requires comprehensive, connected and well organized relational databases that are capable of supporting the various analyses involved in bridge management and reporting this information in a way that can be readily understood by various stakeholders (*2*).

The Michigan DOT has a corporate relational database for storing all bridge data. It is built upon the AASHTOWare Pontis (now Bridge Management) database with state-specific tables added to it. Michigan has their own web-based system, called MiBridge, for collection and reporting of all bridge data.

Network-Level BMS

The Michigan DOT has two levels of BMS: network level and project level. Network-level BMS includes data collection and analysis for the state's population of highway bridges. This includes many of the methods described by MBE Section 3, including development of strategic goals, performance measures and objectives, and regular reporting. They have had strategic goals for their population of bridges for the past 20 years. Like the FHWA's new national performance measures, the Michigan DOT uses the National Bridge Inventory (NBI) General Condition Ratings (GCRs) as performance measures for their freeway and nonfreeway bridges, with one of the goals being to meet and maintain 95% of trunkline freeway bridges in good or fair condition. Dashboards have been created to show the public how the state is doing as shown in Figure 2, and Michigan tracks condition trends of their good, fair, and poor bridges as shown in Figure 3.

To achieve their goals, the Michigan DOT uses a strategy based upon allocating funds to their seven regions for capital preventive maintenance, rehabilitation, and replacement projects per the candidates in each region. Data analysis using current and forecasted condition is done to determine the right mix of fixes to be used to most efficiently manage the network of bridges. To perform this analysis, they identified agency rules to reflect current practice and developed tools to show the deterioration rate of their bridges, track costs (direct and indirect) of bridge projects, and account for construction inflation. Michigan tracks the condition bridges are in when a project is initiated and the resulting condition after the project is completed.

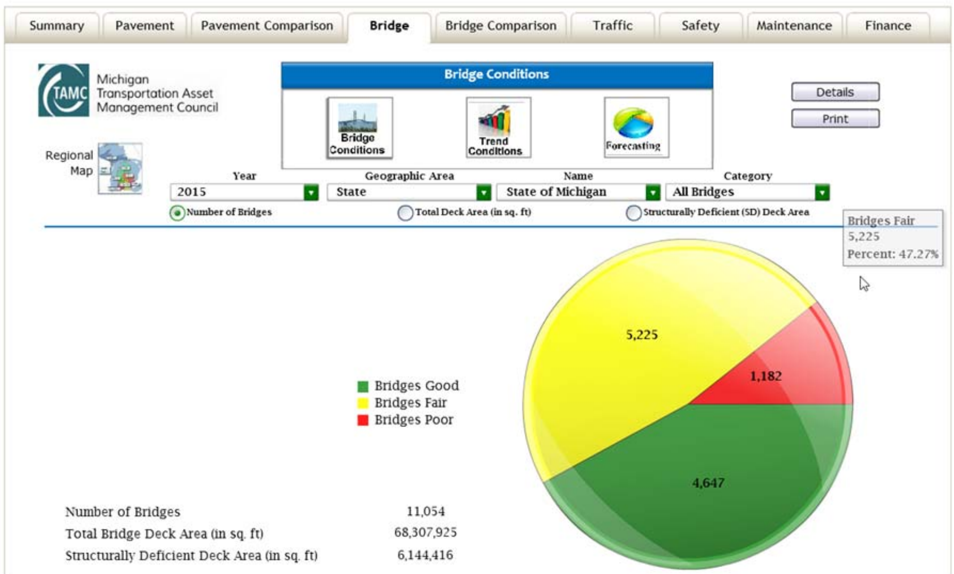

FIGURE 2 Michigan Transportation Asset Management Council bridge dashboard.

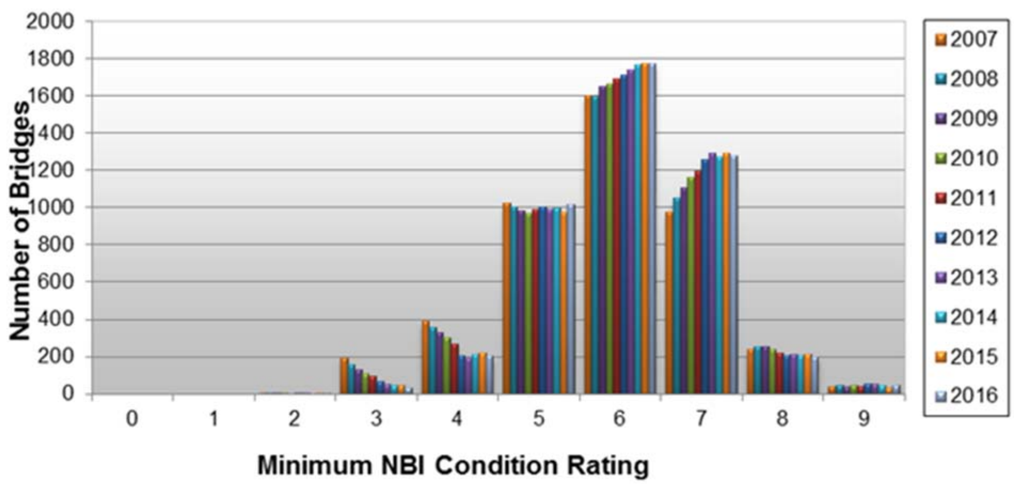

FIGURE 3 Statewide 10-year condition trends for Michigan highway bridges.

The MBE Section 3 discusses agency rules as follows:

> In order for a BMS to make bridge level decisions consistent with agency practice, agency rules need to be developed. The intent of the rules is to translate agency practices and their effects on bridge, program and network level recommendations into the system's modeling approach. These rules should be intuitive and reflect agency business practice and policy (*3*).
>
> Rules may be applied at the bridge, program, or network level…. Program level rules may reflect varying performance measure goals or funding constraints while network rules cover standard agency practice (*2*).

Michigan DOT forecasts bridge condition using their Bridge Condition Forecasting System (BCFS). Agency rules are set for what GCR would cause a bridge to be selected for either preventive maintenance, rehabilitation, and replacement projects and what GCR the completed project will improve the structure to. Results of a BCFS model are shown in Figure 4. In the figure, forecasted bridge condition is shown through 2025 for Michigan DOT's freeway and nonfreeway bridges. By comparing near- and long-term bridge condition for different strategies, an optimal balance of preventive maintenance, rehabilitation and replacement can be identified.

As shown by both historic and forecasted data, the key to achieving the Michigan DOT's bridge strategy with limited funds is a commitment to preservation. Figure 5 shows that many of Michigan DOT's bridges are rated in fair condition (overall NBI rating for the bridge being rated 5 or 6 on the NBI condition rating scale). Management will then dedicate funds to these bridges to correct deficiencies, slow deterioration, and reduce the number of bridges becoming poor each year. A very simple, but helpful measure the Michigan DOT uses to evaluate their preservation program on a network level is counting the number of bridges that become poor each year as shown in Figure 5. This is done statewide and for each region. A successful preventive maintenance program will result in slowing bridge deterioration. By knowing how many bridges are expected to become poor, Michigan DOT bridge managers then know how many bridges

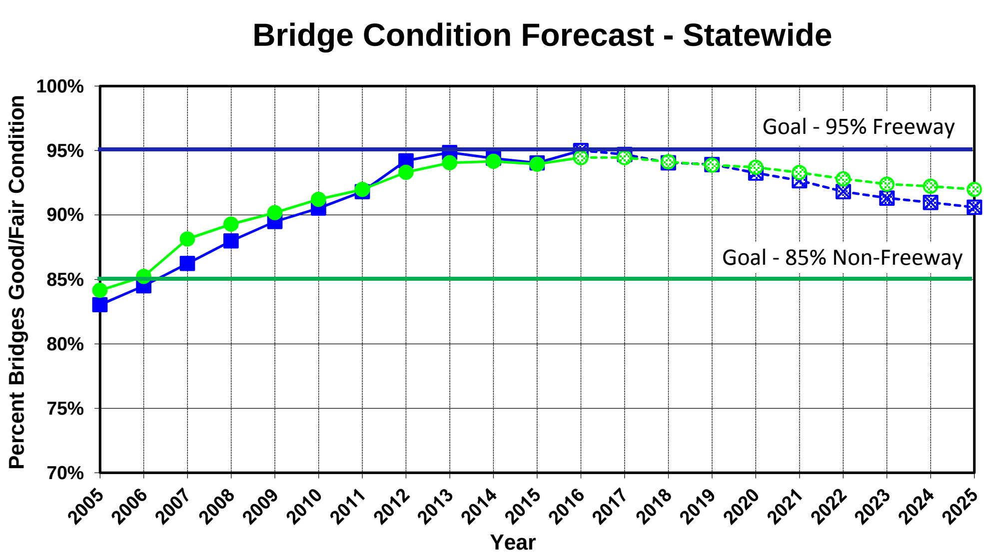

FIGURE 4 Forecasting bridge condition using Michigan DOT's BCFS.

they need to improve just to maintain the current condition state. Additional projects will result in an improvement of bridge condition.

Project-Level Bridge Management

The Michigan DOT has become very good at network level bridge management, and they are now working on enhancing project level management. The draft MBE Section 3 says:

> Advanced BMS analyses requires a more detailed condition assessment to predict and prioritize bridge repair, preservation, or replacement actions (*2*).

For example, Figure 6 helps visualize the circular cycle of network-level bridge management. The entire population of bridges is slowly deteriorating and moving into lower condition states. Projects are done to either slow the deterioration or improve bridge condition. Good bridges are preserved with cyclic maintenance activities. Fair bridges are preserved and improved with preventive maintenance and minor rehabilitation projects. Finally, poor bridges are improved with major rehabilitation and replacement projects.

Using GCRs, the Michigan DOT can categorize projects into work activities, but they are not able to prioritize or optimize individual projects within these categories. To do more refined analysis, Michigan collects the National Bridge Elements (NBEs) and Bridge Management Elements as defined by the AASHTO *Manual for Bridge Element Inspection*. The condition of each element is reported per the quantity or percentage of the element rated in four condition states (CS): CS 1 (good), CS 2 (fair), CS 3 (poor), and CS 4 (severe). Michigan also created and is collecting state specific agency-defined elements. Using the element condition ratings, they can identify more detailed bridge needs, such as identifying protective systems that could be repaired or replaced before deterioration progresses on the underlying element.

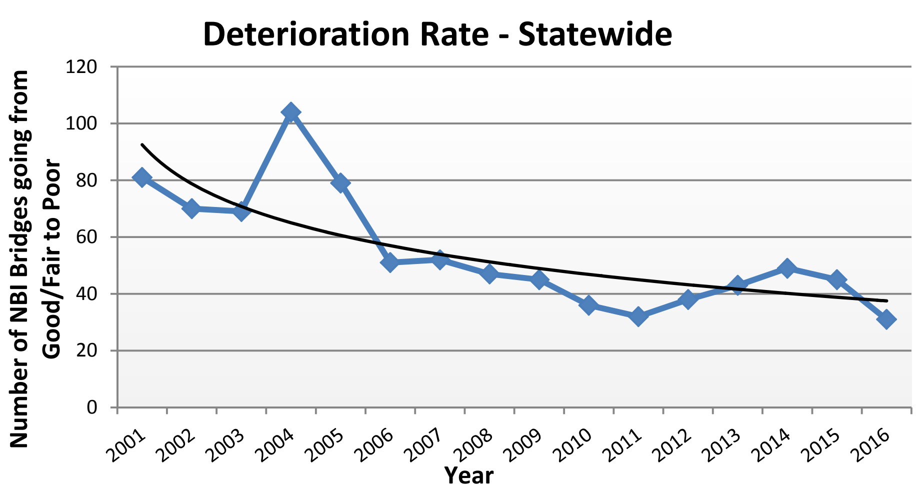

FIGURE 5 Michigan DOT number of bridges going from "good" or "fair" to "poor" each year.

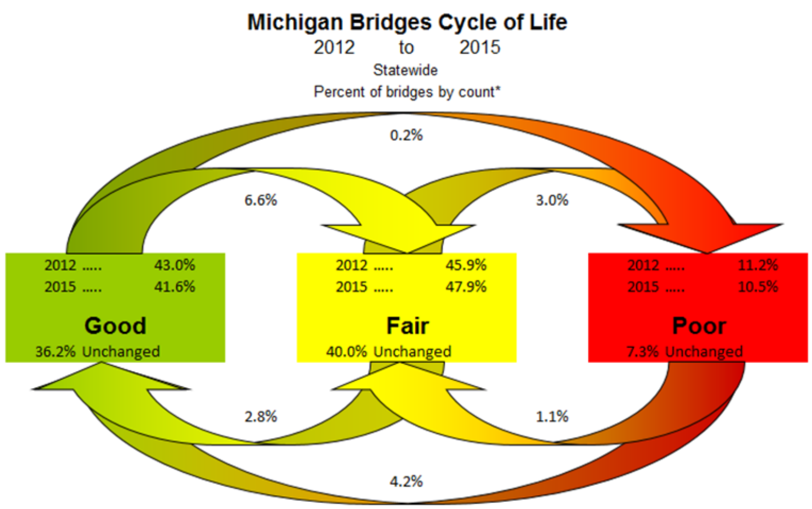

FIGURE 6 Michigan bridges cycle of life.

PRIORITIZATION AND OPTIMIZATION

The MBE Section 3 discusses prioritization and optimization:

> The purpose of optimization at the network level is to select a set of bridge projects in such a way that the total benefit derived from the implementation of all of the selected projects is maximized (costs and risks are minimized). The ability to establish project priorities and optimally allocate limited funds over a predefined planning horizon, both short- and long-run, is a fundamental part of a BMS (*2*).
>
> Bridge owners often need to consider multiple performance criteria and constraints, such as bridge condition, life cycle costs, safety, traffic flow disruption, and vulnerability when making decisions and prioritizing projects. They may need to analyze trade-offs between these performance criteria (*2*).

The Michigan DOT is working towards using multiobjective optimization to produce a prioritized list of projects that the region bridge engineer can use when they select projects each year for the call for projects. MBE Section 3 discusses the different approaches that can be taken for prioritization and optimization. These include top down, where network-level performance measures are addressed first, and then the results are used to guide project selection, resource allocation and scheduling. Another method is bottom up, where by using condition information and inspector recommendations, the most cost-effective option is identified for each bridge, and the results of the analysis are summarized back up to the network level. Michigan uses a combination of the two methods. Figure 7 shows a simple flow diagram of the process. The objective is to use bridge elements and the AASHTOWare BrM 5.2.3 software to do the following: for every bridge not already programed, deteriorate the network 5 years, then using bridge elements and the AASHTOWare BrM software, indicate what the needs are for each bridge, identify what category of work it fits into, estimate the cost for the work, and prioritize the list of possible projects with consideration to fiscal constraints. The region engineers will use

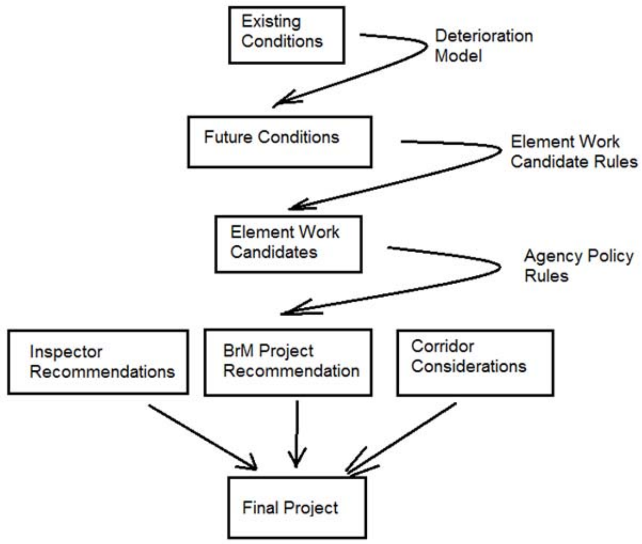

FIGURE 7 Michigan DOT project-level BMS flow diagram.

the element work candidates along with inspector recommendations, corridor considerations, and Michigan DOT bridge objectives to help select projects. Some of Michigan's objectives include:

1. Meet and maintain freeway bridge condition goal (95%) good or fair;
2. Reduce scour critical bridges carrying the Interstate; and
3. Make bridges more resilient to reactive activities resulting from advanced deterioration (reduce need to close traffic lanes because of advanced bridge deterioration).

The Michigan DOT takes different approaches for prioritization of replacement projects and rehabilitation–preventive maintenance projects. Prioritization of Michigan DOT replacement projects often includes a risk assessment when the resiliency of the transportation system is at risk of being impacted due to deterioration that cannot be efficiently repaired or when public safety is at risk such as for scour critical bridges. The MBE Section 3 discusses risk assessment:

> Risk may be understood as the potential for unplanned adverse events to impact one or more transportation facilities in a way that causes unacceptable transportation system performance according to any or all of the agency's performance objectives. In bridge management, the primary concern is disruption of expected or designed service levels, which may cause injuries or property damage, loss of mobility, and immediate expenditures or long-term excess costs (*4*).
>
> Risk assessment evaluates the likelihood and consequence of adverse events. The likelihood of the event includes the probability of the event occurring and may include the vulnerability of the structure to the event. The consequence of the adverse event would quantify the damage to the structure, the impact on the flow of people and goods in the transportation network and the importance (criticality) of the structure (*2*).

At the Michigan DOT, when an inspector identifies a structural element that has distress that may need high priority repair that could impact traffic, a Request for Action (RFA) is submitted. A team of Michigan DOT bridge engineers meets each month to review and prioritize

RFAs based on urgency. For example, a Priority 1 RFA will require the Michigan DOT statewide bridge crew do repairs as soon as possible. For a Priority 2 RFA, a special needs construction contract will be done to complete the repairs. The team considers several factors including location of deterioration, severity of deterioration, structural redundancy, and location of distress in relation to traffic or other loads. Based on these prioritized levels, the structures are temporarily or permanently repaired by internal forces, maintenance contracts or programmed for work within the capital program. Funding is set aside at a statewide level so that action can be taken without delaying or deferring other work within the region's 5-year plan.

Scour risk assessment is performed by the Michigan DOT on an annual basis as part of the bridge call for projects process. Data items that impact scour vulnerability and criticality are queried from the bridge database and imported into a spreadsheet. Data items impacting vulnerability include scour criticality (NBI Item 113), number of substructure units in the water, soil type, and presence of existing scour mitigation. Data items affecting criticality include average daily traffic, detour length, economic importance, and detour length. Many of the items are Michigan DOT-specific and may not be collected by all states. The data items selected, as well as the scoring and weighting of these items were revised at the direction of and approved by the interdisciplinary Michigan DOT Scour Committee. In addition to standard NBI data, the assessment uses data from the Scour Plan of Action to more accurately determine scour vulnerability. The various data items are scored and weighted to arrive at a final vulnerability and criticality score for each structure. These scores are plotted as shown in Figure 8.

Those bridges with high criticality and high vulnerability, as well as scour critical Interstate bridges, are documented in the call for projects submittals and progress is monitored.

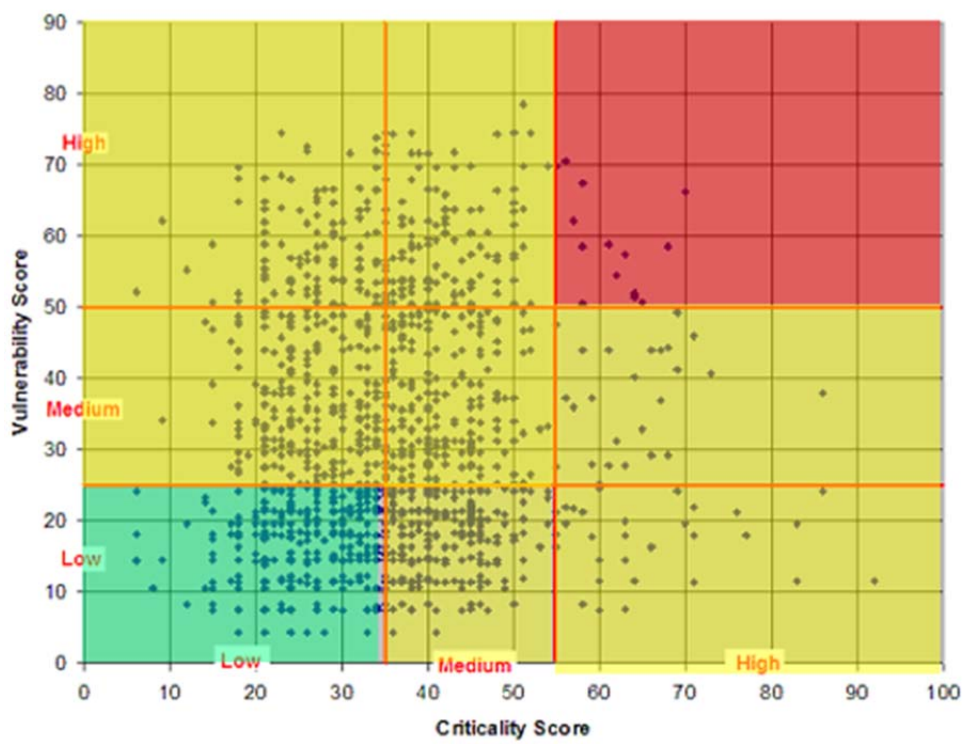

FIGURE 8 Michigan DOT scour risk assessment.

As opposed to the replacement prioritization where the focus is on minimizing impacts to the transportation system by reducing risk, the Michigan DOT's bridge preservation program prioritizes lowering life-cycle cost by maintaining structures in fair condition (lowest major condition rating of 5 or 6) and preventing the need for full replacement. The goal is to prevent bridges rated fair (NBI 5) from deteriorating to poor (NBI 4) condition by performing rehabilitation or preventive maintenance. Prediction models have been developed to identify those bridges that are likely to become poor the earliest. Due to the recent transition from CoRe elements to NBE elements, the models currently rely on NBI data but the process will soon include element data. Years of NBI data collection have made it possible to create deterioration curves for the deck, superstructure, and substructure ratings and the number of years that a bridge can expect to last with a fair condition (NBI 5) can be estimated. By querying past inspection data, the number of years that a major component rating has been fair (NBI 5) is known and thus the number of years remaining fair can be calculated. As part of the call for projects process, a listing of all fair rated bridges, along with their predicted year to become poor, is provided to the region bridge engineers. Those that are due to become poor soon (or even overdue) are given highest priority consideration for preventive maintenance work.

It is important to remember that individual bridges are often part of a corridor of bridges. To minimize the impact on highway traffic, the Michigan DOT coordinates bridge projects within a corridor in the same construction season. Often it is practical to advance a project on a particular bridge if other bridges within the corridor are in more urgent need of preventive maintenance work. This not only lessens the negative impact on traffic, but also allows for more efficient work if similar projects on a series of bridges are done at the same time. For corridors of particular importance, the work should be planned with the long-term goal of minimizing the number of times that traffic will be disrupted and anticipated future needs addressed at the same time as current needs. Additionally, the bridge projects should be coordinated with pavement projects in the corridor.

Decision Support

MBE Section 3 concludes by pointing out that bridge management is not strictly a data analysis or analytical process.

> The function of a BMS is to provide bridge information and data analysis capabilities to improve the decision-making abilities of bridge managers. A BMS should not make decisions. Bridges cannot be managed without the practical, experienced, and knowledgeable input of the engineer/manager. A BMS is never used in practice to find one best policy among the possible choices. Instead, managers should use the BMS as a tool to evaluate various policy initiatives, often referred to as "what if" analysis. The available choices may relate to network-level decisions or project-level decisions (*2*).

The Michigan DOT very much agrees with this. Every bridge has a unique history, location, and impact within the transportation system and the community that it is found. Often, bridge databases do not contain sufficient information to identify the full impact of planning a project, such as the community activities and festivals that might control the construction schedule, the endangered species that could delay the timing of scour mitigation, or the safety and pavement project that will require a bridge replacement to meet alignment requirements. The central office Bridge Management Section within the Michigan DOT recognizes that the bridge program is one part of the overall transportation program. The goal of the section is to provide tools to make the network level decision making as easy as possible so that the region and design staff resources can focus on the more nuanced and unique project level issues to maintain a safe, efficient and effective transportation network.

CONCLUSION

This paper includes highlights of the updates being proposed to the AASHTO MBE Section 3. The updates are being made to reflect progress in state-of-the-art practice and results of research in BMSs. This paper demonstrates the application of these practices by the Michigan DOT. The Michigan DOT leverages a collection of integrated tools to assist the agency in meeting strategic objectives. This paper includes examples of upper management use as a strategic planning tool including performance measurement, the use of dashboards, network-level condition forecasting, and tracking bridge deterioration. The paper also includes examples of the Michigan DOT's use of their integrated BMS at the project level by technical decision-makers through element-level condition data and agency rules to identify and prioritize bridge projects for preventive maintenance, rehabilitation, and replacement. Through this network- and project-level approach to bridge management, the Michigan DOT works to enhance safety, extend the service life of bridges, and serve commerce and the motoring public.

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Bridge-Level Risk and Resilience

Risk Assessment for Bridge Management Systems

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National Cooperative Highway Research Program (NCHRP) Project 20-07, Task 378 was commissioned by the American Association of State Highway and Transportation Officials (AASHTO) Subcommittee on Bridges and Structures to develop a Guideline for Risk Assessment for Bridge Management Systems, to be used within a bridge management system (BMS) to estimate the beneficial effects of bridge risk mitigation and replacement on transportation performance, as a part of methods for project utility and benefit–cost analysis. AASHTOWare Bridge Management was explicitly targeted, but the methodology is intended to be usable with any BMS using the data typically collected by, or available to, U.S. highway agencies.

The final guideline, dated September 2016, which will not be published, but is available from NCHRP as a PDF file, describes methods for developing service disruption scenarios, and then estimating the likelihood and consequences of these scenarios. Likelihood probability models are provided for 16 hazards including earthquake, landslide, storm surge, high wind, flood, scour, wildfire, temperature extremes, permafrost instability, overload, over-height collision, truck collision, vessel collision, sabotage, advanced deterioration, and fatigue. Consequences of service disruption are estimated in dollars for recovery cost, safety, mobility, and environmental sustainability. All of these models are based on published research gathered from a wide variety of sources, and consistent with the AASHTO *Guide for User and Non-User Benefit Analysis for Highways*.

The economic basis for risk assessment is designed to be compatible with existing use of life-cycle cost analysis in BMS, as well as with the utility framework provided in AASHTOWare Bridge Management. This paper introduces the methodology to inform the bridge management community of the new resource soon to be available.

Transportation asset management uses data and analysis to improve decision-making, with the goal of providing the desired level of service in the most cost-effective manner. A bridge management system (BMS) contains features to apply bridge inventory and condition data to assess the costs and benefits of preservation, risk mitigation, and replacement activities, to support management decision-making processes such as project definition, priority setting, resource allocation, and programming.

BMS have long had functionality to estimate the effects of agency actions on the long-term cost of maintaining the desired level of service. Recent BMS innovations have opened the door to multiobjective assessment of additional stakeholder concerns including safety, mobility, and environmental sustainability (*1*). Bridges affect these concerns by means of their functional characteristics, and by means of the risk that natural or manmade hazards might disrupt transportation service.

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BMSs typically provide functions to capture inventory and inspection data for each bridge, and then provide a set of mathematical models to analyze each bridge to forecast future conditions, performance, and costs (Figure 1). As a part of this functionality, BMS apply a set of decision rules to generate one or more alternative projects intended to relieve performance deficiencies or to reduce future costs. The software forecasts future performance and costs conditional on a project alternative and implementation year. A do-nothing scenario is also analyzed using similar models. By comparing each project alternative with the do-nothing alternative, a project benefit is estimated.

Typically a BMS will generate far more project candidates with positive benefits than can be funded under anticipated resource constraints. It then becomes necessary to prioritize. Practically all modern BMS use a benefit/cost ratio as the priority-setting criterion. Given a list of selected projects in a fiscally-constrained program, the BMS estimates future network condition and performance outcomes. Such estimates can be used for evaluating and comparing program alternatives, and for establishing performance targets and resource allocations.

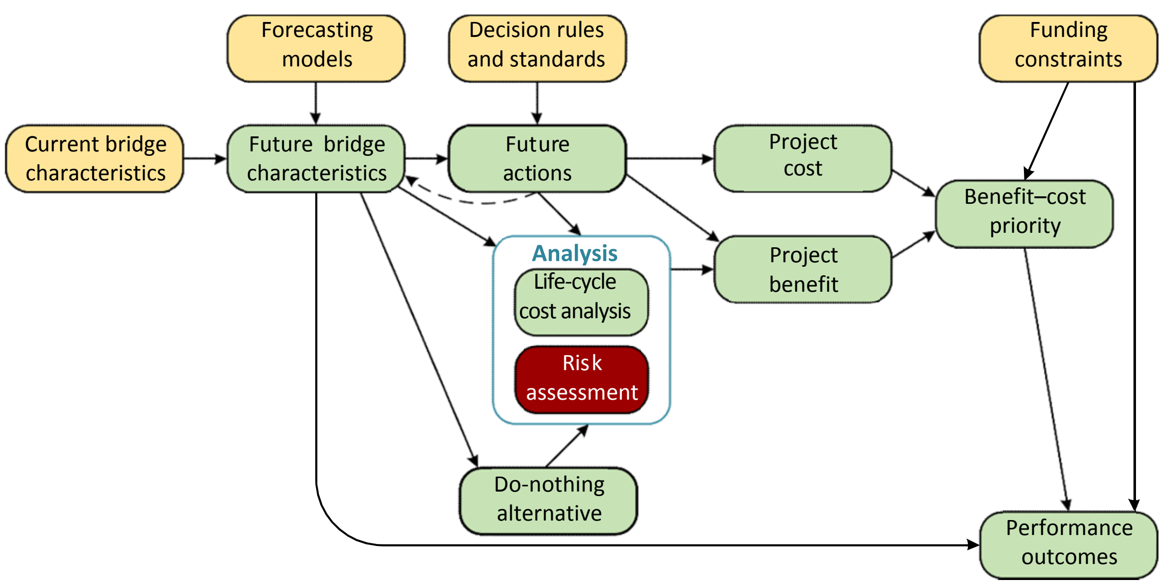

FIGURE 1 Role of risk in a BMS framework.

Most fully developed BMSs compute project benefits using a life-cycle cost analysis, as depicted in Figure 1. In some cases, this life-cycle cost analysis can include the user costs associated with functional deficiencies. Risk assessment that is fully integrated with this BMS analysis framework adds a second model to accompany the life-cycle cost analysis in computing project benefits. The risk assessment uses information about the project and the effects of the project on future bridge characteristics, to compute a portion of the project benefit.

PERFORMANCE CONCERNS AND MEASURES

Transportation agencies typically list their major goals and objectives in their enabling legislation, mission statements, strategic plans, or other broad policy documents that communicate with stakeholders and the public. For transportation asset management in general, a set of national goals have been defined by the Congress in 23 USC 150(b):

- Safety,
- Infrastructure condition,
- Congestion reduction,
- System reliability,
- Freight movement and economic vitality environmental sustainability, and
- Reduced project delivery delays.

Congestion reduction, system reliability, and freight movement are often considered together as "mobility." Elsewhere in the legislation, agencies are also called upon to minimize long-term costs and manage risks. Each state department of transportation (DOT) typically has a similar list of goals. In bridge management decision-making, the most relevant concerns are cost, safety, mobility, and environmental sustainability, with condition and risk potentially influencing all of the other goals.

When conditions of individual bridges are compared with each other, or when one bridge is tracked over time, it is common practice to use a bridge health index or a good–fair–poor classification as in recent federal rules (23 CFR 490) (*2*). To characterize risk on a comparable basis, it is possible to use a concept of resilience or vulnerability (*3*). Network conditions or resilience can be characterized as the percent good or percent poor, perhaps weighted by deck area as in the federal rules.

For benefit–cost priority setting, it is necessary to describe performance of the network as affected by a given bridge. Cost varies from one bridge to another based on size and potentially other factors. Risk mitigation benefits also vary from bridge to bridge because of traffic volume, detour length, and potentially other factors. NCHRP Report 590 explores ways of combining safety, mobility, and other project benefits, taking traffic and detour length into account, to estimate project benefits as a type of utility function (*1*). As an alternative, it is possible to estimate project benefits in the form of user costs, as is done in Florida DOT's analytical process (*4*). This latter approach simplifies the means of combining risk benefits with life-cycle cost reduction. The guideline described in this paper supports both methods.

OVERALL FRAMEWORK

The guideline presents the risk assessment procedure as a series of worksheets. While the worksheets could in principle be filled out by hand, most agencies will want to implement them either by entering corresponding data in AASHTOWare Bridge Management, or by creating a spreadsheet or other software to run the calculations. The worksheet format is intended to make the structure and data requirements as transparent as possible.

Each agency will want to choose which hazards and performance criteria to address, and customize the procedures to fit their own needs and resources. The modular worksheet structure is designed to allow agencies to "mix and match," that is, to plug in the modules which best fit their needs (Figure 2). Depending on the hazards to be addressed, the agency will choose among plug-in modules for the likelihood of extreme events, the likelihood of service disruption, and the consequences of service disruption. The agency defines a set of hazard scenarios and applies the modular analysis to compute social cost or utility for use in the priority setting and resource allocation functions of the BMS.

Hazard Scenarios and Performance Criteria

The disutility of an adverse event depends on the nature and magnitude of the hazard, and on the effect on each performance concern. In order to reflect these influences in a reasonable way, the following concepts are defined:

- Hazard scenarios (denoted in the equations using the subscript h) entail extreme events of a specific magnitude (if applicable), or the cumulative effect of an ongoing adverse process, causing a defined impact on transportation service. For example, a hurricane of at least magnitude 4 that destroys a bridge.
- Performance criteria(denoted using the subscript c) represent agency objectives that may be compromised by a hazard scenario. Examples are cost, safety, mobility, and environmental sustainability.

Each agency will select the hazard scenarios and performance criteria to be analyzed consistently across all bridges. An important decision is the level of disruption that should be incorporated into the threshold for recognition of a hazard scenario. Some of the options include

- The structure is damaged to at least a defined damage level, typically corresponding to the agency's distinction between routine work orders for repair, and programmed capital projects for mitigation, rehabilitation or replacement.
- Near-term or long-term life-cycle costs are increased.
- Transportation service is disrupted, causing a loss of performance in terms of safety or mobility.
- Environmental resources or the property of others are damaged.

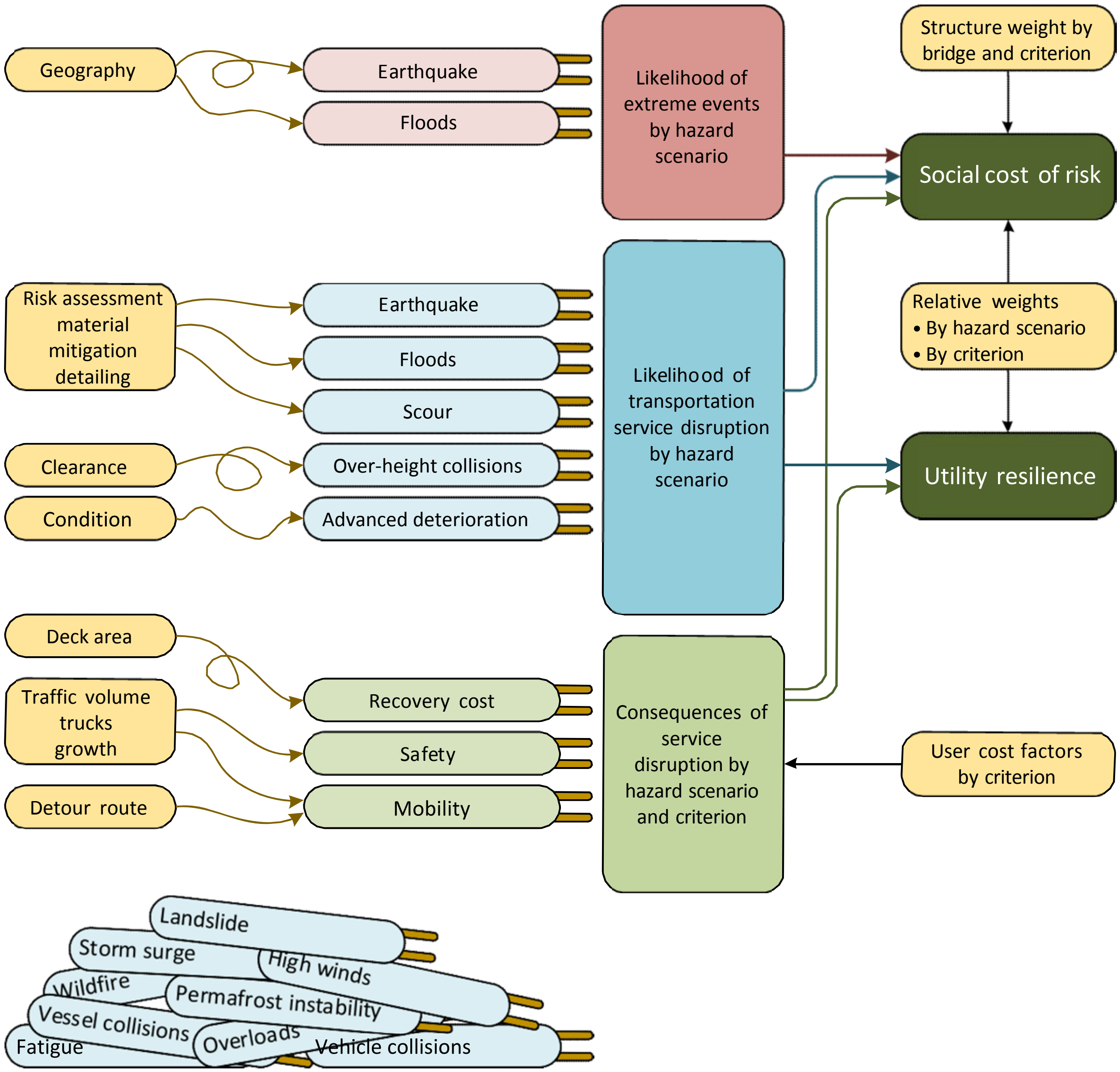

FIGURE 2 Plug-in architecture of the recommended risk analysis.

Any or all of the above could have a role in defining the criteria for a hazard scenario. For an understandable and consistent analysis, however, it is important to be consistent in definitions across all hazard types. The guideline is flexible in allowing agencies to adopt any reasonable set of criteria. However, the service disruption criterion is recommended for primary emphasis, for the following reasons:

- Most of the states having existing risk management capabilities as part of bridge management use this as their criterion.

- For most hazard classes, events that cause service disruption also cause structure damage.
- Service disruption events are typically regarded as more severe than damage-only events, and are more likely to be captured in historical records.
- Damage that is significant enough to disrupt service is typically more expensive to repair and more urgent than damage that does not disrupt service.
- Events belonging to some of the hazard types are not typically recognized as risk consequences unless they disrupt service. Examples are extreme temperature, settlement, advanced deterioration, and fatigue.

In considering which hazards to include in the BMS risk analysis, the following questions should be considered:

- Within the agency's jurisdiction, does the hazard occasionally cause service disruptions or otherwise meet the criteria for a hazard scenario? "Occasionally" should be interpreted in a consistent way, such as at least once every 100 years for a given bridge.
- Does the likelihood or consequences of the hazard scenario differ from one structure to another or one part of the jurisdiction to another? This likelihood could apply to extreme events, to structure damage, or to service disruption. Consequences could apply to any agency objective such as cost, safety, mobility, or environmental sustainability.
- Does the hazard apply to a significant number of bridges? If only a handful of bridges can experience the hazard, then it might be more appropriate to perform site-specific analyses rather than including a model within the BMS.
- Does the agency have treatments available to mitigate the hazard that would be programmed using the BMS? Bridge replacement is a relevant treatment, but in that case the question is, does the magnitude of the hazard make a difference in the choice of replacement or in the priority of replacement?
- Is the hazard significant enough in decision making to justify the additional data collection, particularly field assessment that may be required in order to consider the hazard within the BMS?

The level of detail represented in hazard scenarios can vary based on agency preference. It is likely that most agencies will want to keep the model simple by defining only a small number of scenarios to represent the broader range of possible adverse events. Increasing the number of scenarios increases the development and computational effort, but gives a more precise estimate of outcomes and risk.

If a hazard scenario includes the occurrence of an extreme event, it is desirable to use the event magnitude for which agency's structures are typically designed. For example, if bridges are typically designed to withstand a 100-year flood, then the 100-year flood is the extreme event magnitude to use, and the extreme event probability is 1%.

Project Summary Worksheet

Figure 3 shows the Project Summary Worksheet, presenting intermediate and final results of the risk calculations for a single bridge and project as developed in the Guideline. Input data requirements are provided in the upper two blocks of cells.

The lower two blocks of cells are calculation results. Green cells are results, in thousands of dollars, gathered from worksheets provided for each type of consequences. Red cells are extreme event probabilities gathered from worksheets for each type of hazard scenario. Blue cells are service disruption probabilities gathered from worksheets for each type of hazard scenario. Gray cells are configuration parameters set on a separate worksheet. White cells in the lower two blocks are calculated on the worksheet itself.

NCHRP 20-07 (378) Risk Analysis

Sheet B - Project summary

Bridge ID	**010001**		
Alternative	**Do nothing**	Deck area (sq.ft)	20,000
Program year	**2017**	Program cost ($000)	12,345

Roadways	On structure				Under structure			
Func. class	11–Urban interstate				14–Urban, other principal arterial			
Utilization	ADT	54,000	Trucks	5.50%	ADT	21,000	Trucks	3.00%
Roadway	Length (ft)	200	mph	55	Length (ft)	100	mph	45
Detour	Miles	2.1	mph	45	Miles	1.0	mph	45

From BMS data. If multiple roadways, use the total ADT and most significant roadway, projected to program year. Length on-structure is bridge length. Length under-structure is bridge width.

Hazard scenarios		Consequences ($000)				Likelihood			Risk
I	**Scenario**	**Cost**	**Safety**	**Mobility**	**Environment**	**Extreme**	**Disruption**	**Weight**	**Cost ($k)**
1	Earthquake-100	12,34	50	6,00	600	1.00%	5.00	1.00	9.50
2	Flood 100a	12,34	50	6,00	600	1.00%	10.00	1.00	19.00
3	Flood 100b	100	0	2,00	200	1.00%	20.00	1.00	4.60
4	Flood 500	12,34	50	6,00	600	0.20%	50.00	1.00	19.00
5	Overheight	100	70	200	40	--	5.00	1.00	20.50
6	Deterioration	50	0	200	40	--	10.00	1.00	29.00
7	Fracture	12,34	0	6,00	600	--	0.50	1.00	94.73
8								1.00	0.00
9								1.00	0.00
10								1.00	0.00

Risk cost and vulnerability	Cost	Safety	Mobility	Environment
Struc. weight	20,000	75,000	134,400	134,400
Criteria	1.00	1.00	1.00	1.00
weight Risk	102.79	3.63	79.00	10.90
cost ($k)	5.1394	0.0483	0.5878	0.0811

Risk analysis results	
Maximum unit risk cost:	100.00
Vulnerability index:	0.0586
Utility:	**94.14**
Social cost of risk ($000):	**196.31**

FIGURE 3 Project summary worksheet.

Likelihood of Service Disruption

The likelihood of service disruption in this framework varies by bridge, based on bridge characteristics, and also varies by hazard scenario. It has two parts:

LEbh = likelihood of occurrence of the extreme event of given magnitude that is specified by hazard scenario *h*, estimated for bridge *b*.

LDbh = likelihood of a specific magnitude of service disruption, conditional on the occurrence of the extreme event specified in hazard scenario *h*, estimated for bridge *b*.

The total likelihood of hazard scenario *h* on bridge *b* is LEbhXLDbh . The two likelihood estimates are separated because different data sources and methods are used to calculate them, as described below. These likelihoods are the probability of the indicated event occurring in any one year. Agencies using AASHTOWare Bridge Management may want to use the Assessments feature of that system as a basis for estimating one or both of the likelihoods.

Consequences of Service Disruption

Consequences are defined as an economic quantity that varies by bridge, based on bridge and network characteristics. It also varies by hazard scenario and performance criterion.

CQbhc = consequence, estimated in dollars per disruption event, to performance criterion *c* on bridge *b*, conditional on the occurrence of the service disruption specified in hazard scenario *h*.

Consequences include the agency costs of disaster recovery as well as an economic value assigned to safety, mobility, and environmental impacts. The dollar value of recovery cost is typically estimated using economic models and normal agency cost estimation practices, or by classifying potential losses in ranges using judgment. Methods for other types of consequences are described below.

Performance Measures

The basic ingredients described in the preceding section are used to compute performance measures for decision support purposes. The following performance measures are needed:

RCb = Social cost of risk for bridge *b*. This variable should be structured and scaled so a savings in cost can be used in the benefit of a benefit–cost ratio for priority-setting, and so the BMS resource allocation and optimization models can minimize it network-wide. It may increase over time due to deterioration, traffic growth, or increased hazard likelihood; and it may decrease if an agency action improves bridge characteristics such that life cycle costs, risks, or road user inconvenience are reduced. Its values can range from 0 to positive infinity.

Ub = Utility for bridge *b*. This variable should be structured and scaled so it can be understood as the degree of resilience of an individual bridge. It provides a uniform unitless scale for comparing the status of one bridge with other bridges, or for tracking performance of a bridge over time. Its values can range from 0 to 100, where 0 is the worst possible performance and 100 is the best possible performance.

Social Cost

In the recommended methodology, social cost of risk is the weighted sum of the social costs of all hazard scenarios and all performance criteria:

$$RC_b = \sum_h \sum_c RC_{bhc} \quad (1)$$

RCbhc = statistical expected value of weighted social cost, in dollars per year, of hazard scenario *h* on bridge *b* for criterion *c*.

$$RCbhc = Wc\ XWhXLEbhXLDbh\ XCQbhc \quad (2)$$

The variable *Wc* is a weight given to each performance criterion in the cost equation. It should be 1.0 by default, but can be more or less than 1.0 to increase or decrease the contribution of a criterion in the calculation. For example, if *Wc* = 1.2 for *c*=safety, then safety is given 20% additional cost in the risk calculation. Similarly, *Wh* is a weight given to each hazard scenario. For example, if *Wh* = 1.1 for *h* = earthquakes, then earthquakes are given 10% additional cost, perhaps to reflect the difficulty of incident response and the importance of supporting evacuation plans. The other variables in this equation are computed from bridge and network characteristics as introduced above.

Utility

Utility is a concept related to social cost, but is designed to be used when making a direct comparison between bridges (disregarding their relative size), or when tracking performance over time. It is equivalent to resilience. The scale is intentionally designed so each bridge can potentially score a perfect 100 or a worst-case 0 depending on its ability to resist hazards. By definition, agency actions should be able to improve this resilience to nearly 100 on any bridge, given sufficient resources.

Depending on the structure of the bridge management system, there may or may not be a mathematical relationship between utility and social cost. AASHTOWare Bridge Management, for example, is designed to compute utility first, at the work candidate level, and then convert this to social cost at the program level for computation of the benefit/cost ratio. Other systems may compute utility from social cost, or treat the two concepts as equivalent, or compute the two measures independently. Utility is meant primarily as a communication tool, while social cost is more rigorously defined for priority-setting and resource allocation.

To compute utility, it is common to first compute vulnerability as the product of likelihood and consequence of each separate adverse scenario for each separate performance criterion. Then the results are additive, and utility is:

$Ub = (1 - Vb)X100$ (3)

The quantity Vb can be called the vulnerability index, on a scale where 1.0 is maximum vulnerability and 0 is no vulnerability. One way to compute vulnerability is:

$$V_b = \frac{URC_b}{MaxURC} \quad (4)$$

$$URC_b = \sum_h \sum_c (RC_{bhc}/sW_{bc}) \quad (5)$$

The value URC_b can be called the unit risk cost. It is the same risk cost as in equation 2 except that it is normalized to remove the effects of consequence scale. MaxURC is determined by computing URC_b for every bridge (or a representative set of bridges) in the database and finding the maximum value, which then defines the worst end of the vulnerability scale for the agency. sW_{bc} is called the structure weight, and is computed in different ways for different performance criteria, as follows:

Cost	Deck area (ft^2)	
Safety	Average daily traffic (ADT)	(6)
Mobility	ADT × detour length (miles)	
Environmental sustainability	ADT × detour length (miles	

After an agency first computes or estimates its MaxURC, this quantity is not likely to change very quickly over time. Therefore it might not be necessary for the agency to re-compute this constant unless it makes significant changes in its risk assessment process, such as by adding more hazards.

The advantage of having a linear relationship between vulnerability and social cost is the fact that social cost can be computed from vulnerability, which is a necessity for AASHTOWare Bridge Management and is desirable for keeping any BMS framework relatively simple.

SUBMODEL EXAMPLES

The following sections provide examples of the modules documented in the Guideline for likelihood of extreme events, likelihood of service disruption, and consequences of service disruption. The study collected methods from existing literature and did not have resources to develop new methods. There are significant opportunities for future research to develop new and better methods for many of the hazards and criteria that can be assessed within this framework.

Likelihood of Extreme Events

Certain hazards, specifically earthquake, landslide, storm surge, high wind, flood, wildfire, extreme temperature, and truck collisions, are triggered by short-duration events which are unusual and unexpected at any given site, but which occur with regularity across the inventory. Some of these hazards, such as earthquakes, are so abrupt that they have unavoidable safety consequences. Others, such as floods, occur with some advance warning, allowing operational practices which may improve safety in exchange for a compromise in cost or mobility.

What all the extreme events have in common is that a portion of the likelihood of service disruption is unaffected by normal agency actions, but is related more to bridge location. This can be significant for decision making because, for example, an agency is powerless to prevent earthquakes, but can, with appropriate resource allocation, make programmatic decisions that increase the ability of bridges to resist earthquakes.

Natural Extreme Events

For a given agency, geographically referenced data on extreme event likelihood may be available from several sources. Ideally, such a data set has polygons representing zones where the event return period is estimated to be 100 years. This return period is most appropriate for bridge risk analysis since it is most likely to approximate or exceed the remaining service lifespans of most bridges. It is equivalent to a probability of 1%. Such data sets often have polygons for alternative return periods such as 20 years or 500 years, which can form the basis for defining additional hazard scenarios if this is applicable for decision-making. Alternative return periods also can be used for interpolating extreme event probabilities for locations between polygon boundaries.

As an example, the U.S. Geological Survey (USGS) National Seismic Hazard Maps (Figure 4) display earthquake ground motions for various probability levels across the United States. They represent a uniform probability (either 2% or 10%) that the ground acceleration will exceed the given value over 50 years. These can translate directly to an event likelihood for a corresponding hazard scenario. FEMA, NOAA, USFS, and various state agencies are potential sources of geographic hazard data.

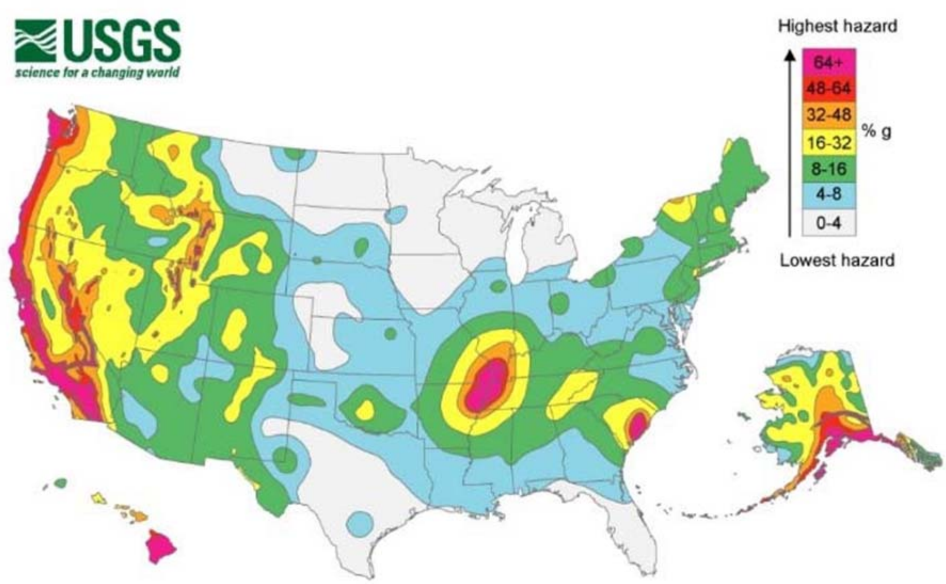

FIGURE 4 USGS National Seismic Hazard Map.

Manmade Extreme Events

The Guideline also provides suggested methods for gathering incident data that might be used for human-caused extreme events such as overloads, over-height truck or tanker truck collisions, vessel collisions, and sabotage. In some cases existing data sources can be used to gain insight into the overall statewide probability of some of these hazards, which can form the basis for individual site estimates.

Likelihood of Service Disruption

For hazard classes that involve extreme events, the likelihood of service disruption is the probability that service is impacted, conditional on the occurrence of the related extreme event scenario. While the extreme event probability depends mainly on location or other exogenous factors, the service disruption probability typically depends on structure characteristics. Certain hazards, such as advanced deterioration, are not associated with extreme events but have service disruption probabilities (e.g., restricted load ratings requiring posting) that depend only on structure characteristics. Models are provided for the following hazards:

Earthquake	Wildfire	Vessel collision
Landslide	Temperature extremes	Sabotage
Storm surge	Permafrost instability	Advanced deterioration
High wind or tornado	Overload	Fatigue
Flood	Over-height collision	
Scour	Tanker truck collision	

The Guideline presents worksheets and examples for several generic methods to approximate the disruption probability that can be applied to most hazard classes even with minimal data availability. For example:

- Assessments. Using the AASHTOWare Bridge Management feature for risk assessments, a probability is assigned to each likelihood category based on judgment, derived perhaps from a Delphi or analytic hierarchy process.
- Scoring tables or decision trees. A set of objective criteria, using BMS data items, are used to group bridges into categories of vulnerability, then those categories are scored and converted to a probability.
- Analogies. Using a Delphi-type process to lead a panel of experts to a likelihood estimate based on comparisons with other hazards having known frequencies.
- Polling. Asking a group of knowledgeable individuals (e.g. area maintenance supervisors) to list past incidents from memory, or to estimate the frequency of such incidents. This can also be used to estimate other needed parameters such as extent of damage and length of closure.
- Risk allocation. Use statewide expenditures, news reports, polling of maintenance personnel, and other historical data to estimate the total damage and disruption statewide. These totals are scaled for network growth and inflation, and then allocated among bridges to reflect bridge characteristics that make each structure more or less vulnerable.

These methods can be used separately or in combination, exploiting whatever data the agency can locate, to bracket reasonable risk estimates for each hazard class. Table 1, for example, is adapted from a scoring table used by Minnesota DOT (*5*) to set the relative likelihood of service disruption based on a field-assessed scour rating. It can be used in combination with statewide frequency estimates to compute a probability for each bridge using the risk allocation method.

The Guideline also documents methods developed from past research studies that are specific to certain hazard classes, most notably for scour, overloads, over-height collisions, tanker truck collisions, terrorism, advanced deterioration, and fatigue. In some cases, such as scour, multiple alternative models can be found in the literature. For example, New York State DOT has its own scour decision tree model (*3*), and an NCHRP study developed a risk allocation model for scour based on national datasets (*6*).

Consequences of Service Disruption

The framework used in the Guideline relies on the clear definition of service disruption scenarios to formulate the likelihood × consequence concept of risk in a way that can reasonably be estimated using quantitative methods. Project benefits related to risk can then be represented by a statistical expected value calculation of avoidable social cost, comparable to existing benefit calculations based on avoidable life cycle agency cost. The likelihood models described above provide the typical probability or frequency, each year that a service disruption event can be expected to take place. Consequence models then assign a dollar value to each disruption event (Figure 5).

TABLE 1 Scoring Table for Scour

Bridge Scour Susceptibility					
		Defect Reduction			
Code	Description	None	2	3	4
A	Not a waterway	100	100	100	100
E	Culvert	100	100	100	100
M	Stable; scour above	90	90	70	40
H	Foundation above water	90	90	70	40
N	Stable; scour in	80	80	60	30
I	Screened; low risk	70	70	50	30
L	Evaluated; stable	70	70	50	30
P	Stable due to protection	60	60	40	20
K	Screened; limited risk	60	60	30	20
F	No eval.; foundation	50	50	40	20
C	Closed; no scour	50	50	25	20
J	Screened; susceptible	40	40	30	10
O	Stable; action required	40	40	20	10
G	No eval.; foundation	20	20	15	10
R	Critical; monitor	10	10	5	0
B	Closed; scour	0	0	0	0
D	Imminent protection	0	0	0	0
U	Critical; protection	0	0	0	0
Defect reduction: Use worst condition state of defect 6000, Scour					

Fortunately, the social cost calculations used in this analysis are already standardized in the AASHTO Red Book, which is widely used in life-cycle cost, value engineering, and regulatory analyses (*7*). The Red Book provides unit costs for accidents, travel time, and vehicle operations. The Guidelines provide worksheets and examples of research-based procedures to estimate the safety and mobility impacts, in terms of excess accidents, hours of delay, and miles of detour. These can then be converted to dollars using AASHTO Red Book parameters.

Figure 5 shows an example worksheet for mobility consequences, which entail detours while a bridge is monitored, repaired, or rebuilt, and may have smaller impacts such as truck restrictions or speed reductions. The mobility cost per disruption event is:

$$CQ_b = ADT_b \times (DD_b DL_b/24) \times (VOC\$ + [TT\$ \times VO]/DS_b) \tag{7}$$

ADT_b = forecast vehicles per day affected;
DDb = duration of the disruption, in hours;
DLb = detour length in miles;
VOC$ = average vehicle operation cost per mile;
DS_b = detour speed in mph;
TT$ = travel time cost per hour; and
VO = average vehicle occupancy rate, people per vehicle.

This formula can be recognized as a method long used in pavement and bridge management systems for functional deficiencies, and relies on the same planning parameters. In fact, agencies using the AASHTOWare Bridge Management software may want to combine the mobility risk model and the benefit model for functional improvements, since the two models are very similar.

In addition to recovery cost, safety, and mobility, the Guideline also presents an environmental sustainability model based on estimates of vehicular emissions. It uses the approach from FHWA's Highway Economic Requirements System (*8*). This methodology, updated from earlier research in California, relies on a study that simulates vehicular air pollution emissions under various scenarios of congestion, speed, and volume. Six pollutants are included in the analysis: carbon monoxide, volatile organic compounds, oxides of nitrogen, sulfur oxides, small particulate matter, and road dust. To establish a dollar value and relative weights of the pollutants, the study uses earlier research on the economic impact on health and property damage caused by these pollutants.

Integration with Bridge Management Systems

In addition to a detailed treatment of AASHTOWare Bridge Management, the Guideline also summarizes alternative approaches to bridge management functions that incorporate risk assessment, highlighting software used in Florida, Minnesota, and New York. It also discusses the mathematical relationship with life cycle cost analysis, for agencies or vendors that may want to develop new spreadsheets or systems that apply the models to support management functions such as treatment selection, priority setting, resource allocation, programming, and target setting.

NCHRP 20-07 (378)Risk Analysis			
Sheet CQ - Mobility			
1	Bridge ID	**010001**	
2	Forecast year	**2018**	
3	Hazard scenario	**Earthquake**	
Prediction of traffic volume			
4	Average daily traffic (NBI 29)		23.0
5	Year of average daily traffic (NBI 30)		201
6	Future average daily traffic (NBI 114)		29.0
7	Year of future average daily traffic (NBI 115)		203
8	Growth rate (g)	1.17%	
9	Projected average daily traffic (ADT)	25,235	
Cost of detoured traffic			
10	Funct class (26)	14 - Urban other principal arterial	
11	Duration of the disruption (DD) (hours)		5.0
12	Detour length (DL, NBI 19) (miles)		2.2
13	Vehicle operating cost (VOC$) ($/mile)		0.20
14	Detour speed (DS) (mph)		45
15	Travel time cost (TT$) ($/hour)		30.6
16	Vehicle occupancy (VO) (persons/vehicle)		1.30
17	Total Social Cost	**12,637**	

FIGURE 5 Consequence submodel for mobility.

CONCLUSIONS

The Guidelines document produced by NCHRP 20-07(378) will be of considerable help to agencies wishing to incorporate realistic risk assessment models into the decision support functions of bridge management systems. Based on a wide range of existing research studies, the models have been simplified as needed so they are compatible with the data and software commonly available to transportation agencies. When bridge management systems such as AASHTOWare Bridge Management are configured to use these models, no significant additional effort is required in order to consider risk routinely in combination with life cycle cost in decision making.

While the models are quite simple when decomposed into their parts as described here, they have the advantage that they work with data that are widely available for all bridges in an inventory, are consistent across the inventory, are sensitive to common classes of risk mitigation and replacement actions, can aggregate to reasonable estimates of system wide risk, respond in reasonable ways to reasonable variations in the input data, and can be weighted according to agency preferences in a transparent way. Because the models are designed to follow real-world engineering and economic relationships, they can be improved through further research when agencies desire more precision. Some examples of potential research topics are:

- More and better applications:
  - An automated tool or spreadsheet, to implement the methods presented in the Guideline.
  - Adaptation to bridge design applications to compare alternatives.
  - Improved guidance on the identification and costing of risk mitigation treatments.
  - Models of the effectiveness of risk mitigation actions in reducing disruption likelihood.
- New or improved submodels:
  - Modeling of agency incident response processes and recovery costs.
  - National-scale risk allocation models similar to the scour example (*6*).
  - Incorporating carbon dioxide into the environmental sustainability model.
  - Modeling sea-level rise as a part of the applicable likelihood models.
  - Improved modeling of flood likelihood in addition to, or combined with, scour modeling.
  - A research-based model of the likelihood of over-height truck collisions.
  - Further development of the likelihood model of advanced deterioration.
  - Effects of bridge characteristics on vessel collision likelihood.
- Implementation:
  - Documentation of case studies based on actual agency use.
  - Training and outreach on implementation of the Guideline.

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Vulnerability
Top-Level Performance Indicator for Bridges Exposed to Flooding Hazards

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The oncoming natural hazards, especially floods, represent a serious threat to users of transportation infrastructure and societies in general. The state-of-the-art bridge management systems still do not comprehensively account for impacts of sudden events and there is a demand for a simplified methodology for quantitative assessment of a bridge performance over time on a network level, which will in turn lead to adequate performance measures with respect to flooding events. As a convenient tool for the assessment, the measure of vulnerability is suggested here as a top-level performance indicator. It is based on two values: the conditional probability of a bridge failure due to a flooding event of a certain magnitude and the related total consequences. The primary culprit for failures inflicted in floods is the local scour at bridge substructures. Here, the estimation of the conditional probability of a bridge failure is a multidisciplinary problem where the combined resistance of the supporting soil at substructures and the bridge is accounted via failure modes. The challenge is in setting the adequate vulnerability thresholds that trigger mitigation and preventative activities. Here the influence of a planned activity or an information update, on the assessment results must be taken into consideration in structuring of adequate quality control plans.

The most common culprit for inadequate bridge performance around the world is the flooding hazard and related local scour at bridge substructures (*1–3*). The painful reminders of a threat this hazard poses to the performance of road networks are the extreme flooding events in Taiwan in 2009 (*4*), and the most recent one in Serbia in 2014 (*5*). However, the transportation infrastructure is not only endangered by low occurrence–extreme intensity floods but also by less-extreme floods with relatively high occurrence rates (*6*). Thus, it is a fundamental responsibility of civil engineers to ensure adequate adaptation of the infrastructure in the face of future weather events. By rule, a validation or an update of bridge management (BM) practices only take place after an extreme event occurrence, which is not an adequate approach for ageing infrastructure. The mitigation of risk of bridge failures due to flooding and related local scour is one of the most extensively elaborated topics in BM in the past two decades, but still there are no comprehensive methodologies to cover this matter.

The 13 state departments of transportations (DOTs) that participate in the long-term bridge performance program agreed that one of the primary research needs is to reliably identify scour-susceptible bridges (*7*). The current methodology of the FHWA is qualitative and based on a specific National Bridge Inventory (NBI) item No. 113 which is related to scour-critical bridges. The ratings for the item are given based on engineering judgement supplemented by: visual inspection, indirect evaluations, and a condition state of applied countermeasures (*8*). There are suggestions to combine the value of item 113 with other relevant NBI items in a procedure which uses weighting factors to introduce an index–bridge sufficiency index for a more-comprehensive ranking of bridges (*9*). In some U.S. states, bridges are specifically ranked using qualitative assessments based on their hydraulic vulnerability and in turn scheduled for a specific plan of action (*10*). The scour vulnerability rating is recognized as one of the key performance measures for development of a multiobjective optimization model for BM systems (*11*).

In the state-of-the-art software for risk analysis of transportation infrastructure exposed to natural hazards, Road Risk, developed by the Swiss federal roads authority (*12*) and the HAZUS-MH (HAZards U.S. Multi-Hazard) (*13*), the resistance of a bridge to flooding scenarios is not adequately accounted for. In the latter case, the probability of a bridge failure due to scour is based on the bridge's structural configuration, relevant ratings from the NBI and a flood return period, while only the direct costs of failure are considered.

The performance of bridges is the key research topic in Europe as well. The ongoing European research project COST TU1406 has a goal to structure the guidelines for development of quality control plans (QCPs) for roadway bridges in Europe, thus enhance preparedness in face of future sudden or slow events (*14*). Within Work Group 3 of the COST project, one of the main tasks is to investigate and consider for the dynamics and uncertainty of the noninterceptable (i.e., sudden) processes, particularly floods, that can significantly affect the bridge performance. Here, the main challenge is selection of adequate performance indicators (PIs) and definition of triggering criteria for detailed inspections and maintenance interventions at bridge sites in respect to required quality levels.

VULNERABILITY AS A PERFORMANCE INDICATOR FOR BRIDGES EXPOSED TO FLOODING HAZARDS: EUROPEAN EXPERIENCE

The PIs relate to a set of observations and data on a bridge structure and bridge site, that can be either assessed, measured or evaluated, and which in turn can be used to assess bridge performance against predefined performance goals. In case of a flooding hazard, the PIs purpose is to point out which bridges are the most vulnerable to a hazard scenario, thus ensuring timely and adequate preventative actions.

Recently, in the research project COST TU1406, the survey for PI for roadway bridges has been performed in 30 European countries by screening of national BM guidelines (*15*). The results of the survey are summarized by Tanasic and Hajdin (*16*). Presented in Figure 1 are the key terms that relate to the reported PIs for flooding–scour. The most of the interviewed countries reported that in the case of flooding–scour their BM procedures solely rely on visual inspections. Some countries additionally perform measurements or monitoring of scour depth, while a few account for hydraulic adequacy of bridge openings. Only one country reported the application of a local scour evaluation formula, while seven countries have not reported that either flooding or scour are considered in their national BM documents. Although the detailed information of PIs for natural hazards were not in the primary scope of the survey, it may be concluded that there are no concise guidelines or QCPs in European BM practice for bridges exposed to a flooding hazard.

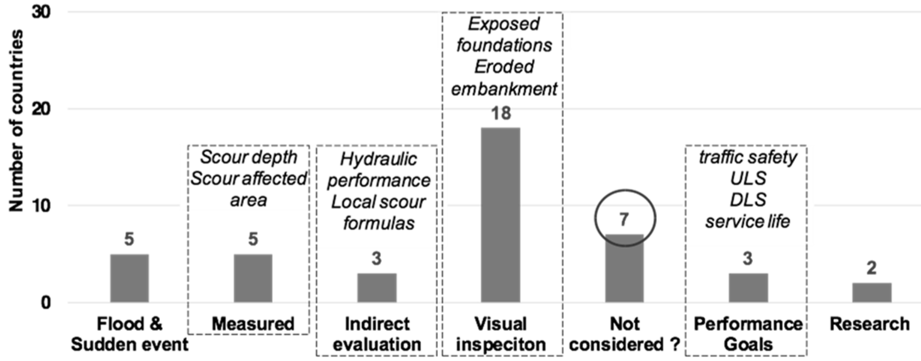

FIGURE 1 The terms related to flooding–scour in national BM guidelines in Europe.

The visual inspections of substructures or the information on measured–evaluated scour depth do not solely provide sufficient information for decision-making. Here the main concerns are eligibility of bridge sites for installing monitoring equipment and refill of scour cavities at substructures. It is evident that a more comprehensive PI must be applied to include all relevant information on a bridge exposure to a flooding scenario, its resistance to the related magnitude of a flooding event (i.e., failure modes) and resulting consequences of a failure:

- Exposure:
  - Flood magnitude and duration (i.e., a hydrograph),
  - Water channel geometry and properties, and
  - Piers and abutments location, geometry and alignment in respect to a water flow.
- Resistance to failure modes induced by local scour at substructures:
  - Properties of a soil at foundations (geotechnics and erodibility),
  - Type and detailing of substructures and superstructure, and
  - Location and severity of damage on relevant bridge elements.
- Consequences related to a specific failure mode:
  - Costs of repairs or replacement and
  - Network and traffic data to include indirect costs of failure: vehicle operating costs, accident costs and loss of travel time.

Clearly, a risk-based approach is the only viable solution to adequately consider an impact of flooding and the related local scour on bridges. In the evaluation of risk, the forecasting of sudden event magnitudes must be performed, which is a complex task, especially for flash flooding. The BM needs efficient procedures for comprehensive screening of an entire bridge population thus the quantitative measure of vulnerability of a bridge failure is suggested as the most adequate top-level performance indicator to account for all relevant information. It represents the product of a conditional probability of bridge failure in a hazard event of a specific magnitude and the total consequences of such event, i.e., it is reflected through monetary units (*17*):

$$V_n^s = P_n^s \cdot (DC_n + IC_n) \quad (1)$$

where:

V_n^s = vulnerability of a bridge with respect to a hazard event of a specific magnitude s and a chosen failure mode $\boldsymbol{n}$

P_n^s = conditional probability of specific bridge failure in the chosen failure mode $\boldsymbol{n}$, with respect to a hazard event of a specific magnitude $\boldsymbol{s}$

DC_n = direct consequences with respect to the chosen bridge failure mode $\boldsymbol{n}$

IC_n = indirect traffic related failure consequences with respect to the chosen bridge failure mode $\boldsymbol{n}$

Unlike the measure of risk, the vulnerability is more convenient to understand since it relates simply to the given hazard magnitude, which is deemed sufficient for the identification of bridges in a network that need to be examined in more detail.

Following the performed survey for PIs in Europe, the next task in COST TU1406 regarding flooding hazard is structuring of a questionnaire, which will reveal availability of the data necessary to conduct quantitative assessments, e.g., risk–vulnerability.

METHODOLOGIES FOR QUANTITATIVE VULNERABILITY ASSESSMENT

The development of BM systems (BMS) is underway in many countries, where one of the main tasks is the establishing of novel risk-based methodologies. The information on 25 BMS from 18 world countries is presented in the report (*18*) which is the outcome of the survey performed by International Association for Bridge Management and Safety (IABMAS). Herein, the findings showed that only a few BMS account for risk of a bridge failure due to hazards. Generally, the current risk-based approaches are mostly qualitative and comprise likelihood–consequences matrices, i.e., risk matrix. In such approaches, the term failure or failure mode is related to a certain level of damage (physical or functional) and following consequences, but neither accounts for the resistance of a bridge to specific hazard scenarios. Although the qualitative approaches are somewhat convenient to use, their outcome (i.e., adequate quality specifications) are vague. The quantitative performance indicators are more valuable, since they may provide more precise information for decision-making.

The benefits of application of a quantitative approach in the assessment of scour critical bridges in North Carolina are reported by Mulla (*19*). Here, a risk-based approach is applied for the management of bridges with unknown foundations in NCHRP Web-Only Document 107 (*20*). The assessment was based on the HYRISK methodology (*8*). Although this methodology may consider the static system of a bridge and type of foundations, the probabilities of failure are based on qualitative data from NBI and the historical frequency of failures. The latter and the fact that neither oncoming flooding magnitudes nor soil resistance are considered, are the main drawbacks of this approach.

Recently, a novel methodology for quantitative vulnerability assessment has been presented in (*21*). It is based on Equation 1, where the analysis of failure modes is done by pragmatic modeling of the local scour action at a pier, considering combined response of a supporting soil and a bridge structure. The scope of the research is set on the reinforced concrete multiple-span girder bridges with piers on shallow foundations which are particularly endangered in a flooding event. The research confirms that the resistance of the soil-bridge system must not be neglected in the vulnerability assessment of bridges exposed to local scour

(*22*). The following evaluation of the direct consequences is straightforward, but the calculation of indirect i.e. traffic related consequences requires a traffic simulation model based on the current transport supply in a road network. An example of such a calculation is given by Tanasic, Ilic, and Hajdin (*23*).

To conduct this vulnerability assessment on a network level, it is necessary to synthesize available information from databases and documentation and systematically collect the missing data from bridge sites. For the latter, it is of the outmost importance to have uniform data level to assess: bridge exposure, bridge resistance, and possible consequences of failure.

STRUCTURING OF QUALITY CONTROL PLANS

The QCPs should be tailored for each individual bridge structure. Besides the adequate PI, the time schedule and analysis of collected data should be defined along with the triggering criteria for initiating preventative procedures. The importance of parameters, which comprise the minimum data set for the quantitative vulnerability assessment, are discussed in Tanasic and Hajdin (*24*). Also, discussed herein are the levels and frequencies of the necessary inspections–data updates, to provide background information for the assessment. The objective information on bridge exposure to flooding hazards is invaluable for structuring a QCP since it provides the facts on possible type of failure modes (e.g., pier related) and the extent of local scour depth (evaluated by local scour evaluation formulas). The reliable information on foundation soil properties (geotechnics and erodibility) as well as on the soil cover at an affected substructure, represent the crucial information to investigate at bridge sites where there is no foundation protection (e.g., Larsen sheets, gabion rock pile).

The relevant bridge elements and related information, which affects the structure of a QCP must be clearly outlined. The main requirement for the quantitative vulnerability assessment is definition of relevant failure modes, and here the influence of specific bridge elements on the type of failure mode (FM) and resistance to local scour is given in Table 1. Complementary to this information, in Figure 2, one of the possible FM type 3 is presented for a multiple span RC girder bridge, where one of its piers with shallow foundations is affected by local scour.

TABLE 1 Key Bridge Elements for Different Types of Resistance to Local Scour at a Substructure

Bridge Element	Attention	Resistance	Failure Mode Type
Affected substructure foundation	Inadequate detailing/condition state	Structure governed	1
Bearing/joint at the top of the affected substructure	Low plastic strength of a bearing/joint (or a poor condition state)	Governed by soil properties, i.e., no/low superstructure resistance	2
Bearings/joints at other substructures	Horizontal displacement is either free or restrained	Combined soil–bridge resistance	3
Main girder	Detailing	Combined soil–bridge resistance	3
		Failure safe	4

The FM type 1 is the most dangerous since it may cause progressive collapse, if the design of the main girder is not failure/collapse safe to a loss of one of the supports (i.e., FM type 4). The FM type 2 may occur e.g. if the top of the pier of an affected foundation is not restrained to movement in horizontal plane. The FM type 3 is the most desired case, since the requirement for failure is that the foundation soil and the structure need to deplete their joint resistance due to the loss of support at the substructure foundation (*21*).

As seen in Figure 2, the crucial set of information for a bridge structure exposed to local scour are related to the detailing of an affected substructure and its foundation. Although the bridges where FM types 1 and 2 have some considerable probabilities of failure (e.g., order of 10-3 and higher) should be mitigated in due time, the consequences of a failure must not be neglected as well as the costs of possible preventative interventions.

The following preventative interventions may be considered to reduce the probability of a failure in a specific hazard scenario:

- Decrease the exposure to the scenario:
  - Soil works at the bridge site and
  - Countermeasures at substructures.
- Monitoring of scour at substructures.
- Increase of structure resistance:
  - Foundation repair/retrofit,
  - Bearings/joint repair retrofit, and
  - Strengthening of a main girder (e.g., fail-safe case).

It must be noted that the actions which are related to the increase of structure resistance also may benefit the overall bridge performance to other sudden or slow (deterioration) processes as well and should be considered in a long-term cost analysis.

CONCLUSION

In sudden events, such as flooding hazards, bridge failures may occur regardless of bridge age, structural system and construction materials. This poses a difficulty to point out the most-vulnerable bridges thus schedule an adequate and timely risk mitigation action. Currently

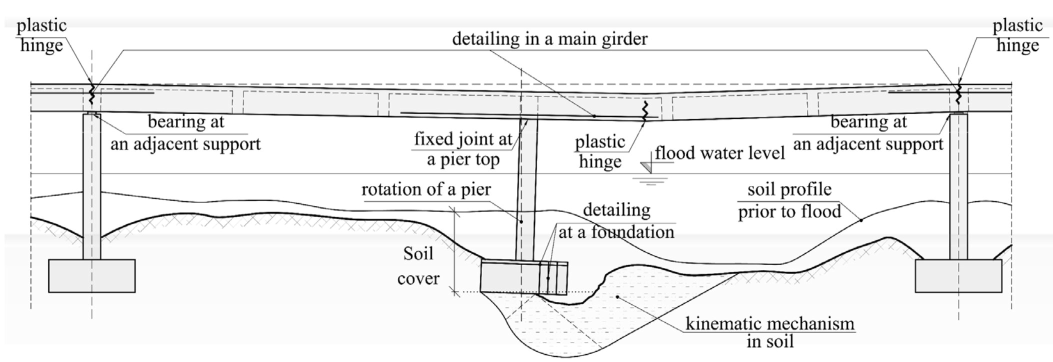

FIGURE 2 A possible failure mode (FM type 3) of a multiple-span RC girder bridge.

implemented qualitative risk-based approaches in BM practice impose constraints in a decision-making process and fail to provide objective information on a risk of a bridge failure. The risk and its progression over time wait to be adequately addressed in the future BMSs, where the desired goal is structuring of an adequate QCP for each structure. There is a need for comprehensive approaches to ensure reliable levels of bridge performance and mobility of goods and people in a society. The accent is on a simplified yet sufficiently accurate procedure, based on a modest data set, eligible for implementation on various bridge types and network topologies.

For quantifying the hazard impact on the transportation infrastructure, it is of the utmost importance to act timely and preventatively by taking into consideration all relevant information on bridge exposure to a hazard, resistance to specific failure modes and related consequences. For this purpose, the adequate PI must be applied, and here the measure of vulnerability is suggested as the most-convenient and -comprehensive PI that will indicate which bridges need specific attention and should be investigated in more detail. Based on a procedure for a vulnerability assessment, a structure of a QCP for a bridge may be elaborated. Here, from a bridge's point of view, it is outlined that the minimum set of information must include condition data and properties of an affected substructure, to account for bridges which are susceptible to critical failure modes (FM types 1 and 2).

Once integrated in the future BMS, the vulnerability assessments will enable timely scheduling of risk mitigation actions and making right decisions for resource allocation. The insight on vulnerabilities in a network would aid in emergency planning as well, since timely warnings could be issued in regions where intensive flooding is expected.

ACKNOWLEDGMENT

This paper is based on work from a research project supported by COST (European Cooperation in Science and Technology).

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Framework for Objective Risk Assessment in Bridge Management

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Many agencies have encountered difficulties in funding robust bridge risk management programs. Part of the problem may be the difficulty of communicating priorities between agencies and legislators; in particular, the lack of usable information for informing tradeoffs among alternative investments. To fill the funding gap, there may be a need to fill the communication gap.

Risk incorporates the uncertainty of exogenous events which may adversely impact an agency's ability to accomplish its program objectives. While uncertainty of events is a given, inclusion of risk in asset management is based on the concept that there are asset characteristics that can be measured and managed. In order to combine bridge risk assessment with other investment needs unrelated to bridges or risk, a common measurement scale for project benefits, tied to program objectives, would be very helpful. This can be defined by identifying objectives that all parts of the program share (such as safety), or by reducing all project benefits to dollars or some other common measure.

The application of risk management methodology is closely tied to the needs of asset management business processes. These include needs identification, project benefit and cost estimation, priority setting, and resource allocation. This paper argues that appropriate measures can closely link risk management into existing asset management processes, and that the information produced in this form may be helpful to decision-makers responsible for allocating resources broadly across infrastructure categories in a statewide context.

Bridge owners face a variety of risks, understood as probability or threat of unexpected outcomes that is caused by external or internal vulnerabilities, and that may be avoided through preemptive action. Risks can have desirable or undesirable consequences, and may be systemic (affecting the agency or inventory as a whole), or site-specific (affecting specific bridges). Uncertainty of planning metrics can be a contributor to systemic or site-specific risk.

In bridge management, risk assessment focuses more specifically on the threat of damage, injury, or loss related to conditions or events occurring on specific structures. Risk is managed by increasing the resilience (or decreasing vulnerability) of individual structures, or of a portion of the network.

Developing and funding a risk management program for bridges is especially challenging because of the uncertain and long-term nature of project benefits, the large number of potential failure points in a transportation network, and the complexity of developing satisfactory programmatic cost estimates for risk mitigation. It is difficult to know when or where an extreme event might strike, but such events can and do happen with regularity across an asset inventory, causing significant amounts of potentially avoidable damage and injury.

When a major disaster strikes, the public naturally asks why a hazard was not recognized earlier and remediated. The inevitability of such questions may establish a form of accountability for managing the resilience of a transportation network. Moreover, legislative and regulatory action, such as 23 CFR 515.9 on Transportation Asset Management Plans, creates a legal requirement for risk management analysis.

FILLING THE FUNDING GAP

Many agencies have encountered difficulties in funding robust bridge risk management programs. To cite just one example, in 2012 the Washington State Department of Transportation (DOT) identified 629 bridges needing seismic retrofit, at a cost of $1.4 billion (*1*). This amount is five times the agency's typical annual budget for pavement and bridge preservation activities. Even with those substantial needs, only $22.4 million was budgeted for the 2011–2013 biennium. Even after passage of a significant gas tax increase, funding for seismic retrofits in the 2015–2017 biennium is only $6.7 million (*2*).

Many reasons could exist for this funding gap, but certainly public awareness of the severity of needs is not one of them. As local media have reported periodically, the risk to the state from major earthquakes has been repeatedly studied, and massive needs have been identified across all types of infrastructure including highways and transit, water and sewer systems, airports and seaports, and schools and other public buildings (*3*). It is apparent that the needs estimates are far beyond the state's ability to fund them, but the legislature thus far has not been able to find a more realistic multiyear funding level.

Part of the problem may be the difficulty of communicating priorities between agencies and legislators. While Washington State DOT is certainly able to prioritize its bridge seismic needs according to relevant technical criteria (such as structure configuration, lifeline routes, traffic volume, and peak ground acceleration), it does not yet have the tools necessary to integrate this priority list with nonseismic programs (such as scour remediation or bridge preservation) (*4*). The legislature lacks appropriate information to balance the risk mitigation needs of highways against those of school buildings and other critical assets (*3*). To fill the funding gap, there may be a need to fill the communication gap.

TOWARD A FRAMEWORK FOR RISK ASSESSMENT

Modern bridge management systems, including AASHTOWare Bridge Management (BrM), have multiobjective performance frameworks for project evaluation, priority setting, and resource allocation. The objectives to be maximized, such as those presented in legislation and agency strategic plans, include safety, mobility, condition, and environmental sustainability. At the same time, agencies are continually called upon to minimize life-cycle costs (LCC) and manage risk.

Over the past three decades, therefore, agencies' bridge management system development and implementation efforts have been focused on LCC estimating and assessment of bridge-level risk. In the United States, research by DOTs such as New York, Minnesota, and Florida has improved on the Federal Bridge Sufficiency Rating, a 1970s legacy measure that emphasizes risk. They have developed improved field assessments, incorporated geographically referenced hazard data, and modern economic models. NCHRP Project 20-07 Task 378 has developed a set of guidelines on quantifying risk for bridge management systems.

Risk incorporates the uncertainty of exogenous events which may adversely impact an agency's ability to accomplish its program objectives. While uncertainty of events is a given, inclusion of risk in asset management is based on the concept that there are asset characteristics that can be measured and managed. In order to combine bridge risk assessment with other investment needs in a larger program for priority-setting or resource allocation, a means must be found to place the bridge risk on a scale that is comparable across all investment categories. This can be by identifying objectives that all parts of the program share (such as safety), or by reducing all project benefits to dollars or some other common measure.

The application of risk management methodology is closely tied to the needs of asset management business processes. These include needs identification, project benefit and cost estimation, priority setting, and resource allocation. Although the overall level of risk is difficult to estimate at the asset and network levels, risk analysis still provides useful tools that serve the more specific needs of these business processes. They can compare the impacts of any two specific projects and direct resources to programs having the most significant likely impact.

RISK ASSESSMENTS USED IN CURRENT PRACTICE

A variety of practices are currently in place to assess risks on highway bridges.

Federal Sufficiency Rating

One of the oldest risk measures used in bridge management is the National Bridge Inventory (NBI) Sufficiency Rating (SR), which was developed in the 1970s and has been a cornerstone of federal management of the national bridge program ever since (*5*). The SR formula can be understood as a proxy for the likelihood of service disruption. The SR is calculated on a scale of 0 (worst) to 100 (best), with the following components:

- 55% of the rating:
  - Condition (deck, superstructure, and substructure ratings) and
  - Load-carrying capacity (inventory rating and its impact on mobility).
- 35% of the rating:
  - Geometrics (lane width, clearances, alignment),
  - Condition and load-carrying capacity (additional weight for overweight truck hazard), and
  - Waterway adequacy (resistance to scour and overtopping hazards).
- 15% of the rating: essentiality for public use (changes the relative weights given to the above factors based on traffic volume and network importance).
- Up to 13% reduction for special safety and mobility deficiencies (increases bridge priority to account for especially long detour routes or substandard safety features, affecting a relatively small fraction of bridges).

The SR does not consider likelihood of natural extreme events, and contains very minimal consideration of traffic volume. It was used for priority-setting in the early days of the bridge program, but was not well-suited for benefit–cost analysis since it disadvantaged the large structures which cost more to repair and replace. It is still used in some states as a performance measure, however.

New York State DOT Bridge Safety Assurance Program

Mandated by the New York Highway Law amended in 1989 (also known as the Graber Law), the New York State DOT embarked on developing comprehensive bridge management and safety assurance programs and its own uniform code of bridge inspection (*6*). It conducted a national survey of bridge failures since 1950 and identified hydraulic, overload, and collision as significant modes of failure for New York state bridges.

Steel and concrete details were considered significant as they presented potential failure vulnerability due to built-in design obsolescence in its existing bridge population. Seismic failure mode was included in this program due to potentially severe consequences if even a single one occurred in the northeastern United States. The six failure modes were then prioritized based on their significance and consequence to New York's transportation network.

In order to assess the vulnerability of its large bridge population, New York State DOT utilizes a multilevel process (Figure 1) with each level successively refining the list of bridges. This enables more detailed evaluation of structures with greater vulnerability. Screening, classifying and rating steps in this process provide increasing understanding of the specific vulnerability of a bridge. Bridges with greater vulnerabilities are progressed first through steps that focus on corrective actions on the most critical bridges in the shortest time. This results in an efficient and staggered progression through the assessment process.

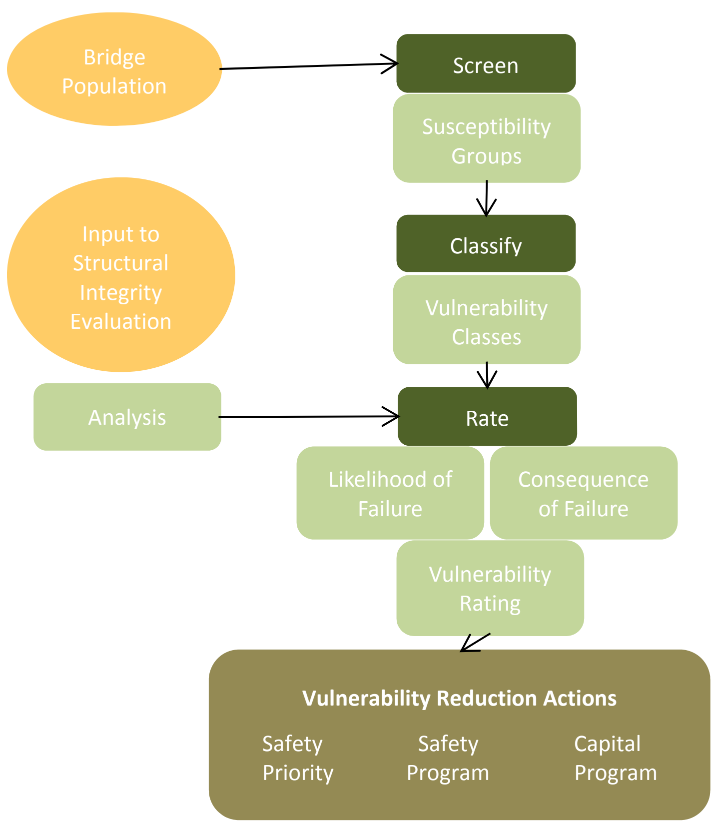

FIGURE 1 New York's multilevel vulnerability assessment.

It is important to note that New York State DOT's vulnerability rating step is common across all failure modes. It is intended to provide a uniform measure of a structure's vulnerability to failure based on its likelihood of failure and its consequences should one occur. New York State DOT accomplishes this by separately assigning vulnerability scores that evaluate the likelihood and consequence of failure. It then adds them together to determine the vulnerability rating. These vulnerability ratings range between 0 (best) and 20 (worst). New York State DOT uses the vulnerability rating for short, mid- and longer term priority-setting for needed remedial actions within its operational and capital program planning.

Minnesota DOT Bridge Replacement and Improvement Management

Minnesota DOT uses a risk-based prioritization tool, developed as an Excel spreadsheet called BRIM, to identify and rank most of the bridge projects that are submitted for its Statewide Transportation Improvement Program (STIP) (*7*). Bridge Replacement and Improvement Management (BRIM) does not develop separate estimates of likelihood and consequence of an event, but instead uses a set of rating tables to convert directly from bridge characteristics in its Pontis database to a measure of utility which it calls the Bridge Performance Index (BPI). These tables were developed entirely from judgment.

Figure 2 shows the table for scour. Minnesota, like many other states, uses a scour classification system that is more detailed than federal standards. The BPI is reduced if certain defects (formerly smart flags) are present. Similar tables were also developed for fracture criticality, fatigue, overweight trucks, over-height trucks, driver loss of control, and overtopping of the bridge or approach.

		SCOUR			
		Smart flag reduction			
Code	Description	None	1	2	3
A	Not a w aterw ay	100	100	100	100
E	Culvert	100	100	100	100
M	Stable; scour above footing	90	90	70	40
H	Foundation above w ater	90	90	70	40
N	Stable; scour in footing/pile	80	80	60	30
I	Screened; low risk	70	70	50	30
L	Evaluated; stable	70	70	50	30
P	Stable due to protection	60	60	40	20
K	Screened; limited risk	60	60	30	20
F	No eval; foundation know n	50	50	40	20
C	Closed; no scour	50	50	25	20
J	Screened; susceptible	40	40	30	10
O	Stable; action required	40	40	20	10
G	No eval; foundation unknow n	20	20	15	10
R	Critical; monitor	10	10	5	0
B	Closed; scour	0	0	0	0
D	Imminent protection reqd	0	0	0	0
U	Critical; protection required	0	0	0	0

FIGURE 2 Minnesota BPI table for scour.

The BPI scores represent bridge qualities that the agency controls, that it spends money to improve over time, that reduce the likelihood of transportation service disruption. The BPI score does not consider the site-specific likelihood of adverse natural events such as earthquakes or floods.

In order to use the BPI score for priority-setting, BRIM further adjusts the BPI by moving scores within the 0 to 100 range based on traffic volume, bridge length, detour length, and network class. The BPI score is used directly for prioritization, without considering project cost or long-term cost, making it a true worst-first framework.

Florida DOT Project Level Analysis Tool

Florida DOT implements the products of its bridge management research in the Project Level Analysis Tool (PLAT), an Excel spreadsheet model built on the AASHTOWare Pontis database to analyze the performance of any one selected bridge (*8*). The PLAT, in turn, contributes estimates of cost and effects to the Network Analysis Tool (NAT), a separate spreadsheet model which is used for priority setting and programming of bridge work on a district and statewide basis.

Philosophically, the performance management approach taken in the PLAT and NAT is to attempt to quantify all costs and benefits in dollar terms at the project and network levels. Each project may affect transportation system performance in a variety of ways: initial cost, LCC, safety, and mobility. These project benefits are considered together in a multi-objective optimization framework. In the Florida DOT models, the utility function for this multi-objective framework is social cost, consisting of agency, user, and nonuser costs.

Florida bridges experience a variety of hazards: hurricanes, tornadoes, wildfires, floods, collisions, advanced deterioration, and fatigue. The causes are, at least in part, outside agency control and subject to random external factors. They are quantified in terms of the likelihood of a hazard event. All of these hazards can cause a bridge to be damaged or destroyed, delivering a consequence to the agency (the cost to repair or replace the structure) and an impact on the public (disruption of transportation service and of the larger economy). Figure 3 shows the basic ingredients.

Hazards are modeled probabilistically. At a given bridge site, the hazard can strike with various levels of severity that can be forecast only with a broad concept of probability distribution. Once a hazard strikes, the damage to the structure and impact on the public are also probabilistic, subject to a limited degree of agency control.

For bridge management purposes, the main decision variable in the Florida risk analysis is the selection and timing of programmed actions to increase the resilience of the department's structures, thus indirectly influencing the social costs caused by hazards. The controllable costs of structure resilience and operational strategies are combined with the more random future outputs of agency, user, and nonuser costs due to hazard events, to produce forecasts of LCC. In effect, the programmed and consequential costs of risk are included within the LCC analysis.

In order to place a dollar value on hazard consequences, regional or statewide historical records of hazards and their dollar-valued recovery costs were summarized and used as a gross indication of future risk. This risk is allocated to specific bridges in a way that is reflective of structure resilience and significance. A bridge is assigned more risk if it has a higher probability

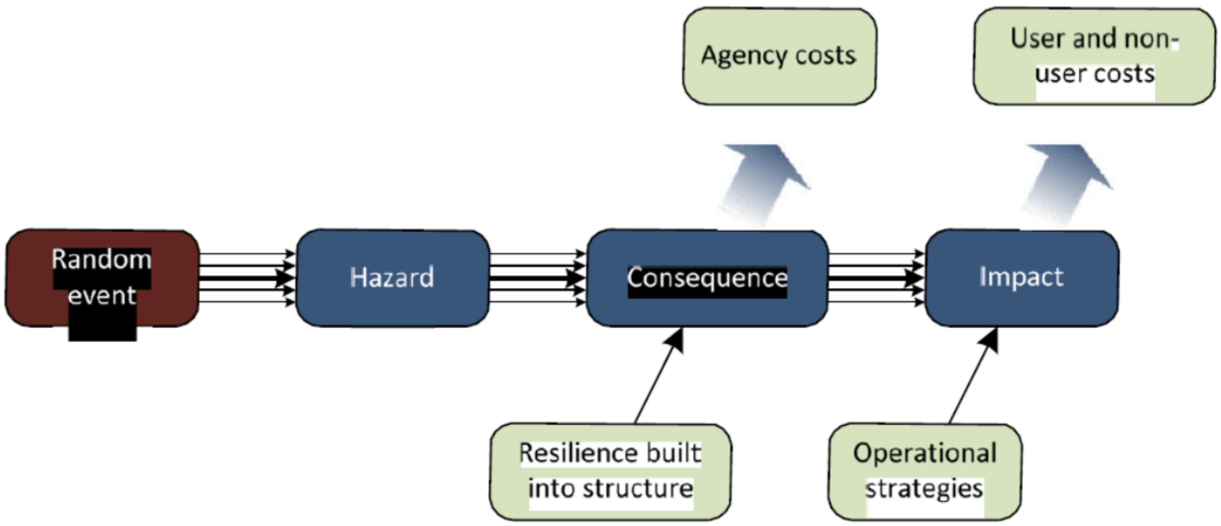

FIGURE 3 Basic ingredients of risk analysis in PLAT.

of an adverse event, if it has less resilience, if it is expensive to replace, or if it is used by a large number of people.

For natural hazards, the probability of an adverse event in most cases is developed from geographically referenced hazard maps maintained by the state and federal governments. Specialized statistical models were developed for the likelihood of fuel truck collisions, overloads, over-height collisions, advanced deterioration, and fatigue. Resilience in most cases was based on data already available in the Florida DOT Pontis database, such as structure type, scour assessment, and condition.

Using this perspective, risk is spread in a consistent manner among bridges, and from year to year over time. Risk may gradually increase over time because of traffic growth and deterioration. If a risk mitigation or replacement action takes place, resilience improves and risk is reduced for the time subsequent to the action. The LCC of this scenario is the sum of discounted social costs incurred throughout the life of the crossing served by the bridge. Risk-related costs are high without the mitigation action, and lower once the action is applied. The action itself also has a cost. If the life cycle that includes the action has lower total LCC than a life cycle without the action, then it is attractive to perform the work.

For project selection purposes in any given year, LCC can be computed for a variety of feasible actions, including doing nothing, to select the action which minimizes LCC. The total benefit of a project is the savings in LCC relative to doing nothing.

If a project is delayed, this lengthens the period of higher risk costs, and thus increases LCC. The benefit of accelerating a project by 1 year is the 1-year savings in LCC. In a priority programming context where a limited budget must be allocated among projects each year, the best projects are those which would save the most in risk costs, relative to each dollar spent, if they are done this year rather than waiting another year.

RISK IN THE CONTEXT OF ASSET MANAGEMENT

Asset management includes procedures to relate decisions to their effects on agency performance goals, such as safety, mobility, and environmental sustainability. For bridges, condition is a special kind of performance goal because it usually affects road users indirectly, if at all, by means of safety and mobility. However, condition directly affects treatment selection and therefore it affects cost. Risk works in a manner similar to condition: it is unknown to road users unless safety or mobility is affected, but it affects the choice of mitigation action.

Resilience and Vulnerability

For certain asset management purposes, it would be useful to have a measure of risk that can be used in the same way that condition is used. Specifically

- It can be assessed in the field using objective, repeatable procedures derived from observable properties of the asset.
- It has a bounded scale where one end is the best possible performance and the other end is the worst possible performance.
- It provides a fair comparison between two assets regardless of their relative size or utilization, on the best-to-worst scale.
- It can be tracked over time as performance changes due to agency actions and exogenous factors.

Transportation agencies are increasingly concerned with transportation network resilience, and asset management can help to maximize this characteristic by improving the resilience of individual assets. Resilience is defined as:

> … the capability of a system to maintain its functions and structure in the face of internal and external change and to degrade gracefully when it must (*9*).

> ‘Vulnerability’ seems largely to imply an inability to cope and ‘resilience’ seems to broadly imply an ability to cope. They may be viewed as two ends of a spectrum (*10*).

“Internal and external change” can be interpreted as changes caused within the asset itself (i.e., normal deterioration) and change caused by external forces (natural extreme events such as floods and earthquakes). “Maintain its functions and structure” can be interpreted as the avoidance of transportation service disruptions.

Reviewing the examples given in the preceding section, it can be seen that the Sufficiency Rating and the measures developed in New York and Minnesota fit this pattern. Resilience is a desirable quality so it could be expressed as a score on a 0 to 100 scale where 100 is best possible, which is the same range as the sufficiency rating and the bridge health index. Vulnerability is an undesirable quality, so it could be expressed on a reversed scale, where 0 would be best.

Good–Fair–Poor and Network Resilience Targets

The analogy between condition and resilience can be taken further. Each potential hazard or hazard scenario can be recorded in a manner similar to structural elements, using resilience states. The resilience states might correspond to the good–fair–poor distinction used in federal performance regulations (23 CFR 490). Alaska's Geotechnical Asset Management Plan (*11*) offers the following guidance for standardizing resilience state definitions among dissimilar hazards and asset classes:

Good: The asset is fully sufficient to resist anticipated hazards and normal deterioration according to current standards.
Fair: The asset is sub-standard, and as a result there is elevated likelihood of mild to moderate disruption to mobility, safety, economic efficiency, or other performance objectives on the corridor. Risk mitigation may reduce this likelihood.
Poor: The asset is ineffective in resisting anticipated hazards, and as a result there is high likelihood of severe disruption to corridor performance objectives. Significant investment such as reconstruction may be needed.

The Risk Assessments feature of AASHTOWare Bridge Management can be configured to support resilience or vulnerability assessments structured in this way.

Network resilience targets can be defined and tracked in the same way as federal condition targets. Since safety and mobility risks are proportional to traffic volume, it would be reasonable to weight the network measures by average daily traffic rather than deck area. To make risks comparable between transportation assets and other types of public facilities, such as schools, resilience could be weighted according to the amount of time spent by people when exposed to the risk. Then an agency's performance dashboard for seismic risk might look like the hypothetical example in Figure 4. The performance measure in this graph is computed from average daily traffic, planning metrics for average speed and average vehicle occupancy, and the field assessment of seismic vulnerability or resilience.

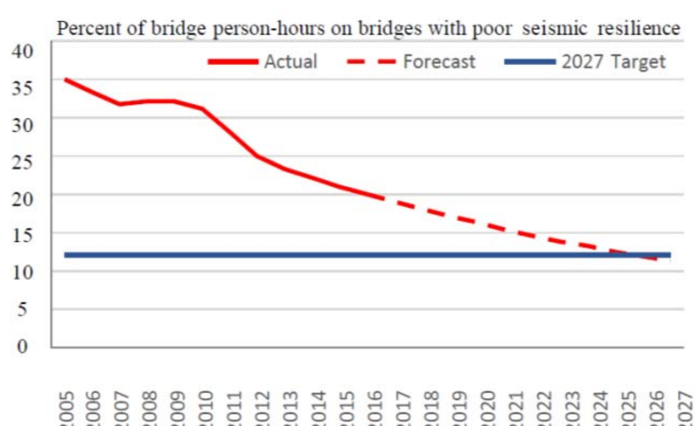

FIGURE 4 Performance dashboard presentation of resilience target tracking.

Framework for Risk Assessment

Figure 5 depicts how New York State DOT utilizes its step by step, multilevel process to conduct hydraulic vulnerability assessment.

Step 1. Screening for Hydraulic Vulnerability

The bridge inventory is screened using information from New York State DOT's bridge inventory and inspection system (BIIS) database to identify bridges that do not span water. These are rated 6 (not applicable) and are eliminated from the assessment process. The remaining bridges are then subjected to a two-part susceptibility screening which consists of a review of bridge plans, construction documents, inspection reports and other available information to place bridges in four susceptibility groups 1 (high) through 4 (low), indicating each bridge's relative susceptibility to damage from hydraulic forces, to prioritize them for the next step.

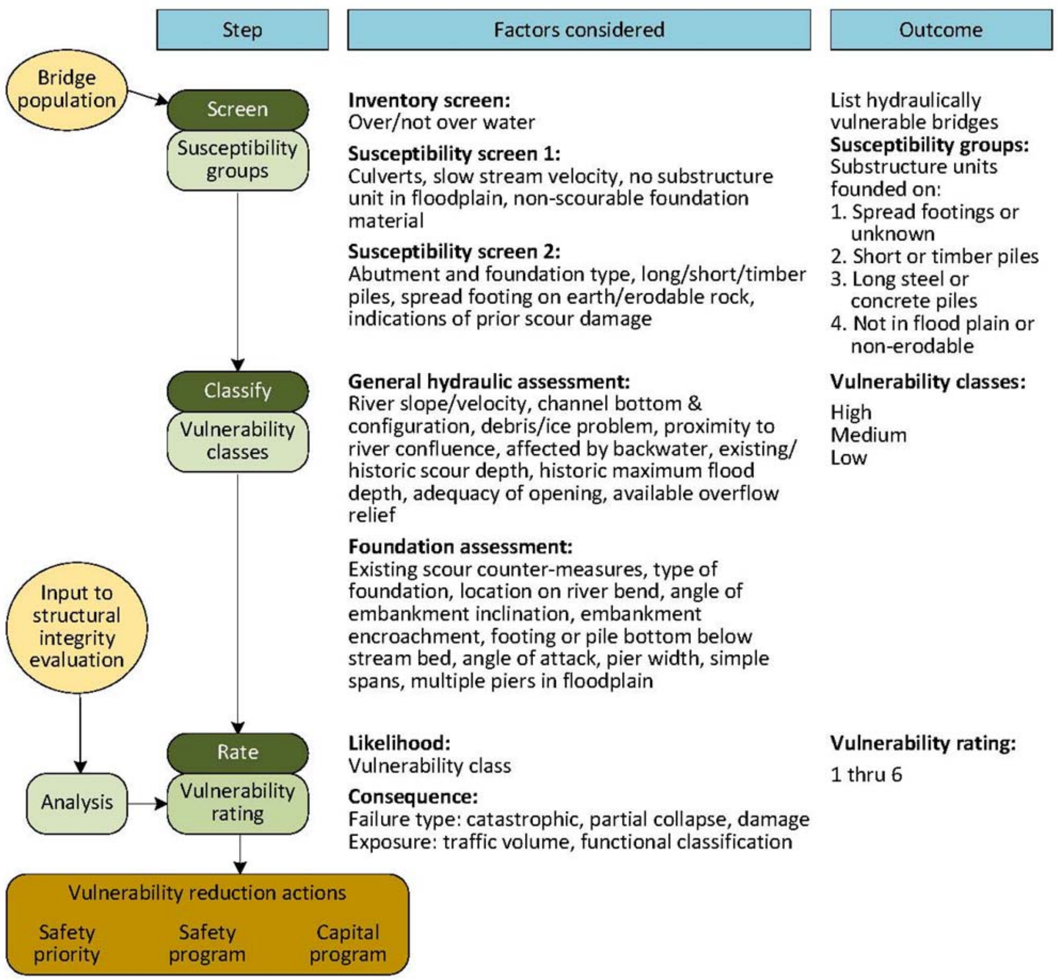

FIGURE 5 Illustration of an assessment of likelihood and consequence factors.

Step 2. Classifying Structure Hydraulic Vulnerability

This step involves evaluation of site hydrology and hydraulic characteristics using general hydraulic and foundation assessment procedures. It quantifies the potential vulnerability of a structure to hydraulic damage relative to other bridges in the classification process and places the structure in the high, medium, or low hydraulic vulnerability class. These classes indicate the likelihood of failure and are used in vulnerability rating of a structure. They are also considered in deciding whether a structure should be placed on a flood watch list or a post-flood inspection list.

Step 3. Rating Structure Vulnerability

The hydraulic vulnerability rating is determined using results of a classification process to assess the likelihood of failure and an evaluation of the consequences of failure in terms of failure type (catastrophic, partial collapse, structural damage) and its exposure (traffic volume and functional classification of route). Rating scores are assigned to the likelihood (1–10) and consequences of failure (0–10) and added together to arrive at final vulnerability rating, which will range between 0 (least vulnerable) and 20 (most vulnerable).

Step 4. Evaluation of Vulnerability

This step conducts a detailed analysis of vulnerable bridges, on a prioritized basis, to provide quantitative assessment of the performance of an existing bridge in comparison to current hydraulic design requirements. Results of this analysis are then used in Structural Integrity Evaluation to determine the stability of a bridge against hydraulic forces. This analysis is also valuable in designing hydraulic improvements and scour protection counter measures to eliminate or mitigate failure vulnerability of the bridge.

Estimating Project Benefits

A field assessment of vulnerability or resilience, such as what was described in the preceding section, can be the foundation for assessment of site-based risk for a wide range of hazards from seismic and scour to over-height trucks and advanced deterioration. They can meet many of the requirements for a risk management framework. Additional normalization may be necessary, however, for the following purposes:

- Setting priorities and allocating resources across dissimilar asset classes that are typically managed independently.
- Establishing a basis for prioritization that consistently and objectively considers the cost of risk mitigation and the magnitude of exposure to risks.
- Combining risk avoidance with LCC savings in an overall assessment of project benefits.
- Quantifying the benefits of projects that combine multiple asset classes.
- Evaluating projects that postpone hazardous conditions for a period of time.
- Suggesting a reasonable starting point for balancing safety, mobility, environmental, and economic concerns.

Many of the business processes that require this kind of functionality are concerned with the allocation of agency funding. As a result, it is useful to adopt a relatively simple and standardized set of procedures to convert all types of risk mitigation benefits to dollar values consistent with the framework of life cycle cost analysis. Once in a dollar-denominated form, all the standard tools of economic analysis are available for prioritization, resource allocation, and optimization.

Where managerial or political judgment is required, such as when balancing safety vs environmental versus economic benefits, the economic model provides a starting point for consistent application of such judgment. For example, if an agency uses the Analytic Hierarchy Process, the data source may be a survey asking a panel of decision makers to express preferences between pairs of alternatives. The survey questions could be structured such that each pair consists of alternatives that have equal benefit–cost ratios according to purely economic criteria. The result would then be more valid in quantifying the extent to which safety or another performance concern should be overweighted.

Converting from an assessment of vulnerability or resilience to an economic project benefit may consist of any or all of the following steps:

1. Estimating the probability of an extreme event of a given magnitude, or hazard scenario.
2. Estimating the probability that a structure will be damaged, if an extreme event occurs, based on the vulnerability or resilience assessment.
3. Estimating the probability that transportation service will be disrupted, if the structure is damaged.
4. Estimating the consequences of a service disruption on outcome performance measures such as accident rate, hours of travel delay, and miles of detours.
5. Converting performance consequences into a dollar amount.

A companion paper (*12*) discusses research-based methodologies that cover these logical steps. Likelihood probability models are provided for 16 hazards including earthquake, landslide, storm surge, high wind, flood, scour, wildfire, temperature extremes, permafrost instability, overload, over-height collision, truck collision, vessel collision, sabotage, advanced deterioration, and fatigue. Consequences of service disruption are estimated in dollars for recovery cost, safety, mobility, and environmental sustainability. These models are based on published research gathered from a wide variety of sources, and consistent with the AASHTO Guide for User and Non-User Benefit Analysis for Highways (the Red Book) (*13*).

In cases where one or more steps cannot be performed due to lack of data, a generic process known as risk allocation may apply. It consists of estimating the total statewide annual losses from the hazard scenario under investigation, in a top-down fashion from agency statistics, news reports, external advocacy groups, polling of maintenance supervisors, or judgment. Then this total loss is divided among all bridges in the inventory according to vulnerability, traffic volume, and any other relevant available data. This creates a risk formula that can be developed quickly and later improved by means of additional research. Florida's risk models apply this approach (*8*).

IMPLEMENTATION OF A RISK FRAMEWORK

Implementation of any new program or process can be easy or difficult. New York State's Highway Law as amended in 1989 (the Graber Law) was in response to the catastrophic failure of the Schoharie Creek Bridge resulting in 13 fatalities. This law mandated creation of a comprehensive Bridge Safety Assurance (BSA) program. The successful implementation of a BSA program by New York State DOT depended upon objectivity, verifiability and transparency of the process that was to be used to assess the vulnerability of New York state bridges to all potential modes of failure. The BSA manuals New York State DOT developed met these criteria and enabled it to vigorously pursue the implementation of its BSA program. This program produces a list of bridges that need Safety Priority Action (short-term), Safety Program (mid-term) and Capital Program (long-term) actions to address the vulnerability to failure (*14*). During the first year, 43 bridges were identified as high Safety Priority, due to vulnerability to hydraulic failure. In response, remedial actions to eliminate/mitigate the risk of failure were designed and completed. Most DOTs can undertake such short-term emergency projects to mitigate, if not eliminate, the hazard to public safety. For Safety Program and Capital Program actions New York State DOT uses its vulnerability rating in conjunction with a bridge condition rating. Funding issues do come into play in these instances as bridge needs compete with other DOT needs and priorities.

Bridge engineers understand their responsibility to assure bridge safety for the traveling public, and the necessity for their analyses to be very detail oriented. However, the general public and legislative leaders can understand likelihood and serious consequences of potential bridge failures, only if it is communicated in a simple and credible manner. New York State DOT experience indicates that there were three critical elements in its successful implementation of the BSA program. They were

1. Commonly shared vision within DOT hierarchy. Collectively, New York State DOT decision makers had a strong and clear vision about the BSA program. This vision was founded on objectivity, verifiability and transparency of the vulnerability assessment process that was developed and being implemented.
2. Authorizing environment. The general public, state legislative leaders and the Governor were supportive and understood that, while technical, New York State DOT's BSA program was objective, verifiable and transparent. New York State DOT's annual reports assured the legislature that BSA goals were set logically and were measurable. This was a convincing example of the Barcelona Principle, "Goal setting and measurement are fundamental to communications and public relations."
3. Organizational capacity. As it passed the Graber Law, state legislature also authorized additional staff positions specifically designated to carry out BSA activities. With this addition, New York State DOT had adequate in-house expertise available in its Structures Design and Construction Division to implement the BSA program.

It can be seen from the description of New York's methodology that it is highly summarized relative to an engineering vulnerability analysis. While the more detailed information is necessary for engineering decision making, the less detailed presentation was equally necessary for informing senior leadership and political decision makers who allocate resources.

CONCLUSION

Although risk assessment methodology is not as standardized as condition assessment, a combination of experiences from several states can provide a complete implementable framework. A simplified scale of vulnerability or resilience, with common well-understood definitions across hazards and asset classes, has been critical for most agencies that have successfully implemented risk management programs. Resilience can be used alongside condition as a means of prioritization, as has been done in New York State DOT.

The ability to convert a vulnerability or resilience assessment into a reasonable estimate of economic project benefits is essential for business processes that involve funding allocation, that must balance economic and noneconomic objectives, and that must consider intertemporal trade-offs, such as advancing or delaying projects, in the face of fiscal constraints. Incorporating risk in the same framework as LCC, safety, mobility, and environmental sustainability represents an application of multiobjective decision-making that includes risk management. The case studies show that these elements are feasible using reasonable data requirements within bridge management systems or spreadsheets.

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Improving Bridge Structure Management Data, Models, and Tools

Novel Cost-Based Performance Index for Condition Assessment of Bridges

Case Study for an Ohio Bridge

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Bridges are key components in transportation systems in Ohio and are essential in supporting various economic activities at local and state levels. These structures deteriorate differently due to differences in exposure to various environmental stressors and service loads, and having diverse ages, configurations, and structural features. Moreover, the amount of budget to maintain, repair and replace bridges in Ohio which has the second largest number of bridges in the United States is limited. These factors, among others, pose a challenge for evaluating the performance of these structures and managing their safety and serviceability. This study presents a practical and efficient measure called bridge condition index (BCI) for reliable condition assessment of Ohio bridges through effective utilization of Ohio Department of Transportation's bridge databases. Ohio BCI (OBCI) is intended to evaluate bridges at element-, component-, bridge-, and network-levels and to reflect the impact of defects as well as condition enhancement of individual elements on the condition-state of the system. In order to compare direct and indirect consequences of various conditions of bridges on users and agencies, a unified metric based on cost is proposed for the OBCI formulation. This index is demonstrated for a real bridge in Ohio. To examine the efficiency of the OBCI, the results are compared to Bridge Health Index which is a common bridge performance metric. Furthermore, the ability of OBCI to account for effects of bridge serviceability features such as average daily traffic is shown. The proposed metric can assist in proper maintenance of transportation systems and enhancement of their serviceability and safety.

Ohio has the second largest inventory of bridges in the United States. These bridges are comprised of various ages, configurations, and structural features, and are exposed to various environmental conditions and service loads. These factors, among others, pose a tremendous challenge for evaluating the performance of these assets and managing their safety and serviceability. A reliable and objective index is needed to effectively utilize available data to evaluate the health conditions of Ohio bridges. The new metric should consider multiple attributes of bridge performance with respect to bridge preservation and vulnerability using a single number. In addition, this measure must be reliable to allow objective assessment of the long-term performance of bridge programs at multiple levels of stakeholders such as county, district, and state levels. It also needs to enable highway agencies to compare and prioritize bridges in a network, identify effective maintenance, repair, and rehabilitation (MR&R) actions, and properly allocate budget over time for a single bridge or a network of bridges. Such a metric should help effective communications about bridge conditions, required budget, and performance of bridge programs with various stakeholders such as the public, legislature, and bridge program directors.

Bridge performance measures are used as a critical tool to manage and operate a large number of bridges in transportation systems. The choice of an appropriate performance measure strongly depends on agency policies, level of decision-making, and bridge type, among other factors (*1*). Consequently, various types of metrics have been developed over the years for different purposes. These metrics are being used to support goals such as preservation maintenance (also sometimes referred to as preventive maintenance) and allocation of funds for rehabilitation–replacement and improvement of bridges. These metrics include, among others, National Bridge Inventory rating (NBI), deficiency rating (DR), sufficiency rating (SR), load rating (LR), Bridge Health Index (BHI), Denver BHI, geometric rating (GR), and vulnerability rating (VR). These performance measures were proposed or implemented by state departments of transportation (DOTs), FHWA, NCHRP, and other researchers. In many indices such as SR (*2*) and DR (*3*), subjective weight factors are considered to account for structural and serviceability failure modes, whereas in reality, the likelihood of these failure modes, as well as their corresponding consequences, depend on the severity of the problems and the environment where bridges are located. In BHI and Denver BHI, first, health indices of elements of similar type (e.g., columns, girders) are determined based on the percentage of elements in each of the condition states. Using the derived health indices and a set of weighting functions, the health index of the entire bridge is evaluated (*4–6*). The weighting functions are subjectively defined for each element to represent the importance and criticality of that element for the safety and serviceability of the entire bridge. However, the criticality of an element should be objectively quantified based on consequences on users and agencies. A solution to improve the objectivity of bridge performance metrics is to account for impacts of various potential consequences of condition states of bridges in terms of expected costs that are expressed in a monetary unit.

In order to address limitations of existing indices and provide a metric with the desired features explained at the beginning of this section, this paper presents a novel cost-based performance metric called Ohio Bridge Condition Index (OBCI). The considered cost categories include (1) implementation costs referring to costs of applying upgrades or repair actions. An important feature of the proposed framework is the incorporation of a comprehensive list of incurred costs to reliably determine consequences of such repair–upgrade actions. (2) structural–serviceability failure costs referring to costs of consequences for the existing condition of bridges. In the rest of the paper, the scope of the OBCI is presented, the involved cost terms are explained, minimum allowable thresholds for the condition state of bridge elements are introduced, formulations of two versions of OBCI are developed, the proposed OBCI formulations are applied to a case study bridge from Ohio DOT's bridge inventory, and conclusion remarks are presented.

OHIO BRIDGE CONDITION INDEX

In the proposed OBCI, direct and indirect consequences of various conditions of bridges for users and agencies, are incorporated through a unified metric based on cost. In bridge management, there are two types of events that have consequences for users and agencies: potential structural–operational failures of bridges and MR&R actions performed on bridge elements; both of these are functions of the condition states of bridge elements, among other factors. Thus, cost terms in OBCI can be classified into two groups:

- Implementation cost. This cost is estimated when MR&R actions are planned to be applied to bridge elements according to the results of routine inspections. It includes element-level costs of implementing MR&R actions. The implementation cost contains user and agency costs. Agency costs are the direct money that is paid by the responsible agency for executing MR&R actions on bridge elements. This cost includes the costs of administration, engineering, crew and equipment mobilization, maintenance of traffic, and costs of executing MR&R actions on bridge elements. User costs are the costs incurred on users, i.e., drivers and passengers, due to the implementation of MR&R actions. This cost may include incurred costs of posting load and clearance restrictions, extra vehicle operation, delay time on users, and excess emission. Implementation costs are elaborated in the next sections.

- Structural–operational failure cost. The sum of all user and agency costs in the foregoing implementation cost is needed to maintain, repair, or replace elements of a bridge. On the other hand, if required MR&R actions are not performed on the bridge, structural or operational failures may occur. Thus, the quantification of consequent failure modes in terms of monetary units helps responsible agencies with the decision-making process through cost–benefit analyses. In addition, each failure mode has a likelihood of occurrence. Thus, for the purpose of quantifying the consequences of failure modes, the concept of risk—i.e., the product of the likelihood and the cost of structural–functional failure modes—can be applied in OBCI. These costs of consequences are expected costs due to structural–operational failures of bridges that can potentially occur as a result of deterioration, fatigue, flooding, and scour, among other factors. When a failure mode occurs, both users and agencies are affected. The responsible agency repairs the damaged elements. Thus, all of the cost terms of the agency costs that were mentioned for the implementation costs, should be considered as the agency costs for the structural–operational failure costs.

Scope of the OBCI Model

OBCI is intended to evaluate bridges at element, component, bridge, and network levels. Each level is defined as follows:

- Element. OBCI evaluates all elements of the same type in a bridge. For instance, OBCI presents a single condition index for all of the pier columns existing in a bridge. Following the new AASHTO-recommended condition rating system (*7*), Ohio DOT provides an overall condition tate rating for elements in a scale from 1 to 4 (*8*). These elements can be any of the 68 element types that are categorized into four groups of: national bridge elements (NBE), bridge management elements (BME), agency-developed elements (ADE), and defects.
- Component. OBCI evaluates the overall condition of a group of different elements that together serve a role in structural integrity or serviceability of bridges. Following AASHTO (*7*) and Ohio DOT (*8*), the subsequent components—approach, deck, superstructure, substructure, culvert, channel, and sign/utility—are available in the new inspection reports.
- Bridge. OBCI evaluates the condition index at the bridge level considering the condition state of the entire constituent elements of that bridge.
- Network. OBCI evaluates the overall condition of a portfolio of bridges in a region, district, county, and the state of Ohio.

This performance measure reflects the impact of defects as well as condition enhancement of individual elements on the condition-state of the system in each of the foregoing levels. In the rest, two versions of the OBCI are presented and the application of these indices are demonstrated for one of the Ohio DOT's bridges.

OBCI Models with Minimum Thresholds

Generally, there is a trade-off between implementation and structural–operational failure costs in the OBCI; the more costly the MR&R action, often the better the long-term performance of benefitted elements. Evaluation of these costs requires failure mode identification and likelihood estimation, which can be very time-consuming considering that each bridge type and configuration may have very different modes of failure. Instead, as a practical alternative for the incorporation of structural–operational failure risk costs in the OBCI, minimum thresholds are established to define unacceptable condition states for bridge elements. This provides an incentive to perform MR&R actions before the state of bridges becomes critical. In addition, these minimum acceptable condition states assure an acceptable level of safety and serviceability of bridges for the public, and reduce the likelihood of failure modes to the extent that the risk costs become fairly negligible compared to implementation costs. Therefore, only implementation costs are incorporated in the OBCI framework. A general flowchart of the proposed framework is shown in Figure 1.

Minimum Required OBCI

On a rational basis, minimum thresholds should be set based on the importance of elements for the safety and functionality of the bridge system. At component or bridge levels, 21 state DOTs have set up target values for the condition of their bridge assets (*9*). For instance, the state of Ohio defines 15% as the maximum allowable percentage for the area of its bridge decks with NBI general appraisal ratings less than 5. In line with the most recent AASHTO-recommended condition state rating system, at element level, authors have defined the following minimum thresholds:

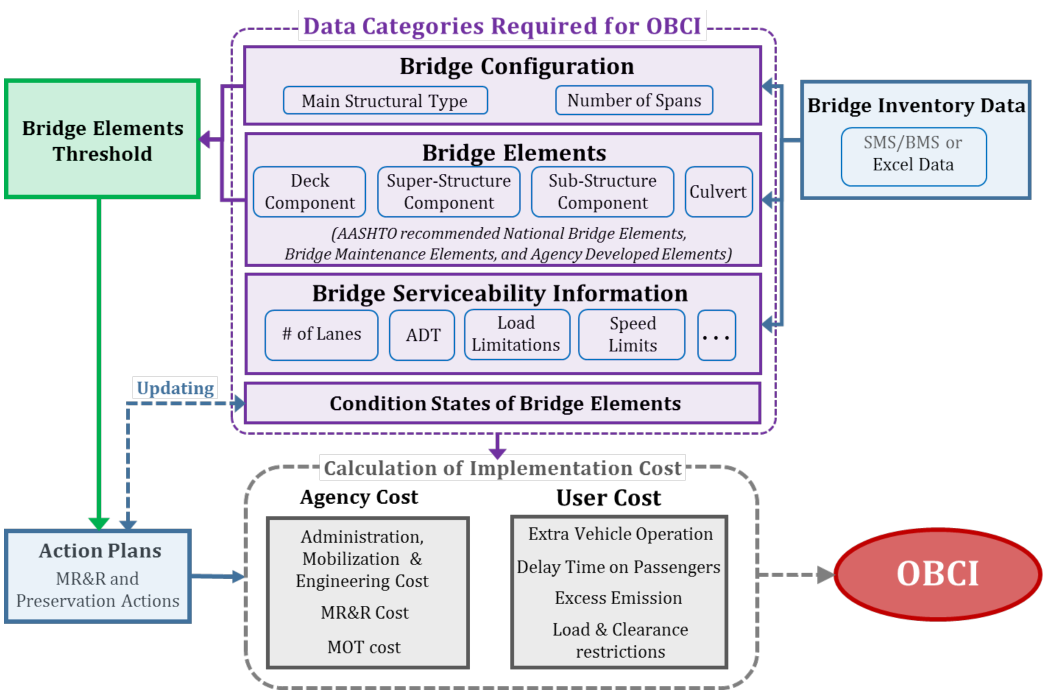

FIGURE 1 General flowchart of the proposed OBCI with minimum condition state thresholds.

- The percentage of NBE, defects and primary elements of ADE in condition states 3 should be less than 2%, while no quantities of these elements should be in condition state 4.
- The percentage of BME and nonprimary ADE elements in condition state 3 and 4 should be less than 10%.

In the future, effects of other factors will be explored to develop more representative minimum thresholds. For bridges where the above criteria are not satisfied, MR&R actions should be taken so that the condition state of all elements is at or above the corresponding minimum threshold. On this basis, minimum OBCI can be expressed as:

$$\mathrm{OBCI}_{\min} = 1 - \frac{\sum \text{cost of meeting minimum thresholds(\$)}}{\text{replacement cost(\$)}} \tag{1}$$

where replacement cost is the total cost of replacing the system including the associated implementation costs. In fact, $\mathrm{OBCI}_{\min}$ represents the proximity of the system to meet all the minimum thresholds. Decomposing the costs into agency and user costs for all the elements, $\mathrm{OBCI}_{\min}$ can be written as:

$$\mathrm{OBCI}_{\min} = 1 - \frac{AC^{\min} + UC^{\min}}{RC} \tag{2}$$

where RC is the replacement cost of the system, and $AC^{\min}$ and $UC^{\min}$ are the incurred agency and user costs (implementation costs) as a result of performing MR&R actions on bridge elements in order to meet the minimum condition-state thresholds. Detailed formulations of $\mathrm{OBCI}_{\min}$ for evaluation at element, component, bridge, and network levels are provided in Table 1.

In this table, $AC_E^{\min}$ and $UC_E^{\min}$, $AC_C^{\min}$ and $UC_C^{\min}$, $AC_B^{\min}$ and $UC_B^{\min}$, and $AC_N^{\min}$ and $UC_N^{\min}$ are the costs incurred on agency and users for reaching the minimum condition states of the constituent elements of the element set E, component C, bridge B, and network N, respectively. The element, component, bridge, and network level agency costs of administration, engineering and mobilization (AEM), and maintenance of traffic (MOT) are denoted by AEM_E, AEM_C, AEM_B, AEM_N, MOT_E, MOT_C, MOT_B, respectively. Furthermore, the element, component, bridge, and network level user costs of load and clearance restriction (LCR), and delay time, vehicle operation, and excess emission (DVE) are denoted by LCR_E, LCR_C, LCR_B, LCR_N, DVE_E, DVE_C, DVE_B, DVE_N, respectively. Where these costs correspond to the cost of reaching the minimum required condition state, they are specified with a superscript “min”, and where these cost represent the associate cost of replacement, superscript “rep” is used. The MR&R cost of bringing the condition state of element k of component c of bridge b to its minimum threshold is denoted by $\mathrm{MR\&R}_{k,c,b}^{\min}$. In order to account for reductions in the MR&R costs as the scale of the project increases, a reduction coefficient, α, is considered for the total MR&R costs for component-, bridge-, and network-level OBCI. These factors range from 0 to 1. The subscript for this factor represents the scope of the project, and the superscript indicates the amount of improvement achieved by the MR&R actions. For example, $\alpha_B^{\min}$ is the reduction coefficient applied for the MR&R cost when all elements of the bridge are improved together to meet their minimum acceptable condition state. Unlike elements and components, the replacement cost

TABLE 1 Formulation of $\mathrm{OCBI}_{\min}$ for element, component, bridge, and network levels.

Scope	$\mathbf{OBCI_{min}}$	
Element	$\mathrm{OBCI}_{\min} = 1 - \frac{(AC_E^{\min} + UC_E^{\min})}{(AC_E^{\mathrm{rep}} + UC_E^{\mathrm{rep}})}$ $AC_E^{\min} = \mathrm{MOT}_E^{\min} + \mathrm{AEM}_E^{\min} + \mathrm{MR\&R}_E^{\min}$ $UC_E^{\min} = \mathrm{LCR}_E^{\min} + \mathrm{DVE}_E^{\min}$ $AC_E^{\mathrm{rep}} = \mathrm{MOT}_E^{\mathrm{rep}} + \mathrm{AEM}_E^{\mathrm{rep}} + \mathrm{MR\&R}_E^{\mathrm{rep}}$ $UC_E^{\mathrm{rep}} = \mathrm{LCR}_E^{\mathrm{rep}} + \mathrm{DVE}_E^{\mathrm{rep}}$	(3)
Component	$\mathrm{OBCI}_{\min} = 1 - \frac{(AC_C^{\min} + UC_C^{\min})}{(AC_C^{\mathrm{rep}} + UC_C^{\mathrm{rep}})}$ $AC_C^{\min} = \mathrm{MOT}_C^{\min} + \mathrm{AEM}_C^{\min} + \alpha_C^{\min} \times \sum_{k=1}^{M_c} \mathrm{MR\&R}_k^{\min}$ $UC_C^{\min} = \mathrm{LCR}_C^{\min} + \mathrm{DVE}_C^{\min}$ $AC_C^{\mathrm{rep}} = \mathrm{MOT}_C^{\mathrm{rep}} + \mathrm{AEM}_C^{\mathrm{rep}} + \alpha_C^{\mathrm{rep}} \times \sum_{k=1}^{M_C} \mathrm{MR\&R}_k^{\mathrm{rep}}$ $UC_C^{\mathrm{rep}} = \mathrm{LCR}_C^{\mathrm{rep}} + \mathrm{DVE}_C^{\mathrm{rep}}$	(4)
Bridge	$\mathrm{OBCI}_{\min} = 1 - \frac{(AC_B^{\min} + UC_B^{\min})}{(\gamma \times A + UC_B^{\mathrm{rep}})}$ $AC_B^{\min} = \mathrm{MOT}_B^{\min} + \mathrm{AEM}_B^{\min} + \alpha_B^{\min} \times \sum_{c=1}^{M_b} \sum_{k=1}^{M_c} \mathrm{MR\&R}_{k,c}^{\min}$ $UC_B^{\min} = \mathrm{LCR}_B^{\min} + \mathrm{DVE}_B^{\min}$ $UC_B^{\mathrm{rep}} = \mathrm{LCR}_B^{\mathrm{rep}} + \mathrm{DVE}_B^{\mathrm{rep}}$	(5)
Network	$\mathrm{OBCI}_{\min} = 1 - \frac{(AC_N^{\min} + UC_N^{\min})}{(\sum_{b=1}^{M_n} \gamma_b \times A_b + UC_N^{\mathrm{rep}})}$ $AC_N^{\min} = \mathrm{MOT}_N^{\min} + \mathrm{AEM}_N^{\min} + \alpha_N^{\min} \times \sum_{b=1}^{M_n} \sum_{c=1}^{M_b} \sum_{k=1}^{M_c} \mathrm{MR\&R}_{k,c,b}^{\min}$ $UC_N^{\min} = \mathrm{LCR}_N^{\min} + \mathrm{DVE}_N^{\min}$ $UC_N^{\mathrm{rep}} = \mathrm{LCR}_N^{\mathrm{rep}} + \mathrm{DVE}_N^{\mathrm{rep}}$	(6)

of a bridge is usually expressed in terms of bridge type and deck area. Therefore, in Equation 5 and Equation 6, γ_b is the unit replacement cost per deck area of bridge b, and A_b is the deck area of the bridge. Finally, M_c, M_b, and M_n are the number of existing elements, components, and bridges, respectively. Following Equations 3–6, if the sum of required costs to improve the condition state of elements, components, or bridges in the system exceeds the replacement cost of the system, it will be replaced with the replacement cost of the system.

The proposed OCBI_{min} has the following features:

- OCBI_{min} evaluates the proximity of the system to meet corresponding minimum thresholds for acceptable condition states considering user and agency costs of implementing MR&R actions.
- OCBI_{min} provides decision-makers with a set of MR&R actions that incur minimum user and agency costs to reach minimum thresholds. This feature is useful for emergency decision-making, and when the available budget is limited (i.e., taking the least-costly decision, while providing the minimum required level of safety and operability).

OBCI Indicating the Current Condition

A true index for the performance of a system needs to compare the state of the system with its like-new condition. On this basis, other than OCBI_{min} that is intended to reflect the minimum necessary amount of work, OCBI_{min} is expressed as

$$\text{OBCI}_{\text{current}} = 1 - \frac{\sum \text{cost of going back to the like new condition(\$)}}{\text{replacement cost(\$)}} \tag{7}$$

According to Equation 7, $\text{OCBI}_{\text{current}}$ ranges from 0 to 1; the more healthy the condition-state of the bridge, the closer $\text{OCBI}_{\text{current}}$ to 1. The structure of the formulation of $\text{OCBI}_{\text{current}}$ at element, component, bridge, and network levels is identical to corresponding formulations of OCBI_{min} presented in Equations 3–6. However, the superscript of the cost terms in the numerators should be changed to 1, indicating the cost to improve to the like-new condition state. Therefore, $\text{OCBI}_{\text{current}}$ compares the current condition of the system with the like-new condition to indicate how close the system is to its desirable condition.

Cost Terms in OBCI

As previously mentioned, cost terms in OBCI are the costs imposed on users and the responsible agency due to performing MR&R actions following routine inspections on bridge elements. These costs include agency costs of MR&R, MOT, and AEM, and user costs of DVE and LCR. The agency and user costs can be estimated based on available information about bridge configurations and inspection data. This information can be categorized into three groups: bridge configuration features, bridge serviceability features, and the types of bridge elements (Figure 1). These three categories of information are mostly available in inspection reports. In the rest of this section, the derivation process of each of the agency and user cost terms using the aforementioned bridge information categories are explained.

Agency Cost

Maintenance, Repair, and Replacement Cost The type and extent of MR&R actions in the OBCI framework depend on the following factors:

- Material and type of elements;
- The current condition state of the elements; and
- The target condition state of the elements: Often more-costly corrective actions result in more improvement in the condition state of an element. Thus, decision-makers may decide to evaluate the performance of bridges under several improving actions, each of which incurs certain cost and imposes certain improvement in the condition-state of elements.

Using the above procedure and Ohio DOT costs for MR&R actions, the unit costs of performing MR&R actions are identified. Then, for calculating the total cost of performing MR&R actions on bridge elements, these unit cost values are multiplied by the amount of elements that are identified to require corresponding actions.

Maintenance of Traffic Cost According to Ohio DOT Office of Estimation, as of January 2016, maintaining traffic using "three laborers, one arrow board, one truck with attenuator, and one truck/flatbed for barrel replacement and removal" costs approximately \$260/h. If any police enforcement should be used for the MOT, an additional cost of \$65/h for each police car will be added to the MOT cost. Police enforcement is assumed to be present at the location of the project, if more than 40% of the bridge lanes are closed for repair actions. Since on weekends no worker is present, the cost of MOT is reduced to the equipment that direct the traffic; for those periods, the \$260/h unit cost is reduced by 60%. In other words, the cost of equipment and labors are considered to be 40% and 60% of the total cost, respectively. In addition, the \$65/h cost of police enforcement is not considered for these days. If $T_l^{t'}$ is the number of working days required for performing l-level project type *t*, with l and t varying among element, component, bridge, or network levels, and project types of 1, min, or rep (explained before), the minimum number of weekends that the project faces is $\left\lfloor \frac{T_l^{t'}}{7} \right\rfloor$, where $\lfloor \quad \rfloor$ is the floor of the ratio $\frac{T_l^{t'}}{7}$. On the other hand, based on information provided by Ohio DOT, the average number of hours that bridge laborers work is 8 h/day. Therefore, MOT cost of an MR&R project of type *t* at l-level, i.e. MOT_l^t, can be calculated as follows:

$$\mathrm{MOT}_l^t = \left(8 \times T_l^{t'} \times \$260 + 8 \times T_l^{t'} \times F^{N_{Cl}} \times \$65 + 16 \times T_l^{t'} \times 40\% \times \$260\right) + \left(2 \times \left\lfloor \frac{T_l^{t'}}{7} \right\rfloor \times 24 \times 40\% \times \$260\right) \tag{8}$$

In this equation, $F^{N_{Cl}}$ is a factor taking a value of 1 or 0, indicating the presence or nonpresence of police officers, as a function of the number of closed lanes, N_{Cl}.

Administration, Engineering, and Mobilization Costs

The cost of administration, engineering, and mobilization for a project of type *t*, at l -level, i.e., AEM_l^t, can be estimated by:

$$\mathrm{AEM}_l^t = \beta \times (\mathrm{MOT}_l^t + \mathrm{MR\&R}_l^t) \tag{9}$$

where β is an overhead factor, and is considered to be 0.25.

User Cost

User Delay, Vehicle Operation, and Excess Emission Costs When MR&R actions are performed on bridge elements, the traffic on or under the bridge may be affected by the assignment of lower speed limits, or partial–complete closure of the bridge. Consequently, user costs due to delay, extra operation of vehicles, and excess emission from vehicles are incurred. The unit cost for such consequences for car and truck users, i.e., ρ_C and ρ_T, in year 2008, were reported as \$19.22/h and \$51.88/h, respectively (*10*). Using the average annual Consumer Price Index values reported in (*11*), the unit user costs for cars and trucks for year 2015 are derived as \$21.13/h and \$57.04/h, respectively. Then, considering an interest rate of 3%, these unit costs are calculated as \$21.76/h and \$58.75/h, for the year 2016.

It should be noted that user cost of DVE is incurred on average uniformly during the entire project time, T_l^t. Considering weekends and weekdays, T_l^t is equal to $T_l^{t'} + 2 \times \left\lfloor \frac{T_l^{t'}}{7} \right\rfloor$. Thus, the DVE user cost due to performing project type *t*, at l level, i.e., DVE_l^t, can be computed as follows:

$$\mathrm{DVE}_l^t = T_l^t \times (t_{ij}^{D/R} - t_{ij}^{O}) \times [(\mathrm{ADT} - \mathrm{ADTT}) \times \rho_C + \mathrm{ADTT} \times \rho_T] \tag{10}$$

where $t_{ij}^{D/R}$ and t_{ij}^{O} are the required time to travel from the start point *i* of the bridge to its end point *j* by taking the detour/bridge with reduced speed limit, and taking the bridge at original posted speed, respectively. These parameters are derived using procedures developed by Bocchini and Frangopol (*12*).

User Costs of Load and Clearance Restriction Load restriction postage to limit heavy vehicles due to poor conditions of bridge elements (mostly structural elements), and restrictions on the allowable horizontal clearance and vertical underclearance of bridges due to performing some MR&R actions are other user cost terms that affect a certain group of users. Generally in these scenarios, the passage of certain types of trucks is restricted. Thus, similar to the process for computing DVE_l^t, the user cost for load and clearance restrictions when performing project type *t*, at l -level, i.e., LR_l^t, can be calculated as follows:

$$\mathrm{LCR}_l^t = T_l^t \times (t_{ij}^{D/R} - t_{ij}^{O}) \times [\mathrm{ADTT}^R \times \rho_T] \tag{11}$$

where ADTT^R is the percentage of restricted trucks that should take the available detour.

CASE STUDY

General Information of the Case Study Bridge

For the demonstration of OBCI, a case study is conducted for a real bridge in Ohio. It is a two-way, two-lane bridge with nine continuous prestressed box beams, passing over a river. The length and width of the deck are 110 and 34.5 ft, respectively. The bridge has a low ADT and ADTT of 50 and 5, respectively, and is on a path with no detour. Therefore, in order to perform any MR&R actions, the bridge should have at least one open lane. Moreover, the bridge is not posted for load and clearance restrictions. Table 2 presents the inspection data for this bridge including the quantity of elements in the four available condition-states.

Calculation of OBCI for the Case Study Bridge

As previously explained, element-, component-, bridge-, and network-level information is required for the calculation of the cost terms in both versions of the OBCI, i.e., $OBCI_{min}$ and $OBCI_{current}$. Some required information is collected from resources provided by Ohio DOT, such as

- Bridge configuration data: width and length, and the type of structural system.
- Type and material of bridge elements and the percentage of those elements in each of the condition states.
- Cost of several MR&R actions together with the condition states before and after performing such actions. For example, as of 2016, patching the defected area of the concrete deck with condition state 3 costs $125/ft^2 and improves these areas to condition state 2. On the other hand, if the entire deck should be replaced, the cost of $100/ft^2 is incurred and the entire deck surface will be improved to condition state 1.
- Bridge serviceability data: ADT, ADTT, number of lanes under and on the bridge.

For other required information, logical assumptions are made when necessary based on engineering judgment and consultation with Ohio DOT. Some of such assumptions are

- Given individual element-level information on the required time for performing MR&R actions, component- and bridge-level duration of work plans are estimated through a reduction factor, which is applied to the sum of individual element-level duration of MR&R actions in the work plan. These factors are considered to be 0.75 and 0.90, for component and bridge levels, respectively.
- As presented in Equations 4–5, reduction factors are incorporated to account for the effect of scale in the computation of MR&R costs in component- and bridge-level OBCI, using element-level cost information (i.e., α factors in Table 1). These factors are considered to be 0.80, and 0.90, for component and bridge levels, respectively.

TABLE 2 Quantity of the Case Study Bridge Elements in Different Condition States

Element	Category of Element	Unit	QTY	Condition State CS1	CS2	CS3	CS4
Approach Items							
Approach wearing surface	ADE	Each	2	0	2	0	0
Approach slab	BME	SF	810	146.5	405	202.5	56
Embankment	ADE	Each	4	0	0	0	4
Guardrail	ADE	Each	4	4	0	0	0
Deck Items							
Floor/slab	NBE	SF	3,795	3783	4	8	0
Wearing surface	BME	SF	2,970	1140	1140	540	150
Curb/sidewalk/walkway	ADE	LF	110	105	5	0	0
Railing	NBE	LF	220	180	30	10	0
Drainage	ADE	Each	2	0	0	2	0
Expansion joint	BME	LF	69	14	15	40	0
Superstructure Items							
Alignment	Defect	Each	3	3	0	0	0
Beams/girders	NBE	LF	990	987	1	2	0
Bearing device	NBE	Each	72	72	0	0	0
Substructure Items							
Abutment walls	NBE	LF	70.06	61.1	9	0	0
Pier caps	NBE	LF	70.1	69.1	0	1	0
Pier columns/bents	NBE	Each	4	4	0	0	0
Wingwalls	ADE	Each	4	4	0	0	0
Scour	Defect	Each	4	4	0	0	0
Slope protection	ADE	Each	2	2	0	0	0
Channel Items							
Alignment	ADE	LF	200	200	0	0	0
Protection	ADE	LF	200	200	0	0	0
Hydraulic opening	ADE	EA	4	4	0	0	0
Sign Items							
Utilities	ADE	LF	220	220	0	0	0

Note: QTY = quantity; CS1 = condition state 1; CS2 = condition state 2; CS3 = condition state 3; CS4 = condition state 4.

- The replacement cost of the bridge (i.e., factor of γ in Table 1) is extracted from Caltrans (*13*); for the case study bridge, this value is \$315/ft^2. In order to update this cost for the state of Ohio, state (adjustment) factors given by U.S. Army Corps of Engineers (*14*) are used.

Based on the aforementioned information, all the user and agency cost terms are estimated for element, component, and bridge levels of the case study bridge. Then, $OBCI_{min}$ and $OBCI_{current}$ for these levels are computed following Equations 3–6, and the results are provided in Table 3. As seen, OBCI is not provided for the alignment of superstructure component. According to Ohio DOT inspection manual (*8*) and AASHTO *Manual for Bridge*

TABLE 3 Element-, Component-, and Bridge-Level OBCI for the Case Study Bridge

Bridge Element	$OBCI_{min}$			$OBCI_{current}$		
	Element	Component	Bridge	Element	Component	Bridge
Approach Items						
Approach wearing surface	1.00	0.78	0.95	0.56	0.57	0.90
Approach slab	0.62			0.42		
Embankment	0.00			0.00		
Guardrail	1.00			1.00		
Deck Items						
Floor/slab	1.00	0.90		0.98	0.82	
Wearing surface	0.76			0.58		
Curb/sidewalk/walkway	1.00			0.87		
Railing	0.93			0.86		
Drainage	0.56			0.56		
Expansion joint	0.70			0.70		
Superstructure Items						
Beams/girders	1.00	1.00		0.96	0.99	
Bearing device	1.00			1.00		
Substructure Items						
Abutment walls	1.00	1.00		0.97	0.99	
Pier caps	1.00			0.97		
Pier columns/bents	1.00			1.00		
Wingwalls	1.00			1.00		
Scour	1.00			1.00		
Slope protection	1.00			1.00		
Channel Items						
Alignment	1.00	1.00		1.00	1.00	
Protection	1.00			1.00		
Hydraulic opening	1.00			1.00		
Sign Items						
Utilities	1.00	1.00		1.00	1.00	

Inspection (*15*), this item is a type of general deficiency for prestressed elements, which is among factors that determine the condition state of concrete elements. The cost of repairing such a defect is considered within MR&R costs of concrete elements of the bridge. However, this does not apply to the scour item in the substructure component. Thus, OBCI is not assessed individually for the “alignment” of superstructure. It should be also noted that the variability of the cost values and other assumptions made in the framework may have nonnegligible impacts on the results of the calculated OBCI values. Effect of these variations will be studied in the future.

As previously expressed, $OBCI_{min}$ compares the condition state of the elements with the minimum allowable thresholds. Based on this index, approach slab and embankment, deck wearing surface, railing, drainage, and expansion joints require immediate repair; among these, approach embankment, which has the lowest index, is the most critical one. In bridge level decision-making, $OBCI_{min}$ of 0.95 indicates that a repair work plan needs to be scheduled for this bridge so that this index becomes 1.0. Based on Equation 5, the minimum agency cost of improving the condition state of the elements of this bridge to exceed the minimum acceptable thresholds, i.e., $AC_B^{\min}$, is estimated to be \$130,810.

In addition, Table 3 indicates that the approach component with $OBCI_{current}$ of 0.57 has the lowest condition index among others, whereas $OBCI_{min}$ for this item is 0.78. This implies that, reaching the minimum acceptable condition-state for the approach component would cost 0.22 times the replacement cost incurred if a repair work plan is chosen for this component. However, the user and agency costs of improving this component to the like-new condition state is 0.43 times the replacement cost which is half of the user and agency costs of replacing the component. Thus, replacing the approach component may be a reasonable plan.

Comparisons of OBCI with BHI for the Case Study Bridge

OBCI can help with decision-making in the presence of budget constraints. An example is provided to support this claim. Three work plan alternatives are investigated:

1. Performing minimum required repair on elements with $OBCI_{min} < 1$.
2. Improving approach elements to like-new, and performing minimum required repair on other elements with $OBCI_{min} < 1$.
3. Improving deck elements to like-new, and performing minimum required repair on other elements with $OBCI_{min} < 1$.

In addition to $OBCI_{current}$, BHI is also calculated at the bridge level for these alternatives. For this purpose, weighting of condition states vary linearly with respect to the average condition state of elements. Element weight factors are also considered as the replacement cost of elements, which are used for the calculation of element-level OBCI.

For each alternative, the incurred agency costs, as well as the number of days required for performing such work plans are derived and presented in Table 4. According to this table, if the minimum required repair is performed on elements with $OBCI_{min} < 1$, $OBCI_{current}$ will be improved by 4%. It should be noted that under this work plan, the bridge will become structurally safe and operationally serviceable since condition-states of all elements will be above the minimum allowable thresholds. If the agency decides to spend more to achieve a better performance for this bridge, alternatives B and C can be chosen. According to Table 2 and Table 3, the elements within approach and deck components have the lowest condition states and OBCI values. Thus, work plans B and C are suggested to primarily improve the condition state of the elements within these components. In more details, alternative B is 63% more costly than

TABLE 4 Proposed MR&R Work Plans for the Case Study Bridge

Work Plan	Description	Agency Cost of the Work Plan	Duration (days)	$OBCI_{current}$	BHI
0	Condition of the bridge after inspection	—	—	0.895	0.944
A	Perform minimum required repair on elements with $OBCI_{min} < 1$	$130,810	9	0.928	0.961
B	Improve approach elements to like-new, and perform minimum required repair on other elements with $OBCI_{min} < 1$	$212,800	12	0.951	0.961
C	Improve deck elements to like-new, and perform minimum required repair on other elements with $OBCI_{min} < 1$	$233,620	13	0.966	0.961

work plan A, while the amount of improvement in OBCI following work plan B is only 3% more than work plan A. If the budget constraint allows, the responsible agency may spend $233,620 on work plan C to achieve an OBCI value as large as 0.966. The required time of performing this project is almost the same as work plan B (i.e., 12 days for work plans B and 13 days for work plan C). The cost of work plan C is $21,000 more than work plan B, while the increase in the OBCI value after performing work plan C is just 2% more than the increase in OBCI under work plan B, when they are compared to the OBCI value after performing merely minimum required repairs (i.e., work plan A). Thus, if the agency decides to select between work plans B and C, comparing the incurred costs, the required time, and the OBCI after executing these alternatives, work plan B may seem to be a better option. Results also show that, while OBCI indicates 6% and 8% improvement in the bridge performance following work plans B and C, BHI of the bridge is improved by only 1.80%. This can be mostly attributed to the fact that BHI considers healthy elements as those with all portions in condition state 1. However, for steel and concrete elements, any improving action other than replacement, improves the state of defected portions of those elements to condition state 2 (*16*). According to OBCI, these portions are considered to be in the like-new state, whereas BHI considers these portions in a state below the healthy state. As a result, BHI becomes insensitive to costly actions that maintain portions of these elements that are already in condition state 2 (work plans B and C compared to work plan A). Furthermore, the required cost to improve condition-state of elements to their like-new state is not necessarily linearly proportionate to the total quantity of defected portions, which is the assumption in BHI. On the other hand, according to Table 4, OBCI is objectively able to reflect the amount of improvements achieved by costly MR&R actions.

Sensitivity of OBCI to Variations in ADT for the Case Study Bridge

A sensitivity analysis is performed to show the ability of the proposed OBCI in reflecting the effect of variations in serviceability parameters such as ADT on the performance of bridges. To this end, $OBCI_{current}$ is evaluated before and after performing work plan A considering four ADT values: (1) 50 vpd (the original ADT of the bridge), and (2) 25%, (3) 50%, and (4) 75% of the bridge maximum traffic capacity (the maximum capacity of each lane is considered as 1,750 vphpl (*17*)). $OBCI_{current}$ is found as 0.90, 0.85, 0.78, and 0.51 for the bridge before conducting work plan A, and 0.93, 0.89, 0.86, and 0.63 after conducting work plan A. As these results show, $OBCI_{current}$ is sensitive to the variation of ADT, which affects the user cost of DVE. As the ADT values increase, the advert consequences on users become more significant compared to the agency costs of improving elements to their like-new state. Furthermore, as the user cost increases, the improvement in the OBCI following work plan A becomes more significant.

CONCLUSION

OBCI is proposed as a reliable performance measure for bridges. This metric has the following features:

- Incorporates condition state based direct and indirect consequences on users and the responsible agency.
- Evaluates the performance of bridges at element, component, bridge, and network levels.
- Reflects the negative effects of defects in bridge elements, as well as positive influences of taking improving actions on the condition index.

Given the objectives of bridge management by DOTs, two variations of OBCI are proposed. The first one is $OBCI_{min}$ which evaluates the proximity of the system to minimum acceptable conditions for its constituent elements. The user and agency costs of implementing repair actions on system elements that do not meet the minimum condition state thresholds are compared with the user and agency costs of replacing the system. $OBCI_{min}$ ranges from 0 to 1, with 0 indicating that the system is in such a severe condition that replacement of the system incurs the least user and agency costs compared to other repair alternatives, in order to have all bridge elements meet their minimum condition state thresholds. On the contrary, $OBCI_{min}$ with the value of 1 implies that all of the system elements have acceptable condition states. The other formulation of OBCI is $OBCI_{current}$ which compares the current condition to the like-new condition of the system. $OBCI_{current}$ ranges from 0 to 1; bridges with healthier elements will have $OBCI_{current}$ closer to 1. A unique feature of OBCI is that it properly incorporates a comprehensive list of user and agency costs that are incurred as a consequence of performing repair–replacement actions. These costs include agency cost of administration, engineering, and mobilization; agency cost of performing repair–replacement actions; agency cost of MOT; user cost incurred from delay time; vehicle operation and excess emission; and user costs incurred from load and clearance restrictions.

The applications of the proposed indices are demonstrated for a case study bridge in Ohio. The inspection report, as well as information regarding configuration, type and the traffic flow of this bridge are provided by Ohio DOT. The calculated $OBCI_{min}$ for this bridge shows that approach slab and embankment, and deck wearing surface, railing, drainage, and expansion joints require immediate repair. In line with that observation, element-level $OBCI_{current}$ indicates that approach and deck components have the worst conditions. These components also contribute the most to the required costs for the bridge to be improved to the like-new condition. Three work plan alternatives are suggested and discussed. Comparing the incurred costs (the required time) and the OBCI value after the application of these alternatives, the best work plans are suggested. Furthermore, it is found that BHI, which is a conventional performance measure being used for management of bridges by many state DOTs, may not be an appropriate metric as it does not properly reflect effects of MR&R actions on the performance of bridges. Finally, the results show that $OBCI_{current}$ is reasonably sensitive to the variation of ADT, indicating the ability of the proposed index to reflect effects of ADT as a significant serviceability feature of bridges. Based on the capabilities provided by $OBCI_{min}$ and $OBCI_{current}$, these metrics can assist in proper maintenance of transportation systems and effective enhancement of their efficiency, safety, and capacity.

ACKNOWLEDGMENTS

This research is supported by the Ohio Department of Transportation. Any opinions, findings and conclusions or recommendations expressed in this article are those of the authors and do not necessarily reflect the views of the sponsor. Authors thank Ohio DOT personnel including Jared Backs, Amjad Waheed, Tim Keller, and Cynthia Jones for providing data and valuable practical insights.

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Modeling of Life-Cycle Alternatives in the National Bridge Investment Analysis System

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The FHWA has developed the National Bridge Investment Analysis System (NBIAS) as a tool to analyze bridge investment needs and predict future bridge conditions and performance at a national level. NBIAS analyzes each bridge in the national inventory for each year in a multiyear analysis period through a program simulation model. In the model the system simulates deterioration, traffic, preservation needs, functional needs, and costs.

The modeling approach used in prior versions of the system was initially based on that implemented in the Pontis Bridge Management System. Though the modeling approach has evolved over time, its fundamentals are well-established and have been discussed previously in the literature. Recently FHWA developed Version 5.0 of NBIAS introducing fundamental changes in the NBIAS modeling approach. This paper details the revised modeling approach implemented in NBIAS 5.0, focusing on the modeling of multiple life-cycle alternatives for each bridge.

In previous versions of NBIAS, the system prioritized bridge alternatives (allowing for the possibility there may be multiple alternatives on a bridge) using the Incremental Benefit–Cost Ratio (IBCR) heuristic. This approach involves calculating the IBCR of each alternative relative to the next cheaper alternative, and prioritizing alternatives in order of decreasing IBCR. Funds are then allocated to the list of alternatives until they are expended. This approach is repeated for each year of the analysis period, which may be up to 50 years. The IBCR heuristic has been shown to yield a near-optimal solution for prioritizing capital projects given certain conditions. However, it is not designed to support multiple budget constraints. Also, because the system simulates in a year-by-year manner, it may produce a suboptimal result in some cases when results are viewed over multiple years, particularly if additional constraints are added to the system.

The basic approach implemented for NBIAS 5.0 was to shift from a simulation model that generates and prioritizes bridge alternatives year-by-year to one that generates and prioritizes multi-year alternatives. In this version the system first generates a set of 21 different life cycle alternatives for each bridge, reflecting different strategies concerning timing of preservation and functional improvement work. The system then uses a modified IBCR heuristic termed "MINCBEN" previously developed for the Virginia Department of Transportation to select life-cycle alternatives given a matrix of budget constraints specified by work type and year. This revised modeling approach provides improved modeling of trade-offs between bridge preservation and replacement, better optimizes resource allocation over time, and allows for flexibility in setting budget constraints by work type and year.

The National Bridge Investment Analysis System (NBIAS) is the tool the FHWA uses to analyze investment needs for U.S. highway bridges. FHWA's analyses appear in the U.S. Department of Transportation (DOT) *Report on the Conditions and Performance of U.S. Highways, Bridges and Transit*, published biennially and termed the "C&P Report" (*1*), as well as in other documents. Although it was designed for use by FHWA for its national-level analyses, NBIAS has been utilized extensively for other national, state, and local bridge needs analysis, often in conjunction with FHWA's Highway Economics Requirements System (HERS) for highway investment needs analysis.

NBIAS was first introduced in the 1999 C&P Report (*2*). The initial version of the system was based on the analytical framework similar to that used in the Pontis Bridge Management System developed by FHWA in 1992 and subsequently adopted by the American Association of State Highway and Transportation Officials (AASHTO). The basic input to the system is National Bridge Inventory (NBI) data, from which the system synthesizes data on representative structural elements. NBIAS models investment needs for element-level maintenance, repair and rehabilitation (MR&R, also termed preservation), and for functional needs such as widening existing lanes, raising, strengthening, and replacing bridges. The basic modeling approach used in NBIAS has been documented previously in the literature (*3–5*).

Over time FHWA has implemented a number of enhancements to the NBIAS modeling approach to improve the quality of the system's projections and the overall usability of the system. In 2014 FHWA identified a need to enhance the system to allow the user to specify investment budgets by type of work or category of bridge to simulate targeting of investment to certain types of work (e.g., replacement of bridges classified as structurally deficient). Previously only one overall budget could be specified in the system. In conjunction with making this enhancement, FHWA sought to improve the functionality of the system for determining the funds required to achieve a targeted level of performance, and to better model trade-offs between performing MR&R work and replacing bridges. Implementing this set of enhancements required both the addition of additional budget constraints to the system's program simulation, and addition of new logic forcing the system to consider additional alternatives for a bridge to better take advantage of available funds. The following sections summarize the NBIAS modeling approach, detail the above enhancements made to Version 5.0, discuss the impacts of the enhancements, and outline future improvements to NBIAS currently under development.

NBIAS MODELING APPROACH

NBIAS analyzes each bridge (excluding culverts) in the national inventory for each year in a multiyear analysis period through a program simulation model. In this model the system simulates deterioration, traffic, preservation needs, functional needs and costs.

An important input to the program simulation is the MR&R policy. MR&R needs are determined through a Markov modeling approach by first developing the MR&R policy, which specifies what actions to perform on individual bridge elements depending on their condition. The MR&R policy is determined using a linear optimization solved for each combination of structural element, condition state, operating environment, climate zone, and U.S. state. The output of the optimization is specification of what action to take in each condition state to minimize life-cycle costs, and the savings in life-cycle costs of performing the recommended work relative to deferring action for 1 year. The modeling approach is similar to that implemented initially in Pontis, but incorporates consideration of user costs (for decks) and includes a penalty function that varies based on condition.

Figure 1 outlines the steps in the program simulation for NBIAS versions prior to Version 5.0. As indicated in the figure, a series of steps is performed for each year of the analysis period. These include generating potential work, sorting the list of project alternatives, allocating the available budget, and simulating the results of the budget allocation. To generate project

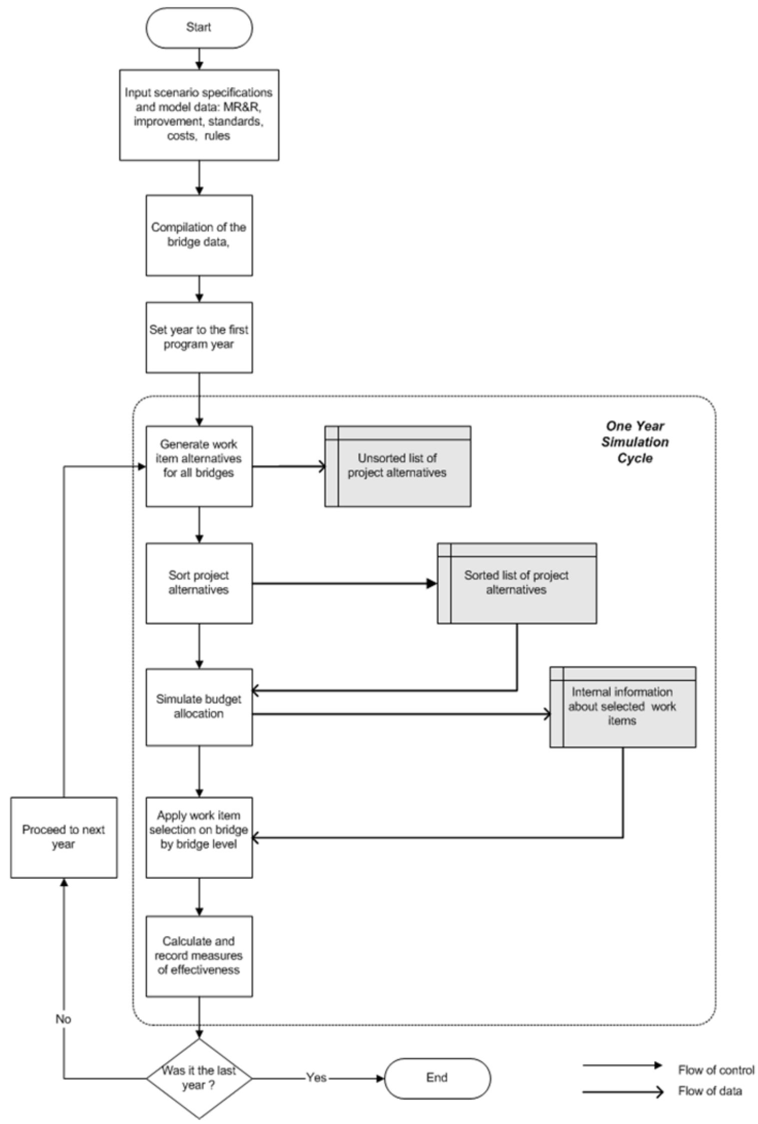

FIGURE 1 NBIAS program simulation steps prior to Version 5.0.
[Source: Robert and Gurenich (*5*).]

alternatives the system uses the MR&R policy to establish needed MR&R work, and applies a set of functional improvement criteria to determine the need for widening existing lanes, raising bridges, or strengthening bridges. Replacement of a bridge may be triggered if functional improvements are needed but infeasible (e.g., widening a truss bridge), if a replacement rule is triggered based on consideration of bridge condition and age, or if replacement is more economically efficient than MR&R or other functional improvements.

Once the set of needs is established, the list of needs is sorted in decreasing order of incremental benefit–cost ratio (IBCR), and projects are selected from the list until the available budget has been expended. The approach of selecting projects in decreasing benefit–cost ratio (BCR) is a heuristic that provides a near-optimal solution to the Capital Budgeting Problem (*6*). The additional step of using IBCR rather than BCR was recommended by McFarland et al. in their description of the INCBEN heuristic for solving the Capital Budgeting Problem for cases where one must select using multiple, mutually-exclusive project alternatives (*7*).

The basic modeling approach is subject to several issues and limitations. These include:

- Generation of MR&R needs is strictly guided by application of the MR&R policy. The system will neither reconsider the policy if funding is chronically short, nor will it "up-scope" work to take advantage of available funds. On the other hand, MR&R needs, are typically accorded high priority, and almost inevitably funded in national-level simulations run with budget levels comparable to expected funding.
- Absent adjustment, the tendency of the system is to allow bridge elements to deteriorate to poor condition, then take action prior to element failure. This tends to result in poor overall conditions, and large numbers of bridges predicted to be structurally deficient. Note this behavior occurs only when allowing an element to deteriorate to poor condition is the lowest life-cycle cost alternative. However, it does not account for agency performance standards and other factors that may result in a different element-level strategy in practice. Further, the tendency to allow elements to deteriorate to poor condition prior to taking action can be overcome to some degree by placing a penalty on poor conditions in solving for the MR&R policy.
- The system allows for specification of replacement rules forcing bridge replacement at specified minimum conditions. However, it can be difficult to predict the impact of adding replacement rules to the program simulation, particularly as the system will recommend replacement only if the BCR of replacing a bridge exceeds a specified minimum threshold.
- Only one overall budget may be specified when performing a simulation.
- The system allocates funds one year at a time, and does not carry unspent funds from 1 year to the next. Thus, particularly if the budget is unbalanced there may potentially be unspent funds in 1 year and unmet needs in others.

NBIAS 5.0 MODELING ENHANCEMENTS

In designing NBIAS Version 5.0 FHWA sought to change the NBIAS program simulation to support multiple budgets by work type, and to make additional changes to support generation of project alternatives to leverage available funds and enable improved performance targeting. To implement this change FHWA made the following enhancements, detailed further in the following subsections:

- Implemented logic for generating a set of life-cycle alternatives for each bridge, with each alternative specifying what action to be taken each 5-year period for up to 50 years.
- Changed the MR&R policy from a 1-year to 5-year policy.
- Altered the program simulation to select project alternatives for each bridge across all periods considering a matrix of budget constraints.

Generation of Life-Cycle Alternatives

The key change made to NBIAS Version 5.0 was to shift from selecting project alternatives on a year-by-year basis to making a single selection of bridge life-cycle alternatives over all periods at once. In order to facilitate this change it was necessary to implement new logic for generating alternative bridge life cycles, with each life cycle describing what actions will be performed on the bridge over a period of up to 50 years.

In concept, generating a life cycle is straightforward: one simply need specify what action to perform each period. However, given there are conceivably three feasible actions for a bridge in any one period (do nothing, MR&R and functional improvement/replacement), one could in theory generate 350 (approximately 7.18E+23) alternatives for each bridge. Managing this many alternatives can best be described as impractical. Thus, to reduce the number of alternatives generated for each bridge, the following rules were implemented in generating life-cycle alternatives:

- The model was changed to consider 10 5-year periods rather than 50 1-year periods.
- A bridge can be replaced or improved only once every 50 years.
- After the first 5-year period, the do nothing alternative is not generated. The "do minimum" option is thus to perform recommended MR&R work.

Based on these rules, in total 21 alternatives are defined for each bridge. These are illustrated in Table 1. This table includes one row for each life-cycle alternative, with the columns of the table indicating the action performed in each 5-year period for the specified alternative. In the table the do nothing action is indicated by "DN" and replacement or functional improvement (whichever is recommended) is indicated by "Improve." Each of the alternatives listed can be characterized by:

- Dollars spent by type of action each period.
- Discounted agency and user benefits obtained each period.
- Discounted future cost of maintaining the bridge following the end of the analysis period (calculated using the MR&R policy).

TABLE 1 Life-Cycle Alternatives Generated for Each Bridge

	Action by Period									
Alt.	1	2	3	4	5	6	7	8	9	10
1	DN	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R
2	DN	Improve	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R
3	DN	MR&R	Improve	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R
4	DN	MR&R	MR&R	Improve	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R
5	DN	MR&R	MR&R	MR&R	Improve	MR&R	MR&R	MR&R	MR&R	MR&R
6	DN	MR&R	MR&R	MR&R	MR&R	Improve	MR&R	MR&R	MR&R	MR&R
7	DN	MR&R	MR&R	MR&R	MR&R	MR&R	Improve	MR&R	MR&R	MR&R
8	DN	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R	Improve	MR&R	MR&R
9	DN	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R	Improve	MR&R
10	DN	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R	Improve
11	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R
12	MR&R	Improve	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R
13	MR&R	MR&R	Improve	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R
14	MR&R	MR&R	MR&R	Improve	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R
15	MR&R	MR&R	MR&R	MR&R	Improve	MR&R	MR&R	MR&R	MR&R	MR&R
16	MR&R	MR&R	MR&R	MR&R	MR&R	Improve	MR&R	MR&R	MR&R	MR&R
17	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R	Improve	MR&R	MR&R	MR&R
18	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R	Improve	MR&R	MR&R
19	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R	Improve	MR&R
20	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R	Improve
21	Improve	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R	MR&R

Revised MR&R Policy

The second major enhancement was to change the MR&R optimization to solve for a 5-year rather than a 1-year period. This change was made without changing the underlying modeling approach of the system described in Transportation Research Circular Number 498 (*4*), and instead required changing only the transition probabilities and discount factor used to account for the 5-year period.

Tables 2 and 3 illustrate the impact of shifting from a 1-year to 5-year policy. In this case, the MR&R model is shown for Element 104: Prestressed Concrete Box Girder, Climate Zone 1 (Wet, Freeze–Thaw) with default costs. Further, the discount rate is 7% (resulting in a discount factor of 0.934) and the unit failure cost is $3,894.66. Four condition states are defined for this element. Actions other than do nothing are feasible in States 2, 3, and 4. In States 2 and 3 the feasible actions are to do nothing or clean and patch. In State 4 the feasible actions are to do nothing, rehabilitate, and replace. The table shows the probability of transition to each state given the indicated action is performed, the unit cost of the action (in this case in dollars per lineal meter of girder), and the long-term cost of performing the action. The long-term cost is the discounted future cost for the element, assuming the indicated action is performed in the current period, and the optimal policy is followed subsequently. The final column of the table indicates which action is optimal in each state (the action with the lowest long-term cost). Here the optimal policy is to clean and patch in State 3 and rehabilitate in State 4.

Table 3 shows a revised version of the model solved assuming a 5-year period rather than a 1-year period. In this version of the model the do-nothing transition probabilities have been revised to reflect the probability distribution resulting from 5 years of deterioration, and the problem is solved with a 5-year discount factor of 0.713 rather than a 1-year discount factor of 0.934. The unit costs and transition probabilities for clean and patch, rehabilitate and replace have been left unchanged. The resulting optimal policy is the same—clean and patch in State 3 and rehabilitate in State 4—but the long-term costs are different, and the relative benefit of the clean and patch action is much greater in State 2 and 3 (the cost differential between do nothing and clean and patch), as this action is now considered once every 5 years rather than annually.

TABLE 2 Example MR&R Model with 1-Year Periods

		Probability of Transition to State					Unit Cost	Long-Term	
State	**Action**	**1**	**2**	**3**	**4**	**Fail**	**($)**	**Cost ($)**	**Optimal?**
1	Do nothing	92%	8%	0%	0%	0%	0.00	87.84	Y
2	Do nothing	0%	98%	2%	0%	0%	0.00	161.48	Y
	Clean and patch	86%	14%	0%	0%	0%	584.25	677.31	
3	Do nothing	0%	0%	87%	13%	0%	0.00	984.32	
	Clean and patch	53%	38%	10%	0%	0%	725.77	910.05	Y
4	Do nothing	0%	0%	0%	87%	13%	0.00	2,127.88	
	Rehabilitate	33%	41%	17%	9%	0%	1,620.42	2,026.86	Y
	Replace	100%	0%	0%	0%	0%	3,953.51	4,035.60	

TABLE 3 Example MR&R Model with 5-Year Periods

		Probability of Transition to State					Unit Cost	Long-Term	
State	Action	1	2	3	4	Fail	($)	Cost ($)	Optimal?
1	Do nothing	65%	28%	7%	1%	0%	0.00	435.74	Y
2	Do nothing	0%	55%	33%	10%	2%	0.00	813.42	Y
	Clean and patch	86%	14%	0%	0%	0%	584.25	933.12	
3	Do nothing	0%	0%	50%	37%	13%	0.00	1,432.17	
	Clean and patch	53%	38%	10%	0%	0%	725.77	1,191.06	Y
4	Do nothing	0%	0%	0%	48%	52%	0.00	2,372.81	
	Rehabilitate	33%	41%	17%	9%	0%	1,620.42	2,259.49	Y
	Replace	100%	0%	0%	0%	0%	3,953.51	4,264.17	

Comparing the long-term costs for State 3 in each table helps illustrate the differences. For State 3 the benefit of performing the clean and patch action in State 3 is $74.27 in Table 2 ($984.32 – $910.05), and $241.11 in Table 3 ($1,432.17 – $1,191.06). Though the optimal action is the same in both cases, the benefit of performing the action is substantially higher in Table 3. Had the unit cost of the action been $100 higher, the action would not have been recommended in the model solved for a 1-year period, but would have remained the optimal action in the model solved for a 5-year period. Note the long-term costs are higher for all actions and states in Table 3 than in Table 2 largely because in the case of the 5-year model there is a small probability of element failure from State 2 (triggering the failure cost) even when the optimal policy is followed.

As illustrated in this example, in general shifting to a 5-year period for the MR&R policy results in projection of increased benefits for taking action. It also in some cases results in a more aggressive MR&R policy, with actions recommended sooner, and tends to reduce the effect of introducing penalties for poor condition.

Revised Program Simulation

The underlying problem the program simulation attempts to solve is a variant of the Capital Budgeting Program discussed above, and can be expressed in the following equations.

$$\max \sum_i \sum_j \delta_{i,j} U_{i,j}$$

such that

$$\forall_i \forall_j \delta_{i,j} = \begin{cases} 0 \\ 1 \end{cases}$$

$$\forall_j \sum_i \delta_{i,j} = 1$$

$$\forall_k \forall_t \sum_i \sum_j \delta_{i,j} C_{i,j,k,t} \leq K_{k,t}$$

where

$\delta_{i,j}$ = 1 if alternative *i* for bridge *j* is programmed, 0 otherwise;
$U_{i,j}$ = benefit obtained from performing alternative *i* for bridge *j*;
$C_{i,j,k,t}$ = cost of performing alternative *i* for bridge *j* for action type *k* in period *t*;
M_t = maximum budget for period *t*; and
$K_{k,t}$ = maximum budget for action type *k,* period *t*.

The problem can be solved exactly using optimization methods, but in practice it is often impractical to solve the problem using an exact approach given limitations in processing speed and memory. Further, the IBC approach used in previous versions of NBIAS has been demonstrated to provide a near optimal solution under certain circumstances, though it is designed to work with a single budget constraint. Thus, for NBIAS 5.0 a different heuristic was used for sorting project alternatives. Specifically, this version utilizes the MINCBEN heuristic documented previously by Robert, Gurenich, and Thompson (*8*) and implemented in an analysis tool designed to work in conjunction with Pontis developed for the Virginia DOT.

To clarify how this heuristic works it is helpful to review the basic steps in the IBC approach originally defined by MacFarland et al. (*7*). These include

- The set of mutually exclusive alternatives is defined for each asset.
- For each asset the alternatives are ordered by increasing cost.
- If a given alternative has benefit less than or equal to that of another alternative with the same or less cost, the alternative is discarded.
- The IBCR for each alternative is calculated as the difference in benefit divided by the difference in cost of the alternative compared to the next cheaper alternative. For the cheapest alternative the IBCR is equal to BCR.
- The IBCR values are examined to verify that the benefit function is well-behaved (i.e., IBCR decreases as cost increases, which implies that the curve of benefits, plotted as a function of costs, is concave). In cases where a higher incremental benefit follows a smaller one, the two are averaged. This process is repeated until the benefit function is well-behaved. If the benefits measure is monetized consistently with costs, then incremental benefits should exceed incremental costs for each alternative, or the alternative should be discarded.
- The IBCR values for all assets are combined into a single list and sorted in decreasing order.
- Projects are selected from the list until the budget constraint is reached.

Figure 2, reproduced from Robert, Gurenich, and Thompson (*8*), provides an example of the calculation of IBCR using this approach. In this case three mutually exclusive alternatives are defined: A, B, and C. If funding is sufficient then C is preferred as this alternative provides the greatest benefit (12 versus six for B and 3 for A). However, if funds are limited then C may not be affordable regardless of its greater benefit. Thus the heuristic first selects A, as it has the highest IBCR, then B, followed by C if funds are sufficient. Note MacFarland recommends adjusting IBCR values as needed to make sure IBCR decreases with increasing costs, hence the

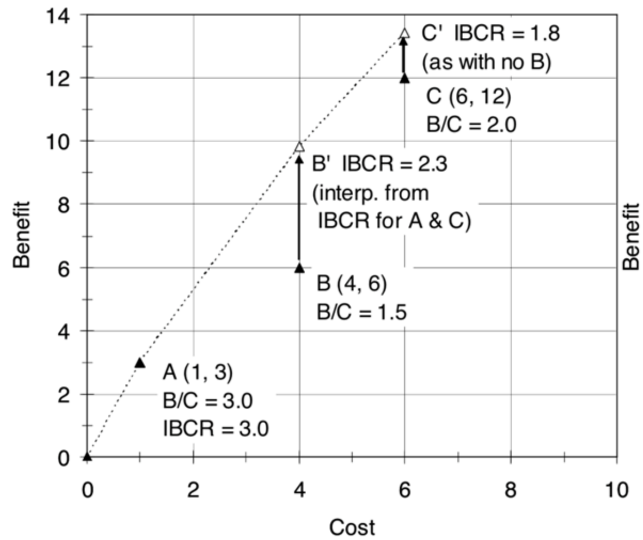

FIGURE 2 Example clculation of IBCR using the INCBEN heuristic. [Source: Robert, Gurenich, and Thompson (*8*).]

adjustment to Alternative B. In practice, in many implementations of the IBCR heuristic—including Pontis and NBIAS—alternatives such as B that fall below the benefit–cost curve are excluded along with any alternatives where benefits decrease with increasing cost.

As noted previously, the above heuristic is not designed to work with multiple budget constraints (e.g., for different work types or multiple periods). In these cases it becomes more important to consider how to handle alternatives such as B, and there may be cases where the optimal solution involves selecting an alternative that has less benefit than a cheaper alternative. For instance, if project C has greater benefit and is cheaper than a hypothetical project D then it is obviously preferred. However, project D may be the preferred alternative if it involves spending money in a year that is less constrained than required for C. The following variation on the IBCR heuristic (termed "MINCBEN") was proposed by Robert, Gurenich, and Thompson (*8*) to address such cases.

- The set of mutually exclusive alternatives is defined for each asset.
- For each asset the alternatives are ordered by increasing cost.
- The IBCR for each alternative is calculated as the difference in benefit divided by the difference in benefit of the alternative compared to the next cheaper alternative. For cheapest alternative is compared to the "do nothing" alternative.
- The IBCR values are examined to verify that the benefit function is well-behaved (i.e., IBCR decreases as cost increases, which implies that the curve of benefits, plotted as a function of costs, is concave). In cases where a higher incremental benefit follows a smaller one, the alternative with the smaller IBCR value is removed from the set of alternatives, and reserved for further consideration. The IBCR is then recalculated for the remaining alternative.
- After the set of alternatives for the asset is examined, analysis proceeds to the reserved set.

- The preceding three steps—recalculating IBCR, examining the benefit function, analyzing the new reserved set—are repeated until multiple sets of alternatives have been defined for each asset, each set having a well-behaved benefit function.
- The IBCR values for all assets and alternative sets are combined and sorted in decreasing order.

Alternatives are selected from the list of alternatives until the budget constraints are met. An alternative is skipped if selecting the alternative would violate a budget constraint, or if a selection has been made from a different alternative set for the same asset.

Figure 3, also reproduced Robert, Gurenich, and Thompson (*8*), illustrates how this heuristic functions. In this case, Alternative B is reserved. The heuristic first selects A, then C if funds are sufficient. Only if neither A nor B is selected will the heuristic consider C. For cases with a single budget constraint alternative C would never be selected, and the algorithm yields the same result as that of the current version of NBIAS. However, for complicated cases with budget constraints for multiple years and work types the modified heuristic "keeps all options on the table" and thus can provide a result that is closer to the optimal solution.

Figure 4 illustrates program simulation approach implemented in NBIAS 5.0, and shows how the generation and selection of life-cycle alternatives described above fits into the overall process. In contrast to previous versions of NBIAS, alternatives are generated once for all periods rather than once each period, and the selection of alternatives is performed in a single step. Alternatives are selected subject to a matrix of budget constraints. The following constraints are specified by period, as well as for nondeficient, deficient and all bridges:

- MR&R (constrained for the first period only),
- Widening,
- Raising,

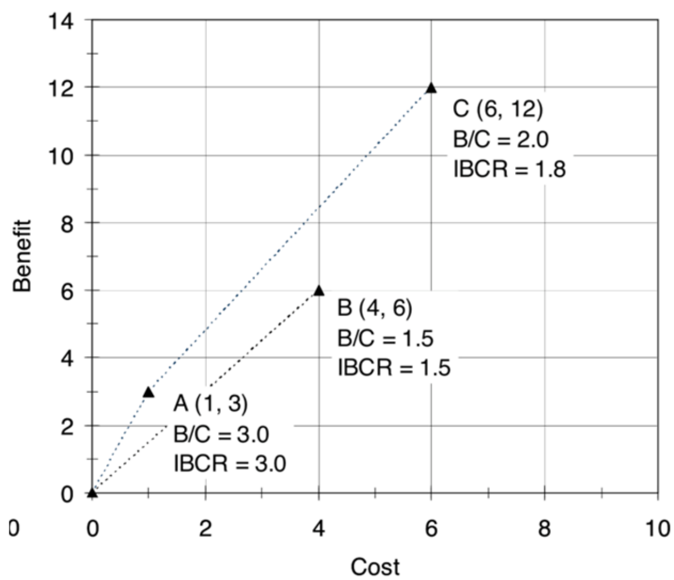

FIGURE 3 Example calculation of IBCR using the MINCBEN heuristic. [Source: Robert, Gurenich, and Thompson (*8*).]

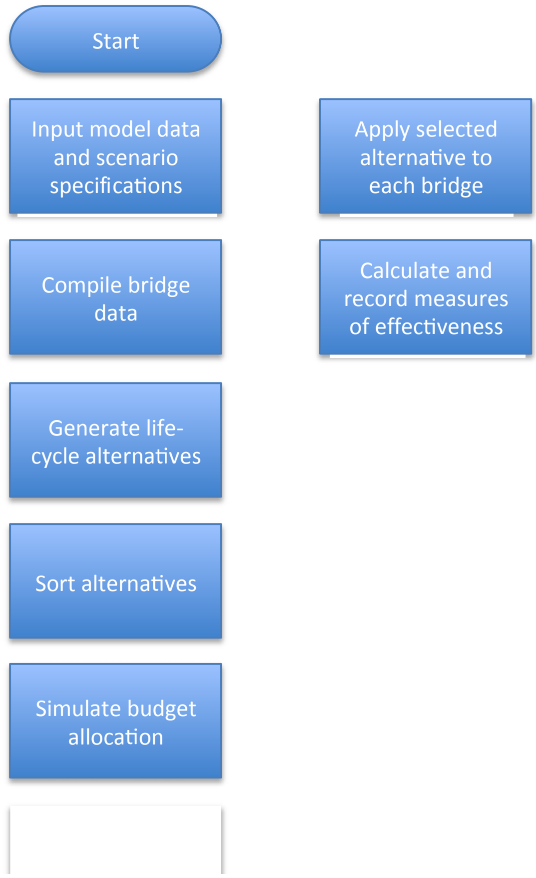

FIGURE 4 NBIAS 5.0 program simulation steps.

- Strengthening,
- Replacement,
- All functional improvements except replacement,
- All functional improvements including replacement, and
- Total budget.

Once life-cycle alternatives have been selected, the system simulates the application of each life-cycle alternative. The results are saved for viewing and reporting using the NBIAS What-If Module detailed in Robert and Gurenich (*5*).

IMPACT OF MODELING ENHANCEMENTS

Initial testing of NBIAS 5.0 indicates that this version of the system does indeed generate different results from prior versions, particularly as budget constraints are introduced by work type. Consistent with its approach to introducing other major modeling enhancements in the tools used to support development of the C&P Report, FHWA is planning to run old and new versions of the system in parallel in developing the next C&P Report to document and clarify the differences. Pending results of this process, initial findings from early tests of the system are:

- The MR&R policy recommended by the system is more aggressive, recommending treatments sooner than that recommended previously. Previously FHWA used an MR&R policy with a penalty on poor conditions to yield better and more realistic results. Adding this penalty had a similar effect, in terms of generating a more aggressive policy. Further testing is needed to determine whether such a penalty is justified in running NBIAS 5.0, and if so how it should be set.
- Generally there is an additional benefit to be obtained by replacing or improving a bridge in addition to performing needed MR&R work, and the tendency of the system, absent budget constraints, is to schedule replacement or improvement at some point over a bridge's life cycle. This reduces—and may even eliminate—the need to create replacement rules to force realistic model behavior. However, if traffic is projected to increase and the accumulation of benefits is limited to a period of 20 years, one can observe cases where greatest benefits are achieved by deferring improvement or replacement as late as possible in the simulation. Further investigation is needed to determine to what extent this occurs in practice, and whether the benefits accrual period should be adjusted.
- Version 5.0 of the system runs somewhat faster than prior versions as a result of the fact that alternative generation and selection is a separate process and need not be repeated when changing budgets and various other scenario parameters. Further speed improvements are nonetheless feasible.

PLANNED NBIAS ENHANCEMENTS

A variety of other enhancements are planned for NBIAS 5.1, scheduled for release in September 2017. This version of the system will extend the enhancements detailed here, adding:

- New element definitions. Transition from use of the AASHTO Commonly-Recognized elements defined in *AASHTO Guide for Commonly Recognized Structural Elements with 2002 and 2010 Interim Revisions* (*9*) to the newer element specification detailed in *AASHTO Manual for Bridge Element Inspection* (*10*). Elements modeled by the system will include those defined in FHWA's *Specification for National Bridge Inventory Bridge Elements* (*11*).
- Support for culverts. The NBU includes a number of bridge-length culverts, but these are screened from analysis in NBIAS. Beginning with NBIAS 5.1 these will be included in the analysis.
- Support for good–fair–poor measures. NBIAS predicts numbers of bridges with specific values for deck, superstructure, and substructure ratings, but provides few measures summarizing overall conditions across rating values. In Version 5.1 the system will calculate percentage of bridge area in good, fair, and poor condition. Consistent with measures defined separately by FHWA, a bridge will be defined to be in good condition if the minimum value of its condition rating is 7 (on a scale from 0 to 9), in fair condition of the minimum is 5 or 6, and in poor condition if the minimum is 4 or less.

CONCLUSION

The modeling enhancements to NBIAS described in this paper offer the potential for FHWA to obtain more accurate and robust projections of highway bridge investment needs and future bridge conditions. However, the work described here raises a number of questions and potential topics for future research. These include the following:

- Increasing the number of alternatives considered. As detailed here, NBIAS 5.0 considers 21 life-cycle alternatives for each bridge over a 50 year period. In concept this number could be increased significantly, particularly in cases where one is analyzing a subset of the nation's bridges, or analyzing a period shorter than 50 years. In these cases it may be valuable to increase the number of periods with a do-nothing alternative defined, or allow for a variable analysis period—both of which would tend to increase the number of alternatives generated.
- Exploring potential for using an exact optimization rather than a heuristic approach. The heuristic approach used in NBIAS for selecting project alternatives is expected to yield near optimal results, but further research is warranted to evaluate how well the heuristic performs, and whether implementation of an optimization approach yielding an exact solution is warranted.
- Implementing parallel processing. NBIAS is architected as a client–server system and does not take advantage of parallel processing or other advanced computational features. However, the change in the program simulation approach of the system, to decouple generation of project alternatives from the year-by-year simulation, enables implementation of parallel processing at a later date to further speed the analysis.
- Other modeling enhancements. FHWA has considered a variety of other potential model enhancements, and may implement these in the future, to the extent they support improved results and can be implemented given available resources. These include but are not limited to modeling other bridge needs besides those triggered by physical condition, such as scour and seismic vulnerability, expanding modeling of widening to consider need for capacity improvements, further improving performance targeting, utilizing the element-level inspection data now being submitted for National Highway System bridges, and various other enhancements.

ACKNOWLEDGMENTS

The authors acknowledge the FHWA Office of Legislative and Governmental Affairs, Highway Needs and Investment Analysis Team for its continuing support of the NBIAS.

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Validating Common Collapse Conjectures in U.S. Bridges

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Bridge collapse is a rare event. However, given the 610,000 plus bridges in the United States and existing level of structural reliability, a certain number of bridge collapses are expected. The New York State Department of Transportation maintains a bridge collapse database, which has been combined with the National Bridge Inventory (NBI) into a new database of the NBI ratings and appraisal for the inspection before collapse occurred. The new database contains 428 bridges that have collapsed and are in record between 1992 and 2014. The compiled–collapse database allows for the evaluation of common conjectures among collapsed bridges. Common conjectures that are studied are structural deficiency and bridge collapse, scour critical rating and hydraulic collapse, age and bridge collapse, and design-provision improvements. Structural deficiency and collapse are associated. The scour critical rating and the condition rating of the substructure indicates that the minor scour is a precursor to accelerated deterioration. Minor scour appears to be a greater hazard to the substructure than it is currently assessed. For collision-caused collapse, newer bridges are built with an improved bridge characteristic that have reduced the chances of a random-event strike. For overload-caused collapse, newer bridges are designed with increased loading requirements that have reduced the chances of overload.

Past investigations analyzed trends among collapsed bridges in the United States by associating the New York State Department of Transportation (DOT) database (*1*–*5*). Cook et al. (*2*) assess trends among collapsed bridges for the state of New York; a frequency of bridge collapse is expected to be ¼,700 annually with additional validation from other states. Wardhana and Hadipriono (*1*) analyzed collapse-trends for bridges that failed between 1989 and 2000. From their study, statistics such as the mean lifespan of a collapsed bridge (52.5 years) is determined. It is also stated, that hydraulic collapse is the number one cause of bridge failure in the United States. Similar investigations with a different database (*6*) have also determined that hydraulic failure is the number one cause of bridge failure in the United States, and Montalvo and Cook (*4*) confirmed it through the analysis of the New York State DOT database.

The Centers for Disease Control and Prevention (CDC) maintains a fatality database, which presents the characteristics of those dying in the United States, to determine life expectancy, and to compare mortality trends (*7*). With the vast amount of data that the CDC collects, this agency is better equipped with data-driven prevention. These qualities are all desirable in the field of structural engineering and in particular bridges in the transportation systems. Unfortunately, the fatality or collapse of bridges has yet to follow suit on the data collection on such a wide scale. This is in part due to the stigmatism and public perception of reporting bridge collapses. As a result, bridge collapse research and data collection are generally limited to significant catastrophic collapses or events. The majority of the bridge collapse events are not considered major events. In this study, with a large sample size of collapsed bridges, analysis of bridge-collapse conjectures is possible.

Common collapsed-bridge conjectures are assessed in an effort to advance the knowledge and predictors of bridge collapse based on the condition and state of bridges from inspection information prior to collapse. The information presented can assist bridge owners and managers in understanding the likelihood of bridge collapse based on mathematical evidence and observed trends.

The investigation presents the databases and statistical methods followed by the analysis preformed. The assessment to date evaluates structural deficiency, scour and scour critical ratings, limited age analysis, and evidence of increased bridge longevity with improved design specifications.

DATABASES

Two databases used to assess common conjectures among collapsed bridges are the National Bridge Inventory (NBI) and the New York State DOT Bridge collapse database. The 2014 NBI database contains inspection data for the more than 610,000 vehicular bridges in the United States (*8*). In-service bridge data and statistics obtained from NBI 2014 act as control data. In addition, the NBI contains bridge inspection data over multiple years dating back to 1992. Bridge inspection ratings are on a scale from 0 to 9 with 0 signifying that the structure is closed or failed and 9 being the best condition (see Table 1 for a breakdown of the rating system). The New York State DOT bridge collapse database contains United States collapsed bridges data acquired through valid sources. For the purposes of this study, failure or collapse is either partial collapse or total collapse. Partial collapse is "severe deformation to several primary members of a span which allows travel but endangers the lives of those passing on or under the structure." Total

TABLE 1 NBI Condition Ratings (*10*)

Code	Description
N	Not applicable
9	Excellent condtion
8	Very good condition: no problems noted
7	Good condition: some minor problems
6	Satisfactory condition: structural elements show minor deterioration
5	Fair condition: all primary structural elements are sound but may have minor section loss, cracking, spalling, or scour
4	Poor condition: advanced section loss, deterioration, spalling, or scour
3	Serious condition: loss of section, deterioration, spalling, or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.
2	Critical condition: advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored it may be necessary to close the bridge until corrective action is taken.
1	Imminent failure condition: major deterioration or section loss present in critical structural components or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic but corrective action may put back in light service.
0	Failed condition: out of service—beyond corrective action

collapse is “severe deformation to several primary members of a span or several spans which leaves the structure unpassable” (*9*). The New York State DOT database generally contains the year built of the bridge, the year it collapsed, the cause of collapse, feature intersection, material of the bridge, and bridge type as well as comments which can further explain the collapse of the structure.

Using the NBI and New York State DOT bridge collapse databases, a new database compilation associates the NBI data of bridges for the inspection ratings prior to collapse and collapse data. There are 428 vehicular bridges (excludes pedestrian, railroad, etc.) that have collapsed and are associated with precollapse NBI data between the period of 1992 and 2014. With the data, several assessments on common conjectures are investigated through mathematical processes. The large sample size in this study provides control on the variability of the data.

ANALYTICAL METHODS

The majority of data fields assessed in the compiled data are nonparametric or skewed and are not normally distributed. The control data (*8*) is also nonparametric, from a normality check, for the same data fields. Nonparametric statistics (i.e., median instead of mean) and statistical methods enable assessment of common conjectures among collapsed bridges.

One statistical test used in this investigation is the Kruskal–Wallis H test. The Kruskal–Wallis H test is a rank-based nonparametric that can be used to determine if there are statistically significant differences between two or more groups of an independent variable on a continuous or ordinal dependent variable (*11*). The Kruskal–Wallis H test is a nonparametric test which does not require or assume normality in the data. A Kruskal–Wallis H test is similar to a one-way analysis of variance (ANOVA), but considered the nonparametric alternative to it.

Another statistical method used in this investigation is the Chi-squared test. The Chi-squared test examines independence of binary variables at 1 degree of freedom.

STRUCTURAL DEFICIENCY

Structurally deficient (SD) are bridges generally in poor condition and have a rating of 4 or less for the deck, superstructure, substructure or a 2 for waterway adequacy (*12*). Out of the 428 bridges that collapsed, 197 (46.0 %) are SD (Table 2). For all the bridges currently in service in the United States, 53,354 (9.0 %) out of more than 610, 000 are SD. The significant amount of bridges that are SD and failed suggest that there is a possible association between structural deficiency and bridge failure. A chi-squared test of independence (Table 2) assesses the association between structural deficiency and collapse and indicates that the two variables are associated. The result concludes that structural deficiency or poor condition in inspection element condition rating in the United States is a possible indicator of bridge failure. An analysis of structural deficiency per collapse–cause yields different conjectures. As per Table 3, overload–caused collapsed bridges are 53.3% structurally deficient in the superstructure. Hydraulic-caused collapsed bridges are 32.5% structurally deficient in the substructure. Deterioration-caused collapse has similar quantities of structural deficiency in each bridge component, and they all have equal median condition ratings.

TABLE 2 Contingency Table for Structural Deficiency Versus Collapse

	Failed Bridges	In-Service Population
SD bridges	197 (46.0%)	53,354 (8.7%)
Non-SD bridges	231 (54.0%)	557,073 (91.3%)
∑	428	610,427

TABLE 3 Structural Deficiencies per Type of Collapse

Type of Collapse	SD Deck	SD Superstructure	SD Substructure	Median Age
Overload	14 (23.3%)	32 (53.3%)	25 (41.7%)	68
Hydraulic	38 (16.0%)	48 (20.2%)	77 (32.5%)	54
Deterioration	12 (33.3%)	14 (38.9%)	14 (38.9%)	48
Collision	6 (7.3%)	14 (17.1%)	13 (15.9%)	43
In-service population	4,968 (0.8%)	22,264 (3.6%)	29,189 (4.8%)	41

Collision-caused collapse is lower all around in SD but, remains higher than the in-service population. From the comparison the component leading SD also appears to relate to the cause of collapse, more discussion on this topic is located in the Age and Bridge Collapse section.

SCOUR CRITICAL RATING

There are 237 (55.4%) bridges that have collapsed due to a hydraulic-induced failure. Given that hydraulic-caused collapse is the number one cause of bridge failure, it is critical to gain a deeper understanding of trends for this cause of collapse. Since the majority of hydraulic collapses are a result of a scour-induced failure (*13*) the scour critical rating (NBI Item 113) is assessed. Scour is erosion of streambed or bank material due to flowing water; often considered as being localized (*13*). A chi-squared test of independence (see Table 4 for the contingency table) performed between hydraulic failure and the scour critical rating yields a p-value of less than 0.001, which indicates that the two variables are associated. See Figure 1 for the distribution of the scour critical ratings (6, *U*, *T*, and *N* are omitted for simplicity) and see Table 5 for a breakdown of the ratings. For the scour-critical rating, an elemental rating given with 9 through 4 excluding a 6, signifies that the substructure is rated scour stable, a 6 indicates that the scour evaluations have not been made, and 3 through 0 indicate that the substructure is rated scour critical. Upon the inspection of Figure 1, the majority of the bridges in the in-service population and hydraulic-caused collapsed bridge are given a scour critical rating between 8 and 4. A rating between an 8 and a 4 indicates that bridges have been evaluated as scour stable. Bridges are rated scour stable, even though scour causes the majority of bridge failures in the United States. It is evident that there is a discrepancy between the scour critical rating given and the cause of collapse.

TABLE 4 Scour Critical Rating Contingency Table

	Hydraulic Failure	In-service Population
Scour critical	22	22,387
Nonscour critical	109	448,572
∑	131	470,959

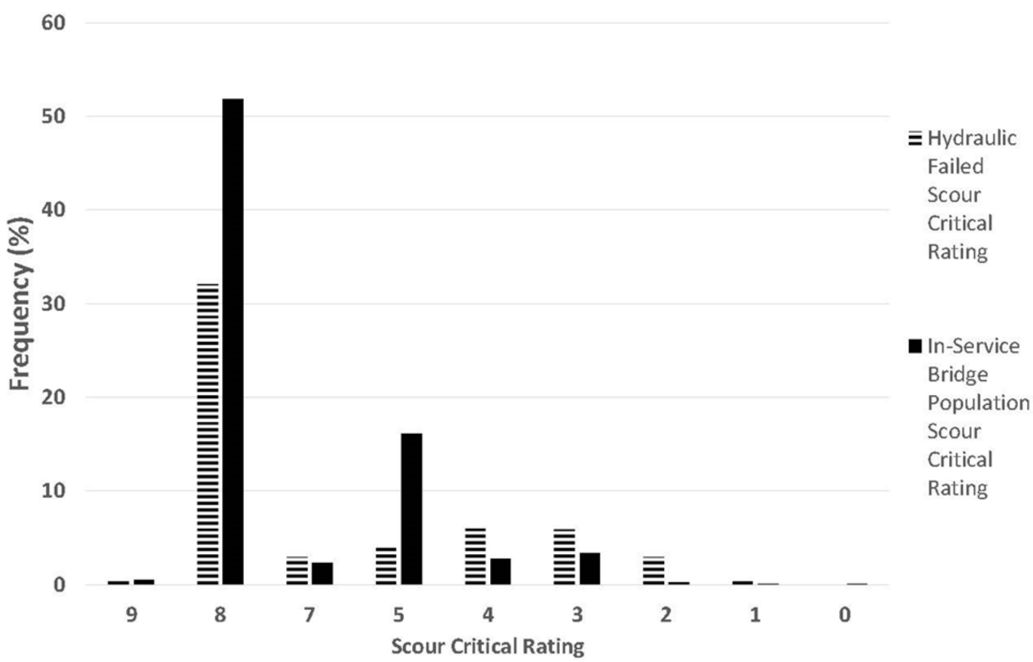

FIGURE 1 Histogram of the scour-critical rating for hydraulic failure.

As per Table 6, hydraulic-caused collapse has a median condition rating of a 5, and the in-service population has a median condition rating of a 7 (see Table 1 for the condition rating descriptions). A rating of a 5 represents minor section loss, cracking, spalling, or scour. Hydraulic-caused collapsed bridges experience an age-induced deterioration for the deck and the superstructure. For the substructure, there is an accelerated deterioration compared to the deck and the superstructure. Since hydraulic-caused collapse is the number one cause of bridge failure, assuming that a rating of a 5 has been given due to minor section loss, cracking, or spalling is not rational. With the accelerated deterioration of the substructure, a better approach for hydraulic collapse is to assume that the substructure has a median rating of a 5 because of the presence of minor scour. The hazard that minor scour represents for the substructure is more critical than it is currently assessed to be.

TABLE 5 NBI Scour-Critical Bridge Ratings (*9*)

Code Description	
N	Bridge not over waterway.
U	Bridge with "unknown" foundation that has not been evaluated for scour. Since risk cannot be determined, flag for monitoring during flood events and, if appropriate, closure.
T	Bridge over "tidal" waters that has not been evaluated for scour, but considered low risk. Bridge will be monitored with regular inspection cycle and with appropriate underwater inspections.
9	Bridge foundations (including piles) on dry land well above flood water elevations.
8	Bridge foundations determined to be stable for assessed or calculated scour conditions; calculated scour is above top of footing.
7	Countermeasures have been installed to correct a previously existing problem with scour. Bridge is no longer scour critical.
6	Scour calculation–evaluation has not been made. (Use only to describe case where bridge has not yet been evaluated for scour potential.)
5	Bridge foundations determined to be stable for calculated scour conditions; scour within limits of footing or piles.
4	Bridge foundations determined to be stable for calculated scour conditions; field review indicates action is required to protect exposed foundations from effects of additional erosion and corrosion.
3	Bridge is scour-critical; bridge foundations determined to be unstable for calculated scour conditions.
2	Bridge is scour-critical; field review indicates that extensive scour has occurred at bridge foundations. Immediate action is required to provide scour countermeasures.
1	Bridge is scour-critical; field review indicates that failure of piers/abutments is imminent. Bridge is closed to traffic.
0	Bridge is scour-critical. Bridge has failed and is closed to traffic.

TABLE 6 Age versus Median Condition Ratings

Cause of Collapse	Age (Years)	Deck	Superstructure	Substructure
Collision	43.0	6	6	6
Deterioration	47.5	5	5	5
Hydraulic	53.5	6	6	5
Overload	68.0	6	4	5
In-service population	41.0	7	7	7

With the discrepancy between the scour-critical rating and hydraulic collapse, underwater inspections are evaluated for hydraulic-caused collapsed bridges. Only 16 (6.8%) of the hydraulic-caused collapsed bridges require an underwater inspection; while the in-service population only requires underwater inspections for 19,267 (3.2%) of the bridges in the United States. In addition, 520,000 plus (85.2%) bridges in the United States are over a waterway. Increasing the number of bridges that require an underwater inspection has the potential to provide a better assessment for the condition rating of the substructure and scour critical rating.

Another method that can address the discrepancy between the scour critical rating and hydraulic collapse is to revise the current rating system. As per the compiled–collapse database, 112 (47.3%) of the hydraulic collapses are classified as a hydraulic–flood collapse. Even though a flood is considered a random event, the scour-critical rating inspection system should account for the hazard that minor scour represents in case a flood occurs. Modifying the scour critical rating to account for the probability of failure due to flood events can help preserve bridges in the United States.

AGE AND BRIDGE COLLAPSE

An analysis of age and collapse indicates that the mean age of bridges in the collapse database is 55 years with a standard deviation of 27 years, and median of 51 years. Age is assessed per type of collapse. As per Table 6, there is an age-induced deterioration for all collapse-types (*4*). However, the age deterioration is not sufficiently rapid to be a serious hazard to the condition of the structure. In general, for each cause of collapse there is an alternate-cause–induced accelerated deterioration. For overload-caused collapse, the superstructure has a lower condition rating than the deck and the superstructure. The accelerated deterioration of the superstructure in overload-caused collapsed bridges requires further investigation. Hydraulic-caused collapsed bridges have a lower condition rating for the substructure. The lower condition rating is attributed to the accelerated deterioration caused by the presence of minor scour. Deterioration-caused collapse differs from the other causes of collapse as it experiences an even deterioration between components; the median age is lower than hydraulic-caused collapsed and overload-caused collapsed bridges. Collision-caused collapsed bridges fail due to a random-event induced strike. For all causes of collapse, there are apparent variables besides age that deteriorates bridges at a faster rate than age does.

EVIDENCE OF IMPROVED DESIGN SPECIFICATIONS

Improvements to the bridge design specifications are continuous and evidence of increased bridge longevity; however, post implementation can be difficult to measure. Through this retrospective analysis, two improvements to the design specifications that show association with bridge collapse are decreased minimum vertical clearance and decreased operating rating. These two areas are compared with collision-caused and overload-caused collapses, respectively.

Minimum Vertical Under-Clearance

As per Table 7, collision-caused collapse has the highest-frequency usage [average daily traffic (ADT) and annual ADT (AADT)] from all of the causes of collapse indicating bridges with higher usage are more likely to experience high-impact loads.

The geometric characteristic of the collision-caused collapsed bridges is analyzed to understand the impact that the improvement to the design provisions have on collision-caused collapse. The bridge characteristic analyzed is the minimum vertical underclearance. A Kruskal–Wallis H test compares the median minimum vertical underclearance bridge characteristic for the compiled database and the in-service population. The Kruskal–Wallis H test's result for the minimum vertical clearance (NBI Item 54) yields a p-value of 0.016 at 1 degree of freedom. See Table 8 for the median minimum vertical clearance for collision-caused collapse and the in-service population. The test indicates that a lower vertical under-clearance increases the chances

TABLE 7 ADT and AADT per Cause of Collapse

Cause of Collapse	Median ADT (NBI Item 29)	Median AADT (NBI Item 109)
Collision	3,500	9
Deterioration	1,104	8
Hydraulic	150	6
Overload	123	6
In-service population	840	6

TABLE 8 Bridge Geometric Characteristic

Category	Median Minimum Vertical Clearance (NBI Item 54)
Collision-caused collapse	4.9 m (16.1 ft)
In-service population	5.1 m (16.9 ft)

of a random over-height–induced collision. The test also indicates that older bridges tend to have lower minimum vertical clearances. An additional correlation test known as the Spearmen's rank coefficient evaluated the NBI 2014 data to verify older bridges have lower minimum vertical clearances. The results yield a correlation of −0.28, the negative indicates an inverse relationship and the closer the coefficient is to negative one the stronger the inverse correlation. With the result being −0.28the correlation appears poor; however, the statistical power due to a sample size of over 100,000 bridges over roadways the value is statistically significant. The result does verify that as age increases minimum vertical clearance decreases. The change in minimum vertical clearance (*14*) changed from 4.3 m (14 ft) to 4.9 m (16 ft) in 1960. Although collision-caused collapse can be thought of as random events, a statistically significant difference is found in the bridge height and age. The results indicate that the improvements done to the design provisions, such as the increase in minimum vertical clearance, have decreased the chances of collapse in newer bridges.

Design Load and Operating Rating

Inspection of age and the type of collapse (Table 6) show overload-caused collapsed bridges tend to be older. An analysis of the design load (NBI Item 31) and age is of interest; however, the median design load for overload-caused collapse is 0, meaning "other or unknown." Where the median age is high, and the design load interpreted as unknown, the lack of information on design loading is compensated by using the operating rating (NBI Item 64). Two Kruskal–Wallis H tests compare age and the type of collapse and the Operating Rating, an evaluated condition, and the type of collapse. The result of each Kruskal–Wallis H test yields a *p*-value of less than 0.001 at 1 degree of freedom. There is a statistical significant difference between the median ages and their respective cause of collapse, and the median operating rating and their respective cause of collapse (shown in Figure 2). The results indicate that overload-caused collapse has the highest median age, even above deterioration-caused collapse, and the lowest operating rating. While the in-service bridge population has the lowest median age and the highest operating rating; newer bridges have a higher operating rating than older bridges do, which can be attributed to the increase of the minimum design load and a lower median age (less deterioration). Lower operating ratings can be attributed to lower minimum design loads, a higher median age, and greater deterioration.

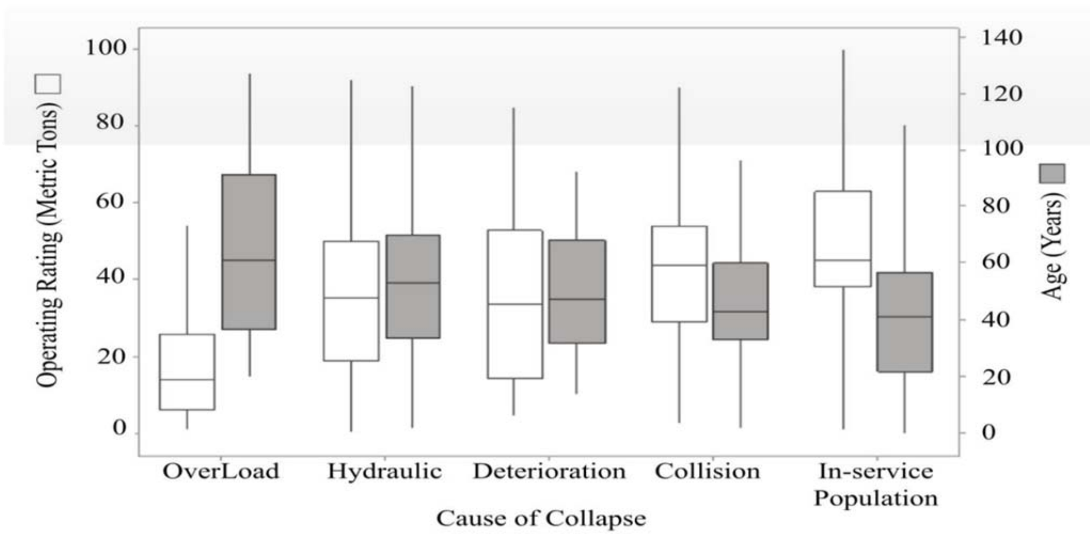

FIGURE 2 Boxplot of median age and median operating rating per cause of collapse.

The design load and age is also evaluated through correlation Spearmen's rank coefficient with the NBI 2014 data. The results yield a correlation of −0.34, the negative indicate an inverse relationship. The correlation appears poor; however, the statistical power due to a sample size of over 400,000 bridges with nonzero design load values; the correlation is statistically significant. The result does verify that as age increases design loading decreases or older bridges are designed with lower vehicle loads.

CONCLUSION

There are 428 bridges that have collapsed and are in record between 1992 and 2014. The mean age of collapse bridges is 55 years with a standard deviation of 27 years, and a median of 51 years. The assessment of common conjectures yields that structural deficiency and collapse are associated. Another test of association indicates that the scour critical rating and hydraulic collapse are associated. The majority of hydraulic-caused collapsed bridges are rated as scour stable at the time of inspection. It is evident that there is discrepancy between the scour critical rating and hydraulic collapse. There is potential for accelerated deterioration due to the presence of minor scour at the substructure that the current inspection system is not accounting for. Possible solutions for the discrepancy in the current inspection system for the scour critical rating are to require more underwater inspections, which have the potential to provide a better assessment for the condition rating of the substructure and scour critical rating. Adjusting the scour critical rating to account for the probability of a flood event is a probable solution to the discrepancy between the scour critical rating and hydraulic failure. Collision-caused newer bridges are being built with higher vertical underclearance that have reduced the chances of a random event strike. In addition, new bridges have higher design loadings.

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Implementation of Road Structure Management System KUBA
Experience Report

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KUBA is a comprehensive road structure management system, developed for the Swiss Federal Roads Office. KUBA relies heavily on the inspection data to obtain deterioration functions and on data on performed maintenance interventions to obtain unit cost data. The collection of inspection data is well established and proceeds quite smoothly. The collection of maintenance data poses a severe problem due to organizational and technical problems.

In this experience report the data collection for KUBA is described with the focus on the measures to ensure data quality and work efficiency.

In the first part, the lack of data in "bad" condition states is discussed, which proves to be a serious obstacle to obtain meaningful deterioration functions. The paper describes the consequences if the raw data is used to obtain deterioration functions.

In the second part, the agency organization is described and the organizational issues are addressed that hinder the meaningful exploitation of data on maintenance interventions. The split in responsibilities between the asset management and construction management seems to pose an obstacle to obtain data that can be used for planning purposes. The possible organizational measures are described – some of them are implemented – that can improve the work flow and consequently facilitate the accessibility of necessary information.

In the third part, a method for the monitoring of workload related to inspections and the analysis of the monitoring results are presented. The influence of different properties was analyzed in order to determine the ones that govern the inspection workload.

The Swiss Federal Road Office (FEDRO) is responsible for high-volume road infrastructure, of approximately 12,500 road structures, which include 4,300 Bridges and 220 tunnels. Each of its five regional offices is in charge with the operational asset management and the construction management.

For road structures, FEDRO performs visual inspections every 5 years. Within FEDRO's inspections, a condition assessment is performed for the whole structure, its elements (like pillars, bearings, joints, etc.), and damage areas of the elements. The data on damages can be also stored (Figure 1). In some cases, elements can be further divided into segments to account for different deterioration process or exposition. The inspection results are stored in the road structure management system (RSMS) named KUBA. For this purpose, the inventory data on road structures and its elements have to be collected before the first inspection. For the main inspections, FEDRO spends approximately US$3 million per year.

Based on the stored data, the RSMS KUBA furnishes the condition forecast and financial needs for the period of 40 years and proposes maintenance interventions.

KUBA uses Markov chains for modeling the condition development. Each combination of deterioration process and exposition is modeled by its characteristic Markov chain. At first the transition matrices of the Markov chains were estimated by a pool of experts. As time goes by,

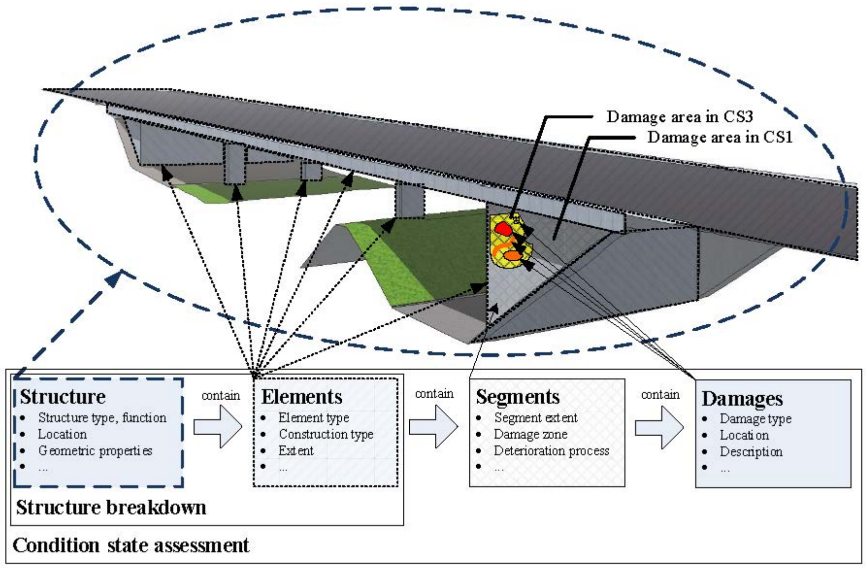

FIGURE 1 Structure breakdown and condition state assessment within RSMS KUBA.

the matrices are updated using suitable statistical analysis of the condition data collected during inspections.

Possible interventions are classified in a relatively small number of intervention types. Each of it is characterized by unit costs and effectiveness. KUBA's cost forecasts depend on these values. The unit costs refer to a specific unit in which the extent of an element is measured. The unit for maintenance work on reinforced concrete, for example, is the square meter of the surface area.

A maintenance intervention results in a condition state improvement expressed by transition probabilities, which represent the effectiveness of the intervention. This approach incorporates the empirical knowledge that a maintenance intervention often does not restore an element into the best condition state. The transition probabilities representing intervention effectiveness are calculated and updated by a statistical analysis of the condition data collected during inspections before and after the interventions (*1*, *2*).

In this experience report the data collection for KUBA is described with the focus on the measures to secure data quality and work efficiency.

The lack of data in bad condition states is a serious obstacle for the determining of condition development in KUBA. The estimation of the Markov chains relies on observations in each condition state and since FEDRO seldom allows structures and elements to deteriorate into the worst two condition states, reliable calibration can hardly be performed. Even if the calibration algorithms are able to bridge data voids (*2*), they cannot overcome the problem of lack of almost all data in bad condition states. In the next section the results of just using the raw data for the calibration are presented.

Within FEDRO, the two areas of responsibility of the asset and the construction management are clearly separated. Since the asset management does not have direct managerial authority over the construction management and the design and construction phases are under the responsibility of the construction management, the construction management is almost free to decide which interventions are to performed and how they are to be documented, respectively, which data is to be collected. The asset management defines the standards and control mechanisms, but due to cost and time pressure as well as due to the lack of manpower, the control mechanisms are not effective and the data on performed interventions is not collected in sufficient quality. The lack of this data hinders the tracking of the road structure's history and the quality improvement of the unit costs.

In the third section of this paper, these issues are described in detail. Furthermore, possible organizational measures are described—some of them are implemented—that can improve the work flow and consequently facilitate accessibility of information.

Finally, although the standards defined for KUBA as well as the software itself are well-documented and known in Switzerland, the inspections, which are contracted out to private consultants, has to be supervised in order to ensure data quality. The stiff competition among private consultants and related cost pressure may tempt consultants to assign inspectors with little experience in structure diagnostics and expertise in working with KUBA. To overcome the lack of expertise and experience, the employees of the private consultants had to attend a training course. Private consultants are required to register the amount of work for each structure in monthly time sheets. By this, the workload can be evaluated in combination with the data stored in KUBA. Results of the evaluation—e.g., determining factors influencing the workload for inspections—will be presented in the last section of this paper. The workload on almost 500 road structures was analyzed.

DATA COLLECTION OF DETERIORATION

Collected Data and Missing Data in Bad Condition States

KUBA provides decision support in the planning of maintenance interventions. In order to compare maintenance strategies, the system forecasts the deterioration using discrete Markov chains. Estimating the transition probabilities of a discrete Markov chain is rather straightforward when observational data are available at each discrete time instance. KUBA algorithms used to obtain transition probabilities are described in detail in (*2*).

For corrosion of reinforced concrete with average exposition, the number of transitions between any two condition states (CS) observed in two consecutive inspections is presented in the matrix below. The row CS3 in the matrix, for example, is to be read as follows: the total of 49 transitions from CS3 to CS4 was observed; whereas –1,201 damage areas stayed in CS3. The initially mentioned exposition is used to consider that portions of an element may behave differently. In order to consider these differences, each segment is attributed a so-called "exposition indicator." Three exposition indicators are used: favorable, average, and unfavorable, which are correlated to the segment having slow, moderate, or fast deterioration.

Observed Number of Transitions					
	CS1	CS2	CS3	CS4	CS5
CS1	41,491	14,798	4,520	370	10
CS2	0	2,572	329	23	0
CS3	0	0	1,201	49	0
CS4	0	0	0	75	3
CS5	0	0	0	0	3

In Figure 2, on the left-hand side, the polygonal lines represent the deterioration pattern of each observed damage area. The transitions between identical condition states are ignored, so that the lines connect the points of the first observations in each CS. In the figure on the right-hand side, the total number of transitions from starting CS is shown. It can be understood as sample size in each starting CS.

In order to calculate the deterioration matrix, the year in which the transition is observed has to be taken in account. It is therefore not possible to simply divide the total number of transitions from the CS (respectively the sample size) by the number observed damage areas that stay in the same or change to the next worse CS.

For deterioration of expansion joints with average influence, the number of transitions between any two CSs observed in two consecutive inspections is presented in the matrix below.

Observed Number of Transitions					
	CS1	CS2	CS3	CS4	CS5
CS1	1,071	843	493	70	1
CS2	0	104	52	2	0
CS3	0	0	89	6	0
CS4	0	0	0	2	0
CS5	0	0	0	0	0

As already described, in Figure 3, on the left-hand side, the polygonal lines represent the deterioration pattern of each observed damage area. On the right-hand side of Figure 3, the total number of transitions from starting CS is shown.

The figures and matrices show that few transitions are observed from CS2, very few from CS3, and almost none from CS4 (Figures 2 and 3). It is supposed that the reason is to be found in the common practice in Switzerland, according to which interventions are mostly performed in CS3. Unfortunately, these observations are missing since no inspections are stored immediately before performing the intervention.

The described analysis was made for all relevant deterioration processes in KUBA. The results are very similar to the ones above and present the same problem.

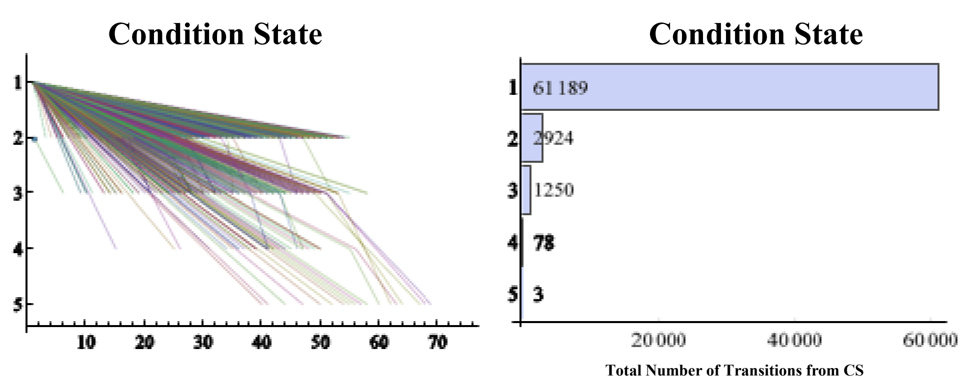

FIGURE 2 Transitions of CS by age and number of transitions by CS for corrosion of reinforced concrete with average influence.

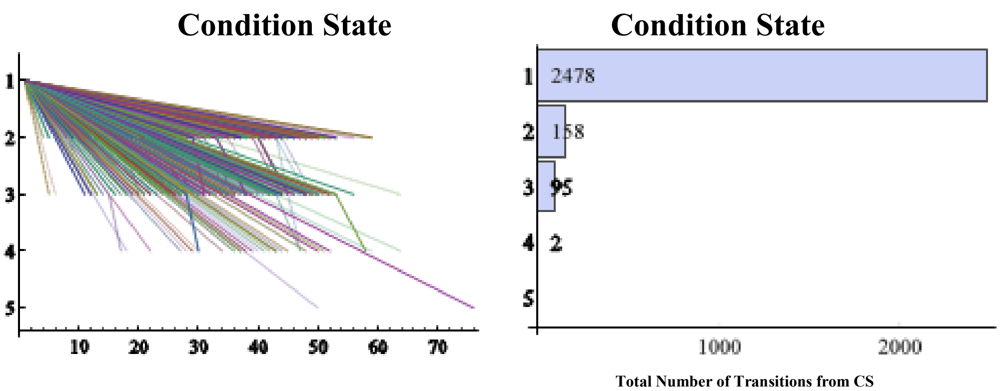

FIGURE 3 Transitions of the CS by age and number of transitions by CS for corrosion for expansion joints with average influence.

Issues of Using Raw Data for Calibration

In order to obtain the deterioration functions, the number of transition from a CS is set into relationship with the number of observations staying in a CS or switching to the next-worse CS. The deterioration is therefore governed by the number of observed transitions into the next CS. If raw data would be directly used for calibration, the condition development as presented in the following figures would result.

The figures are to be read as displayed in Figure 4 on the left-hand side: after 40 years, for instance 42% will be in CS1, 30% will be in CS2, 24% will be in CS3, and 30% will be in CS4; this also means that after 80 years, 27% are in CS3 or worse. Furthermore, according to Figure 4 and Figure 5 almost no or no deterioration is observed to CS5.

The raw data can be used directly for the calibration of KUBA's management system. It can be argued that the data corresponds to the practice and thus the results for the commonly applied maintenance strategies are correct.

Transition probability at t+5years

$$
\begin{array}{c|c}
 & \begin{array}{ccccc} CS1 & CS2 & CS3 & CS4 & CS5 \end{array} \\
\begin{array}{c} CS1 \\ CS2 \\ CS3 \\ CS4 \\ CS5 \end{array} &
\begin{bmatrix}
0.947 & 0.053 & - & - & - \\
- & 0.926 & 0.074 & - & - \\
- & - & 0.968 & 0.032 & - \\
- & - & - & 0.986 & 0.014 \\
- & - & - & - & 1.000
\end{bmatrix}
\end{array}
$$

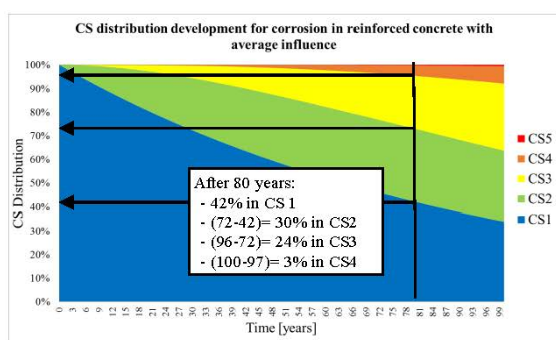

FIGURE 4 Matrix and graph of the development of the CS distribution for corrosion in reinforced concrete with average influence using raw data.

Transition probability at t+5years

$$
\begin{array}{c}
\\
CS1 \\ CS2 \\ CS3 \\ CS4 \\ CS5
\end{array}
\begin{array}{c}
\begin{array}{ccccc} CS1 & CS2 & CS3 & CS4 & CS5 \end{array} \\
\begin{bmatrix}
0.927 & 0.073 & - & - & - \\
- & 0.848 & 0.152 & - & - \\
- & - & 0.946 & 0.054 & - \\
- & - & - & 0.993 & 0.007 \\
- & - & - & - & 1.000
\end{bmatrix}
\end{array}
$$

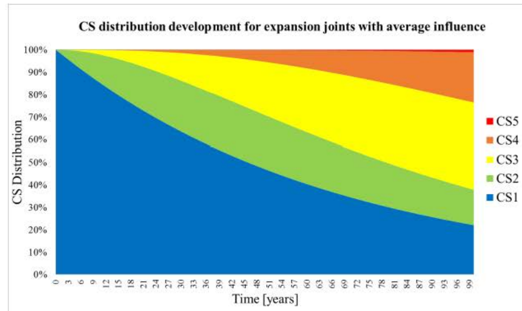

FIGURE 5 Matrix and graph of the development of the CS distribution for deterioration of expansion joints with average influence using raw data

The calibration results present the cause for concern that the deterioration speed—even from CS1—appears to be very slow. It can be possible that individuals perceive the lifetime expectation shorter than the actual one since they mostly have to deal with damaged elements. Nevertheless, the discrepancy between the commonly expected and the calibrated lifetime expectation is huge. The common lifetime expectation of expansion joints is for example 10 to 40 years (*3*). According to the calibration after 40 years only 20% of the expansion joints are in a CS that is worse or equal to 3; and even after 100 years this percentage raises only to 40%.

A reason for the slow deterioration is that the CS (normally CS3) immediately before performing interventions is not stored. Since the calibration algorithm is based on stored data it will yield slow deterioration from CS2. According to Figure 2 and 3, the same explanation could be given for the transitions from CS1. Nevertheless, it's not likely that interventions are already performed in CS2. Plausible explanations could be that either the calibration algorithms don't deliver correct results or CS assessments weren't stored. Since the calibration algorithms were tested and gave robust results, it is assumed that almost no assessments were performed before interventions. This could have been the case since in 2008 the ownership of the road infrastructure was transferred from the Swiss Cantons to the Swiss Confederation. In order to verify this, a sample of elements with apparently long lifetimes ought to be analyzed.

As described, for the application in the management system, the transition matrices of the Markov chains were initially estimated by the pool of experts. As time goes by, the matrices are updated based on inspection results. The analysis shows that the estimation by the pool of experts is still necessary in order to model a realistic deterioration.

COLLECTION OF DATA ON PERFORMED MAINTENANCE INTERVENTIONS

Issues for Data Collection Because of the Organization

The spilt in responsibilities between the asset management and construction management seems to be a main obstacle to obtain the needed data on performed maintenance interventions.

The idea behind the split is to separate the client (which is played by the asset management) and the contractor (which is played by the construction management). The main issue related to this split is that the asset management and the construction management stand below the same managerial authority. By consequence, the asset management has no direct managerial authority towards the construction management.

Furthermore, the budgets, i.e., the resources of the asset management, are by far lower than those of the construction management. Due to this circumstance, the organizational weight of the construction management is larger than the one of the asset management. The concerns of the asset management tend to be treated with lower importance.

The construction management sets the priority in the tasks of design, building supervision, and partially to as-built documentation. Since the construction management doesn't perceive an advantage in the data collection, its priority is low.

Additionally, FEDRO's policy is to contract-out most of the task. The asset management contracts out on-site monitoring like inspections to regional units (Cantons) or to private consultants. Generally, the supervision of inspections is also contracted out to third parties. The construction management contracts-out the design and the on-site building supervision as well as the supervision of these tasks to private consultants. In order to achieve good results, this kind of policy requires a precise specifications and strict controlling of the task execution.

Finally, FEDRO's organization is relatively young. The ownership of the road infrastructure was transferred from the Swiss Cantons to the Swiss Confederation January 1, 2008. Since then, FEDRO is responsible for the strategic and operational task related to high-volume road infrastructure. The organization is well set but—considering the service life of road structure—has relatively modest experience. Furthermore, the standards and the awareness for the importance of data collection aren't completely established yet.

The described issues and their combination lead to a lack of data on performed maintenance interventions.

Possible Ways to Overcome the Organizational Issues

In order to overcome the organizational issues, the following solutions are possible.

- Give the asset management direct managerial authority over the construction management. By doing this, the asset management could enforce that its requirements related to the data on performed maintenance interventions are fulfilled. This would be a major organizational change with manifold consequences and would have to be examined in detail.
- Give the asset management the competence for acceptance of work and release funds for the as-built documentation of performed maintenance interventions. This would give the asset management managerial authority to enforce the collection of the needed data. Additionally, it would be needed that the persons involved in the documentation know from the very beginning about the requirements and collect the needed data at the right moment. Otherwise it wouldn't be assured that when the as-built documentation is approved it fulfills the requirements. If the needed data is stored months after the maintenance intervention was performed, its quality is likely to be too low.
- Raise awareness from the construction management of the need for data on maintenance interventions and related advantages. The statistical analysis of performed maintenance interventions yields the unit costs of different interventions on element types that can be very useful for construction management to improve bid evaluations. Furthermore, making the construction management aware that the data on performed maintenance interventions has influence on the financial need calculated by the management system and the

future funds, which will be available, could raise overall awareness. Nevertheless, this solution relies heavily on the insight that this data need to be collected.

Technical Issues of Data Collection

Within KUBA, the calibration of maintenance intervention costs is done in two steps. In the first step, the unit costs of typical intervention were determined by collecting experts' opinions. In the second step, KUBA automatically calibrates the unit costs based on the stored intervention costs.

Consequently, the more performed interventions are stored in KUBA the more reliable unit costs in KUBA can be expected. The issue with this very promising approach is that the breakdown of cost in practice is different from the cost breakdown in KUBA. In KUBA element unit costs are needed and these costs have to be stored, but in practice—during project realization—the costs are related to the type of work.

To overcome this issue, a research project was carried out and will soon be published. A main result of the research project is that there is no way to bypass collection of element costs during project realization. In order to obtain these costs, the contractor has to track them and the awarding authority has to pay for it (*4*).

ORGANIZATION OF THE DATA COLLECTION

FEDRO contracts out the inspections and spends yearly about US$3 million for it. Following internal guidelines, FEDRO has to commission the work based on the workload. Besides other quality requirements, FEDRO has to control the workload that is declared by the private consultants. The difficulty of controlling the workload is that each road structure is requiring different workload and no detailed quantitative data exists by which the workload can be forecasted: a bridge with 3,000 m of length in bad condition will require considerably more workload compared to a common bridge with a length of 20 m, which is almost new. Since the road structures vary considerably from network section to network section, it is not or just very roughly possible to assess the workload. In Figure 6 the average workload is presented for different network sections.

In order to improve the planning and controlling of the workload, the private consultants were required to file—in addition to the usual timesheet data like the name of the person, date, work time, work item, etc.—the ID of road structure related to the work item. The IDs of the road structures are unique and these are stored in KUBA. This allows one to link all data stored in KUBA (e.g., the length of the road structure, the number of elements, the condition state of the structure, the number of observed damages) to the workload and to perform a correlation analysis. For instance, one can determine which parameters govern the workload. By analyzing different parameters, one can determine the ones with low scatter as ones that are likely to govern workload. In the first step the workload is plotted as a function of the parameter, which is analyzed and, based on this, meaningful cohorts are built. In the second step the scatter is analyzed by analyzing box-plot diagrams in order to determine which cohorts or parameters have to be analyzed in more detail. A low scatter characterizes itself by low difference between the upper and lower quantile, in which the lower quantile corresponds to 20% of the values and the upper quantile corresponds to 80% of the values. The lower whisker is set to 5% quantile and the

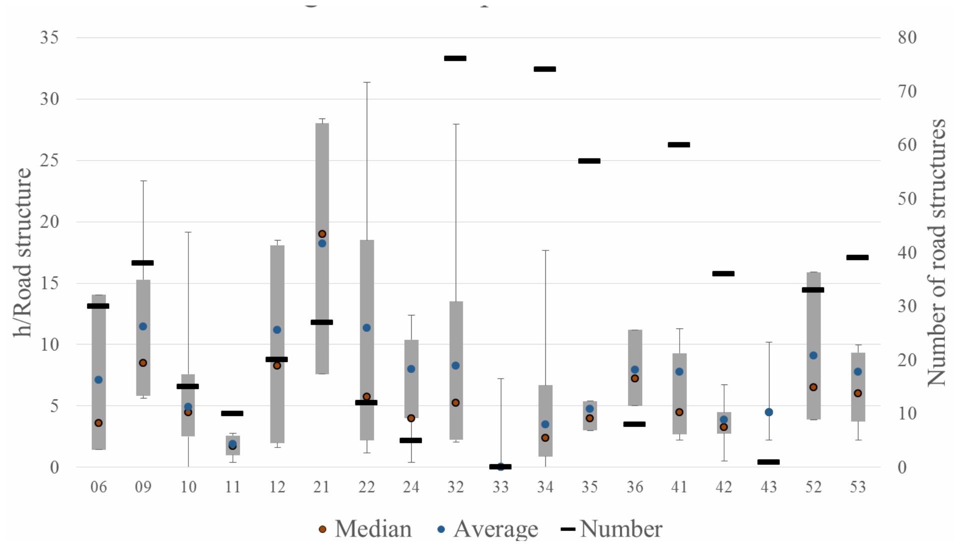

FIGURE 6 Average workload for inspections per network section.

upper to 95% quantile. A quasi normal distribution is characterized by little difference between the values for average and median so as a symmetrical boxplot.

In a third and final step the scatters of the different cohorts are compared between each other and discussed.

In the following paragraphs the results of the analysis are presented.

Analysis of Scatter and Distribution of the Workload in Function of Different Properties

Analysis of the Influence of Structure Types

In a first step the scatter and the distribution of the average workload per road structure type was analyzed.

As can be seen in the upper part of Figure 7, the scatter for noise protections, retaining walls, and sewerages is low and the workload is quasi-normally distributed. For culverts this is less the case but the values are still acceptable. For bridges, in contrast, the scatter is big and the workload isn't normal-distributed. Consequently, the bridges are analyzed in more detail in order to reduce the scatter and achieve a distribution which approaches the normal distribution.

Analysis of the Influence of Length, Deck Area, Number of Elements, Number of "Newly" Collected Damages and Condition States

In the second step, cohorts were defined for bridges as a function of the length, deck area, number of elements, number of "newly" collected damages and condition states. This was done in order to determine which properties govern the workload.

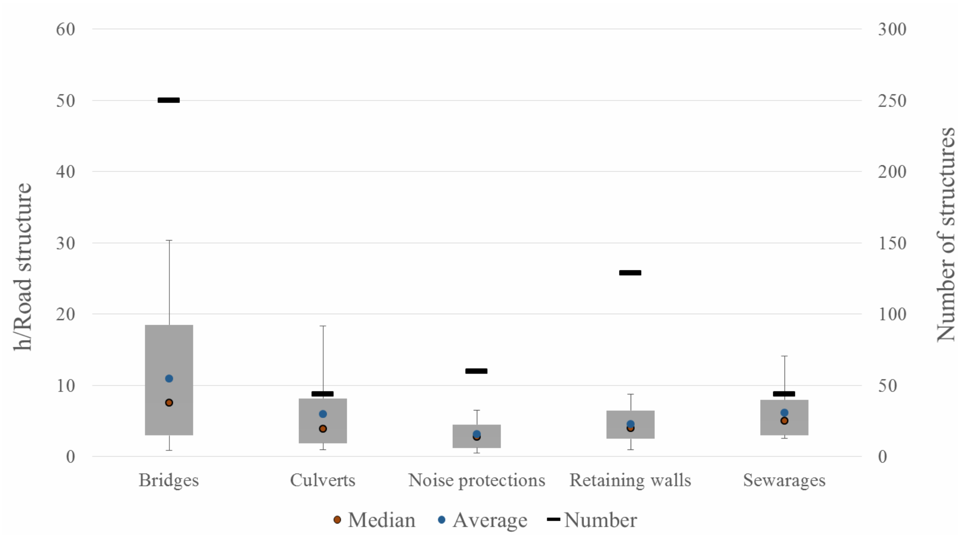

FIGURE 7 Average workload per road structure type.

Based on the average workload per bridge length, obvious cohorts couldn't be the defined. For the purpose of representation, cohorts were defined for bridges with 0 to 80 m, bridges with 80 to 340 m, and bridges with 340 to 3,155 m. The same analysis was also done for the deck area of bridges.

Following the same procedure, i.e., by analyzing the average workload over the number of elements, cohorts were defined for bridges with 20 or less elements, bridges with more than 20 and equal or less than 50 elements, and more than 50 elements.

The comparison of the box-plox from Figure 8 shows that the scatter for bridges with 20 or less elements is low and the workload is quasi-normally distributed. For bridges with more than 20 and equal or less than 50 elements this is less the case but the values are still acceptable. For bridges with more than 50 elements the scatter is significant.

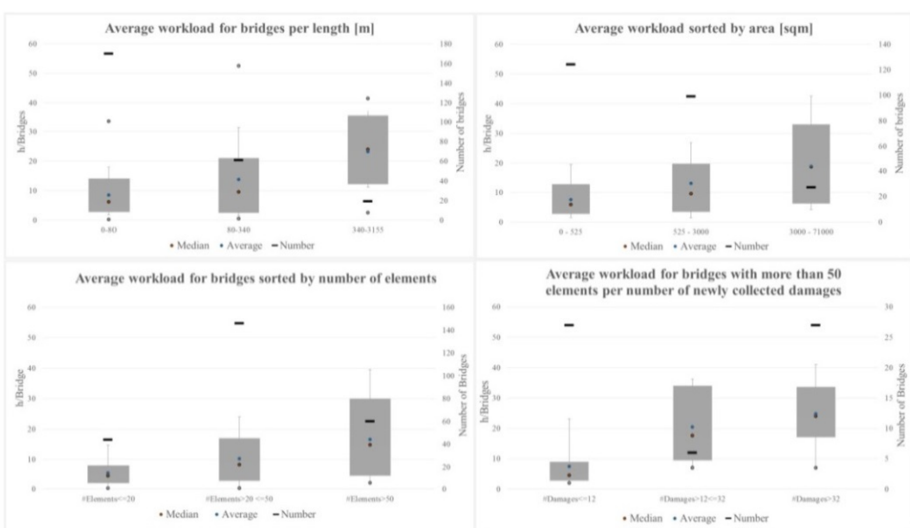

FIGURE 8 Average workload for bridges.

In order to reduce scatter for bridges with more than 50 elements the number of newly collected damages, respectively damages which were stored for the first time during the inspection were analyzed. Based on this, cohorts were defined for bridges with 12 or less damages, bridges with more than 12 and equal to or less than 32 damages, and more than 32 damages.

As can be seen in Figure 8 the scatter of bridges with more than 50 elements can be reduced significantly by differentiating by number of damages. The scatter for bridges with more than 12 and equal or less than 32 damages is still quite big, but it has to be considered that the number of considered bridges is low.

The consideration of newly collected damages was analyzed in the last step since it can just be for controlling of the performed workload but not for the workload forecast. Since it seems that the number of newly collected damages significantly influence the workload, the workload per number of damages and per condition state is analyzed. Based on this analysis, cohorts for bridges with 12 or less damages, bridges with more than 12 and equal to or less than 32 damages, and more than 32 damages are built. Since the number of damages is related to the CS, the box plot diagrams were also plotted for the CS.

As can be seen in Figure 9, the scatter of workload for bridges can be reduced significantly by differentiating by number of damages and especially by differentiating over CS. Especially the CS is a good parameter for benchmarking in order to control the declared amount of work.

Based on the analysis the average workload for the inspection and its scatter is summarized in Table 1.

CONCLUSION

Given that the paper addresses three distinct topics related to the deployment of RSMS KUBA within FEDRO, the conclusions are also threefold.

The first topic addresses the estimation of the deterioration functions with calibration algorithms. It can be concluded that even with the calibration algorithms that are able to bridge data voids (*2*), the problem of lack of almost all data in bad condition states cannot be overcome. Since FEDRO seldom allows structures and elements to deteriorate into the worst two CSs, the

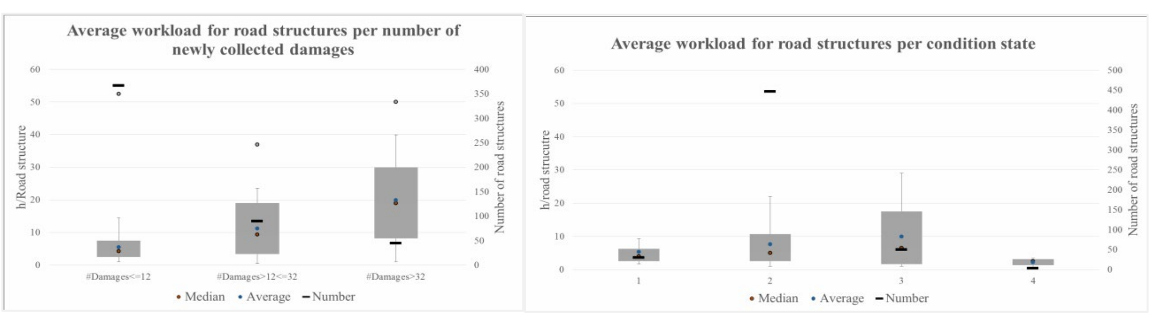

FIGURE 9 Average workload for road structures per number of newly collected damages and average workload for road structures per CS.

TABLE 1 Average Workload for Inspection and Its Scatter

	Average	Δ	Median	Lower Quantile (20% of values)	Δ	Upper Quantile (80% of values)	Number of Considered Structures
Type of Road Structure							
Noise protections	3.1	0.3	2.8	1.3	3.2	4.5	60
Retaining walls	4.5	0.5	4	2.5	4	6.5	129
Sewerages	6.1	1.1	5	3	5	8	44
Culverts	5.9	2	3.9	1.9	6.3	8.2	44
Bridges	10.9	3.4	7.5	2.95	15.55	18.5	250
Number of Bridge Elements							
≤20	5.3	0.9	4.4	2	6	8	44
>20 and ≤50	10.3	2	8.3	2.8	14.2	17	146
>50	16.6	1.8	14.8	4.4	25.6	30	60
Number of Newly Collected Damages for Bridges							
≤12	7.3	1.5	5.8	2.8	7	9.8	133
>12 and ≤32	12.1	0.2	11.9	3	16.5	19.5	72
>32	19.9	1.1	18.8	7.9	22.1	30	44
Number of Newly Collected Damages for Bridges with > 50 Elements							
≤12	7.4	2.9	4.5	2.75	6.25	9	27
>12 and ≤32	20.5	2.9	17.6	9.5	24.5	34	6
>32	24.8	0.8	24	17.1	16.6	33.7	27
Condition States for Bridges							
CS1	6.9	2.9	4	2.5	3.8	6.3	11
CS2	10.9	3.4	7.5	3	15	18	212
CS3	13.2	0.6	13.8	1	20	21	26
Condition States for Bridges with > 50 Elements							
CS1	7.9	1.9	6	3.2	3	6.2	7
CS2	34	17	17	4.4	26.7	31.1	50
CS3	33.7	12.7	21	15.8	14.6	30.4	7
Number of Newly Collected Damages for Road Structures							
≤12	5.5	1.2	4.3	2.5	5	7.5	367
>12 and ≤32	11.2	1.8	9.4	3.4	15.6	19	90
>32	19.9	0.9	19	8.2	21.8	30	45
Condition States for Road Structures							
CS1	5.3	1.3	4	2.5	3.8	6.3	30
CS2	7.6	2.6	5	2.5	8.2	10.7	447
CS3	9.9	3.4	6.5	1.6	15.9	17.5	50
CS4	2.2	0.3	2.5	1.3	1.8	3.1	3

deterioration function from these worst two condition states rely on the estimates of a pool of experts. The failure to collect condition data immediately before performing intervention is an additional drawback to reliably estimate deterioration functions. The awareness that these data has to be collected needs to be reinforced.

The second topic addresses the organization focusing on the split in responsibilities between the asset and the construction management. This split seems to be an obstacle to obtain valuable data that can be used for planning purposes. The proposed solution of giving to the asset management the competence for acceptance of a performed intervention and for funds release for the as-built documentation of performed maintenance interventions would directly address the issue and thus be an effective measure. Furthermore, additional research projects should be conducted in order to overcome the issues related to the difference between the breakdowns of cost in construction practice from the one in asset management.

The third topic addresses the collection of inspection data that is well-established and proceeds smoothly. As the workload for inspections varies considerably, the performed inspections are analyzed to determine the parameters that influence the workload. Based on this analysis, the type of road structure, the number of newly collected damages, and especially the CS seem to govern the workload. For bridges, the newly collected damages clearly govern the workload. Since the information about the CS and the newly collected damages isn't available à priori, it just can be used for controlling the registered workload and not for planning purposes. For planning purposes, the number of elements can be used, but they don't provide accurate values for large bridges. It is clear that the workload heavily depends on the data which is collected during inspections. It is therefore important to be aware of the workload related to the data which is required to be collected. On the other hand, the improvements in the processes and tools are more effective if done for data on damages. Another possibility for analyzing the data would be to perform regression analysis and plot the regression graphs. Regression graphs have the advantage to give a better overview of the scatter. On the other hand, regression graph doesn't show so clearly cohorts and their distribution. Since a goal of the analysis also was to provide benchmarks, the representation with box-plots was chosen.

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Migration Probability Matrix for Bridge Element Deterioration Models

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Transportation agencies are required in 23 USC 119(e) to develop Transportation Asset Management Plans featuring life-cycle cost analysis for bridges. Final rules in 23 CFR 515.17 elaborates that this analysis and related bridge management systems must contain deterioration models. Element-level bridge inspection data suitable for deterioration models have been collected by most state department of transportation (DOTs) since the mid-1990s, but in 2013 AASHTO significantly modified the inspection process. FHWA has proposed adding the modified inspection language to the National Bridge Inventory in compliance with element inspection requirements in 23 USC 144(b). This presents a serious problem for all DOTs because none yet have sufficient element inspection data under the 2013 AASHTO manual to support deterioration modeling.

Research completed in 2016 for the Florida DOT suggests one readily implementable solution to this problem. A migration probability matrix was developed to encapsulate the differences in definitions between Florida's bridge element inspection data gathered under AASHTO's 1998 *Guide to Commonly Recognized Structural Elements*, and the new 2016 Florida DOT inspection manual, which is based on the 2013 AASHTO *Manual for Bridge Element Inspection*. Deterioration models previously developed using the older data can be easily multiplied by this migration probability matrix to develop reasonable models that are compatible with inspection data gathered under the new manual. Ultimately this migration matrix can be validated and improved once sufficient element inspection data are gathered under the new manual.

The Florida Department of Transportation (DOT) began gathering bridge element condition data as part of its routine biennial inspections in the late-1990s, implementing the AASHTO *Guide to Commonly Recognized Structural Elements* (CoRe) (*1*) for use with its AASHTOWare Pontis bridge management system. Over time Florida DOT augmented its bridge inspection process to incorporate the specialized elements of movable bridges, to add elements that are of particular maintenance concern in Florida (such as pile jackets, drainage systems, fenders, dolphins, and seawalls), and to add nonbridge structures such as sign supports, high-mast light poles, mast arms, and certain retaining walls (*2*).

Using its element inspection standards, Florida DOT conducted innovative research to develop its own deterioration models, action cost and effectiveness models, and decision-support tools for life-cycle cost analysis and risk analysis at the project and program levels. In particular, Florida DOT has a statistically rigorous bridge deterioration model that it uses for many purposes in planning of bridge work (*3*).

As many states gained experience with the element inspection process, a number of potential improvements were identified. Among them include the following:

- A more-precise definition of the specific types of defects that are considered in condition state assessments;

- Separate assessment of certain types of protective systems from their underlying elements, especially deck wearing surfaces, coating systems, and cathodic protection systems; and
- Standardization of the number of condition states possible for each element.

In response to these issues, a new AASHTO *Manual for Bridge Element Inspection* was published in 2013 (*4*). Florida DOT prepared its own version of this manual, containing its agency-defined elements, in 2016 (*5*). Bridge inspections in accordance with the new manual commenced that same year. Florida DOT needed to adapt its decision support models to the new standards in order to use them with the new inspection data in AASHTO's new software.

Legacy Deterioration Model

Florida DOT developed its bridge element deterioration model using a statistical analysis of 14 years of CoRe Element data completed in 2011 (*3*). A database of 884,678 element inspection records was filtered for various error conditions, and then self-joined using database analysis tools to produce 614,699 inspection pairs. Each pair consisted of two inspections spaced approximately 2 years apart on the same element of a bridge.

Since the desired model was intended to predict the outcome of a do-nothing scenario, a related database of maintenance activity, combined with a set of heuristics, was used to remove 55,388 inspection pairs where agency preservation or improvement activity may have occurred.

In the 2011 research, a novel algebraic method was used in order to compute an estimate of the probability that a given element will transition from a given condition state to the next-worse state in a 1-year period, based on the filtered inspection pairs. This estimate formed the basis for a Markovian model of bridge deterioration compatible with AASHTO's Pontis bridge management system and with Florida's Excel-based Project-Level Analysis Tool (PLAT). The same research then proceeded to develop a Weibull model of the onset of deterioration, which was incorporated into PLAT and was designed to be compatible with AASHTO's upcoming Bridge Management software, the successor to Pontis.

Because of small populations for many of the elements, the researchers grouped the required 755 element models into 72 element groups for which there was a sufficient population of inspection pairs to produce a statistically reliable model. Within each element group, the grouped elements were similar in the number of defined condition states, units of measure, material, and expected deterioration behavior. The condition states defined for elements within each element group generally had identical or very similar definitions according to the Florida DOT inspection manual.

The models produced by this analysis exhibited strong statistical performance, with an average R^2 value of 0.73. It was found that, as a rule-of-thumb, a population of 500 or more inspection pairs was sufficient to produce a robust model for a given element or element group.

Potential for a New Model

Given the substantial differences between the 1998 and 2016 inspection manuals, including the change in the number of defined condition states, it would not be possible to use the 2011 models directly with future inspection data gathered according to the 2016 manual. Somehow, a new model would need to be developed that would be compatible with new inspection data going forward.

Markovian models such as those used in Pontis and the new AASHTOWare Bridge Management software are cross sectional, so they do not require a long time series of inspection data (*6*). However, the estimation process does require a database of inspection pairs. The two inspections in each pair must have the same condition state definitions in order for the model to be applicable to future inspections that comply with the same definitions. Such models could be developed using the same methods as in the 2011 research, after two complete cycles of inspections under the new definitions are completed, which would take 4 years. However, the less common element groups may require more than two cycles in order to amass the needed population of 500 inspection pairs. This fact would leave Florida DOT without a rigorous deterioration model for a significant length of time.

As an interim measure, one alternative would be to develop a new deterioration model based on expert judgment, as many of the states did in the early years of using Pontis (*7*). Florida DOT had conducted such an analysis in 2001, and attempted to validate the results as part of its 2011 research. Unfortunately, the expert judgment predictions of deterioration rates substantially overstated the deterioration probabilities and underestimated transition times by an average factor of 1.97 (*3*). For example, if the expert panel had judged the median transition time from state 1 to state 2 of a substructure element to be 20 years, the actual inspection data showed that this median transition time turned out to be 40 years.

The magnitude of the error in expert judgment estimates of transition times ranged from 1.6 for railings to 3.3 for deck slabs. The reasons for the error are speculative based on psychological research, and it is not clear to what degree future expert elicitation methods would be able to correct the discrepancy.

It was clear from the 2011 research that the model based on statistical analysis of inspection data was much more reliable than the one based on expert judgment, so Florida DOT wanted to preserve the statistical model if possible. This meant finding a way to migrate the model to make it compatible with the 2016 inspection manual.

The AASHTO Visual Element Migrator

In order to assist agencies in making the transition to the new element inspection process, AASHTO developed a software program called the Visual Element Migrator, distributed with its Bridge Management software. The Migrator operates on a database of CoRe Element inspections from Pontis, and attempts to convert them into a form compatible with the 2013 Element Manual for use in AASHTOWare Bridge Management. Using the Migrator software, an engineer can design a script to specify how new elements are to be created, based on old elements and other characteristics of each bridge. The Migrator also attempts to translate CoRe Element condition states so they are compatible with the new manual.

The migration process is necessarily inexact, because the Pontis database does not contain enough information to identify the new elements and condition states precisely. Analysis of the preliminary Migrator output for Florida bridges showed that the program was not able to identify 27% of the new element groups defined in the 2016 Florida DOT manual and was not able to populate 43% of the condition states. This was after Florida DOT engineers had made an initial attempt to customize the script to incorporate the Florida DOT agency-defined elements.

As an example of elements that were not identified, the Migrator was unable to determine whether a bridge deck was reinforced concrete or prestressed, nor whether it consisted of the top flange of girders or was a separate slab component. As an example of unidentified condition states, the Migrator had no basis for populating four condition states of expansion joints when only three states are provided in the Pontis data. These are not necessarily shortcomings of the Migrator software, but are merely a consequence of the fact that the new inspection process is more detailed than the old one.

When an element's condition states are redefined, or the number of states is changed, there is, in principle, a probabilistic relationship between the old states and the new ones. For example, if a large group of expansion joints are inspected under both the old and new systems, all four of the new condition states will be populated even though only three states existed under the old system. The AASHTO Migrator program is a deterministic simplification of this transition. However, for the deterioration models it is necessary to approximate more closely the actual correspondence between old and new, since the deterioration models must produce realistic transitions of real inspection data gathered under the new definitions.

One way this correspondence could be modeled would be a statistical analysis comparing inspection results of the same set of bridges, at the same time, under both systems of elements. This would require a dual inspection, recording two sets of results under different standards for the same observations. Another similar and somewhat more practical approach is to apply the existing deterioration model to a set of recent CoRe Element inspections and compare the result to actual element inspections, 2 years later, under the new system. Unfortunately for the present study, neither type of data set would be available until 2018 or later.

Overview of Approach

The selected approach was based on expert judgment, informed by the substantive definitions of elements and condition states in the old and new manuals. The input and output of the AASHTO Visual Migrator program were summarized to assist in this process.

To make the application of judgment feasible and consistent, a common denominator was developed to aggregate both manuals into a set of element groups, based on the same 72 element groups used in the 2011 Florida DOT research to develop deterioration models. The elements within each element group have the same number of condition states with the same or very similar definitions, and are expected to deteriorate at similar rates.

Using expert judgment, a migration probability matrix was developed for each element group, to relate the three to five condition states of the CoRe Elements to the uniform four condition states of the new AASHTO manual. This was based primarily on interpretation of the definitions in the Florida DOT manuals, with assistance from the Migrator data. The Migrator data is assumed to incorporate the previous judgments of the developers of the program and the Florida DOT engineers who customized it. Florida DOT provided a summary of the rationale it intended when configuring the Migrator program.

The migration probability matrix was multiplied by the vector of deterioration transition times developed for each element group in the earlier research. This process allocated segments of the lifespan of each element, previously associated with the old CoRe Element definitions, to the new condition states for future use. The result was that all four condition states of each element group were given reasonable estimates of their transition times.

DATA PREPARATION

Although the number of element definitions is nearly unchanged between the old and new Florida DOT element inspection manuals, there are significant differences. In addition to the differing classification of bridge decks noted earlier, there is the separation of wearing surfaces, coatings, and cathodic protection from substrate elements, and the addition of more prestressed, masonry, and "other material" elements. It should also be noted that only 151 elements were assigned deterioration models in the 2011 deterioration research. The other elements either did not occur in the Florida DOT inventory, or had only recently been inspected for the first time (for example, traffic signal mast arms).

The element grouping system developed in the 2011 research was equally useful for the present research, since it assured that the migration of the deterioration model would be reasonably concise and consistent. Each of the 168 new element definitions was assigned to the same element groups developed in the earlier study. The result at the element level is shown in Table 1, and the listing of groups is shown in Table 2.

Some of the groups were defined by the existence of protective systems. For example, the old inspection manual had an element for uncoated metal railings (4 condition states) and a separate element for coated metal railings (5 states). As a result, some of the groups did not have any corresponding elements in the new definitions, where coatings are inspected separately. In general the choice of group was based on interpretation of condition state definitions and examination of the deterioration model, to determine which one would be most applicable based on professional judgment.

Some of the new element definitions, such as mast arms, were not addressed in the 2011 research. These were handled by assigning them to the group whose deterioration model was judged to be most applicable. Protective system elements were associated with new groups of their own, but inherited deterioration models from the old CoRe elements which were judged to be most applicable:

- Deck wearing surfaces were based on a weighted average of concrete decks and concrete slabs, considering that both groups were influenced by the condition of asphalt concrete wearing surfaces.
- Paint on steel and stain on concrete were based on the model for painted steel girders and floor beams.
- Weathering steel patina was based on the model for unpainted steel superstructures and substructures.
- Galvanized and metallized coatings were based on the model for metal culverts.
- Reinforcing steel protective systems (such as cathodic protection) were based on the model for pile jackets with cathodic protection.

ANALYSIS AND RESULTS

The remaining analysis work was performed at the level of element groups as in Table 2. Since the new inspection manual has only 74 element groups, it is feasible to elicit expert judgment to generate the migration probability matrix. Many of the groups pertain to Florida custom elements and might not be included in the inspection process in other states.

TABLE 1 Grouping of Elements from the 2016 Florida DOT Manual (see Table 2 for the names of the groups)

Group	Element	Group	Element
A1	12 reinforced concrete deck	F1	202 steel column
A1	13 prestressed concrete deck	F3	203 other column
A1	15 prestressed concrete top flange	F2	204 prestressed concrete column
A1	16 reinforced concrete top flange	F3	205 reinforced concrete column
A4	28 steel deck - open grid	F8	206 timber column or pile extension
A4	29 steel deck - concrete fill grid	F1	207 steel tower
A4	30 steel deck - orthotropic	F8	208 timber trestle
A5	31 timber deck	F3	210 reinforced concrete pier wall
A2	38 reinforced concrete slab	F3	211 other pier wall
A5	54 timber slab	F8	212 timber pier wall
A1	60 other deck	F3	213 masonry pier wall
A2	65 other slab	F5	215 reinforced concrete abutment
D2	102 steel closed box girder	F8	216 timber abutment
D6	104 prestressed closed box girder	F5	217 masonry abutment
D7	105 reinforced closed box girder	F5	218 other abutments
D1	106 other closed web/box girder	F1	219 steel abutment
D2	107 steel open girder/beam	F7	220 reinforced concrete pile cap/footing
D6	109 prestressed open girder/beam	F1	225 steel pile
D7	110 reinforced open girder/beam	F2	226 prestressed concrete pile
D8	111 timber open girder	F3	227 reinforced concrete pile
D1	112 other open girder/beam	F8	228 timber pile
D3	113 steel stringer	F3	229 other pile
D6	115 prestressed concrete stringer	F1	231 steel pier cap
D7	116 reinforced concrete stringer	F2	233 prestressed concrete pier cap
D8	117 timber stringer	F6	234 reinforced concrete pier cap
D1	118 other stringer	F8	235 timber pier cap
D5	120 steel truss	F6	236 other pier cap
D8	135 timber truss	G2	240 steel culvert
D1	136 other truss	G1	241 reinforced concrete culvert
D5	141 steel arch	G2	242 timber culvert
D1	142 other arch	G2	243 other culvert
D6	143 prestressed concrete arch	G1	244 masonry culvert
D7	144 reinforced concrete arch	G1	245 prestressed concrete culvert
D7	145 masonry arch		
D8	146 timber arch		
D2	147 steel main cables		
D2	148 secondary steel cables		
D1	149 other secondary cable		
D2	152 steel floor beam		
D6	154 prestressed floor beam		
D7	155 reinforced concrete floor beam		
D8	156 timber floor beam		
D1	157 other floor beam		
D2	161 steel pin or pin/hanger		
D2	162 steel gusset plate		

Continued on next page.

TABLE 1 (*continued*) Grouping of Elements from the 2016 Florida DOT Manual (See Table 2 for the names of the groups)

Group	Element	Group	Element
B1	300 strip seal expansion joint	K1	8480 mast arm foundation
B2	301 pourable joint seal	K1	8481 vertical mast arm member, metal
B3	302 compression joint seal	K1	8483 vertical mast arm member, concrete
B4	303 assembly joint with seal	K1	8484 horizontal mast arm member, metal
B5	304 open expansion joint member, metal	K1	8487 overlane sign struct horiz
B5	305 assembly joint without seal	K1	8488 overlane sign struct vert member metal
B6	306 other joint	K1	8489 overlane sign structure foundation
E1	310 elastomeric bearing	K1	8491 rc overlane sign vertical
E2	311 moveable bearing	K1	8496 high mast light poles metal coated
E2	312 enclosed bearing	K1	8499 high mast light pole foundations
E2	313 fixed bearing	P2	8516 painted steel
E2	314 pot bearing	P3	8517 weathering steel
E2	315 disk bearing	P4	8518 galvanized steel
E2	316 other bearing	P4	8519 other steel coating
A6	320 prestressed concrete approach slab	L1	8540 open gearing
A6	321 reinforced concrete approach slab	L1	8541 speed reducers
C2	330 metal bridge railing	L1	8542 shafts
C3	331 reinforced concrete bridge railing	L1	8543 shaft bearings and shaft couplings
C4	332 timber bridge railing	L2	8544 brakes
C5	333 other bridge railing	L3	8545 emergency drive and back up power system
C3	334 masonry bridge railing	L3	8546 span drive motors
P1	510 wearing surfaces	L4	8547 hydraulic power units
P5	520 concrete rebar protective system	L5	8548 hydraulic piping system
P2	521 concrete protective coating	L4	8549 hydraulic cylinders/motors/rotary actuators
A3	8097 ps/rc hybrid slab	L6	8550 Hopkins frame
A2	8098 concrete deck on pc panel	L7	8560 span locks/toe locks/heel stops/tail locks
A3	8099 sonovoid	L8	8561 live load shoes/strike plates/buffer cylinders
D6	8199 duct	L6	8562 counterweight support
F2	8207 hollow core pile	L6	8563 access ladder and platforms
H1	8290 channel	L9	8564 counterweight
I1	8298 pile jacket bare	L9	8565 trunnion/straight and curved track
I3	8386 steel fender/dolphin system	M1	8570 transformers and thyristors
I3	8387 prestressed concrete fender/dolphin	M2	8571 submarine cable

Continued on next page.

TABLE 1 (*continued*) Grouping of Elements from the 2016 Florida DOT Manual (See Table 2 for the names of the groups)

Group	Element	Group	Element
I3	8388 reinforced concrete fender/dolphin	L5	8572 conduit and junction boxes
I3	8389 timber fender/dolphin system	M1	8573 programmable logic controllers
I3	8390 other fender/dolphin system	M3	8574 control console
I3	8393 other material bulkhead/seawall	M4	8580 navigational light system
I4	8394 reinforced concrete slope protection	M5	8581 operator facilities
I5	8395 timber slope protection	M6	8582 lift bridge specific equipment
I6	8396 other slope protection	M6	8583 swing bridge specific equipment
I7	8397 metal drainage system	M7	8590 resistance barriers
I7	8398 other material drainage system	M7	8591 warning gates
J1	8474 metal wall	M8	8592 traffic signal
J2	8475 reinforced concrete wall		
J3	8476 timber wall		
J4	8477 other material wall		
J5	8478 mechanically stabilized earth wall		

Migration Probability Matrix

The linkage between the old deterioration model and the new one was conceived as a migration probability matrix, which is similar to a transition probability matrix. For each condition state under the old definitions, the migration probability matrix contains a probability, in percent, that the same bridge elements would be assessed in each of the new condition states. If the migration probability matrix is well formed, the process should guarantee that all condition states are populated if element inspection data were to be generated using the matrix. However, it is emphasized that the purpose is only to migrate deterioration, action effectiveness, and cost models, not element inspection data.

Table 3 reports the complete migration probability matrix. The four major sets of columns are the four new condition states defined for each new element in the 2016 inspection manual. Within each set are five columns, representing the up to five condition states of old CoRe elements from the 1998 manual. Some of the element groups have fewer than five old states, in which case any excess states show 100% probability of transition to new state 4. If the definition of a new condition state is found equivalent to a corresponding old state, then a 100% transition probability is shown. Otherwise, a probability of less than 100% is assigned, and the remainder is assigned to one or more other new condition states. The sum of each row is 500%, indicating that all five possible CoRe condition states are fully assigned to new states. The footnotes at the end of the table describe in detail the rationale for each decision.

Seventeen of the element groups, representing just 20% of the element inspections, were able to migrate directly across from the old to new definitions without adjustment. In most other cases the definitions of new element condition states did not exactly match the corresponding old element condition states. Examples of common issues include:

TABLE 2 Element Groups: Names of Element Groups Used in Tables 1 and 3

A1 Concrete deck	I1 Pile jacket w/o cathodic protection
A2 Concrete slab	I2 Pile jacket with cathodic protection
A3 Prestressed concrete slab	I3 Fender/dolphin/bulkhead/seawall
A4 Steel deck	I4 Reinforced concrete slope protection
A5 Timber deck/slab	I5 Timber slope protection
A6 Approach slabs	I6 Other (incl. asphalt) slope protection
B1 Strip Seal expansion joint	I7 Drainage system other materials
B2 Pourable joint seal	I7 Drainage system metal
B3 Compression joint seal	J1 Uncoated metal wall
B4 Assembly joint/seal	J2 Reinforced concrete wall
B5 Open expansion joint	J3 Timber wall
B6 Other expansion joint	J4 Other (incl. masonry) wall
C1 Uncoated metal rail	J5 Mechanically stabilized earth wall
C2 Coated metal rail	K1 Sign structures/hi mast light poles
C3 Reinforced concrete railing	K1 Sign structure/hi mast light poles (coated)
C4 Timber railing	L1 Moveable bridge mechanical
C5 Other railing	L2 Moveable bridge brakes
D1 Unpainted steel super/substructure	L3 Moveable bridge motors
D2 Painted girder/floor beam/cable/pin and hanger	L4 Moveable bridge hydraulic power D3 Painted steel stringer
L5 Moveable bridge pipe and conduit	L6 Moveable bridge structure
D4 Painted steel truss bottom	L7 Moveable bridge locks
D5 Painted steel truss/arch top	L8 Moveable bridge live load items
D6 Prestressed concrete superstructure	L9 Moveable bridge counterweight/trunnion/track D8 Timber superstructure
D7 Reinforced concrete superstructure	M1 Moveable bridge electronics
E1 Elastomeric bearings	M2 Moveable bridge submarine cable
E2 Metal bearings	M3 Moveable bridge control console
F1 Painted steel substructure	M4 Moveable bridge navigational lights
F2 Prestressed column/pile/cap	M5 Moveable bridge operator facilities
F3 Reinforced concrete column/pile	M6 Moveable bridge misc. equipment
F5 Reinforced concrete abutment	M7 Moveable bridge barriers/gates
F6 Reinforced concrete cap	M8 Moveable bridge traffic signals
F7 Pile cap/footing	P1 Deck wearing surface
F8 Timber substructure	P2 Paint on steel or stain on concrete
G1 Reinforced concrete culverts	P3 Weathering steel patina
G2 Metal and other culverts	P4 Galvanized / metalized /other
H1 Channel	P5 Reinforcing steel protective system

TABLE 3 Migration Probability Matrix [In the column headings, P_{xy} is the migration probability (percent) from old condition state *x* to new condition state *y*. See Table 2 for the names of element groups. Footnotes in the Note column follow at the end of the table.]

Group	P11	P21	P31	P41	P51	P12	P22	P32	P42	P52	P13	P23	P33	P43	P53	P14	P24	P34	P44	P54	Note
A1	100	0	0	0	0	0	80	30	0	0	0	20	70	70	0	0	0	0	30	100	1
A2	100	0	0	0	0	0	80	60	20	0	0	20	40	70	50	0	0	0	10	50	1
A3	100	0	0	0	0	0	80	60	20	0	0	20	40	70	50	0	0	0	10	50	1
A4	100	100	0	0	0	0	0	100	0	0	0	0	0	100	0	0	0	0	0	100	2
A5	100	0	0	0	0	0	60	0	0	0	0	40	70	0	0	0	0	30	100	100	3
A6	100	0	0	0	0	0	100	0	0	0	0	0	100	60	0	0	0	0	40	100	4
B1	100	0	0	0	0	0	50	0	0	0	0	50	30	0	0	0	0	70	100	100	5
B2	100	0	0	0	0	0	50	0	0	0	0	50	30	0	0	0	0	70	100	100	5
B3	100	0	0	0	0	0	50	0	0	0	0	50	30	0	0	0	0	70	100	100	5
B4	100	0	0	0	0	0	50	0	0	0	0	50	30	0	0	0	0	70	100	100	5
B5	100	0	0	0	0	0	50	0	0	0	0	50	30	0	0	0	0	70	100	100	5
B6	100	0	0	0	0	0	50	0	0	0	0	50	30	0	0	0	0	70	100	100	5
C1	100	0	0	0	0	0	100	0	0	0	0	0	100	0	0	0	0	0	100	100	6
C2	100	0	0	0	0	0	100	0	0	0	0	0	100	0	0	0	0	0	100	100	6
C3	100	0	0	0	0	0	100	0	0	0	0	0	100	0	0	0	0	0	100	100	6
C4	100	0	0	0	0	0	100	0	0	0	0	0	50	0	0	0	0	50	100	100	7
C5	100	0	0	0	0	0	50	0	0	0	0	50	30	0	0	0	0	70	100	100	5
D1	100	0	0	0	0	0	100	0	0	0	0	0	100	0	0	0	0	0	100	100	6
D2	100	50	0	0	0	0	50	100	50	0	0	0	0	50	0	0	0	0	0	100	8
D3	100	50	0	0	0	0	50	100	50	0	0	0	0	50	0	0	0	0	0	100	8
D4	100	50	0	0	0	0	50	100	50	0	0	0	0	50	0	0	0	0	0	100	8
D5	100	50	0	0	0	0	50	100	50	0	0	0	0	50	0	0	0	0	0	100	8
D6	100	0	0	0	0	0	100	20	0	0	0	0	80	0	0	0	0	0	100	100	8
D7	100	0	0	0	0	0	100	30	0	0	0	0	70	0	0	0	0	0	100	100	9
D8	100	0	0	0	0	0	100	0	0	0	0	0	60	0	0	0	0	40	100	100	10
E1	100	0	0	0	0	0	50	0	0	0	0	50	0	0	0	0	0	100	100	100	12
E2	100	0	0	0	0	0	50	0	0	0	0	50	0	0	0	0	0	100	100	100	13
F1	100	50	0	0	0	0	50	100	50	0	0	0	0	50	0	0	0	0	0	100	13
F2	100	0	0	0	0	0	100	20	0	0	0	0	80	0	0	0	0	0	100	100	8
F3	100	0	0	0	0	0	100	30	0	0	0	0	70	0	0	0	0	0	100	100	11
F5	100	0	0	0	0	0	100	30	0	0	0	0	70	0	0	0	0	0	100	100	10
F6	100	0	0	0	0	0	100	30	0	0	0	0	70	0	0	0	0	0	100	100	10
F7	100	0	0	0	0	0	100	30	0	0	0	0	70	0	0	0	0	0	100	100	10
F8	100	0	0	0	0	0	100	0	0	0	0	0	60	0	0	0	0	40	100	100	10
G1	100	0	0	0	0	0	100	0	0	0	0	0	100	50	0	0	0	0	50	100	12
G2	100	0	0	0	0	0	100	0	0	0	0	0	100	50	0	0	0	0	50	100	14
H1	100	0	0	0	0	0	100	0	0	0	0	0	100	0	0	0	0	0	100	100	14
I1	100	0	0	0	0	0	100	0	0	0	0	0	100	0	0	0	0	0	100	100	6
I2	100	0	0	0	0	0	100	0	0	0	0	0	100	0	0	0	0	0	100	100	15
I3	100	0	0	0	0	0	100	0	0	0	0	0	100	0	0	0	0	0	100	100	15
I4	100	0	0	0	0	0	100	30	0	0	0	0	70	0	0	0	0	0	100	100	6
I5	100	0	0	0	0	0	100	0	0	0	0	0	60	0	0	0	0	40	100	100	10
I6	100	0	0	0	0	0	100	0	0	0	0	0	100	0	0	0	0	0	100	100	12
I7	100	0	0	0	0	0	100	0	0	0	0	0	100	0	0	0	0	0	100	100	6
I7	100	50	0	0	0	0	50	100	50	0	0	0	0	50	0	0	0	0	0	100	6
J1	100	0	0	0	0	0	100	0	0	0	0	0	100	0	0	0	0	0	100	100	8
J2	100	0	0	0	0	0	100	30	0	0	0	0	70	0	0	0	0	0	100	100	6
J3	100	0	0	0	0	0	100	0	0	0	0	0	60	0	0	0	0	40	100	100	10

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TABLE 3 (*continued*) Migration Probability Matrix [In the column headings, P_{xy} is the migration probability (percent) from old condition state *x* to new condition state *y*. See Table 2 for the names of element groups.]

Group	P11	P21	P31	P41	P51	P12	P22	P32	P42	P52	P13	P23	P33	P43	P53	P14	P24	P34	P44	P54	Note
J4	100	0	0	0	0	0	100	0	0	0	0	0	100	0	0	0	0	0	100	100	12
J5	100	0	0	0	0	0	100	0	0	0	0	0	100	50	0	0	0	0	50	100	6
K1	100	0	0	0	0	0	100	0	0	0	0	0	100	0	0	0	0	0	100	100	16
K1	100	50	0	0	0	0	50	100	50	0	0	0	0	50	0	0	0	0	0	100	6
L1	100	0	0	0	0	0	100	0	0	0	0	0	100	0	0	0	0	0	100	100	6
L2	100	0	0	0	0	0	100	0	0	0	0	0	100	0	0	0	0	0	100	100	6
L3	100	0	0	0	0	0	100	0	0	0	0	0	100	0	0	0	0	0	100	100	6
L4	100	0	0	0	0	0	100	0	0	0	0	0	100	0	0	0	0	0	100	100	6
L5	100	0	0	0	0	0	100	0	0	0	0	0	100	50	0	0	0	0	50	100	7
L6	100	50	0	0	0	0	50	100	50	0	0	0	0	50	0	0	0	0	0	100	8
L7	100	0	0	0	0	0	100	0	0	0	0	0	100	0	0	0	0	0	100	100	6
L8	100	0	0	0	0	0	100	0	0	0	0	0	100	50	0	0	0	0	50	100	7
L9	100	0	0	0	0	0	100	0	0	0	0	0	100	0	0	0	0	0	100	100	6
M1	100	0	0	0	0	0	100	0	0	0	0	0	100	50	0	0	0	0	50	100	7
M2	100	0	0	0	0	0	100	0	0	0	0	0	100	50	0	0	0	0	50	100	7
M3	100	0	0	0	0	0	100	0	0	0	0	0	100	50	0	0	0	0	50	100	7
M4	100	0	0	0	0	0	50	0	0	0	0	50	100	0	0	0	0	0	100	100	17
M5	100	0	0	0	0	0	50	0	0	0	0	50	100	0	0	0	0	0	100	100	17
M6	100	0	0	0	0	0	50	0	0	0	0	50	100	0	0	0	0	0	100	100	17
M7	100	0	0	0	0	0	50	0	0	0	0	50	100	0	0	0	0	0	100	100	17
M8	100	0	0	0	0	0	50	0	0	0	0	50	100	0	0	0	0	0	100	100	17
P1	50	20	0	0	0	40	70	50	20	0	10	10	50	60	10	0	0	0	20	90	18
P2	100	0	0	0	0	0	100	0	0	0	0	0	50	0	0	0	0	50	100	100	19
P3	100	0	0	0	0	0	100	50	0	0	0	0	50	0	0	0	0	0	100	100	20
P4	100	0	0	0	0	0	100	50	0	0	0	0	50	0	0	0	0	0	100	100	21
P5	100	0	0	0	0	0	100	0	0	0	0	0	100	0	0	0	0	0	100	100	22

NOTE: These footnotes describe the rationale for the assigned probabilities, based on the definitions of element condition states.

1. Concrete decks and slabs. The definitions have changed from extent-based to severity-based, so there is little correspondence except for state 1. State 4 warrants a structural review, which is seen as less common on bridge decks than the old condition state 5.
2. Steel decks. The old states 1 and 2 both clearly fit within the new state 1, and the old state 5 clearly fits the new state 4. The remaining two states also have a reasonable correspondence with each other.
3. Timber decks and slabs. The old state 4 requires that serviceability be affected, but the new state 4 only warrants structural review, a lower standard. So a portion of the old state 3 is also allowed to be in new state 4. State 1 is a more direct match.
4. Approach slabs. Old condition states 1, 2, and 3 seem to correspond reasonably well with the new ones, though the old language is not very precise. Old state 4 is more permissive than the new one, since it doesn't warrant structural review; so only a portion of old state 4 was assigned to the new state 4, the rest to new state 3.
5. Expansion joints. Condition state 1 is essentially the same in both the old and new language. The severity range of distresses covered by the remaining two CoRe element states appear to be evenly spread over the remaining three new states. New state 4 includes conditions more severe than those described in old state 3.
6. Various elements, mostly Florida DOT custom. Condition state language appears to be equivalent between the old and new elements.
7. Various movable bridge elements. Condition state 1 is essentially the same in both the old and new language. Old condition state 3 contributes to both states 3 and 4 in the new system, and state 2 is roughly unchanged. In these cases there was no basis for splitting the transition time between states 3 and 4, so they were arbitrarily split evenly.

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TABLE 3 (*continued*) Migration Probability Matrix. (In the column headings, P_{xy} is the migration probability (percent) from old condition state *x* to new condition state *y*. See Table 2 for the names of element groups.)

NOTE (*continued*):

8. Most steel elements. Part of old state 2 is included in the new state 1. Old condition state 5 is essentially the same as new condition state 4. The severity range of distresses covered by the remaining three CoRe element states appear to be evenly split between the remaining two new states.
9. Prestressed concrete superstructures. Mostly the old and new condition states are equivalent. One difference is that old state 3 has no deterioration of the prestress system, while new state 3 has section loss in the prestressing (that doesn't warrant review).
10. Various reinforced concrete elements. Mostly the old and new condition states are equivalent. One difference is that old condition state 2 does not allow exposed reinforcing while new state 2 does. Therefore a part of state 3 is moved to state 2.
11. Prestressed substructure elements. Mostly the old and new condition states are equivalent. One difference is that old state 3 has no deterioration of the prestress system, while new state 3 has section loss in the prestressing (that doesn't warrant review).
12. Various timber elements. The old and new condition states are roughly equivalent, with the exception that old state 4 asserts that serviceability is affected, while new state 4 only warrants a review. Therefore part of old state 3 must be allocated to new state 4.
13. Bearings. Old condition state 1 agrees with new state 1. Old state 3 agrees with new state 4. Old state 2 appears to be divided between new states 2 and 3.
14. Culverts. The old and new condition states are roughly equivalent except for old state 4, which is much broader than new state 4 (which warrants structural review).
15. Pile jackets. It is difficult to relate the old and new condition states because the old language is quite vague. But they appear roughly equivalent.
16. Mechanically stabilized earth walls. The old and new language focus on different distresses, making them difficult to compare. There is little reason to believe they aren't equivalent, with the exception of the new state 4, which is much broader than the old state 4.
17. Various movable bridge elements. States 1 and 3 are roughly equivalent between old and new. Old state 2 is divided between new states 2 and 3.
18. Deck wearing surface. Relied mainly on old elements 13 and 39, belonging to element groups A1 and A2, which are the most common elements having wearing surfaces. The condition state language for these elements mainly describes the wearing surface. However, there is very little correspondence between the old and new language since the old language is purely extent-based and the new language is purely severity-based.
19. Paint on steel or stain on concrete. Relied mainly on painted steel superstructure elements of group D2. State 1 is equivalent between the old and new language. Old state 2 remains in new state 2. Old state 3 feeds into both new states 3 and 4. Old states 4 and 5 are included in new state 4.
20. Weathering steel patina. Relied mainly on unpainted steel super/substructure of group D1. States 1 and 2 remain in the same condition. Old state 3 is divided between new states 2 and 3. State 4 remains in state 4.
21. Galvanized/ metalized/other. Relied mainly on metal culverts of group G2. States 1 and 2 remain in the same condition. Old state 3 is divided between new states 2 and 3. State 4 remains in state 4.
22. Reinforcing steel protective system. The old condition state language did not address cathodic protection system condition directly, but in terms of evident corrosion the states roughly correspond to old element 299 in element group 12.

- The CoRe elements with five states which needed to be merged to four.
- The CoRe elements with three states which needed to be divided into four.
- Differences in whether structural review is warranted in the worst condition state.
- Differences in whether reinforcing steel is exposed in the second condition state.

Bridge decks were most difficult to interpret since the old definitions are largely based on extent of distress, while the new elements are based on severity.

Transition Times

A way to use the migration probability matrix to estimate a new deterioration model is to assume that the allocation of transition times among condition states is roughly proportional to the allocation of element quantities. This is not the same thing as saying condition is uniform with age. It says rather than if a change in condition state definitions causes 10% of an element quantity to be reclassified into the next condition state, that it is reasonable to reclassify 10% of the transition time also. There isn't an easy way to prove or disprove this assumption, without repeating a full-scale study as was done in 2011. It makes intuitive sense, however, and can be validated once a full set of inspections is completed under the new 2016 manual.

Accepting this assumption, new transition times can be readily computed by multiplying the old vector of transition times (from the 2011 research) by the migration probability matrix. The old and new transition times thus computed are reported in Table 4. This computation is readily performed using a spreadsheet or SQL query.

There were six element groups where this matrix multiplication resulted in a transition time of zero from state 3 to state 4. When this occurred, the transition time from state 2 to state 3 was arbitrarily divided, with half reassigned to the 3 to 4 transition. In most cases this occurred with three CoRe element condition states when the model didn't provide clear guidance on the division of old state 3 into new states 3 and 4.

The new transition times in Table 4 were expanded using the correspondences given in Table 1 to yield a deterioration model for every element in the 2016 Florida DOT Manual. The result was imported directly into the new deterioration model table in AASHTOWare Bridge Management, which expresses its deterioration model in the form of median transition times.

CONCLUSION

More than half of the state DOTs are participating in the AASHTOWare Bridge Management project, and all states are required by 23 CFR 515.17 to implement some type of bridge management system using deterioration models. Because of the transition to a new AASHTO inspection manual, none of the states will be able to use inspection data gathered under the new manual to develop deterioration models until several years of data are collected. However, most of the states have plentiful element level inspection data gathered under earlier AASHTO element inspection guides.

The methodology described here will enable state DOTs to take advantage of their valuable CoRe Element data to develop deterioration models that are based on valid inspection data but adapted to the latest inspection procedures. The large populations of CoRe elements in many state DOT Pontis inventories would help agencies develop high-quality models. From the 2011 Florida research that attempted to validate the 2001 judgment-based models, there is strong reason to believe that such models would be significantly more reliable than models based purely on expert judgment.

The biggest shortcoming with the new models is the fact that the migration probability matrix had to be developed from judgment. Once Florida DOT completes a cycle of inspections under the new manual, a better approach will be possible. The most recent inspection on each bridge can be projected forward 2 years using the existing CoRe Element deterioration model.

TABLE 4 New Deterioration Model

Element Group	Median Years from State 1 to State 2	Median Years from State 2 to State 3	Median Years from State 3 to State 4
A1 Concrete deck	6	48	51
A2 Concrete slab	4	47	25
A3 Prestressed concrete slab	5	79	50
A4 Steel deck	5	11	11
A5 Timber deck/slab	5	7	15
A6 Approach slabs	12	25	28
B1 Strip Seal expansion joint	13	23	23
B2 Pourable joint seal	10	4	4
B3 Compression joint seal	6	5	5
B4 Assembly joint/seal	14	7	7
B5 Open expansion joint	18	15	15
B6 Other expansion joint	19	30	30
C1 Uncoated metal rail	74	3	3
C2 Coated metal rail	18	10	4
C3 Reinforced concrete railing	68	24	38
C4 Timber railing	12	4	4
C5 Other railing	37	8	8
D1 Unpainted steel super/substructure	13	9	13
D2 Painted girder/floor beam/cable/pin and hanger	14	40	28
D3 Painted steel stringer	19	150	137
D4 Painted steel truss bottom	15	19	3
D5 Painted steel truss/arch top	10	90	76
D6 Prestressed concrete superstructure	293	16	11
D7 Reinforced concrete superstructure	32	16	15
D8 Timber superstructure	41	27	3
E1 Elastomeric bearings	96	121	121
E2 Metal bearings	14	24	24
F1 Painted steel substructure	12	9	2
F2 Prestressed column/pile/cap	16	40	62
F3 Reinforced concrete column/pile	41	46	84
F5 Reinforced concrete abutment	87	164	347
F6 Reinforced concrete cap	145	68	139
F7 Pile cap/footing	9	38	55
F8 Timber substructure	24	18	3
G1 Reinforced concrete culverts	7	37	138
G2 Metal and other culverts	8	29	34
H1 Channel	9	17	26
I1 Pile jacket w/o cathodic protection	13	17	18
I2 Pile jacket with cathodic protection	19	56	43
I3 Fender/dolphin/bulkhead/seawall	11	9	27
I4 Reinforced concrete slope protection	56	16	10
I5 Timber slope protection	62	17	82

Continued on next page.

TABLE 4 (*continued*) New Deterioration Model

Element Group	Median Years from State 1 to State 2	Median Years from State 2 to State 3	Median Years from State 3 to State 4
I6 Other (incl. asphalt) slope protection	35	13	9
I7 Drainage system other materials	8	2	3
I7 Drainage system metal	8	3	1
J1 Uncoated metal wall	9	6	71
J2 Reinforced concrete wall	50	31	46
J3 Timber wall	24	9	8
J4 Other (incl. masonry) wall	10	18	19
J5 Mechanically stabilized earth wall	76	10	17
K1 Sign structures/hi-mast light poles	15	18	7
K1 Sign structures/hi-mast light poles (coated)	14	40	28
L1 Moveable bridge mechanical	12	34	12
L2 Moveable bridge brakes	5	7	6
L3 Moveable bridge motors	9	7	10
L4 Moveable bridge hydraulic power	8	15	13
L5 Moveable bridge pipe and conduit	6	14	14
L6 Moveable bridge structure	13	10	6
L7 Moveable bridge locks	4	6	15
L8 Moveable bridge live load items	6	11	11
L9 Moveable bridge counterweight/ trunnion/track	13	14	81
M1 Moveable bridge electronics	38	10	10
M2 Moveable bridge submarine cable	10	3	3
M3 Moveable bridge control console	9	8	8
M4 Moveable bridge navigational lights	9	5	5
M5 Moveable bridge operator facilities	14	19	19
M6 Moveable bridge misc. equipment	1	5	5
M7 Moveable bridge barriers/gates	10	10	10
M8 Moveable bridge traffic signals	30	3	3
P1 Deck wearing surface	12	57	36
P2 Paint on steel or stain on concrete	10	8	4
P3 Weathering steel patina	13	15	7
P4 Galvanized/metalized/other	8	46	17
P5 Reinforcing steel protective system	19	56	43

Then a migration probability matrix can be computed by comparing the new inspections against the projected estimates, using an algebraic method similar to the 2011 research (*3*). In the longer term, after two complete cycles of inspections are completed under the new manual, a new set of deterioration models can be developed as was done in the 2011 study.

ACKNOWLEDGMENT

This work was funded by the Federal Highway Administration of the U.S. Department of Transportation, and the Florida DOT, under a contract with Florida State University (FSU). The Florida DOT Project Manager was Richard I. Kerr, and the FSU Principal Investigator was John O. Sobanjo. Special thanks are extended to Chris Laughlin of the Florida DOT State Maintenance Office for data and technical assistance.

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Exploring the U.S. Interstate Highway Bridge Maintenance Expenditure Versus Condition Trade-Off Relationship Using Aggregate Data

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In the United States, oversight agencies such as the U.S. Department of Transportation, the Federal Highway Administration, and the Government Accountability Office are responsible for the measuring and monitoring the overall accountability of state highway agencies. To help these oversight bodies to carry out this task, it is often useful to show the extent to which the infrastructure condition at a given year influences the repair expenditure in a subsequent year. Further, oversight agencies typically seek to establish a methodology to assess how well the individual states are doing compared to each other. In response to these two issues, this paper first establishes an empirical relationship between the average deck condition at a given year and the normalized expenditure in a subsequent year. This is illustrated using Interstate highway bridge decks as a case study. The unit of observation in this paper is the state level (each state contains a collection of bridge decks whose average annual expenditures and average condition rating are known). Thus, the analysis is aggregate in nature and does not consider site-specific variables. The paper recognizes that there exist jurisdiction-specific variables that affect Interstate bridge deck condition, and therefore attempts to remove some of this bias by normalizing the expenditure as a ratio of the inventory size and by considering state-specific values of the key deterioration variables. Secondly, the paper identifies the factors found significant in the condition–expenditure relationship, and uses these factors as a basis for assessing the performance of state highway agencies relative to others.

In any country, there are oversight bodies that are responsible for tracking the overall performance of each provincial, state, or regional infrastructure agency in their jurisdiction. This often means that the oversight body assesses whether each agency's outcomes were consistent with their spending levels, and the extent to which the outcomes in a previous year can influence the expected spending in a subsequent year. In the United States, for example, oversight bodies including the FHWA and the U.S. Government Accountability Office, regularly measure and monitor the overall accountability of state highway agencies. These oversight bodies carry out their functions by, among others, (*a*) quantifying the extent to which the repair expenditure in a given year influences the infrastructure condition at a subsequent year; (*b*) quantifying the extent to which the infrastructure condition at a given year influences the repair expenditure in a subsequent year; and (*c*) assessing the performance of the individual states relative to each other, in terms of their spending levels and infrastructure performance.

STUDY OBJECTIVES AND SCOPE

In addressing two of the three issues raised above, this paper first seeks to use the expenditure and condition data of 2012 in Indiana to establish an empirical statistical relationship between the average deck condition at a given year and the normalized expenditure in a subsequent year. This is illustrated using Interstate highway bridge decks as a case study. The unit of observation in this paper is the state (each state contains a collection of bridge decks whose average annual expenditures and average condition rating are known). Thus, the analysis is aggregate in nature and does not consider site-specific variables. The paper recognizes that there exist jurisdiction-specific variables that affect interstate bridge deck condition, and therefore removes some of this bias by normalizing the expenditure variable as a ratio of the system size and by considering state-specific values of the key deterioration variables. In the second part of the paper, the paper identifies the factors found significant in the condition–expenditure relationship established in the first part, and uses these factors as a basis for assessing the performance of state highway agencies relative to others. Then the paper offers plausible explanations of the observed differences in the resulting overall performance across the states.

DATA

The paper used three sets of databases. FHWA's Office of Highway Policy Information (*1*) maintains a database of state highway expenditures on highway construction and maintenance. The FHWA also maintains a database of highway bridge features that include the average daily traffic, deck condition rating (Figure 1), deck width, structure length, and average daily truck traffic (*2*). The metadata in this database is available in FHWA's Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges (*3*). The climate data were obtained or derived from the National Climate Data Center (NCDC) database (*4*). Of the several primary and secondary indicators of climate (*5*), the freeze index is used in this paper. For each state, the average freeze index over the 1992 to 2012 period was used in this paper. In short, the variables used in this paper include the following:

- Strength factors:
  - Total expenditure per ft^2 (the total expenditure was for all components: deck, superstructure, and substructure). The authors recognize that this introduces bias in the analysis. Therefore, in this paper, the assumption has been made that the amount of deck maintenance is directly related to the amount of overall bridge maintenance. In ongoing research work, authors are isolating the deck maintenance amounts for each bridge; this is being done using an algorithm that tracks the deck performance jump [in terms of National Bridge Inventory (NBI) rating] as a result of any deck work implemented.
- Stress factors:
  - Traffic (truck) loads and
  - Climate severity indicated by freeze index in degree-days. (Freeze index is calculated by subtracting the number of degree-days between the highest point and lowest point on a cumulative degree-day curve. The calculation takes place for one freezing season. And, a degree-day is the change between the average daily air temperature and 32°F.)

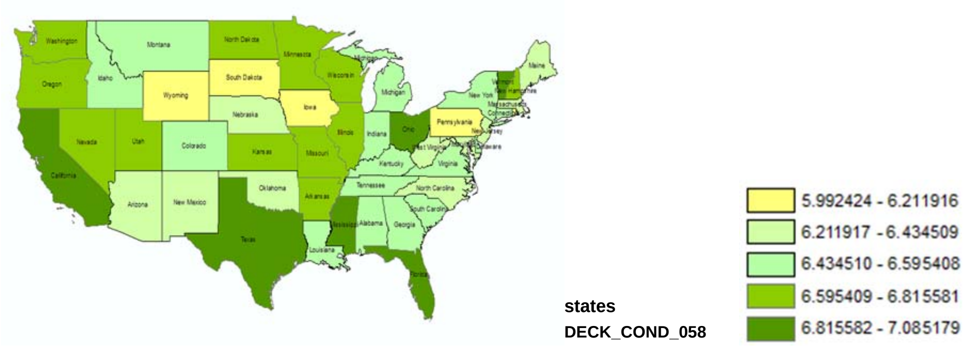

FIGURE 1 Distribution of the average Interstate bridge deck condition across the states—average for all states is satisfactory, i.e., condition rating over 6.

METHODOLOGY

Part 1: The Relationship Between Condition and Expenditure

This part of the paper quantifies the extent to which the deck condition at a given year influences the repair expenditure in a subsequent year, using a regression model. The model's response variable is the rehabilitation and maintenance expenditure (EXP). The explanatory variables are deck condition (Cond), freeze index (FRZ), area of the bridges (AREA), and annual average daily truck traffic (ADTT). Deck condition is rated from 1 to 9, where 9 is the best condition and 1 is the worst condition. The model form (Equation 1) is:

$$\mathrm{EXP} = \beta_0 + \beta_1\,\mathrm{Cond} + \beta_2\mathrm{FRZ} + \beta_3\mathrm{ADTT} + \beta_4\mathrm{AREA} \tag{1}$$

The choice of these explanatory variables can be justified based on past research findings. In states with mild climate, such as California, the decks suffer less exposure to freezing conditions, free–thaw cycles, snow, and ice-control chemicals. The average truck traffic experienced by the Interstate decks in the state also influences the repair expenditures. For example, South Carolina generally has comparatively low truck traffic on average. Furthermore, states with large bridge inventories such as Texas are expected to have far greater expenditure compared to those with smaller inventories such as Delaware. These differences in state inventory sizes are not expected to influence the average deck condition (so there is no need to normalize this variable). However, the size differences will affect the deck expenditure; as such there was a need to normalize the expenditure variable by dividing this amount by the state inventory size (total area of decks in the state).

Part 2: Assessing the Relative Performance Across the States

This paper's methodology for assessing the performance of states is based on the inputs (normalized expenditure, average truck traffic, and average climate severity). A performance plot was developed to visualize the relative performance across the states. The *y* axis (ordinate) represents the average expenditures on bridge work divided by the average deck condition. Therefore, states with a large ordinate value are exhibiting poor performance because they are spending more than other states but yielding low deck conditions. The *x* axis (abscissa) represents the overall stress experienced by the decks in terms of the truck traffic and climate severity. Therefore, states with a large abscissa value are facing challenging conditions in terms of the stressors of deck condition.

The average value of the ordinate can be determined for all the states in the United States and can be calculated and plotted as a horizontal line. Similarly, the average value of the abscissa can be determined for all the states can be determined and plotted as a vertical line. These lines together yield four quadrants on the performance plot. The quadrant location of each state can be determined.

The first quadrant (high values of the ordinate and low values of the abscissa) represents the states that have low deck condition despite their favorable environment (low truck traffic and mild climate) and high spending levels ($\$/ft^2$ of deck). These are the poor performers. The second quadrant (high values of the ordinate and high values of the abscissa) represents the states that have low deck condition and unfavorable environment (high truck traffic and severe climate) and high spending levels ($\$/ft^2$ of deck). These are the good–fair performers. The third quadrant (low values of the ordinate and high values of the abscissa) represents the states that have high deck condition despite their unfavorable environment (high truck traffic and severe climate) and low spending levels ($\$/ft^2$ of deck). These are the good performers. The fourth quadrant (low values of the ordinate and low values of the abscissa) represents the states that have good deck condition, favorable environment (low truck traffic and mild climate) and low spending levels ($\$/ft^2$ of deck). These are the fair–good performers. Thus, the position of a state in a quadrant can be a reflection of the prudent use of the taxpayer funds by the state agency.

RESULTS

Part 1. The Relationship Between Condition and Expenditure

Figure 2 shows the histogram of the residual for EXP and log(EXP). This shows that MLR.6 (normality) is satisfied in this regression for log(EXP) and data is not skewed.

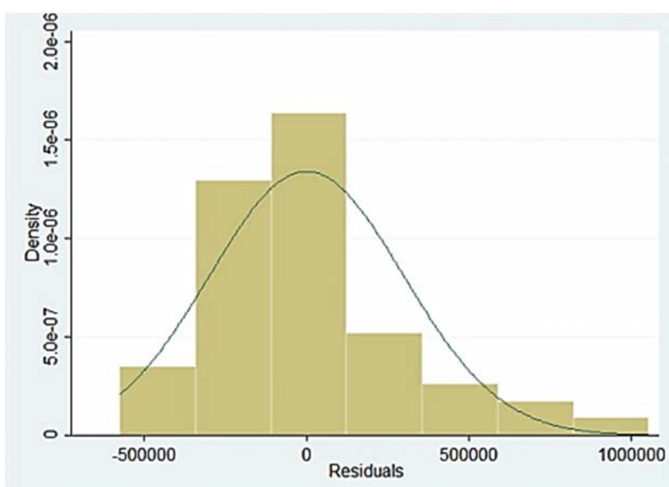

FIGURE 2 Normal distribution of the residuals.

The model results (Table 1) show that there is a reverse effect between the deck condition and the expenditure: in other words, a lower average condition in one year leads to higher expenditures the following year. In addition, the higher the total area of bridge deck, the higher the expenditure per ft^2, this is suggestive of diseconomies of scale. The results also suggest that a higher freeze index is generally associated with higher expenditure.

Part 2: Assessing the Relative Performance Across the States

Considering all the statistical data described above, an analysis on the expenditure, area of the bridge, deck condition versus freezing index and ADTT (average values for each state in the year 2012) leads us to the following categories of the states (Figure 3):

- Lowest performers (states with relatively high spending per ft^2, low deck condition, low truck traffic, or mild climate): Rhode Island, District of Columbia, New Jersey, Alaska, Vermont, North Dakota, Oregon, West Virginia, Oklahoma, Idaho, Washington, and Mississippi.
- Fair–good performers (states with high spending per ft^2, low deck condition, high truck traffic, or mild climate): New York, Connecticut, Illinois, Massachusetts, Michigan, and Pennsylvania.
- Fair–good performers (states with relatively low spending per ft^2, high deck condition, low truck traffic, or severe climate): Delaware, Kentucky, Arkansas, Arizona, Louisiana, Texas, Georgia, Tennessee, Missouri, Kansas, Nevada, South Dakota, South Carolina, Alabama, Virginia, New Hampshire, New Mexico, Florida, Montana, Maryland, Hawaii, Maine, North Carolina, and Nebraska.
- Highest performers (states with relatively low spending per ft^2, high deck condition, high truck traffic, or severe climate): Indiana, Utah, Iowa, Colorado, Minnesota, Ohio, Wisconsin, Wyoming, California

It must be stated that these results are only exploratory, and further more-detailed studies need to be conducted to reach a more definite statement about the relative performance across the agencies.

TABLE 1 Regression Model for EXP, Deck Condition, Area, Freeze Index, and ADTT

Number of Observations	50	***F* (4,45)**	6.71	**Prob. >*F***	0.0003
R^2	0.3734	**Adj.** R^2	0.3177	**RMSE**	3.1E+5
EXP	**Coef.**	***t***	***P*>|*t*|**	**95% Conf. Interval**	
DECK	–382373.4	–2.17	0.035	–736482	–28264.75
AREA	0.004872	2.91	0.006	0.0015	0.0882
FRZ	29.7635	0.35	0.730	–143.1713	202.6985
ADTT	0.01444	1.32	0.194	.007628	0.0365110
Condition	2562771	2.27	0.028	289499	4836043

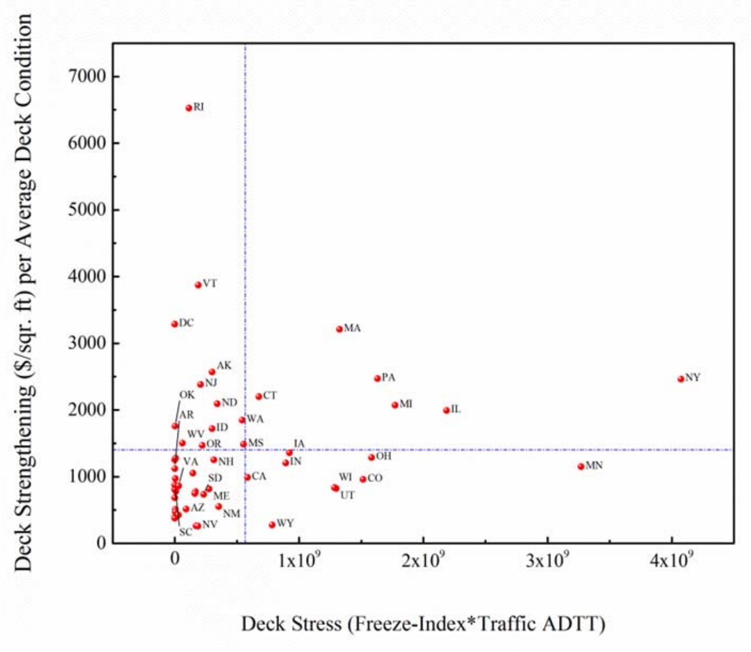

FIGURE 3 Relative performance across the states.

CONCLUSION

In summary, the framework and results shows how oversight agencies can monitor the overall accountability of individual highway agencies. The observed differences in the state performance could be due to extreme differences in construction cost across states, differences in agency audit quality, work culture, poor geotechnical conditions in a state, unfavorable design–construction practices, and possibly, poor quality of quarry or borrow pit materials available in or near a state. The relative rankings could also prompt those agencies seen as not well performing, to carry out critical self-assessment to identify the possible causes of such performance as a first step towards their resolution.

Future studies could address a number of limitations and areas of this paper. The current paper does not address a quantification of the extent to which the repair expenditure in a given year influences the infrastructure condition at a subsequent year; this is the reverse of what was investigated in this paper, but is also of great interest to all agencies. In order to do this, the following model form (Equation 2) can be used:

$$\mathrm{COND} = \beta_0 + \beta_1\,\mathrm{EXP} + \beta_2\mathrm{FRZ} + \beta_3\mathrm{AADT} + \beta_4\mathrm{AREA} \tag{2}$$

where the symbols are as defined in Equation 1.

In addition, future work could use actual deck expenditures and not all expenditures. Also, in this paper, one of the key assumptions was that 1 degree-day of FRZ and 1 truck have equivalent effects on bridge work expenditure. Future papers relax the analysis assumptions made in the present study, for example, by establishing appropriate weights between the deterioration factors and use these weights to determine the quadrant positions of the agencies. In addition, future studies could consider other model specifications such as the lagged panel model, not just a one-year lag ($t - 1$) as done in this paper but also $t - 2$, $t - 3$, and so on. Future work could also consider average statewide values of other design variables that constitute stressors or strengtheners, measure the stability of the state quadrant position (performance rankings) across the years, and extend the work to the other bridge components (superstructure and substructure).

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Optimal Retrofit Decision-Making for Bridge Systems Based on Multihazard Life-Cycle Cost Analysis

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Various types of hazards each with the potential to occur multiple times during the long service life of bridges may threaten the functionality of transportation systems and significantly impact the society. In hazard-prone areas, as the recovery time becomes longer, the likelihood of other hazard events occurring before the system is recovered increases. This can result in the accumulation of damage and higher vulnerability of the infrastructure. This study presents a multihazard life-cycle cost assessment framework to find optimal solutions for retrofit strategies. The possibility of multiple occurrences of multiple types of hazard incidents is probabilistically incorporated in the framework. This methodology accurately determines the expected life-cycle cost of hazard-induced consequences by comprehensively including direct and indirect incurred costs. The presented framework is applied to a realistic multispan reinforced concrete bridge in California that is exposed to flood and earthquake hazards. The total life-cycle cost of several practical retrofit strategies are evaluated and compared for a wide range of bridge service lives. A sensitivity analysis is also performed to characterize the impacts of several key variables on the expected life-cycle cost of the bridge and the optimal retrofit plans.

Bridges are vital components in transportation systems. Various types of hazards each with the potential to occur multiple times during the long service life of bridges may threaten the functionality of transportation systems and significantly impact the society. Depending on the extent of induced damage, type of retrofit and repair strategy, and socioeconomic factors, the recovery time after each hazard incident may vary from short to long periods. Especially in hazard-prone areas, as the recovery time becomes longer, the likelihood of other hazard events occurring before the system is recovered from the previous incident increases. This can result in the accumulation of damage and higher vulnerability of the infrastructure.

In the literature of risk analysis for infrastructure systems, life-cycle cost (LCC) that expresses the risk of extreme events in terms of monetary loss over the service life of the system is considered as one of the most appropriate performance measures for infrastructure decision-making (*1–4*). These studies appraised system performance primarily for single hazard occurrences. For instance, LCC was applied to identify optimal decisions for the management of infrastructure systems under a single type of hazard (*5–9*). Some studies considered multiple types of hazards for decision-making of bridge systems. As an example, Patidar et al. (*10*) introduced a utility function that includes risk of hazard types as one of the weighted performance criteria for management of bridges. In these studies that consider multiple hazard incidents, it is assumed that repairs following hazard occurrences are instantaneous or there are no repair actions after each incident of hazards. In reality, however, the time required for repairing damage to infrastructures depends on the extent of damage, type of repair action, availability of materials and crew, and socioeconomic factors, among others. When repair times are long, the possibility of next hazards happening before the damage arising from previous hazards are repaired, increases. This results in accumulation of damage and represents a vulnerable condition for infrastructures. For example, in September 2010, an earthquake with the magnitude of 7.1 caused widespread damage to structures and infrastructure systems in Christchurch, New Zealand (*11*). Six months later, an aftershock with the magnitude of 6.3 shook the same region and induced further damage in already damaged structures and infrastructure systems, and caused 185 casualties (*12*).

When looking at infrastructures located in regions that are exposed to more than one type of hazard, many studies, such as Wen and Kang (*13*) and Decò and Frangopol (*14*), disregard the dependency between damage conditions induced by various hazard types. Jalayer et al. (*15*) attempted to address such dependencies for multiple hazard types in a framework that requires simulating all possible scenarios for the order of hazard events of various types and intensities. In addition, each of these scenarios requires time-consuming structural pushover and dynamic analyses. These make the framework computationally prohibitive for a comprehensive LCC analysis. Moreover, there are a number of assumptions in that framework that may not accurately represent the performance of actual systems. For example, when calculating the probability of exceeding a particular damage state *i* at *j*th hazard occurrence, the dependency of damage state *i* to prior exceeded damage states other than *i* is disregarded. Conversely, any extent of prior damages directly affects the probability of exceeding damage state *i* at the current occurrence (i.e., *j*th occurrence) of the hazard event.

This study, which is an extension to the methodology developed by the authors for a single type of hazard (*16*), proposes a multihazard LCC assessment framework to find optimal solutions for retrofit strategies. The possibility of multiple occurrences of multiple types of hazard incidents are probabilistically incorporated in the LCC analysis framework through a recursive function that utilizes damage state-dependent fragility models and repair times. This methodology accurately determines the expected LCC of hazard-induced consequences, including repair costs of structural damage, human casualties, damage to the environment, user costs of traffic delay, vehicle operation and excess emission, and indirect economic losses due to interruptions of affected businesses. The computed LCC of hazards is discounted over the years, and added to the initial cost of applying retrofit actions and the discounted expected LCC of maintenance, to estimate the total LCC of the bridge system under study. In the rest of this paper, the analytical formulation of the suggested multihazard LCC is introduced. Then, the framework is demonstrated for a realistic case study bridge subjected to multiple occurrences of two types of hazards: earthquakes and floods. Finally, optimal retrofit decision-making for the case study bridge is discussed and sensitivity analysis is performed to identify some factors that significantly influence optimal retrofit decisions.

MULTIHAZARD LIFE-CYCLE COST FRAMEWORK

Net present value (NPV) of the total LCC of an infrastructure can be typically expressed as:

$$C_{T,\mathrm{NPV}} = C_0 + C_{M,\mathrm{NPV}} + C_{R,\mathrm{NPV}} \tag{1}$$

where $C_{T,\mathrm{NPV}}$, $C_{M,\mathrm{NPV}}$, and $C_{R,\mathrm{NPV}}$ are the discounted NPV of C_T (total LCC), C_M (LCC of maintenance), and C_R (LCC of repair). If the LCC is evaluated for an existing system, C_0 will be zero. In case of planning to upgrade the system, this cost is equal to the cost of such upgrade. In terms of performing annual maintenance actions in the lifetime of infrastructures to keep them functioning in a healthy condition, $C_{M,\mathrm{NPV}}$ can be represented as follows:

$$C_{M,\mathrm{NPV}} = \sum_{t=1}^{T_{LC}-1} \gamma^t \times C_{m,t} \tag{2}$$

where $C_{m,t}$ is the maintenance cost at year t, T_{LC} is the expected service lifetime of the infrastructure, and γ is the annual discount factor equal to $\frac{1}{1+\delta}$, with δ as the interest rate.

In the lifetime of an infrastructure, the system may experience multiple occurrences of multiple types of hazards. For instance, six types of hazards have been identified significant for bridges in the state of New York: earthquake excitations, collisions, details of steel structures, details of concrete structures, hydraulic, and overload (*17*). After each such incidents, the system may experience damage or stay intact. Each condition state is followed by consequences that are typically expressed in cost terms. These costs comprise agency cost of repairing the system, user costs such as the delay cost associated with the reduced serviceability of the system during the repair process, impacts on the economy and related environmental costs, and even injuries and human casualties. In this article, these costs are referred to as repair cost.

Similar to $C_{M,\mathrm{NPV}}$, in order to account for the discounted repair costs that are likely to incur at different times in the future, NPV of the life-cycle repair cost can also be split into yearly repair costs as follows:

$$C_{R,\mathrm{NPV}} = \sum_{t=0}^{T_{LC}-1} \gamma^t \times C_{R,t} \tag{3}$$

where $C_{R,t}$ is the repair cost incurred at year t. $C_{R,t}$ can be further expanded to:

$$C_{R,t} = \sum_{n=1}^{N_{CS}} C_r(CS_n) \times P(CS_n, [t, t+1]) \tag{4}$$

where N_{CS} is the total number of condition states, $C_r(CS_n)$ is the repair cost when the infrastructure experiences condition state n, and $P(CS_n, [t, t+1])$ is the probability of the structure sustaining condition state n between time t and t+1. Expanding on the latter term, Equation 4 can be written as:

$$C_{R,t} = \sum_{n=1}^{N_{CS}} \{C_r(CS_n) \times P(CS_n, [0, t+1]) - C_r(CS_n) \times P(CS_n, [0, t]\} \tag{5}$$

Using the total probability theorem, considering that i number of hazards of various types may happen during the lifetime of an infrastructure, $C_r(CS_n) \times P(CS_n, [0, t+1])$ can be expanded as:

$$C_r(CS_n) \times P(CS_n, [0, t]) = \sum_{i=0}^{\infty} P(i, t) \times \sum_{j=0}^{i} C_r(CS_n) \times P\left(CS_n^j | i, t\right) \tag{6}$$

where $P\left(CS_n^j | i, t\right)$ is the probability that condition state *n* is experienced by the infrastructure at *j*th hazard incident if *i* hazards take place during [0 *t*]. $P(i, t)$ stands for the probability that *i* hazards occur during [0, *t*]. Equation 6 calculates cumulative repair costs for the entire *i* events that are likely to occur. Assuming independent hazard events, $P(i, t)$ is represented by Poisson distribution function as:

$$P(i, t) = \frac{\left(\sum_{h=1}^{N_H} \upsilon_h \times \mathrm{t}\right)^i e^{-\left(\sum_{h=1}^{N_H} \upsilon_h \times \mathrm{t}\right)}}{i!} \tag{7}$$

where υ_h stands for the occurrence rate of hazard type h, and N_H represents the total number of hazard types that may hit the infrastructure throughout its lifetime.

In terms of available information from fragility curves, which is a common practice in structural reliability, $P\left(CS_n^j | i, t\right)$ for one type of hazard can be written as:

$$P\left(CS_n^j | i, t\right) = P\left(LS_n^j | i, t\right) - P\left(LS_{n+1}^j | i, t\right) \tag{8}$$

where $P\left(LS_n^j | i, t\right)$ is the probability that limit state *n* is exceeded by the infrastructure at *j*th hazard incident, if *i* hazards take place during [0, *t*]. This information can be extracted from structural fragility curves. Considering uncertainties in structural response, structural repair status at the time of *j*th hazard incident (whether complete or yet incomplete), the condition state of the structure at the time of *j*th hazard incident, and the intensity of the *j*th hazard incident, $P\left(LS_n^j | i, t\right)$ can be articulated as follows (*15*):

$$\begin{aligned} P\left(LS_n^j | i, t\right) = \sum_{n'=1}^{N'} \sum_{RP} \sum_{IM} & P\left(LS_n^j \middle| \left[RP_{n'}, CS_{n'}^{j-1}\right], IM, i, t\right) \times P\left(\left[RP_{n'} \middle| CS_{n'}^{j-1}, i, t\right]\right) \\ & \times P\left(CS_{n'}^{j-1} | i, t\right) \times P(IM) \end{aligned} \tag{9}$$

where N' is the total number of condition states, RP is the repair status (either complete or incomplete), and IM is the intensity measure of the hazard incident. $\left[RP_{n'}, CS_{n'}^{j-1}\right]$ represents the condition state of the infrastructure at the time of *j*th hazard incident, which is considered as intact if the repair process is complete, or condition state n'otherwise. Extending Equation 9 to multiple hazard types with the possibility of the infrastructure experiencing multiple types of damage, $P\left(LS_n^j | i, t\right)$ is modified to $P\left(LS_{[n_1, \ldots, n_M]}^j | i, t\right)$ which can be expressed as:

$$P\left(LS^{j}_{[n_1,\ldots,n_M]}|i,t\right) = \sum_{n'_1=1}^{N'_1}\cdots\sum_{n'_M=1}^{N'_M}\sum_{h=1}^{N_H}\sum_{RP}\sum_{IM_h} P(LS^{j}_{[n_1,\ldots,n_M]}\left|\left[RP_{[n'_1,\ldots,n'_M]}, CS^{j-1}_{[n'_1,\ldots,n'_M]}\right], HT_h, IM_h, i, t\right) \times P\left(\left[RP_{[n'_1,\ldots,n'_M]}\middle|CS^{j-1}_{[n'_1,\ldots,n'_M]}, HT_h, i, t\right]\right) \times P(CS^{j-1}_{[n'_1,\ldots,n'_M]}|i,t) \times P(HT_h) \times P(IM_h) \quad (10)$$

where $P\left(LS^{j}_{[n_1,\ldots,n_M]}|i,t\right)$ is the probability of exceeding condition state $[n_1,\ldots,n_M]$ at *j*th hazard occurrence given *i* hazards take place within time [0, *t*]. These terms are called limit state transition probabilities in this paper. N'_M is the total number of condition states for damage type *M*, N_H is the total number of hazard types that may hit the system, RP is the repair status (either complete or incomplete) for each of the *M* damage types, and IM_h is the intensity measure of hazard type *h*. Having the knowledge of the repair status for each probabilistic realization in Equation (*10*), $P(LS^{j}_{[n_1,\ldots,n_M]}\left|\left[RP_{[n'_1,\ldots,n'_M]}, CS^{j-1}_{[n'_1,\ldots,n'_M]}\right], HT_h, IM_h, i, t\right)$ can be calculated based on fragility curves. For some realizations, this information should be available when the infrastructure is in a damaged condition. Thus, damage state-dependent fragility curves should be available for the infrastructure under study. It can be shown that $P(CS^{j-1}_{[n'_1,\ldots,n'_M]}|i,t)$ in Equation 10 can be expressed in terms of exceedance probabilities of limit states as follows:

$$P\left(CS^{j}_{[n_1,\ldots,n_M]}|i,t\right) = P\left(LS^{j}_{[n_1,\ldots,n_M]}|i,t\right) - \sum_{\substack{i_1\in\{0,1\}\\(i_1,\ldots,i_M)\neq(0,\ldots,0)}}\cdots\sum_{i_M\in\{0,1\}} P(LS^{j}_{[n_1+i_1,\ldots,n_M+i_M]}|i,t) + \sum_{k=2}^{2^M-1}(-1)^k\times\binom{2^M-1}{k}\times P\left(LS^{j}_{[n_1+1,\ldots,n_M+1]}|i,t\right) \quad (11)$$

Then, based on Equation 10 and inserting the right hand side of Equation 11 in Equation 10, $P\left(LS^{j}_{[n_1,\ldots,n_M]}|i,t\right)$ can be recursively calculated. This procedure is the key to the time efficiency of the proposed framework, while the realizations of a wide range of uncertain variables are comprehensively integrated. Since hazards of different types are considered independent, $P(HT_h)$ is enumerated as:

$$P(HT_h) = \frac{\upsilon_h}{\sum_{h'=1}^{N_H}\upsilon_{h'}} \quad (12)$$

Finally, in Equation 10, $P\left(\left[RP_{[n'_1,\ldots,n'_M]}\middle|CS^{j-1}_{[n'_1,\ldots,n'_M]}, HT_h, i, t\right]\right)$ stands for the probability of a given repair status, i.e., complete or incomplete. This term is calculated depending on the events of:

- Condition state of the structure that hazard of type *h* will affect;
- The likelihood of the hazard that is happening at *j*th hazard incident; and
- The time span [0 *t*] during which *i* number of hazards should take place.

IMPLEMENTATION OF THE FRAMEOWRK FOR A CASE STUDY BRIDGE

The suggested framework is implemented for a realistic five span reinforced concrete bridge located in the city of Sacramento, over American River. The bridge model was developed and analyzed by Prasad and Banerjee (*18*). The bridge is vulnerable to both seismic-induced damages and flood-induced scour accumulations. Following National Institute of Building Sciences (NIBS), FEMA (*19*) and Prasad and Banerjee (*18*) categorized seismic-induced damages based on the displacement ductility capacity of bridge piers. As the scour depth of bridge piles increases, the capacity of the bridge against seismic-induced damage decreases. However, the accumulation of seismic damage in the bridge does not affect the scour depth induced by flood events.

For the suggested LCC framework with the discount rate of 5%, combinations of retrofit alternatives including no retrofit, applying steel jacketing, and performing scour countermeasures are considered, and optimal retrofit decisions for various service lifetimes of the case study bridge are determined. The repair process for any seismic-induced damage starts following each earthquake event. For the case of flood hazard, the scour countermeasure, if implemented, will be applied at the start of decision-making time horizon. For retrofit alternatives where no scour countermeasures are performed initially, scour depth accumulates as the number of flood events increases. The required input information for the implementation of the proposed framework for the case study bridge is briefly discussed hereafter.

Hazard Curves

Flood and earthquake hazard curves for the location of the bridge are adopted from Prasad and Banerjee (*18*) and Peterson et al. (*20*), respectively. These curves are shown in Figure 1, and are required for the generation of limit state transition probabilities according to Equation 10.

Damage State-Dependent Fragility Curves

Considering four scour levels of 0.0, 0.6, 1.5, and 3.0 m, and five ductility capacity levels for bridge piers [0.0 (no damage), 2.25 (minor), 2.90 (moderate), 4.60 (major), and 5.0 (complete or collapse)], there are 20 limit states for hazard-induced damages. The statistical characteristics of these damage state-dependent fragility curves for the case study bridge are taken from Prasad (*21*), Prasad and Banerjee (*18*), and engineering judgment of the authors. Flood-induced damages, on the other hand, only contain the foregoing four scour levels. Expected scour depths caused by various flood discharge levels were determined in Prasad and Banerjee (*18*) for the case study bridge, and are utilized in this study.

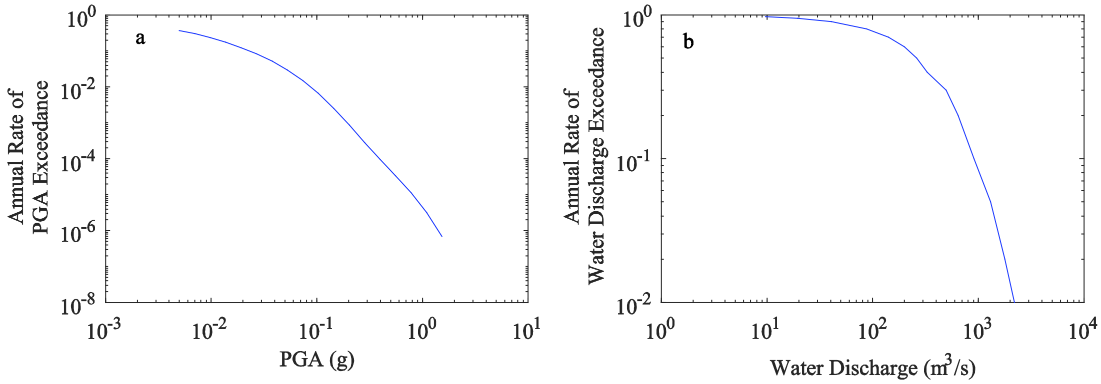

FIGURE 1 (*a*) Seismic (*20*) and (*b*) water discharge (*18*) hazard curves for the location of the case study bridge.

Required Repair Times for Damage States

One of the major features of the proposed framework is the ability to consider damage state-dependent repair times in the LCC calculations. Based on the repair path for each hazard-induced damage-state, lognormal mean and standard deviation of the required repair times are identified from NIBS, FEMA (*19*), Gordin (*22*), Shinozuka et al. (*23*), Burton et al. (*24*), and authors' judgement.

Cost Terms

Initial Cost of Retrofit Actions

Retrofitting or performing a scour countermeasure plan adds an initial cost to the total LCC of the bridge. Based on California Department of Transportation (Caltrans) historical data, Venkittaraman and Banerjee (*25*) reported \$2/lb for the cost of retrofitting piers of a bridge with steel jacketing. On this basis, the total cost of steel jacketing for all piers of the case study bridge is estimated as \$383,420.

The scour countermeasure plan considered for this bridge is concrete grouting the voids of the loose soil underneath each pile foundation and the soil surrounding bent foundations together with 1-m layer of rock slope protection material. Following Caltrans (*26*), the cost of performing this countermeasure plan for the four pile foundations of the case study bridge is estimated as \$195,000.

Annual Maintenance Cost

The annual cost of maintenance for the case study bridge is determined from the average cost of major repairs and rehabilitations for a sample of bridges in the city of Rancho Santa Margarita, California (*27*), considering these costs are repeated every 15 years. On this basis, annual maintenance cost is estimated as 7.5% of the bridge replacement cost. According to Caltrans construction statistics, the expected cost of replacement of the case study bridge is \$1833/m^2 (*8*).

Cost of Consequences for Damage States

Agency Cost of Repairing the Physical Damage Following NIBS and FEMA (*19*) and in line with the seismic damage states considered by Prasad and Banerjee (*18*), the cost of repairing minor, moderate, major, and collapse damages are 0.03, 0.08, 0.25, and 1.00 times the bridge replacement cost, respectively. In addition, 10% and 20% of the bridge repair cost are added to incorporate mobilization and contingency costs, respectively.

Cost of Delay on Users, Vehicle Operations, and Excess Gas Emission As a consequence of partial/complete closure of the bridge for repairing the physical damage, cost of delay on users, extra vehicle operations, and excess gas emission (emission of hydrocarbons, carbon monoxide, and nitrogen oxide) are incurred (*28*). The total cost of such consequences are denoted by C_{DVE}. The unit cost of these consequences after updating to year 2016 is \$21.79/hour and \$58.83/hour for unit cars and trucks, respectively (Ohio DOT, 2010). The general formulation for the calculation of these costs is:

$$C_{\text{DVE}} = \tau_n \times (t_{ij}^{D/R} - t_{ij}^{O}) \times [(\text{AADT} - \text{AADTT}) \times \rho_C + \text{AADTT} \times \rho_T] \tag{13}$$

where AADT and AADTT are the annual average daily traffic and annual average daily truck traffic of path *ij* that the bridge is part of, τ_n is the recovery time for damage state *n*, t_{ij}^{O} is the original time for passing path *ij* using the main bridge with no partial/complete closure and speed reduction, and $t_{ij}^{D/R}$ is the time for passing path *ij* using the main bridge/detour with partial/complete closure and speed reduction. The AADT of the bridge is considered as 77,000 for the three lane Capital City highway (*29*), which crosses the American River in the Sacramento County. The terms $t_{ij}^{D/R}$ and t_{ij}^{O} are calculated following the procedure presented by Bocchini and Frangopol (*30*).

Indirect Cost of Economic Losses As a result of interruptions due to complete/partial bridge closure for repair actions, business activities neighboring the bridge get affected. Following a study by Kliesen (*31*), twice the C_{DVE} is considered for the indirect cost of economic losses.

Cost of Human Casualties Human injuries and deaths are potential consequences of incurred damages to bridges. Dividing the severity of these consequences into four levels, according to NIBS, FEMA (*19*), the general formulation to quantify these adverse consequences, C_H, is:

$$C_H = \sum_{i=1}^{4} C_{SL_i} \times CR_n^{SL_i} \times \text{NPAR} \tag{14}$$

where C_{SL_i} denotes the cost of human casualty for severity level *i* (extracted from Porter et al. (*32*), $CR_n^{SL_i}$ stands for the casualty rate for severity level *i* [given in NIBS, FEMA (*19*)] and condition state *n*, and NPAR is the total number of people at risk [estimated based on relations presented by Caltrans (*33*)].

Cost of Damage to Environment Air pollution, consumption of energy, and the possibility of global warming due to excess emission of carbon dioxide is a consequence of extra gas consumption by vehicles that are delayed by partial/complete bridge closure. The cost of these implications, C_E, can be generally formulated as:

$$C_E = C_{Env} \times \tau_n \times \text{AADT} \times \left[l_{ij} \times En_{V_{ij}} + \sum_{b \epsilon ij} s_{b,ij} \times l_{b,ij} \times En_{V_{b,ij}} - l_{ij} \times En_{V_{ij}^O} \right] \quad (15)$$

where l_{ij} and $l_{b,ij}$ are the length of the path *ij* through the main highway and detour *b*, respectively. $En_{V_{ij}^O}$, $En_{V_{b,ij}}$, and $En_{V_{ij}}$ denote the unit value of carbon dioxide emission at speeds V_{ij}^O, V_{ij}, and $V_{b,ij}$, which are the average velocity of vehicles traveling from point *i* to *j* passing through the main highway before interruption by partial/complete road closure, the main highway after interruption by partial/complete road closure, and detour *b*, respectively. These values are extracted from the study conducted by Gallivan et al. (*34*). Finally, C_{Env}, the unit cost of environmental damage, is considered as \$33.49 per ton for year 2016.

NUMERICAL RESULTS

The framework is implemented for the case study bridge to determine optimal retrofit actions for a wide range of decision-making time horizons. In this regards, four retrofit alternatives are considered:

- Status quo: the bridge is planned to operate as is.
- No ScC with SJ: no scour countermeasure plans are performed on the bridge, while all bridge piers are strengthened using steel jacketing.
- With ScC and No SJ: no steel jacketing is performed on bridge piers, while the scour countermeasure plan described in the previous section will prevent bridge foundation from undermining throughout its lifetime.
- With ScC and with SJ: both steel jacketing and scour countermeasure retrofit plans are implemented on the bridge.

The numerical results are provided in Figure 2. Figure 2*c* shows that the maintenance costs of the four retrofit alternatives are relatively close, however as expected, this cost is slightly more for costlier retrofit actions. Figures 2*b* and 2*c* also show that if no retrofit plan is performed on the bridge, the LCC of repair is more than the LCC of maintenance; this indicates the significance of considering the risk of hazards in the total LCC calculations. Figure 2*b* also indicates that retrofit alternatives with ScC and with SJ, no ScC with SJ, and with ScC and no SJ are the most effective strategies in reducing the risk of hazards in the lifetime of the bridge. However, since performing these plans are initially costly, none of these plans are optimal if the lifetime of the bridge is less than 30 years (see Figure 2*a*). That is, within the range of [0, 30] years of decision-making time horizon, status quo is optimal, which results in the least LCC among all alternatives. It is worthy to mention that if the effect of multi-hazard is ignored in the LCC analysis, i.e., equivalent to zero LCC of repair, the agency is not motivated to take any

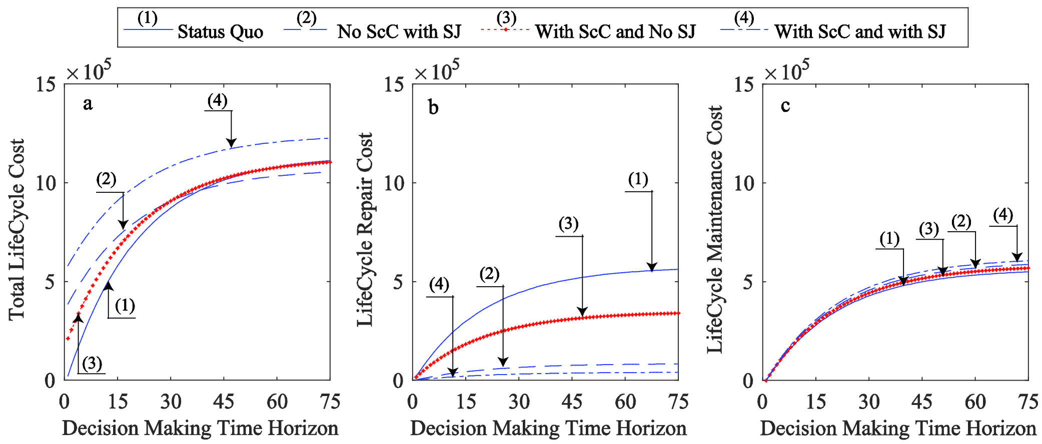

FIGURE 2 Expected (*a*) total; (*b*) repair; and (*c*) maintenance LCC of various retrofit alternative for the case study bridge.

retrofit action for the bridge for any decision-making time horizon, since retrofit plans are initially costly. However, the proposed multihazard framework identifies no ScC with SJ as the optimal strategy for lifetimes beyond 30 years. This results in $79,000 less incurred LCC, if the decision-making time horizon is 75 years.

As the discount rate increases, the effect of future costs on the LCC of the bridge diminishes. This might affect optimal decisions for long-term decision-making time horizons. Considering three values of discount rate, ranging from 1% to 7% (suggested by Beck et al.), variation of the total LCC and optimal retrofit plans for the four retrofit alternatives are evaluated in Figure 3. As a general trend, increasing discount rate reduces the total LCC. This reduction is more significant for the status quo retrofit plan. Since the long-term LCC of repair for this alternative is more than other retrofit plans, reduction in discount rate reduces these long-term costs more for the status quo plan compared to other strategies. This makes status quo the optimal policy for the entire considered decision-making lifetimes, if the discount factor is 7%. On the other hand, if the discount rate is as low as 1%, implementing no ScC and with SJ results in minimum LCC.

In Figure 4, effects of the variation of repair times and AADT on the total LCC of the bridge are shown. The repair times, as a function of each damage state, may vary based on the availability of crew and materials, damage to the nearby infrastructure, and preparedness of the agency in responding to the incurred damages, among others. Based on the statistical characteristics of the repair times described in the previous section, using a Latin Hypercube sampling technique, the required repair times corresponding to non-exceedance probabilities of 0.00, 0.50, and 0.95 are calculated for each damage state *n*, i.e., T_0^n, $T_{0.5}^n$ and $T_{0.95}^n$. In other words, T_α^n is calculated such that the probability of the required repair time at condition state *n* less than T_α^n is α, where $\alpha = \{0, 0.5, 0.95\}$. Figure 4*a* shows as the required repair times increases, the incurred LCCs grows significantly. For example, the total LCC corresponding to $\alpha = 0.95$ is almost twice the total LCC in the case where $\alpha = 0.00$ (representing instantaneous repairs), when the decision-making time horizon is 75 years. In Figure 4*b* the variation of LCCs with respect to three AADT values corresponding to 0.00, 0.40, and 0.80 times the traffic capacity of the bridge reported by Zegeer et al. (*36*), i.e., AADT_δ with $\delta = \{0.00, 0.40, 0.80\}$, is depicted. The results show that LCC values increase considerably with the passing traffic on the bridge in such a way

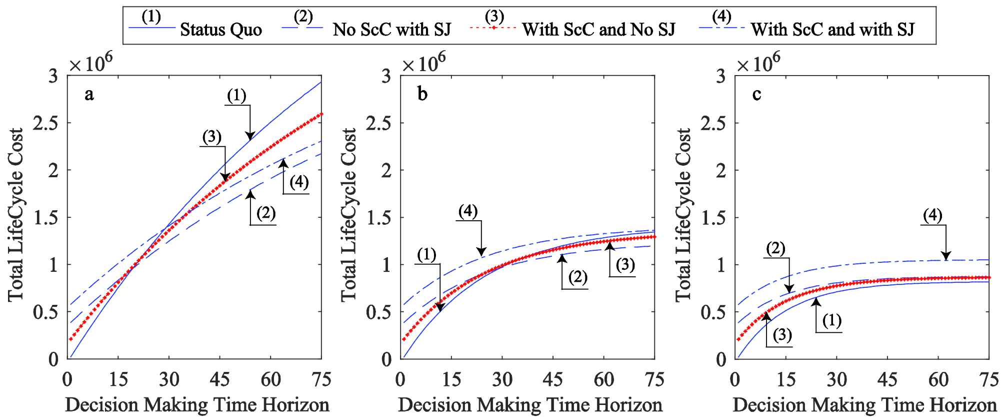

FIGURE 3 Total LCC of various retrofit alternatives with respect to discount rates of (*a*) 1%; (*b*) 4%; and (*c*) 7%.

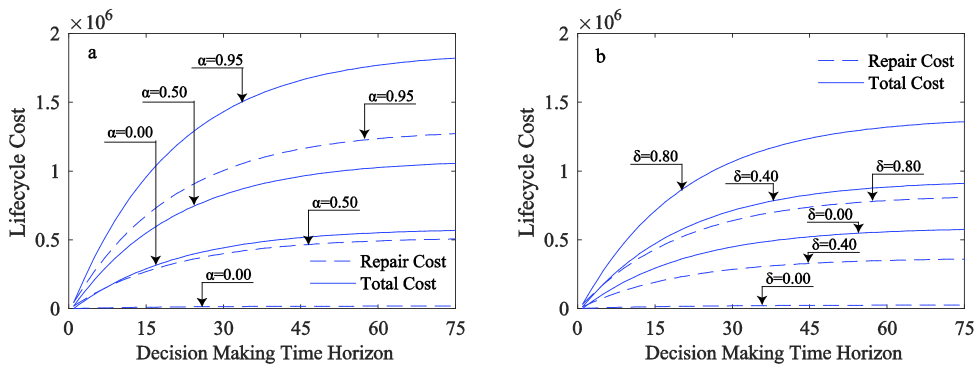

FIGURE 4 Sensitivity of the total and repair LCC of the status quo retrofit alternative with respect to variations in (*a*) repair time durations and (*b*) AADT.

that the total LCC corresponding to $AADT_{0.80}$ becomes as high as three times the total LCC associated with $AADT_{0.00}$, when 75 years of service lifetime is expected from the bridge.

CONCLUSION

This study proposes a multihazard LCC assessment framework to find optimal solutions for retrofit strategies. The possibility of multiple occurrences of multiple types of hazard incidents are probabilistically incorporated in the LCC analysis framework through a recursive function that utilizes damage state-dependent fragility models and repair times. The proposed recursive algorithm is the key to the time efficiency of the framework, which makes it feasible for applications in practice. This methodology accurately determines the expected LCC of hazard-induced consequences, including repair costs of structural damage, human casualties, damage to the environment, user costs of traffic delay, extra vehicle operation and excess emission, and indirect economic losses due to interruptions of affected businesses. The computed LCC of hazards is discounted over the years, and added to the initial cost of applying retrofit actions and the discounted expected LCC of maintenance, to estimate the total LCC of the bridge system under study.

The presented framework is applied to a realistic multispan RC bridge in California exposed to flood and earthquake hazards. The total LCC of several practical retrofit strategies, including steel jacketing of the entire columns and scour countermeasures are evaluated and compared. Considering a wide range of decision-making lifetime horizons for the bridge system under study, the optimal strategies are found as:

- Performing no retrofit action, when the expected lifetime is less than 30 years.
- Applying steel jacketing to bridge columns, if the decision-making service lifetime of the bridge is between 31 and 75 years.

These optimal policies result in the least total LCC. This optimization scheme assures optimality in both the incurred costs and safety of the bridge users. A sensitivity analysis is also performed to characterize the impacts of several key variables on the lifecycle cost values and the optimal retrofit plans. It is shown that lower discount rates, higher required repair times, and larger traffic volumes on the bridge significantly increase the total LCC of the bridge. It is also found that the optimal plan may change as the above variables change. Given the capabilities offered by the proposed methodology, it can greatly help decision-makers in identification of optimal retrofit strategies with higher confidence, and enables them to invest on factors that reduce the lifecycle costs the most.

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Developing a Program to Rank New York City Bridges by Benefit–Cost Ratio

Lessons Learned from New York City Department of Transportation

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The New York City Department of Transportation (DOT) maintains a portfolio of 789 bridges within the five boroughs of the city of New York. Because funding from all levels of government is limited, the DOT anticipates the need to make difficult decisions regarding the allocation of its resources. In an effort to inform this decision-making process, the DOT has begun a study to rank its bridges by benefit–cost ratio (BCR). The DOT's emphasis is not on whether the projects show a BCR of above or below 1.0 in an absolute sense, but on developing a consistent benefit–cost methodology based on readily available data that can produce dependable and replicable rankings across hundreds of projects. This effort has required that the DOT think creatively with regards to travel-time savings, safety, and social benefits. It is the DOT's hope that by developing a proper BCR metric for our bridges, the DOT can better inform decision-makers regarding the relative importance and cost-effectiveness of each of our infrastructure investments. The DOT feels that the lessons learned in producing this analysis would be of great interest to other municipalities facing similarly difficult decisions regarding their aging infrastructure.

The New York City Department of Transportation (DOT) maintains a portfolio of 789 bridges within the five boroughs of the city of New York. The majority of the New York City DOT's capital budget is spent on the repair and reconstruction of bridges—approximately $10 billion in the current 10-year plan. Yet the New York City DOT may require up to twice this amount to meet the need for bridge reconstruction in the next ten years. Because funding from all levels of government is limited the New York City DOT will need to make difficult decisions regarding the allocation of its resources. In an effort to inform this decision-making process, the New York City DOT has begun a study to rank all of its bridges by benefit–cost ratio (BCR)—an ambitious goal that has triggered an intense data gathering effort with regards to bridge usage, user mode, alternate routes, and costs of construction. This effort has also required the New York City DOT to think creatively with regards to assumptions regarding travel-time savings, safety, and social benefits. While still in its early stages, the investigation has already yielded valuable information beyond its initial goals. As of this writing, the New York City DOT has developed rankings for approximately 153 bridges. It is the New York City DOT's hope that by developing a proper BCR metric for its bridges, it can better inform decision-makers at the highest levels regarding the relative importance and cost-effectiveness of each of our infrastructure investments and make the best decisions possible with the taxpayer dollars available. The New York City DOT feels that the lessons learned in producing this analysis would be of great interest to other municipalities facing similarly difficult decisions regarding their aging infrastructure.

BACKGROUND AND HISTORY

In 2014, under the leadership of New York City DOT Commissioner Polly Trottenberg and (now) Executive Deputy Commissioner Joseph Jarrin, the New York City DOT embarked on an ambitious attempt to perform economic analysis on its entire capital project portfolio. The seeds of this initiative were born from the agency's success in using benefit–cost analysis of capital projects to support its federal grant applications for those projects. Due in part to those efforts, the agency was awarded $25 million by the federal government under the TIGER VI grant to support capital investments in Vision Zero, the city's ambitious initiative to reduce annual traffic deaths to zero. By working on this grant application, key city personnel gained experience in the methods and best practices required for successful economic analysis of transportation projects. Agency leadership then made a decision to build upon this capacity and develop a program of economic analysis that would assist the agency in making difficult, yet necessary, future funding decisions. The city's bridge portfolio was chosen as the first focus of economic analysis because it accounted for the largest share of the New York City DOT's capital budget and the largest shortfall in funding. The New York City DOT hopes that an objective economic analysis will help rank projects with the greatest benefits and the lowest costs, and will thereby maximize the benefits of each dollar spent.

Goal of the Study

The New York City DOT's goal was to develop a method to rank its bridge projects using benefits and costs. This is slightly different than the typical objective of a benefit–cost analysis. In a typical benefit–cost analysis the benefits and costs of a "build" scenario are compared to the benefits and costs of a "no build" scenario and a project whose benefits outweigh its costs is generally considered a good investment worthy of construction. Its benefit to cost ratio is above 1.0. However, the goal of this study is to prioritize and rank rebuilding projects for funding purposes using benefits and costs as our criteria, not to determine whether these projects should be rebuilt at all. As such, the New York City DOT's emphasis is not on whether the projects show a BCR of above or below 1.0 in an absolute sense, but on developing a consistent benefit–cost methodology based on readily available data that can produce dependable and replicable rankings across hundreds of projects.

The Study Generally

This study asks whether a bridge should be rebuilt or demolished given its estimated cost, its current usage, and the added distance of the alternate routes available. It monetizes the current usage and the time inconvenience that would be caused by permanent demolition of the bridge. Because it evaluates bridges that have already been built, it allows the New York City DOT to take measurements of how much each bridge is being used and by whom. This allows the New York City DOT to gauge benefits with a greater degree of accuracy than economic studies of future construction or future expansion that are often based on assumptions of future usage and users.

In addition to providing greater accuracy in the prediction of benefits (since they are not predictions but actual measurements of current use), this approach to analyzing already-built infrastructure allows the New York City DOT to think creatively about infrastructure renewal projects in already-built environments like New York City. For example, if an existing four-lane bridge receives a relatively low ranking because it does not have enough users to justify its high reconstruction cost, perhaps the New York City DOT can explore reconstructing it as a two-lane bridge (reducing costs) and thereby improve its BCR. If the majority of a bridge's monetized benefits are due to pedestrians and cyclists, perhaps more space can be devoted to those users in the next incarnation of the bridge and less to motorized vehicles.

It is important to note, however, that this is intended to be a broad analysis to create preliminary rankings for further investigation, and in no case should this analysis by itself be used to justify the demolition or closure of a bridge simply based on its BCR without additional study or analysis. As will be explained herein, this study is based on an analysis of a limited (but important) category of benefits, and it in no way captures all of the benefits and costs that should be taken into consideration before making a decision to close a bridge.

Benefits Generally

A bridge provides many different benefits to society. Some of these benefits are easier to measure and monetize than others. For example, the amount of time saved by using a bridge to cross a river is easier to measure than the value of the scenic view of the river from the middle of the bridge. Table 1 is a list of some of the benefits that bridges provide, and the ones that the New York City DOT has measured for this analysis:

As Table 1 shows, the New York City DOT has monetized five types of benefits for this study. Four of these benefits are quantified and included in the primary BCR metric discussed herein. We have also created a secondary BCR metric called the "safety BCR" which is a combination of the quantitative data included in the primary BCR metric and qualitative data regarding safety.

It is important to note that certain benefits, with potentially large monetary impacts, were not included in this study—chief among these are utilities benefits. Some New York City bridges carry (1) high-voltage power lines that provide electricity, (2) telecommunications infrastructure that provides television, Internet, and phone access, as well as (3) water mains that provide fresh potable water to entire neighborhoods. In addition, each bridge carries a different mix of utilities. While the New York City DOT is aware of the types of utilities on each of its bridges, it does not have information regarding the magnitude and effects of the impacts or the industry specific expertise to monetize these impacts. Therefore, due to the complexity of including this category of benefits in such a broad study, the New York City DOT has decided to not include them in this phase of the analysis but to revisit them once this study has flagged a smaller number of bridges for more rigorous analysis.

Utilities notwithstanding, travel time savings are the primary benefit produced by a bridge. It is the reason why most bridges are built. In our sample of 153 bridges, monetized benefits from vehicle operating costs, emissions reductions, and pavement maintenance savings were less than 5% of total monetized benefits; travel-time savings accounted for the rest. Travel-time savings were not calculated using a traffic model. This could take several days of computer time per bridge due to the complexity of our local roadways, even if complete data was available. A different, more practical approach was required due to the large number of bridges requiring analysis. Instead, an approximation was developed based on alternate distance traveled (in the no build scenario) and average speed per user, such that travel time savings could be computed by spreadsheet. This method will be described below.

TABLE 1

Benefit	Description	Included in DOT Analysis		
		Monetized	Quantified	Qualitative
Faster travel (travel time savings)	Faster time to reach destination when bridge is open	☑	☑	No
Lower vehicle operating costs	Lower vehicle operating costs when extra distance is avoided	☑	☑	No
Emissions reductions	Lower greenhouse gas emissions when additional distance is avoided	☑	☑	No
Pavement maintenance savings	Lower cost of maintaining roads when vehicles travel less distance	☑	☑	No
Safety	Difference between the number and severity of injuries on the alternate route and the build route	☑	No	☑
Utilities	Carrying utilities infrastructure over less distance	No	No	No
Resilience	Faster evacuations during emergencies	No	No	No
First responder access	Faster emergency response time	No	No	No
Recreation	Enjoyment of crossing bridge, view of bridge, or view from bridge, etc.	No	No	No
Property values	Change in nearby property values depending on presence of bridge	No	No	No

Choice of Candidate Bridges

Despite the fact that the benefits analysis was limited to five categories of benefits, data collection (pedestrian–bicycle counts and measurements of alternate route distance) for nearly 800 bridges would have been unrealistic. The data requirements of this analysis are discussed in further detail in the "Number of Users" section and the "Alternate Route" section below. Therefore, the New York City DOT narrowed the list of candidates by first focusing on those bridges which were unfunded or underfunded in its 10-year plan. The New York City DOT narrowed this further by removing critical bridges such as the East River bridges, bridges supporting arterial roadways, and bridges on emergency escape routes since it was thought that these bridges, because of their critical nature, would most likely find funding at some point in the future. Due to available funding and time constraints during the New York City DOT's initial data gathering phase, this number was further reduced to 194. Intensive data gathering on these bridges began in early May of 2015 and was completed in late June 2015.

METHODOLOGY

As previously stated, this study's emphasis is not on whether projects are truly above or below 1.0 in an absolute sense, but on developing a consistent benefit–cost methodology based on readily available data that can produce dependable and replicable rankings across hundreds of projects. This distinction heavily influences our BCR methodology and so it is important to understand its impact. Because of this distinction, it is far more important that the methodology used herein prioritize uniform standards for measuring costs and benefits for each bridge over methods that may potentially be more accurate for some bridges but cannot be applied consistently to all bridges. For example, if the New York City DOT was in possession of detailed traffic studies which would allow it to calculate traffic delays for some bridges but not for others, they would not be used because they would violate the need for uniformity of measurement. Instead, the New York City DOT tried to develop a travel-time savings methodology that would approximate these results and could be applied to all of the bridges uniformly. In this manner it could guarantee that all of the bridges had been rated according to the same standards and that it was comparing apples to apples. As a result, what the DOT has developed can be more accurately described as a benefit–cost *ranking* rather than a benefit–cost *ratio*. This metric is an excellent tool for prioritizing hundreds of projects quickly with easily obtainable data, but should not be used to decide whether a project's costs outweigh its benefits in an absolute sense. As discussed earlier, many benefits, such as utility benefits, were left out of the calculation due to the difficulty in gathering such a large amount of data on so many projects simultaneously. Nevertheless, despite the omission of these important benefits, over 85% of the 153 projects analyzed produced a benefit-to-cost ratio of greater than 1.0.

This analysis follows a standard benefit–cost methodology with the present value of the benefits being divided by the present value of the costs. As with all benefit–cost analyses, the BCR is derived by comparing the costs and benefits of the build condition to the costs and benefits of the no-build condition. In the New York City DOT's case, all of the bridges studied were already built, therefore the no-build condition had to account for the demolition of the existing bridge since a deteriorating bridge cannot be allowed to collapse of its own accord. For this reason the no-build condition is also referred to as the demolition scenario. More detail on the no-build condition can be found below. The formula for this benefit–cost analysis is set forth below (Equation 1). The assumptions used in the formula will be discussed first, followed by a brief discussion of the data gathering process. The formula will then be applied, and its components explained, using a real-world example.

$$\text{BCR} = \frac{\text{Present Value (Benefits of Reconstruction – Benefits of Demolition)}}{\text{Present Value (Cost of Reconstruction – Cost of Demolition)}} \tag{1}$$

The No-Build Condition

In most benefit–cost analyses the no-build condition indicates no action was performed to improve or change the existing condition. The benefits and costs of no action are measured so that they can be compared with the benefits and costs of the build condition which by definition involves an action of building or changing the existing condition. A typical example of this is where a highway project is being considered. The no-build condition assumes no highway is built and cars have to travel on the local road. The benefits of the local road are compared to the benefits of the highway and the costs of the highway are compared with the costs of the local road. The costs of the no-build are typically low, since nothing was built, however, depending on the no-build condition, maintenance might be high over the applicable life cycle. The no-build scenario used is this study is unusual in that it starts with an already built asset that is nearing the end of its useful life and must be closed in the near future for safety reasons if nothing is done to rehabilitate it. Moreover, this no-build scenario includes the cost of the demolition of the asset, since it was thought that a scenario where the bridge is merely allowed to collapse of its own accord was unrealistic. Whereas the build scenario (reconstruction scenario) assumes construction of the new bridge 2 years in the future (2 years for design before construction occurs). The demolition scenario assumes closure and demolition 6 years in the future, since it is assumed that demolition will be more politically contentious than rebuilding. Yearly maintenance costs are included in both scenarios, except that in the demolition scenario maintenance ends on the demolition year, as do benefits that accrued while the bridge was still in operation. It is because of this demolition no-build scenario that this analysis also includes an alternate route measurement which is used to determine the time value lost by the users of the bridge over the course of the no-build scenario.

Present Value

The present value is the current worth of a future sum of money. That future sum of money is discounted by the discount rate. The discount rate can be understood to be the cost of capital or the minimum return on the investment (over the long run) to meet expectations. For New York City, the cost of capital is the coupon on New York City Capital Bonds. That coupon in 2015, when the first batch of bridges was analyzed, was 3.6%. While this number is rather low when compared to the discount rate required for federal grant applications (7%), the value of the discount rate matters very little when the goal is to provide a relative BCR ranking. The discount rate is extremely important where being above or below 1.0 can make or break a project, but when the goal is to rank projects amongst each other the discount rate isn't as critical since it won't change the ranking of the projects relative to each other. For the purpose of this study the New York City DOT applied a discount rate of 3.6% to all projects.

Expected Useful Life

This model assumes an expected useful life of 40 years for a vehicular bridge and 70 years for a pedestrian bridge. The New York City DOT's bond commitments require a 40-year minimum useful life for its bridges before they can be substantially rehabilitated. However, its bridges are built to withstand 50 years of use before they require substantial rehabilitation. This study opted for the more conservative measure of 40 years. Pedestrian bridges are expected to last longer without the need for substantial rehabilitation. New York City DOT's Bridges Division expects the typical pedestrian bridge to perform safely for at least 75 years without major rehabilitation. The model conservatively assumes 70 years for pedestrian bridges.

Benefits of Reconstruction

The primary benefit derived from any bridge is the travel time savings enjoyed by the users of the bridge over the life of the bridge. In order to calculate this value New York City DOT needed to determine (1) the average dollar value of time for New York City residents; (2) the number of vehicles, bicycles, and pedestrians crossing each bridge; (3) the average velocity of each mode type; (4) the bridge length; and (5) the alternate route length. By identifying the alternate route and calculating the time saved by using the bridge, New York City DOT can place a monetary value on the travel time savings created by the bridge.

Value of Time

On June 29, 2016, the U.S. Bureau of Labor Statistics released a New York City Economic Summary Report. It states that the average wage for all occupations for New York City was $28.84/h as of May 2015. According to federal government BCA guidance this number should be halved for nonbusiness travel. Inflating this number from May 2015 to June 2016 by the Consumer Price Index resulted in an hourly wage of $14.56/h. It should be noted that when calculating the value of time for trucks, the hourly value of time does not need to be halved since it is assumed that truck travel is business travel. During the data-gathering portion of this study motor vehicle classifications where not obtained, therefore truck travel was valued at the same hourly rate as private motor vehicle travel.

Number of Users

It was important that the benefit analysis reflect not just motor vehicle data, but data for all users including pedestrians and cyclists. Since there was no recent pedestrian or cyclist information on the 194 bridges in the first round of the study, the New York City DOT used a consultant to obtain the data. Miovision video processing was used to count vehicles, cyclists and pedestrians at each bridge. Counts were conducted from 7 a.m. to 7 p.m. on a weekday (Tuesday, Wednesday, or Thursday) and on a weekend day (Saturday or Sunday) on each bridge for a total of 24 h of video footage. Some wider bridges required multiple cameras to capture all of the lanes and they produced more video footage. Video collection was not conducted on rainy days as this would obviously distort the data.

The results were converted into annual average daily traffic (AADT) by assuming that overnight usage (which was not recorded) was approximately 25% of daytime usage. Some highlights of these results: the highest pedestrian count was at East 167th Street and Grand Concourse in the Bronx with an AADT of 37,102; the highest bicycle count was at the Grand Army Plaza entrance to Prospect Park in Brooklyn with 10,062; the lowest pedestrian count was at a small pedestrian bridge in Inwood Park in Upper Manhattan with an AADT of 9; the highest vehicular count was on Northern Boulevard over Alley Pond Creek in Queens with 47,453; the lowest vehicular count was on 44th Avenue over the CSX train tracks in Queens with 601.

Average Speed per Mode

New York City DOT designs its roadways, crossings and signal timings based on an average pedestrian walking speed of 3 ft/s or 2 mph. In order to remain consistent with the DOT's policy this speed was used. For the purpose of this study motor vehicle and bicycle speed was assumed to be 9.5 mph. This is slightly faster than speeds measured for both bicycles and motorized vehicles in the Manhattan Central Business District (CBD), but since most of the bridges were located outside of the CBD the New York City DOT assumed a slight improvement in speed (*1*).

Bridge Length

Here the New York City DOT used the length of the deck area which is a record kept by the Bridges Division.

Alternate Route Length

The alternate route used for each bridge is the shortest route available between the two ends of the bridge using any roadway (other than the bridge itself) regardless of street directionality. See Figure 1. This methodology sets a consistent and replicable standard for determining alternate routes for vehicles, bicycles and pedestrians. In the absence of a rigorous traffic analysis for each bridge, such as an origin–destination study (which would be conducted as part of a rigorous benefit–cost analysis conducted by an urban planner or an economist if the demolition of any one bridge was seriously being considered), any other start and end points would appear arbitrary and more importantly would reduce the consistency of comparison that is necessary for this study. It also provides a practical alternative to computer traffic models which can take several days to run (for just one bridge) given currently available technology. Running a model on 153 bridges would be impractical. Finally, computerized traffic models typically measure motor vehicle impacts and fail to address impacts to other users such as pedestrians and cyclists. These can be substantial oversights, especially in an urban environment. The methodology described herein allows New York City DOT to take into account not only the motor vehicle travel-time benefits, but also the pedestrian and cycling benefits so that impacts to all users can be taken into account.

Despite the benefits listed above, there are some shortcomings to be aware of when using this methodology instead of a traffic model. For one, it does not measure delays to motor vehicle users caused by traffic congestion on the alternate route. This may be particularly pronounced in areas with already high traffic volumes along the alternate route, but less so in areas where alternate routes are not at capacity. In order to compensate for this, the New York City used an average vehicular speed only slightly faster than that of the typically congested CBD (9.5 versus 8.7 mph). While this may adequately adjust for delays to existing users of the bridge, it does not take into account delays to existing users of the alternate route. This may result in undercounted motor vehicle delays. However, the methodology counteracts this undercounting by overestimating the length of the alternate route and resulting motor vehicle delays by assuming that all former bridge users will double back to the other side of the bridge as shown in Diagram A. It is believed that along all but the most congested alternate routes these competing impacts all but cancel each other out. However, this has yet to be rigorously proven.

FIGURE 1 Alternate route measurement: measurement of the alternate route on the 55th Avenue pedestrian bridge.

In sum, the author believes that this method is the most consistent, objective, accurate, and efficient method of measuring travel time savings short of performing a time consuming origin–destination study for all users.

APPLICATION OF METHODOLOGY TO SAMPLE BRIDGE

Diagram A above shows a pedestrian bridge in Elmhurst, Queens. This is one of the 153 bridges the New York City DOT analyzed using the methodology described above. This bridge will serve as an example of how to utilize this methodology. It is a simple example as there is only one type of user: pedestrians. The amount of the time needed to cross this bridge will be calculated first. The amount of time for one crossing will be monetized. We will then use this to determine the value of all of the crossings performed in a day (using AADT), then a year, then over the expected life of the bridge. The results will be discounted to their present value. This will then be compared to the no build or demolition scenario where the distance to be crossed (the alternate route) is much greater, takes longer, and therefore has a greater monetized value per crossing.

Benefits of Reconstruction

The bridge length, as indicated by the New York City DOT's records and verified by Google Maps, is 138 ft. At 2 mph or 3 ft/s it will take a pedestrian 46 s to cross this bridge. At a value of \$14.56/h this crossing has a time value of \$0.186 per person. The pedestrian bridge pictured above has an AADT of 853 which when multiplied by \$0.186 equals \$158.66 in crossing time per day. This number in turn multiplied by 365 days equals \$57,910. The present value of \$57,910 over 70 years (this is a pedestrian bridge) is \$1,473,325. See calculations below:

Calculation 1:

Present value of travel time over bridge = $PV\ (L_y\ D_y\ A((D_B/V_P)T_v\ ))$

Where:		Example:	
PV	= present value	PV	= present value uses discount rate of 3.6%
L_y	= assumed bridge life in years	L_y	= 70 years
D_y	= number of days in a year	D_y	= 365 days
A	= AADT	A	= 853 pedestrians
D_B	= distance to cross bridge in feet	D_B	= 138 ft
V_P	= velocity at which pedestrians walk	V_P	= 3 ft/s
T_v	= value of time	T_v	= \$14.56/h

When this value is inserted into Equation 2 it should be stated as a *negative* because this is not a gain in time, but time *lost* by pedestrians in crossing the bridge. If the time lost crossing the bridge is less than time lost on the alternate route, then there is a *positive* benefit. See below:

$$\text{BCR} = \frac{(–\$1{,}473{,}325) – (\text{PV of Benefits of Demolition})}{\text{PV (Cost of Reconstruction – Cost of Demolition)}} \tag{2}$$

Benefits of Demolition

The benefits of demolition in Equation 3 were calculated exactly the same as benefits of reconstruction, except that pedestrians must now walk 2,700 ft (15 min) rather than 138 ft (43 s) to reach the other side of the bridge. All other values remain the same. The PV of the benefits of demolition is \$28,832,805. Again, this should be stated as a negative or a *disbenefit,* see Equation 3.

$$\text{BCR} = \frac{(–\$1{,}473{,}325) – (–\$28{,}832{,}805)}{\text{PV (Cost of Reconstruction – Cost of Demolition)}} \tag{3}$$

Solving for this:

$$\text{BCR} = \frac{\$27{,}359{,}480}{\text{PV (Cost of Reconstruction – Cost of Demolition)}} \tag{4}$$

The PV of the travel time savings benefits of this pedestrian bridge is \$27,356,559. This method was also followed for vehicular bridges. The travel-time savings from vehicles and bicycles were also added to the pedestrian totals. Finally, motorized vehicles were assumed to have an occupancy of 1.4 persons (the New York City average), therefore the AADT was multiplied by that amount in order to determine the number of actual users being impacted.

Cost of Reconstruction

Cost of Reconstruction = PV (Bridge Replacement Cost) + PV (Annual Maintenance Cost) (5)

The cost of the reconstruction scenario is the sum of both the one-time cost to replace the bridge and the recurring annual cost to maintain the bridge throughout its useful life. The cost of replacement is based on a standard cost-per-square-foot of deck area developed by the Bridges Division. This per-square-foot-cost includes both hard and soft costs, and is an average based on 15 years of bid price data adjusted for time. It is based on average bid prices prior to 2014. The same per-square-foot-cost of construction is applied to both pedestrian and vehicular bridges. Deck area data for all bridges was provided by the Bridges Division. In addition, a cost equivalent to 2,000 ft^2 of deck area is added to all non-Americans with Disabilities Act (ADA) compliant bridges in order to account for the cost of ADA accessible ramps. This cost does not apply to all pedestrian bridges since some of these bridges cross at grade over sunken railroad and expressway trenches. The pedestrian bridge in Diagram A is estimated to cost $5,200,000 to replace.

The New York City DOT estimated yearly maintenance costs since actuals were not available on a per bridge basis. The annual cost of maintenance was assumed to be 1% of the cost of replacement. This includes but is not limited to sweeping, clearing storm drains, snow plowing, painting, and minor repairs such as replacing joints and bearings. For the pedestrian bridge in Diagram A, this would result in a yearly maintenance cost of $52,000. For the PV calculation, it was assumed that all of the bridges would be built 2 years in the future; therefore, the PV of the bridge replacement is $4,676,534. The PV of the maintenance costs over the 70 year life of the bridge is $1,322,961. See Equation 5a below.

Cost of Reconstruction = $4,676,534 + $1,322,961 (5a)

The PV of the maintenance cost and the replacement cost was summed ($5,999,495) and entered into Equation 6 below:

$$\text{BCR} = \frac{\$27{,}359{,}480}{\$5{,}999{,}495 - \text{PV Cost of Demolition}} \qquad (6)$$

Cost of Demolition

The cost of the demolition scenario is the sum of the PV of demolition and the present value of maintenance through the year of demolition. Based on previous bridge demolitions in New York City, the cost to demolish a bridge was assumed to be 4% of the cost to replace the bridge, with a minimum cost of $500,000. An increase in this percentage would have the effect of increasing the BCR of reconstruction. Intuitively, this makes sense: if the cost to demolish increases and the cost to rebuild remains the same, then the rebuild scenario becomes more appealing. The DOT assumed demolition would occur 6 years from the date of the study; therefore the PV of this bridge demolition was $404,400. Maintenance was calculated to accrue for the next 6 years until demolition: $52,000 per year for 6 years. The PV of the demolition cost and maintenance through the demolition date was $680,577. See Equation 7.

$$\text{BCR} = \frac{\$27{,}359{,}480}{\$5{,}999{,}495 - \$680{,}577} \qquad (7)$$

Solving for the above:

$$\text{BCR} = \frac{\$27{,}359{,}480}{\$5{,}318{,}918} = 5.14 \qquad (8)$$

This pedestrian bridge has a benefit cost ratio of 5.14. This places it in the 60th percentile of the rankings.

Additional Benefits

Given the AADT and alternate route data that was compiled for the analysis above, it was also easy to calculate vehicle operating costs savings, emissions reductions benefits, and pavement maintenance savings using TIGER grant methodology and valuations (*2*, *3*). These benefits were included in the BCRs for vehicular bridges. However, their impact is miniscule compared with travel-time benefits. For example, on average they amounted to no more than 5% of total benefits combined. For this reason we will sidestep a discussion of these benefits and refer those who are interested in further detail to the TIGER guidance documentation (*2*, *3*).

Safety Benefits

The New York City DOT has a deep commitment to safety, starting with our commitment to Vision Zero, Mayor de Blasio's ambitious goal to eliminate all pedestrian and traffic deaths in New York City. The New York City DOT felt it was necessary to attempt to measure the relative safety implications of closing a bridge and find a way to include it in the rankings. For example, if a pedestrian bridge were to be closed, would this put pedestrians on a particularly dangerous alternate route? Would closing a vehicular bridge divert cars to areas with a heavy pedestrian presence? Would the injury rate increase or would the added traffic bring vehicles to a crawl and thereby reduce injuries to all users? Answering these questions would require a level of analysis that would take months to undertake for one bridge, let alone for 200. Therefore, the New York City DOT looked for a way to approximate the potential safety impacts with data that could be readily attained.

The New York City DOT maintains a computerized database of traffic-related injuries and fatalities called Traffic Safety Data Viewer. The data is mapped and joined via GIS to the intersection nearest where the crash occurred. The data contains severity and mode of travel. Using Traffic Safety Data Viewer, the DOT was able to calculate the number and severity of injuries along the alternate routes and the existing bridge routes. Using 5 year averages, the New York City DOT then monetized the yearly cost of these injuries using TIGER methodology for the monetization of safety benefits. For each bridge, the PV of injuries on the build route was subtracted from the PV of injuries on the no-build route. This calculation provides a relative measure of the safety benefit of the build condition.

$$\text{Safety Benefit} = \text{PV of Injuries on No-Build Route} - \text{PV of Injuries on Build Route} \qquad (9)$$

When the safety benefits were being calculated for pedestrian bridges, only pedestrian and bike injuries on the alternate route were counted. If the alternate route for pedestrians was a continuous sidewalk without exposing the pedestrians to a street crossing, the safety benefit was set equal to zero.

As one would expect, once these benefits were added most bridge BCRs increased. The New York City DOT called this BCR the "safety benefit BCR" and it was tracked separately from the "plain BCR" (the travel-time savings-based BCR). The safety BCR boosts the rank of those bridges which have particularly dangerous alternate routes. It emphasizes that a bridge not only has travel-time impacts, but its existence also has a safety impact to its users. While most bridges remained within 3 or 4 places of their former ranking, some bridges jumped significantly in the rankings once safety was included. One bridge in Brooklyn, where the alternate route included a particularly crash prone intersection, moved 48 spots higher in the rankings, and another in Manhattan moved 30 spots higher for similar reasons. This indicates a particularly dangerous condition along the alternate route, and is a red flag alerting us that if the bridge is demolished, remedial safety measures should be considered along the alternate route so as to lessen safety impacts. This is an important finding and it is the DOT's hope that the safety BCR will be used in conjunction with the plain BCR to determine prioritization and its impacts.

HOW THE RANKINGS ARE BEING USED

As discussed earlier, the ranking methodology used herein should not be used to determine whether to eliminate a project but only to rank them; however, it may be used to flag those projects where a thorough benefit–cost analysis should be considered. The DOT is now conducting an in-depth benefit–cost analysis of the bridges in the bottom 10% of the plain BCR rankings. Traffic modeling will be used in these in-depth analyses, and a greater range of economic benefits will be explored—such as benefits to property values, utilities, economic benefits to the local economy, etc. The presence of certain bridges in this category has reinforced the opinions of some of the DOT's senior engineers regarding the need (or lack thereof) for some smaller bridges. For example, the metric supports the DOT's recent decision to close the 216th Street pedestrian bridge (ranked 148 out of 153 bridges). The fact that the BCR ranking is accurately predicting the conclusions of the DOT's senior engineers is an encouraging sign. It shows that the BCR ranking method is producing a credible and useful metric. The rankings are also forcing us to ask hard questions about the cost of the bridges in the bottom 10th percentile. Perhaps the answer is not to find new benefits or demolish a bridge, but to reconstruct at a scale that reflects their actual usage. By reducing their size and cost these low-performing bridges may be saved.

CHALLENGES

The travel-time savings methodology on which this model relies cannot be applied to bridges whose sole function is recreational, not utilitarian. Of the 193 bridges the New York City DOT originally set out to rank, 57 were small bridges, many of them foot bridges located within parks. The alternate routes were often meandering and several orders of magnitude longer than the build routes because paths within parks are not designed in a grid with efficiency of mobility in mind, rather they are purposely designed to lengthen one's enjoyment of the park experience and to channel visitors through certain limited paths. Blind application of the BCR ranking methodology results in astronomical BCRs. A different methodology, yet to be devised, is required. An accurate valuation of recreational time is also necessary. During recreational time, a longer route is not necessarily a less-desirable route. The value of one's time is different. The proper cost of a footbridge overlooking a lake is more than just the time saved by crossing the bridge. But how much is it worth? Such questions might never be resolved by resorting to economics and may properly lie in the policy making or political sphere. There are limits to what can or should be measured, and perhaps that is a positive result.

CONCLUSION

The New York City DOT's benefit–cost ranking metric is a reliable measure based on readily available data that can produce dependable and replicable rankings across hundreds of projects with minimal data inputs. It is a powerful tool that can help decision-makers with difficult choices regarding the allocation of scarce resources.

ACKNOWLEDGMENTS

The author of the study thanks Commissioner Polly Trottenberg and Executive Deputy Commissioner Joseph H. Jarrin for their support of this effort, and to the members of the Bridges Division Executive Committee for their feedback and assistance.

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Implementation of AASHTOWare Bridge Management 5.2.3 to Meet Agency Policies and Objectives for Bridge Management and Address FHWA Requirements

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Ensuring a functional, safe, and resilient transportation network is a vital objective of transportation agencies. Two major challenges that make it difficult to accomplish this goal are (1) limited resources, specifically funding, and (2) the increase in maintenance and preservation needs of bridges as they age. Additionally, construction and maintenance costs have continued to rise over the last decade and when combined with steady or declining revenue, this results in a reduction in purchasing power. To maximize the impact of maintenance and preservation work, bridge managers, planners, and decision-makers must have the data and tools available to determine the optimal allocation of resources between competing bridges in a transportation network. Furthermore, they must identify the optimum timing to do the work. Faced with this need for data-driven decision-making it is crucial that transportation agencies have effective decision-making processes, procedures, and tools, such as bridge management systems, to manage their network of bridges.

While bridge managers recognize that aggregate information is advantageous for providing high-level executive reports and general estimates for funding requirements, they acknowledge the limits of bridge management practices that only use a single criterion to predict needs. Identifying several key bridge management criteria, with well-defined goals, enables decision-makers to clearly distinguish differences in the functionality and other attributes of bridges that may otherwise go unnoticed. Therefore, there is a clear need to pursue and develop supplemental bridge management practices that utilize multiple criteria and trade-off analyses.

AASHTOWare Bridge Management (BrM) 5.2.3 is an excellent bridge management software solution that assists engineers, managers, and decision-makers in the selection and timing of preservation, rehabilitation, and replacement projects for their structures. BrM 5.2.3 provides a robust, data-driven approach to project selection and therefore, it is imperative that the software is configured to meet the specific needs, policies, and practices of the agency. These configurations include, but are not limited to, the utility tree, deterioration rates, benefits and actions performed, funding, and performance measures. This paper serves as a high-level guide of the functionality and bridge management modules in BrM 5.2.3

AASHTOWare Bridge Management (BrM), formerly known as Pontis, is a robust bridge inspection and management system licensed by a majority of the state departments of transportation (DOTs) as well as other transportation organizations as shown in Figure 1.

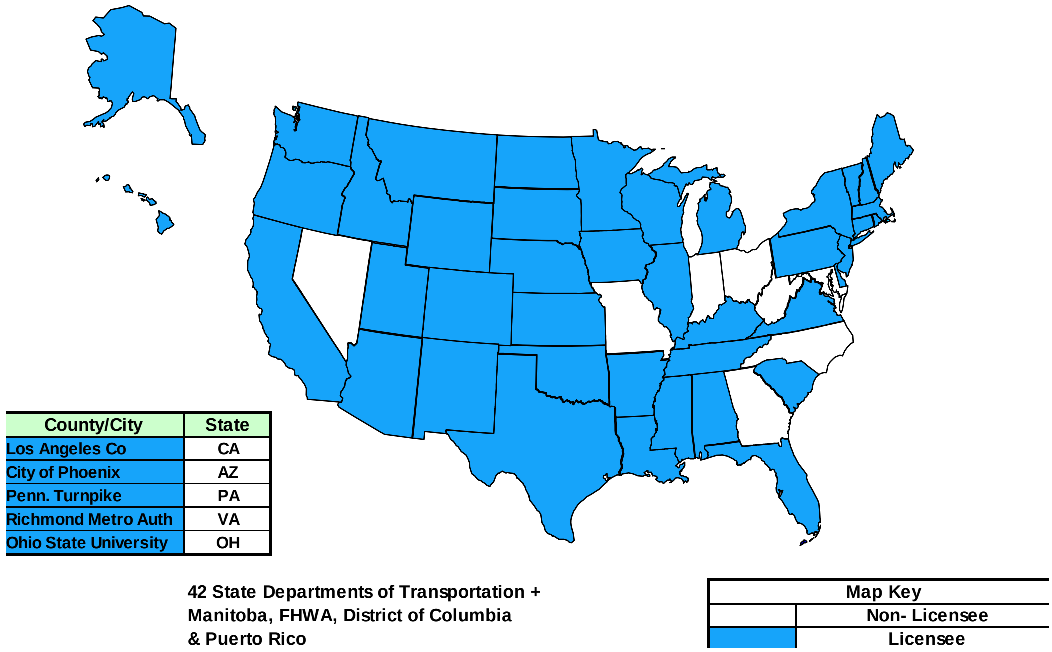

FIGURE 1 Agencies participating in BrM.

BrM, and the complete AASHTOWare software suite, is unique in that the software is designed by transportation professionals for transportation professionals, supporting a collaboratively developed bridge management solution. Pooling knowledge and resources enables costs to be distributed across many users, so agencies procuring AASHTOWare realize vast cost savings and receive quality software that matches precise needs. The licensing agencies form the product user group, and five members of the user community serve as members of the governing task force which oversees the enhancement, maintenance, and support activities. AASHTO contracts with Bentley Systems, Incorporated, to perform software development, maintenance, and support.

BrM along with AASHTOWare Bridge Design and Rating (BrDR), cover the entire bridge life cycle as shown in Figure 2.

There are two versions of BrM, an enterprise version and a workstation version. In the enterprise version, the software is hosted from a server and is accessed through the Internet or agency intranet. In the workstation version, the application is served up from a local laptop. Bridge data is easily shared back and forth between the two versions. In both versions, the database is open to the agency licensee to allow easy access to their data using either Oracle or SQL Server. Both versions support reporting capabilities, using Crystal Reports to produce report templates, and both versions can run the optimization module.

BrM has undergone a significant overhaul from the old Pontis, specifically upgrading it to a multicriteria analysis rather than just a condition-based optimization, as well as a new user interface. BrM is comprised of several modules to address the aforementioned life cycle which include bridge inspection, tunnel inspection, work history, projects, programming, performance measure dashboards, and a scenario explorer to allow agencies to do trade-off analysis.

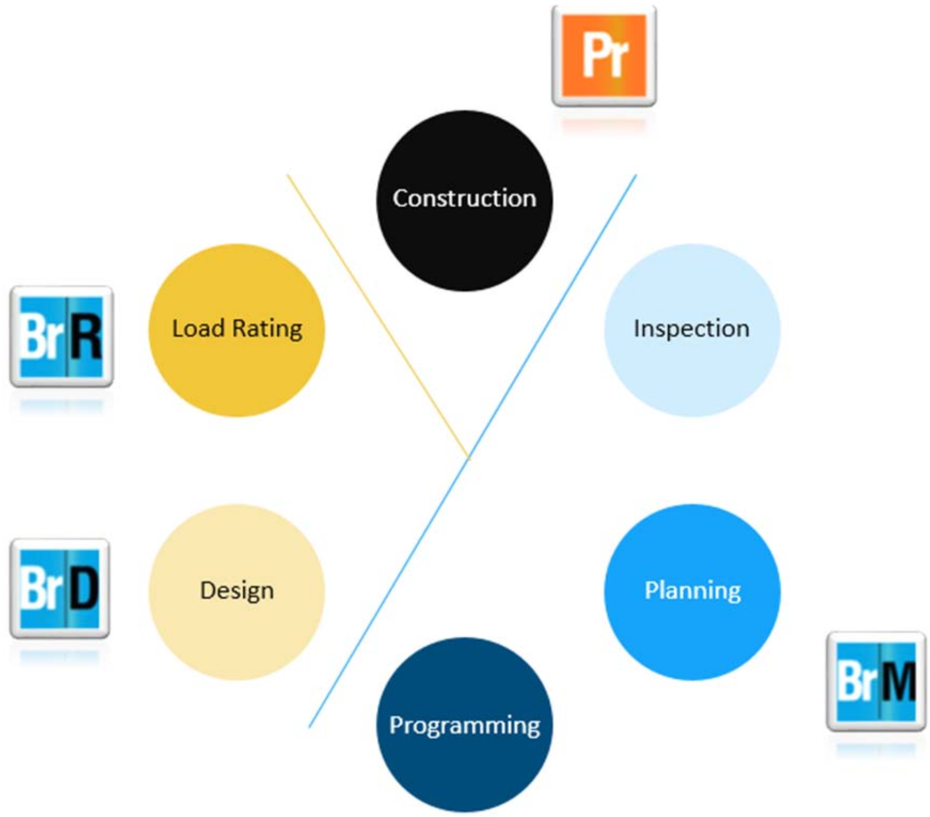

FIGURE 2 AASHTOWare and the bridge life cycle.

BRIDGE MANAGEMENT MODULES

Figure 3 is an illustration of all the components and modules that comprise the management portion of BrM. The graphic is shown as a pyramid to represent the typical work flow required to create a program and run an optimization in BrM with the items at the base completed first. In addition, a brief description of each part is provided and several will be discussed in more detail in subsequent sections.

- **Inspection Data.** Inspection > Condition (and others). Supports inspection data about the current condition of the structure.
- **Work Candidates.** Inspection > Work. Enables the user (inspectors, planners, managers, etc.) to define recommendations for work to be performed on a structure.
- **Default Utility Tree**. Admin > Modeling Config > Utility. Defines the multiple criteria (conditions, risks, and other attributes of a structure) that are used during the optimization process.
- **Utility Weight Profiles.** Admin > Modeling Config > Weights Profile. Allows the user to temporarily reweight the utility tree to target certain objectives.
- **Element Deterioration Rates**. Admin > Modeling Config > Weights Profile. Enables the user to determine and model the deterioration curves for bridge elements.

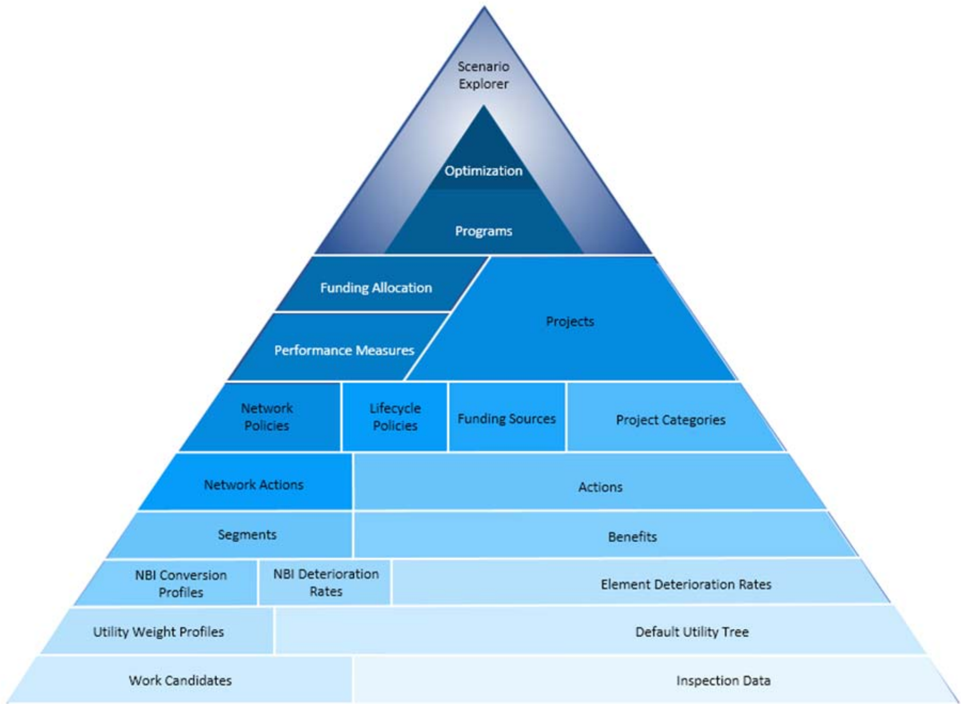

FIGURE 3 Illustration of the modules involved in the BrM optimization.

- **National Bridge Inventory (NBI) Deterioration Rates**. Admin > Modeling Config > NBI Deterioration Models. Enables the user to determine and model the deterioration rate for NBI components using a time-in-state method.
- **NBI Conversion Profiles.** Admin > Modeling Config > NBI Conversion Profiles. Enables the user to determine and model the deterioration rate for NBI components by converting the deterioration curves of the elements related to each component.
- **Subdivisions**. Admin > Modeling Config > Subdivision Profiles. User defines how to break up structure groups for data in programs.
- **Benefits**. Admin > Modeling Config > Benefits. User defines how the condition or other attributes should change when work is done.
- **Actions and Network Actions**. Admin > Modeling Config > Actions. User defines which benefits correspond to work done (actions), and the costs associate with that work (action).
- **Network Policies**. Admin > Modeling Config > Network Policies. Enables the user to identify any number of valid combinations of actions and the conditional logic that determines when those actions will be performed.
- **Life-Cycle Policies**. Admin > Modeling Config > LCCA Policy Rules (and others). Enables the user to define how their agency would normally plan for and program work over the life cycle of a bridge to determine the future benefit from performing work today.
- **Funding Sources.** Projects > Manage Funding > Funding List. User defines the various funding sources.
- **Project Categories**. Admin > Modeling Config > Project Categories. Allows users to filter projects and work candidates.

- **Projects**. Projects > Create/Edit Project. User identifies work to be done on one or multiple structures.
- **Programs**. Programs > Create/Edit Programs. User identifies and groups a set of projects into programs.
- **Performance Measures**. Programs > Performance Measures. User determines the targets, thresholds, and other metrics to achieve during the optimization.
- **Funding Allocation**. Programs > Funding Allocation. User defines how much funding the optimizer should use for each year of a program.
- **Project Allocation**. Programs > Assign Projects. Enables the user to assign and/or freeze projects to a program.
- **Optimization**. Programs > Program Planning. The user selects which parameters, which have been setup in the previous modules, will be used during the optimization process. The computer then performs the optimization process using the selected parameters and objectives.
- **Scenario Explorer**. Programs > Create/Edit Scenarios. Allows the user to run multiple optimizations using multiple values for a given parameter or objective.

Additionally, transportation agencies are concerned about meeting the transportation asset management plan (TAMP) requirements set forth by the FHWA. Figure 4 is the same pyramid illustration but with TAMP requirements mapped to specific portions that address FHWA requirements.

FIGURE 4 Illustration of the modules involved in the BrM optimization with links to TAMP requirements.

Finally, it is imperative that users review and update the configuration parameters of BrM to ensure that the optimization is providing valid results specific to their needs. Users should also be cognizant of the fact that the parameters set upon initial configuration to provide boundary conditions for the optimization (such as deterioration models and life-cycle policies), may need to change over time as their agency collects more data. These configurations need to be periodically verified and updated with the most current data and policies to ensure valid results. Figure 5 shows pyramid highlighting the configurable modules.

INSPECTION

The bridge inspection module, as seen in Figure 6, allows inspectors to track their notes for each element of the bridge and to define defects and protective systems. This module follows the FHWA guidelines for NBI and National Bridge Element (NBE) submittals. Agencies can also copy this screen, make configuration changes, and then use this customized screen to track the data important to them.

The 5.2.3 release of BrM includes a tunnel inspection module which was incorporated via an underlying framework to support varied asset types. The tunnel inspection module (Figure 7) follows the FHWA National Tunnel Inventory (NTI) standards.

Inspectors can identify work needs with their inspections and as identified during planning, through the work candidates module as shown in Figure 8. The software can help generate approximate costs for that work or the user can define the costs. Some agencies use these fields to inform inspectors of the plan for the structure, which in turn helps reduce the probability that specific work and needs are duplicated.

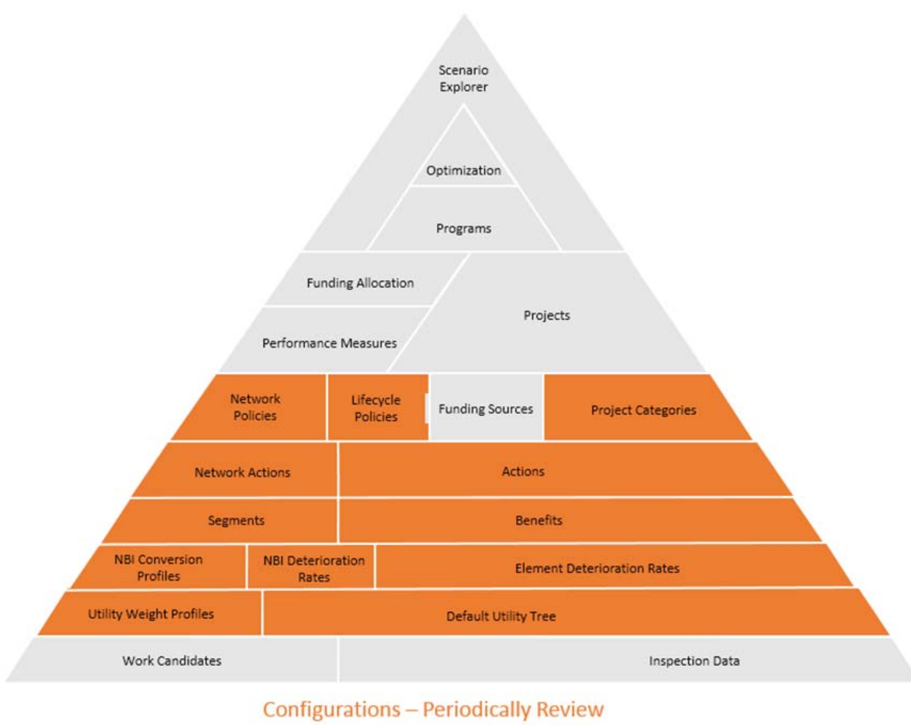

FIGURE 5 Illustration showing modules that require configuration for optimization.

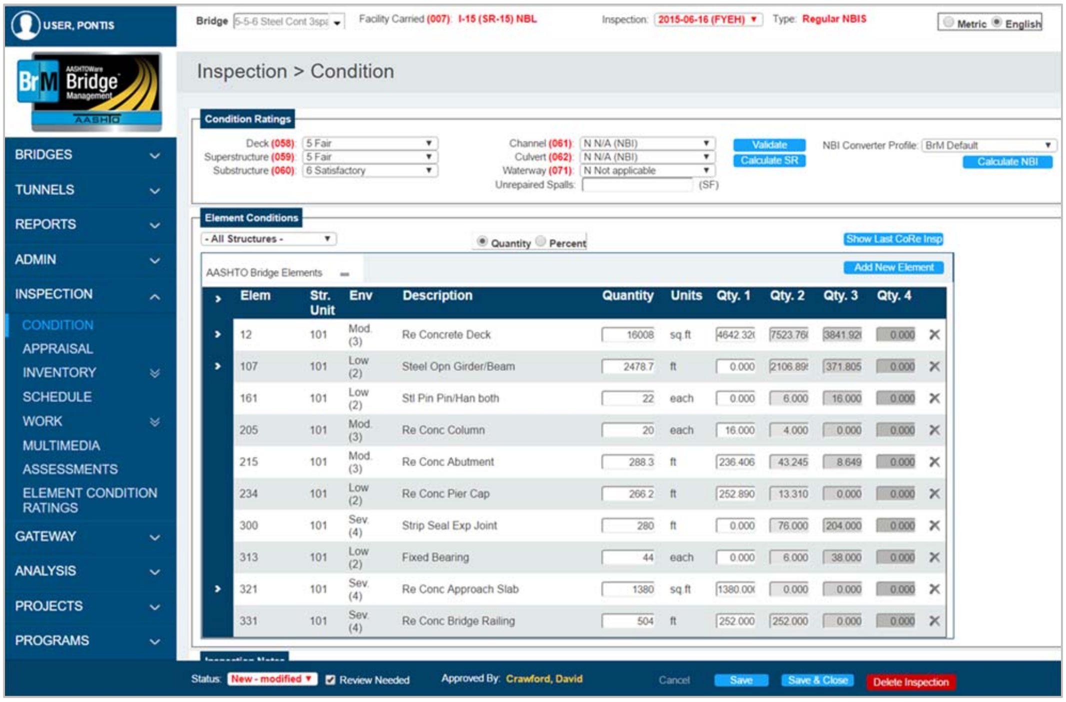

FIGURE 6 Inspection module.

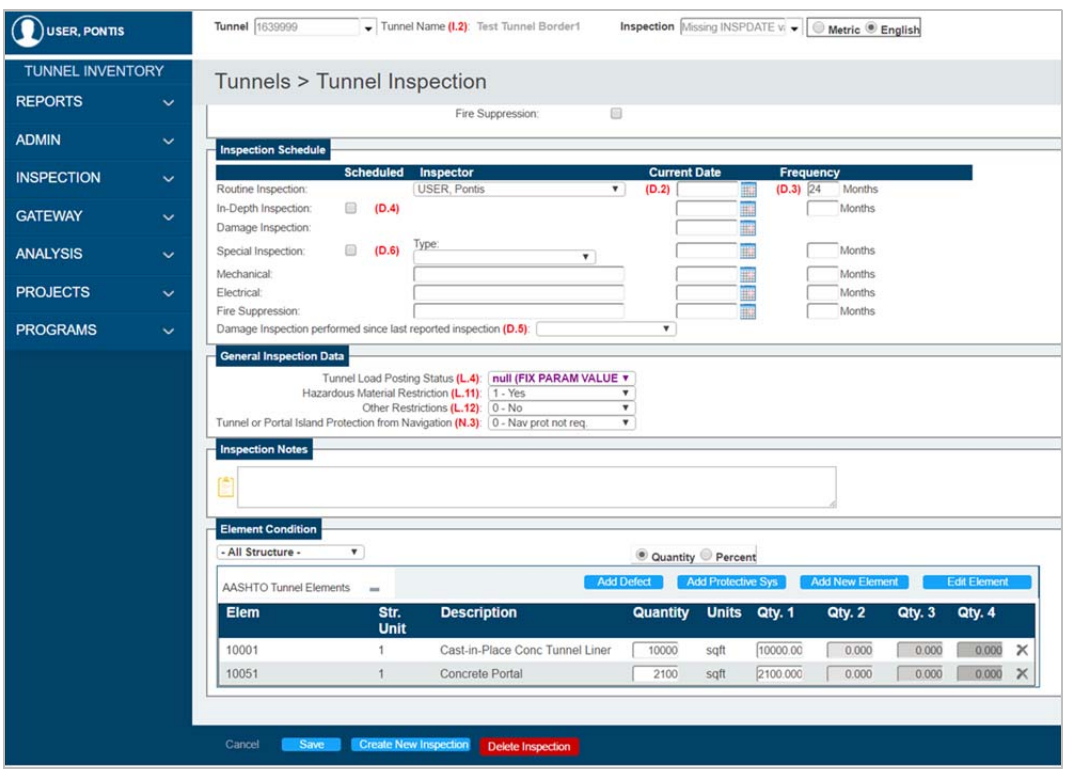

FIGURE 7 Tunnel inspection module.

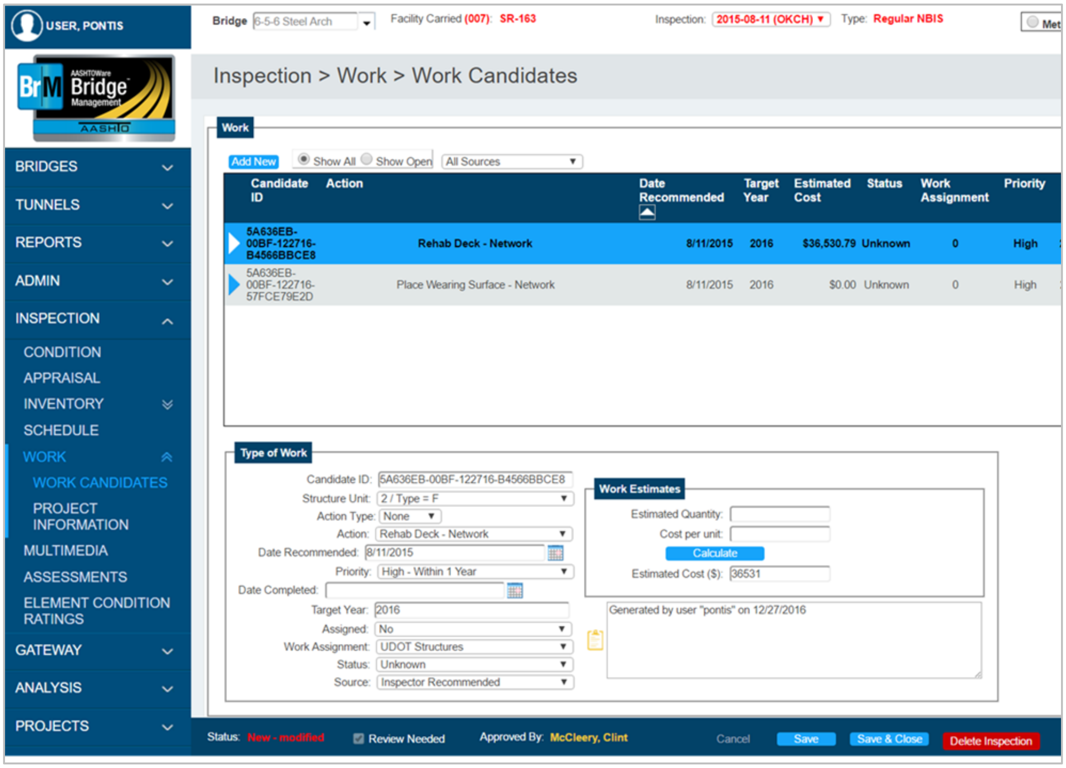

FIGURE 8 Work history and needs module.

Furthermore, BrM has tools to take the inspector recommendations and the system generated recommendations to compare their benefit relative to the cost for a given structure, or a subset of structures. The user can compare their immediate benefit, their life-cycle cost impact and add selected work to a project as shown in Figure 9.

UTILITY AND MULTICRITERIA DECISION ANALYSIS

As previously mentioned, bridges have several attributes that determine the need for repairs or improvements; therefore, decision-makers need to be able to capture multiple criteria to manage them effectively. This was the message from the Pontis (now BrM) User Group and the reason for the changes to how the software optimizes potential work for bridges. Furthermore, agencies have realized that they have a need to make bridges the driving factor and have therefore developed state- or districtwide bridge projects.

Multicriteria decision analysis in BrM works through utility. Each of the criteria go into the overall utility value, which is simply an amalgamated and weighted score for a given bridge. As the bridge ages, the software models the deterioration of elements and the utility value will decrease as the value for the condition based criteria are reduced. Conversely, as the software models improvements, the utility value will increase. The benefit part of cost–benefit analysis for doing work is calculated from the incremental increase in the utility value.

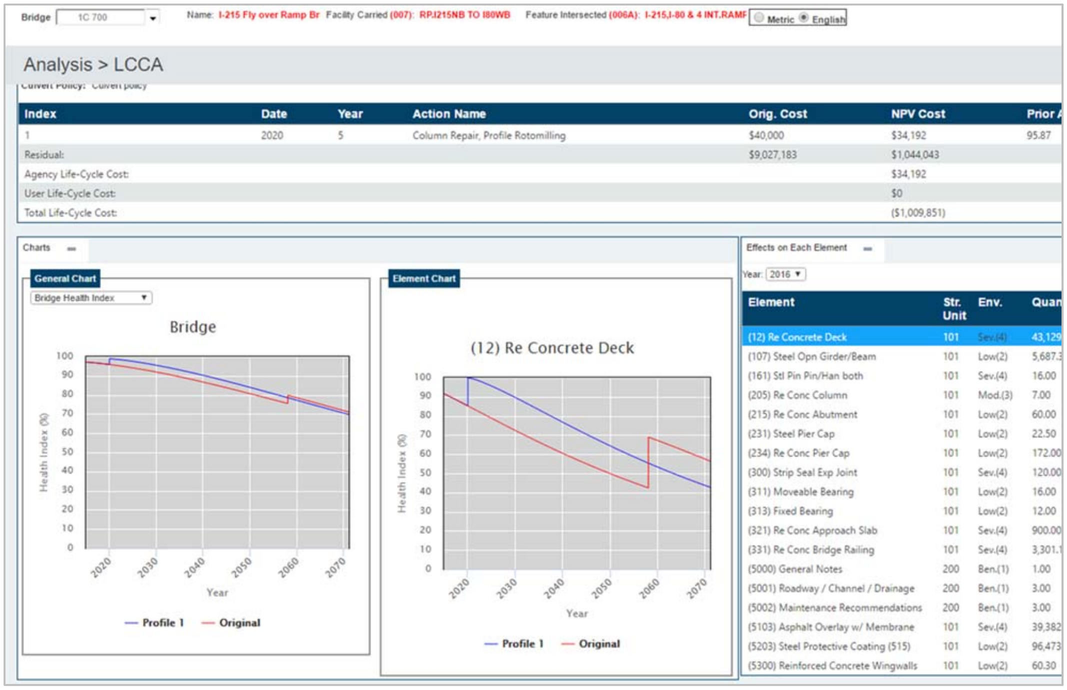

FIGURE 9 Projects module.

The criteria in the utility tree are determined through user input. The user reviews all the factors that impact bridge management, isolates these factors, categorizing them under distinct criterion and then determines the corresponding relative weight/importance of the criteria. There are four primary factors serve as the default criteria in the multiobjective optimization in BrM (Figure 10):

- Condition measures the structural adequacy of a bridge;
- Life-cycle cost (LCC) evaluates the timing of when work occurs to provide the least cost over a given period;
- Mobility evaluates how bridge attributes affect the traveling public; and
- Risk evaluates how bridge attributes and external factors affect the vulnerability of a bridge.

Although these are the out-of-the-box criteria, an agency can define and setup the utility tree to meet their specific goals and business practices. The primary criteria and subcriteria could be a major or minor element of a bridge (e.g., deck, girder, column), a characteristic of the bridge (e.g., vertical clearance, span length, roadway width) or an external attribute that is associated with the bridge (e.g., seismic category, detour length, traffic volume). Some of the criteria may be included under multiple components with the purpose of addressing the specific goal for each component. For example, the NBI Item 70–Posting may be included under both the condition and mobility criteria with the respective goals of quantifying how deficiencies influence a bridge's structural adequacy and how it could affect the traveling public by inducing route restrictions on heavy vehicles.

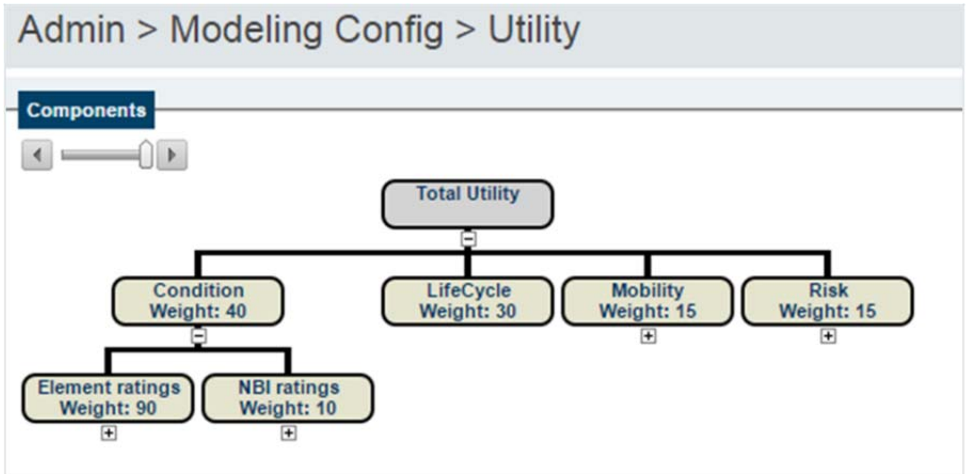

FIGURE 10 BrM utility tree with out-of-the-box criteria.

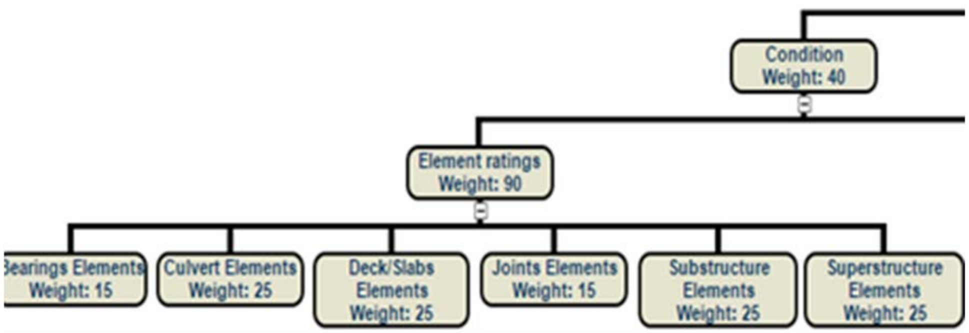

FIGURE 11 Subcriteria for condition and element ratings.

As previously mentioned, to determine the weighted average utility value, each of the primary criteria is given a set of subcriteria (Figure 11). The bottom level of the subcriteria is where the actual scores are assigned. For example, deck elements will have a utility score based on their condition state (CS). This score is then multiplied by the relative weight. The scores are then added and multiplied again by the relative weight of the next higher level in the hierarchy, and so on until there is a total combined utility value.

The relative weights don't have to add up to 100 as the software will normalize the relative weights. The user can assign any value to each of the criteria; however, a recommended methodology for determining relative weights is the Analytic Hierarchy Process (AHP). This process requires the user perform a series of pairwise comparisons and translates those comparisons to a normalized relative weight for each item. This method can easily be used by any agency to customize the relative weights to their needs.

Each criterion has two parts: the base value and the scaler (Figure 12). The base value can be from any field of the user's database, any element or group of elements, any specific type of risk assessment, or even derived from a formula. The scaler will then dictate how intermediate values are weighted. This can be done by the graphs or scaling formulas, which are used to get everything on a common 100-point scale. A few examples of the different scales include the

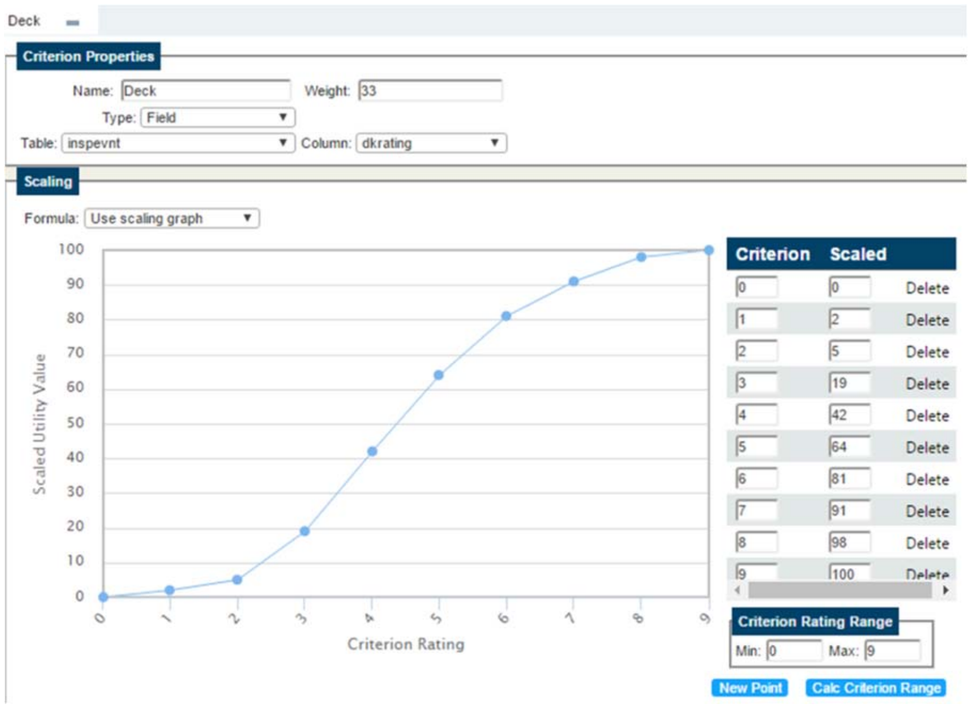

FIGURE 12 Scaling function for deck.

deck NBI value (NBI Item 58, a 1-9 scale), a structure's posting status (NBI 70, a 1-5 scale), and the health index of the deck NBE elements (a 1-100 scale).

Although there is only one utility tree, the software allows the user to define utility weight profiles so that an optimization can be performed for a given objective or goal. For example, if an agency has a specific set of funds dedicated for scour mitigation, they would want to focus the optimization on scour risk and condition and would not be as concerned with mobility. Using weight profiles, they can reweight the utility tree to focus on these criteria as shown in Figure 13.

DETERIORATION MODELING

An important aspect of any asset management system is the deterioration modeling of the asset. In previous versions of BrM (Pontis), deterioration was predicted through a Markov model, which required a parameter to dictate how quickly the deterioration proceeded between each CS. The parameters were median years to transition (T), or the number of years it would take 50% of the element currently in a CS to proceed to the next one.

A pure Markov model begins deterioration at a steady rate from day 1, which is unrealistic for most elements. Therefore, BrM was updated to utilize a Weibull model for deterioration between CS1 and CS2, and a Markov model for the rest of the deterioration. A Weibull model is a Markov model with an adjustment factor Beta. A Beta value of 1.0 matches a Markov model, while larger numbers delay the onset of initial deterioration but speed later deterioration. Figure 14 shows the effects of the Beta parameter on the shape of the Weibull curve. The blue line is a Beta value of 1.0, which is identical to a Markov model. The other lines

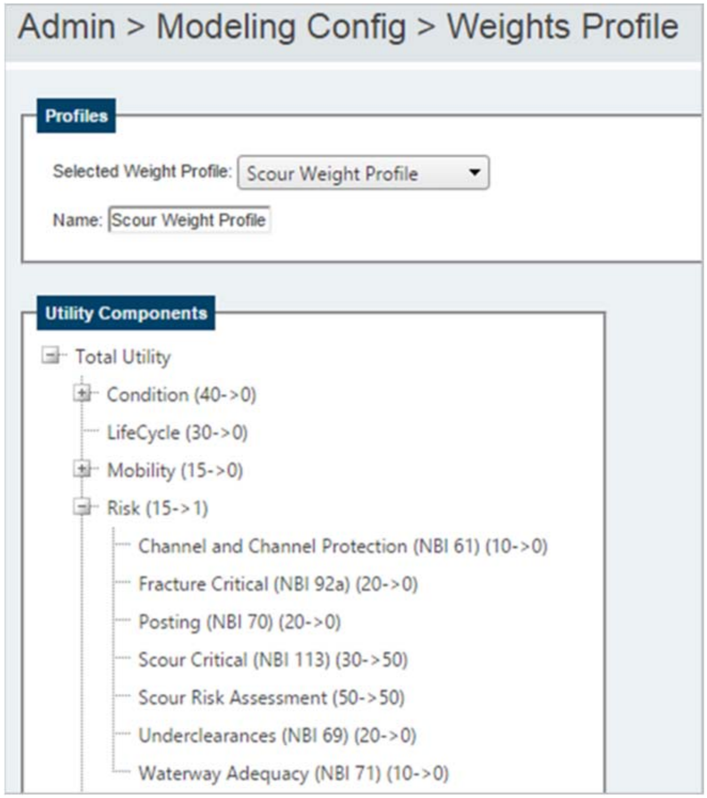

FIGURE 13 Utility weight profile for scour.

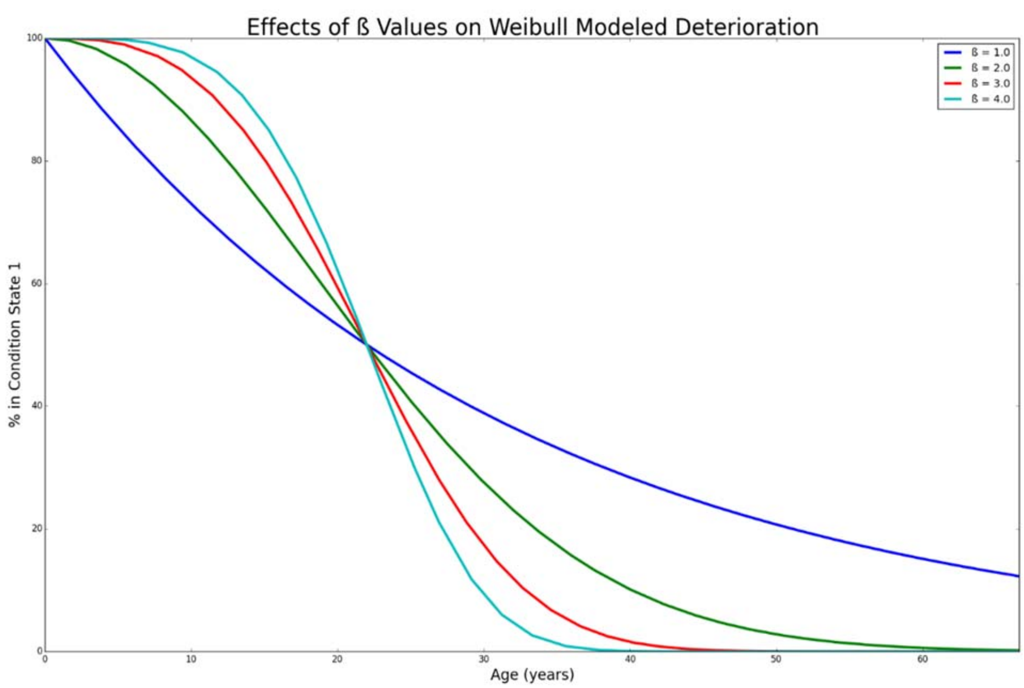

FIGURE 14 Effects of Beta factor on bridge deterioration.
(Note: image is not from software; shown only for illustrative purposes.)

show a Beta value of 2.0, 3.0, and 4.0. Note that deterioration of the Weibull has a comparatively delayed onset.

In addition, to the Beta factor the deterioration model can be impacted through other modification factors which include protection factors, environmental factors, and formula factors. Protection factors are meant to model the existence of a protective system on a primary element. For example, the Figure 15 illustrates the existence of a protective coating on a metal bridge railing and the different factors that modify the deterioration curve.

Environmental factors are meant to model the fact that elements deteriorate at different rates based on the surrounding conditions and exposure. For example, a steel girder in a dry, arid climate will have a much slower deterioration rate than a steel girder that is exposed to moisture and salt in a coastal environment. Figure 16 illustrates how different environmental factors impact the deterioration curve.

Finally, the formula factor is a user defined formula that will modify the deterioration curve for a factor that is not covered by the protection or environmental factors.

In addition to element deterioration, BrM has the ability to perform deterioration at the NBI component level. This allows agencies to manage their structures at a higher level to meet their objectives and performance measures. One method BrM uses to estimate how NBI ratings will deteriorate over time is to use element level deterioration and convert the predicted element level data into NBI component-level data.

As seen in Figure 17, the conversion method uses a table where the user would input their preferences for the cutoffs between each NBI rating. After the distribution of CSs in the elements that make up the component are calculated, each NBI rating is evaluated. In this case, if the

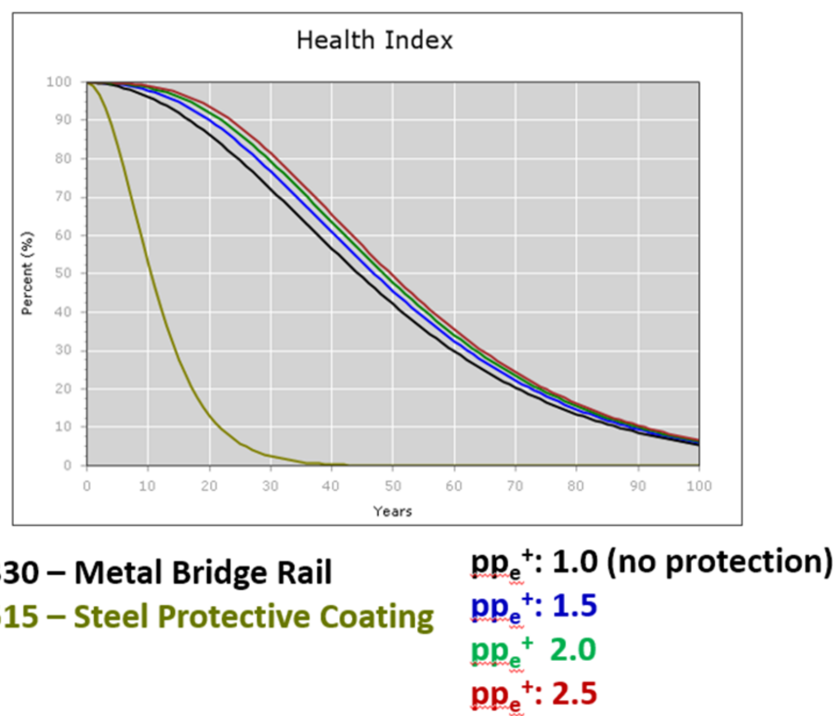

FIGURE 15 Protection factors for element deterioration.
(Note: image is not from software; shown only for illustrative purposes.)

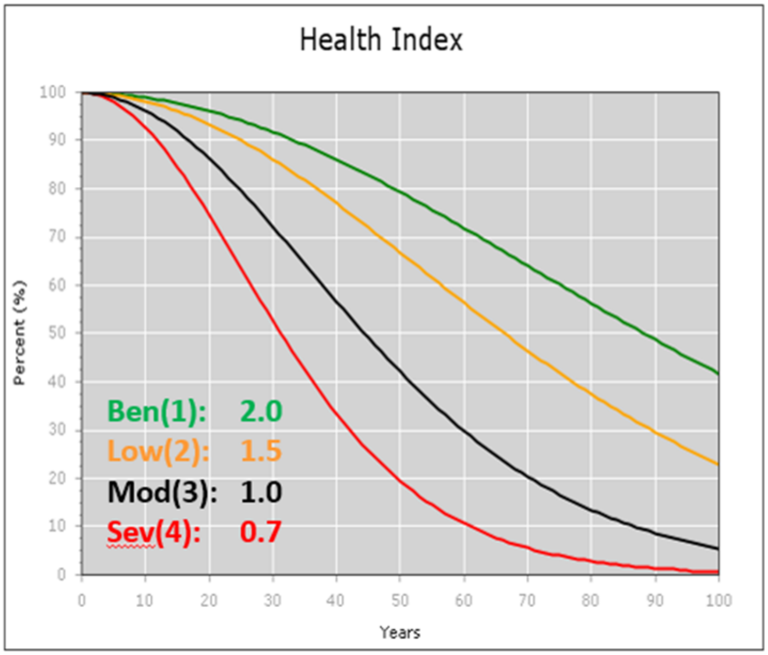

FIGURE 16 Environmental factors for element deterioration.
(Note: image is not from software; shown only for illustrative purposes.)

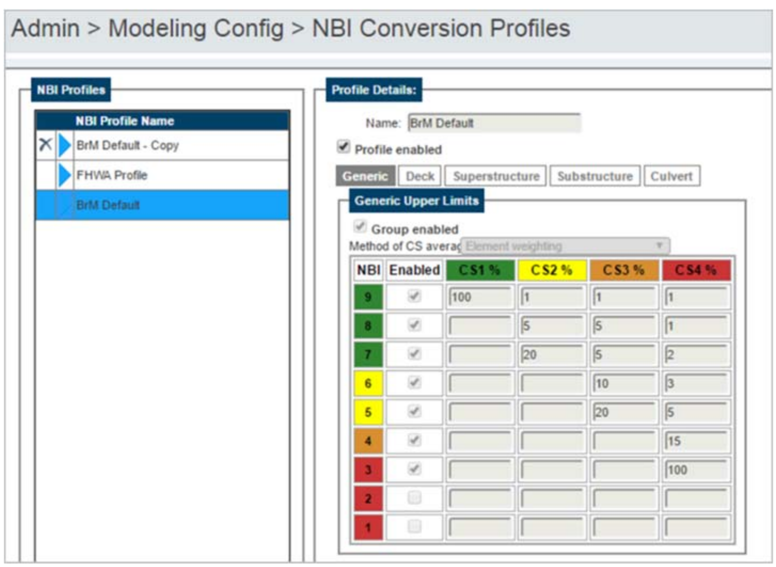

FIGURE 17 Element Conversion for NBI Component Deterioration

component has more than 0% in CS2, CS3, or CS4, it does not meet the criteria and is evaluated for NBI 8, where it is allowed as much as 5% in CS2, 5% in CS3, and 1% in CS4. This process continues until the component satisfies the criteria for an NBI rating. A graphical representation of this can be seen in Figure 18.

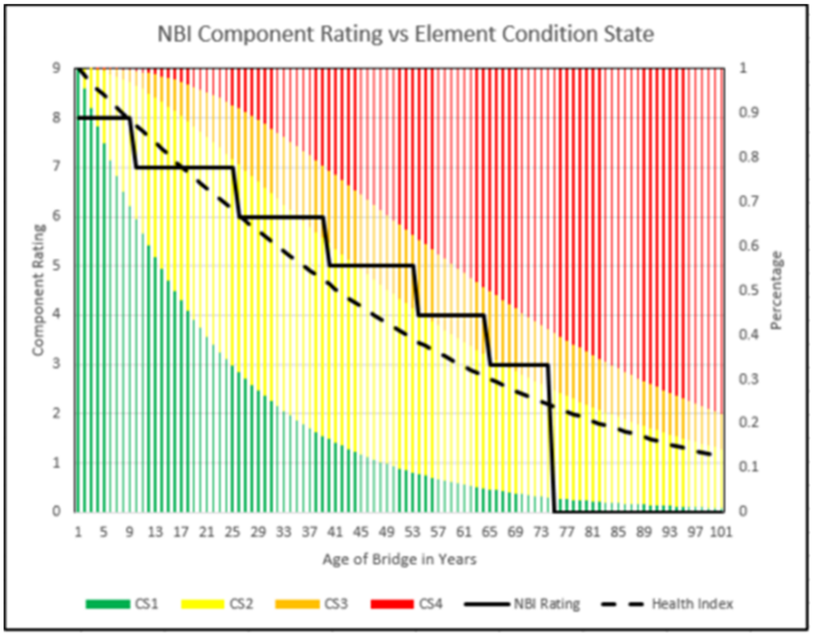

FIGURE 18 NBI component rating stepping down over time with element CS curves. (Note: image is not from software; shown only for illustrative purposes.)

If the user does not wish to use element level deterioration, they can assign a number of years for the bridge to spend in each NBI rating before transitioning to the next NBI rating. This results in a very predictable deterioration pattern as illustrated in Figure 19 and Figure 20.

BENEFITS AND ACTIONS

In order for work to be considered, it has to provide some improvement in the utility of a bridge. This improvement is captured through the benefits and actions modules of BrM. The benefits are the changes to a structure as a result of work. For example, a bridge may have a deck replacement and the benefit of this replacement may be that the NBI rating for deck goes from a 4 to a 9. This increase in the deck rating would provide an increase in the overall utility of the bridge by improving the related criterion scores in the utility tree. This increase in the utility is the benefit portion of the cost/benefit analysis used in the optimization process.

A benefit can be connected to multiple actions, and an action can have multiple benefits. This means an agency can define something like "Replace Super" and use the same benefit for a "Replace Super" action and as part of a "Replace Structure" action. Benefits can be defined in several ways as follows:

- The "Changed Elements" section defines a benefit where all or part of an element in a certain CS is moved to another CS. The costs will be modeled as cost per unit fixed. For example, pothole patching a concrete approach slab moves all CS3 and CS4 quantities to CS2.
- The "Removed Elements" section defines a benefit which removes an element. This can be for record-cleanup and remove defects, or this could be for removing elements like

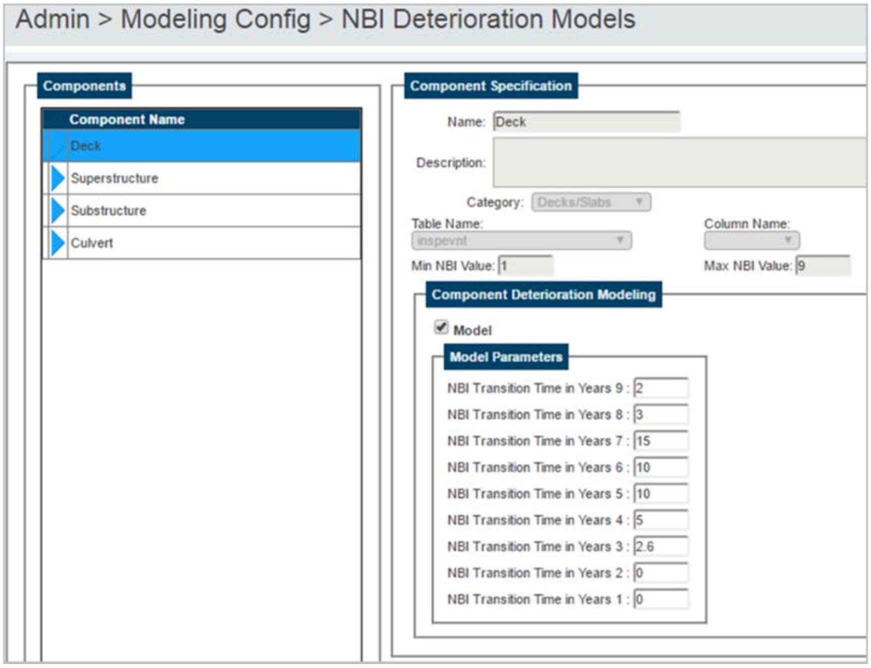

FIGURE 19 Direct NBI component deterioration.

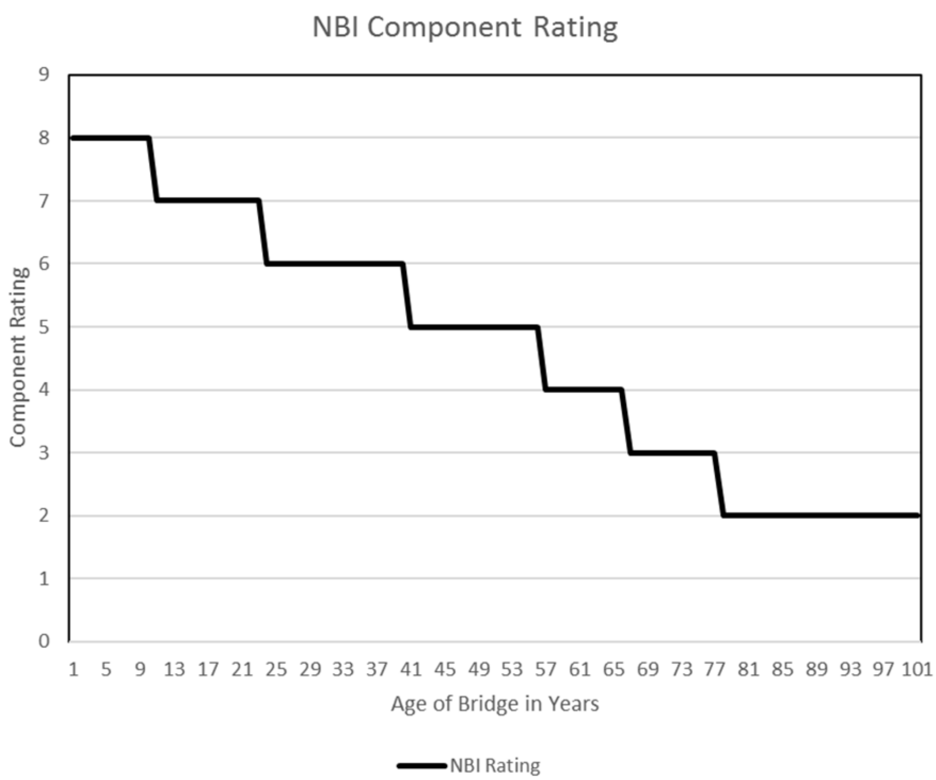

FIGURE 20 NBI component rating stepping down over time.
(Note: image is not from software; shown only for illustrative purposes.)

temporary shoring or asphalt overlays to exclude them from further deterioration analysis. The costs will be modeled as cost per total quantity of units.

- The "Replaced Elements" section defines a benefit in which one element is replaced with the same or different element. Costs will be estimated in terms of cost per unit replaced.
- The "Create Protective Systems" section lets the user add elements to the structure. However, due to the complexity of new quantities, it is limited to just protective systems. The cost is modeled as cost per unit added.
- The "Fields" section lets users model changes to other fields such as NBI fields and scour ratings. All tables and columns of bridge data can be modified. The cost is not modeled by elements, so users must use other methods.
- The "Risk" section lets users model changes to the risk assessments. Seismic or scour countermeasures can be modeled. There are no direct costs associated with these benefits.

It is imperative that the benefits defined effect on the criteria in the utility tree. As previously mentioned the benefit is calculated as the change in utility, and therefore, in order for an action to provide an improvement in the utility, it must affect a related criterion.

After the user, has defined the benefit groups, the benefit groups must be linked to actions. Figure 21 is an example of the actions page in BrM.

FIGURE 21 Actions page.

Action types allow users to categorize actions so that they have the ability to filter actions based on a specific type because an action may only be applied to a specific subset of bridges or it may be applied to the entire network.

As seen in Figure 21, the lower left is where the benefits are linked to an action. Multiple benefits can be linked to an action. For example, a deck replacement benefit can be used as part of a deck replacement action and as part of a bridge replacement action.

The costs can be modeled in a few ways. First, there is an option to override the costs on a per-element basis with a cost per square foot of deck. This can be useful if an agency has a square foot cost estimate for structure replacement or standard collections of deck work. Another way of modeling costs is the element per-unit costs. As benefits are added to the action, the elements affected will be added to this list. The parenthesis indicates if it's a removal, improvement, replacement, or creation. Finally, there is an option to define a minimum cost, which will keep the optimizer from recommending that work until the cost reaches a minimum threshold.

Deferment rules are used primarily by the life-cycle cost analysis (LCCA) modules, and are helpful for modeling how work would actually be performed. An action can have deferment effects on many actions, for example replacing the bridge can defer any other actions for several years.

NETWORK ACTIONS

The concept of network actions and how they differ from actions centers around the difference between two general approaches to optimization, generally referred to as "top down" and "bottom up". The top down approach uses an average project and applies it to all bridges in the network. The bottom up approach finds the optimal program for each bridge individually and aggregates all the bridges in the network for the network program. The fundamental challenge with the Bottom Up approach is the number of possible combinations of actions. Figure 22 illustrates common work items using the bottom up approach as well as the fact that it would take a very long time for the optimization to find the optimal solution for every bridge in the network.

For the top down approach, BrM uses higher level actions that encompass several work items based on typical agency policies to help reduce the number of possible combinations as shown in Figure 23.

To account for additional work that may be considered when performing an action, there are follow-up actions that are used to define a network policy. For example, if an agency performs work on a deck, logically they would not defer work on the superstructure or substructure and come back to that same bridge in a year or two. They would logically perform all the work at the same time. Also, agencies do not typically patch a deck but replace the superstructure. Policies can reduce the number of possible combinations by removing options that would not be performed in practice. Figure 24 illustrates the concept of follow-up actions.

The network policies allow the user to define the combinations of actions to be used in optimization. Network polices are allowed to go three levels deep, and there's no limit to how many actions are placed on each level; however, everything added will add run-time to the analysis. Figure 25 is the network policies page in BrM.

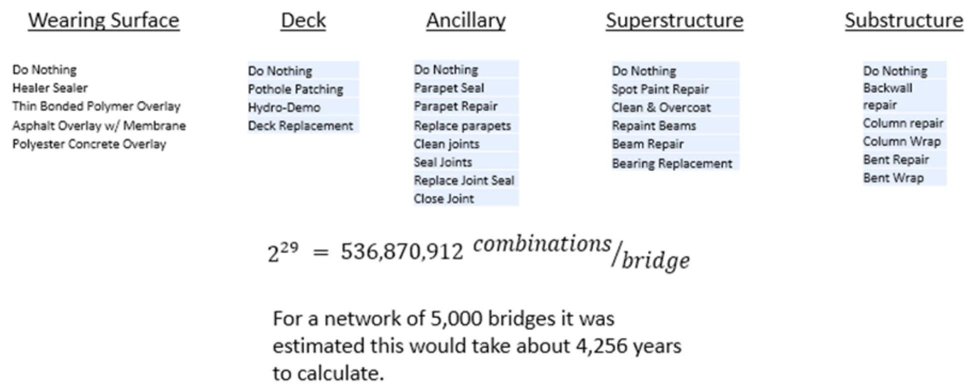

FIGURE 22 Bottom-up approach for network analysis.

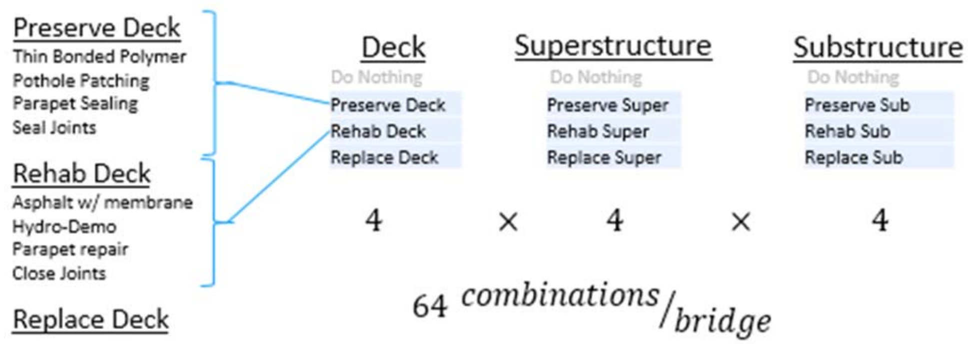

FIGURE 23 Consolidation of work items for network actions.

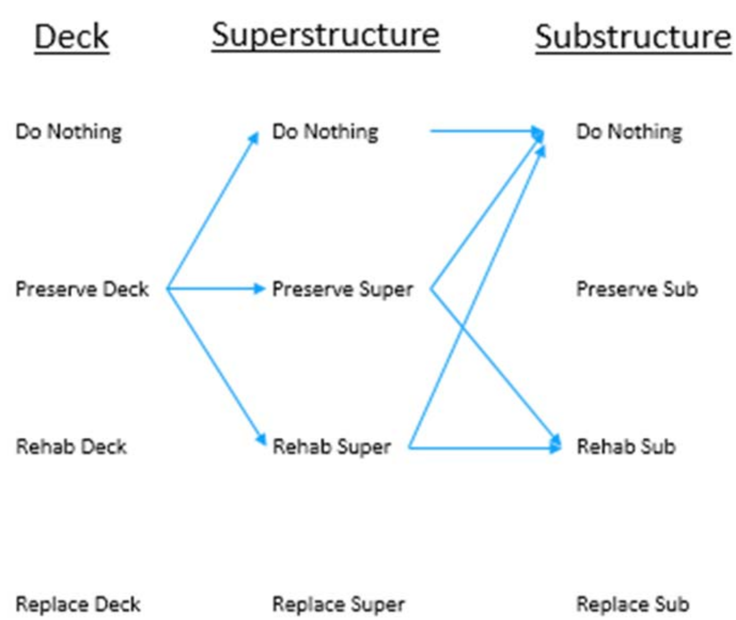

FIGURE 24 Example of follow-up actions.

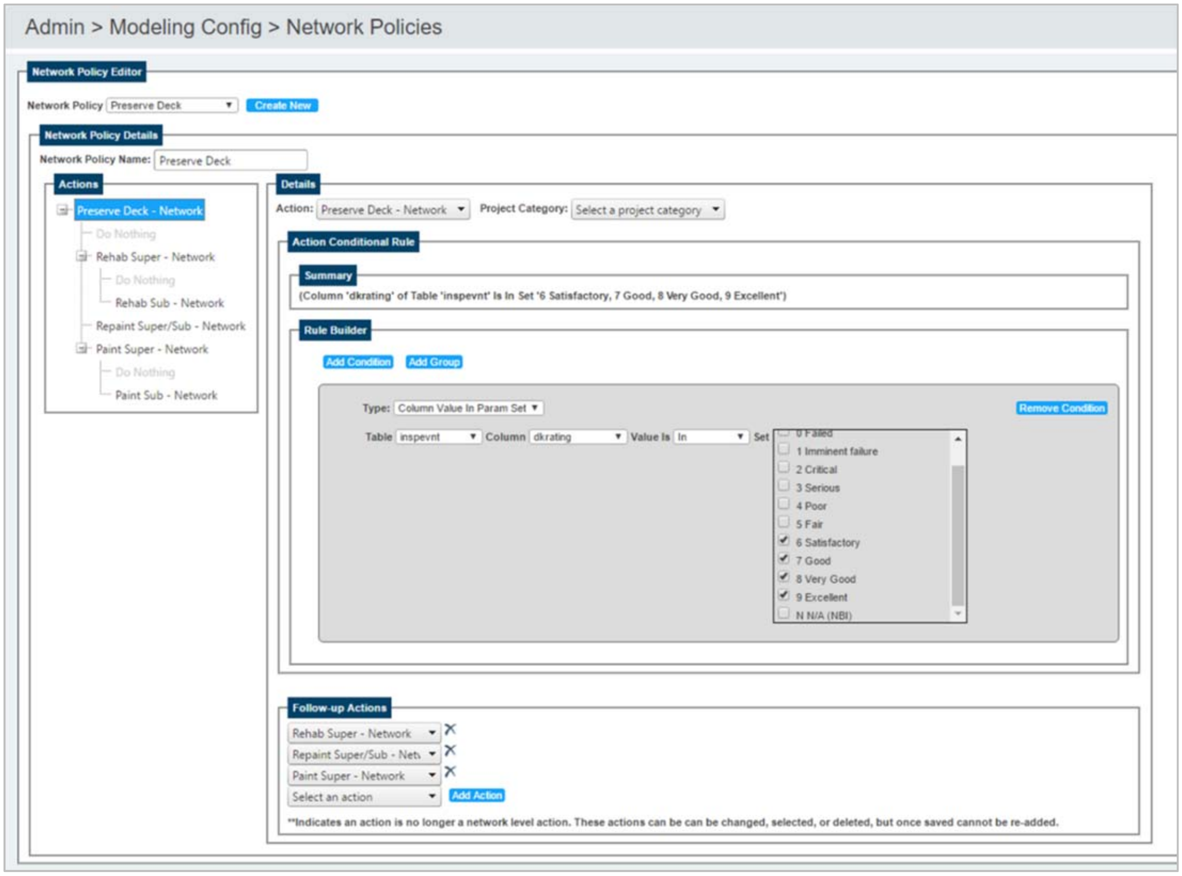

FIGURE 25 Network policy page.

To limit the number of conditions even further, users can add conditions based on any database field related to when this work should be considered. As an example, a policy could be added that would prevent the optimizer from recommending preservation work on a bridge deck with an NBI deck component rating of 3.

LIFE-CYCLE COST ANALYSIS IN BRM

Another crucial component in bridge management is the LCCA for given treatments or work to ensure the lowest practical cost over the life span of a structure. BrM computes the LCCA through a combination of a short-term (e.g., 5 years) and a long-term (e.g., 75 years) analysis. The short-term analysis is the effect/benefit and cost of a project (e.g., deck rehab, desk preservation, super rehab) will have over the short-term period as defined by the user. The long-term analysis considers what impact the work/project has on a bridge beyond the short-term period. The software uses preservation policies that are defined by the user to apply future work to the bridge. Table 1 is an example of preservation policies.

TABLE 1 Example Preservation Policies for LCCA

Component	Conditions	Action
Deck	Deck NBI = 6	Epoxy Overlay (Thin Bonded Polymer)
Deck	Deck NBI = 5	Concrete Deck Overlay (Polyester Concrete Overlay)
Deck	Deck NBI <=4 and Super NBI >= 5 and Sub NBI >= 5	Deck replacement
Super	(515) - Steel Protective Coating < 40 and (107) Steel Opn Girder/Beam > 60	Paint
Super	(107) Steel Opn Girder/Beam < 50	Repair Beams
Sub	Substructure HI < 50	Substructure Rehab
Bridge	Super NBI <= 4 or Sub NBI <= 4	Bridge Replacement

The example in Table 2 illustrates how the LCCA module works in BrM. In this example, a bridge rehab is conduced and therefore that cost and benefit are applied at year zero and no other work is considered for the next 5 years, the short-term analysis period. After the short-term analysis period, the bridge continues to deteriorate and work is applied based on the preservation policies in Table 1. Table 2 shows the work conducted, when it is conducted, the cost and resulting net present value (NPV).

The LCC benefit from doing the bridge rehab today is captured in the increase in utility value as shown in the following calculations based on Table 2 and the example bridge having a utility value of 75.83 prior to the bridge rehab project.

TABLE 2 Example LCCA in BrM

Index	Date	Year	Action Name	Orig. Cost	NPV Cost
1	2016	0	Bridge-Rehab	$260,570	$260,570
2	2021	5	Paint-General, Thin Bonded Overlay	$45,844	$37,681
3	2036	20	Beams Rehab, Concrete Deck Overlay	$176,863	$80,718
4	2048	32	Beams Rehab, Deck-Replace	$292,525	$83,386
5	2058	43	Beams Rehab	$18,861	$3,493
6	2064	48	Substructure-Rehab	$21,984	$3,346
7	2069	53	Beams Rehab	$17,747	$2,220
8	2079	63	Beams Rehab	$17,768	$1,502
9	2080	64	Bridge-Replacement	$1,625,000	$132,044
10	2093	77	Paint-General	$13,176	$643
Remaining Life	52 years				
Residual				$1,242,647	$53,911
Total Life-Cycle Cost					$551,691

$$LCC = \text{ST} + \text{LT} - Residual$$

$$Residual = \left(\frac{Remaining\ Service\ Life}{Service\ Life}\right) \times Replacement\ Cost$$

$$LCC = ST + LT - R = \$260{,}570 + \$345{,}032 - \$53{,}911 = \$551{,}691$$

$$LCC_{Utility} = \left(1 - \frac{LCC}{2 \times replacement\ cost}\right) \times 100$$

$$LCC_{Utility} = \left(1 - \frac{\$551{,}691}{2 \times \$1{,}625{,}000}\right) \times 100 = \mathbf{83.02}$$

$$Benefit = 83.02 - 75.83 = \mathbf{7.19}$$

PLANNING, PROGRAMMING, AND OPTIMIZATION

One of the major new features included in version 5.2.3 is the multicriteria optimization. After setting up the rules and boundary conditions for modeling, the software will consider all the alternatives for a program and recommend work that would most optimally help an agency meet their performance measures.

Performance measures can contain limitations, targets or simply exist to track the data through optimization. BrM then has many configurable charts to help bridge managers isolate the data they want to focus on, as shown in Figure 26.

TRADE-OFF ANALYSIS AND SCENARIO EXPLORER

The crowning aspect of BrM 5.2.3 is the Scenario Explorer, which allows agencies to run many optimizations and then compare the results if they change inputs, boundary conditions, targets, etc. Figure 27 illustrates how spending more money helps the agency reduce their percentage of structurally deficient bridges faster. These graphs are very useful for presentation to legislatures and commissions who set spending limits and priorities. This module can also be a tool used by states to develop the alternate strategies required by the Performance Measure Rule Making.

A trade-off is a situation where one quality is lost to gain another. In the case of BrM, trade-offs typically involve the probability of gaining or losing condition or other qualities of a bridge and comparing that to cost and other such scenarios. When combining, and plotting two variables, the user can identify combinations where one costs less than the other, or one provides better outcomes for the same cost. The most optimal combinations create a line which is referred to as the Pareto Front, or Pareto horizon. Figure 28 is an illustration of the cost to take care of a bridge versus the condition of the bridge. Notice there are some combinations which achieve the same condition for less cost, or better condition for the same cost. Given the limitations defined in the model, there is a horizon of ‘most optimal’ combinations.

FIGURE 26 Performance measures and dashboards.

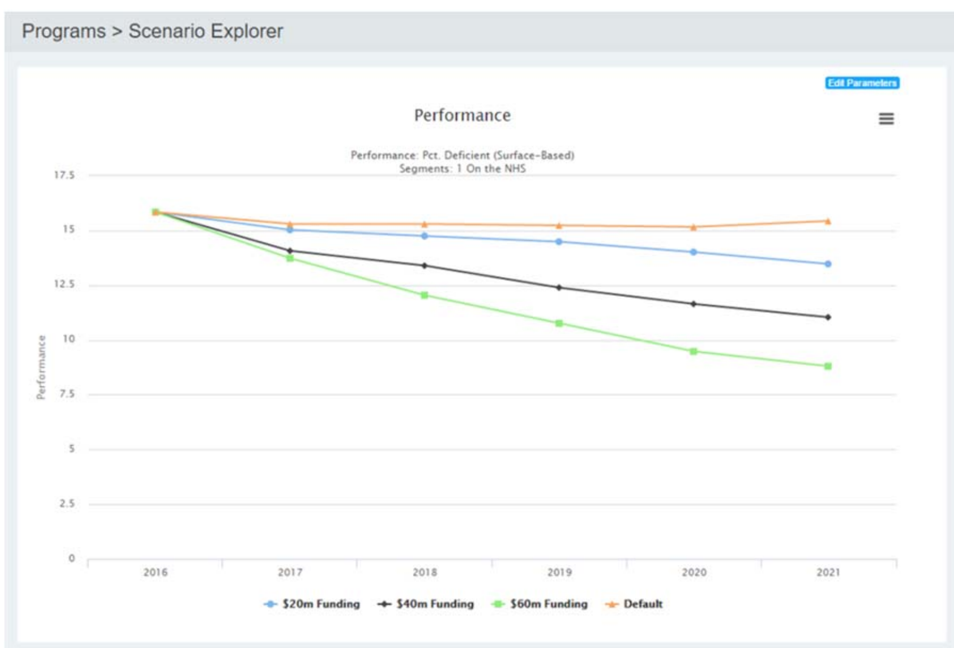

FIGURE 27 Example showing the results of different levels of funding for the same program.

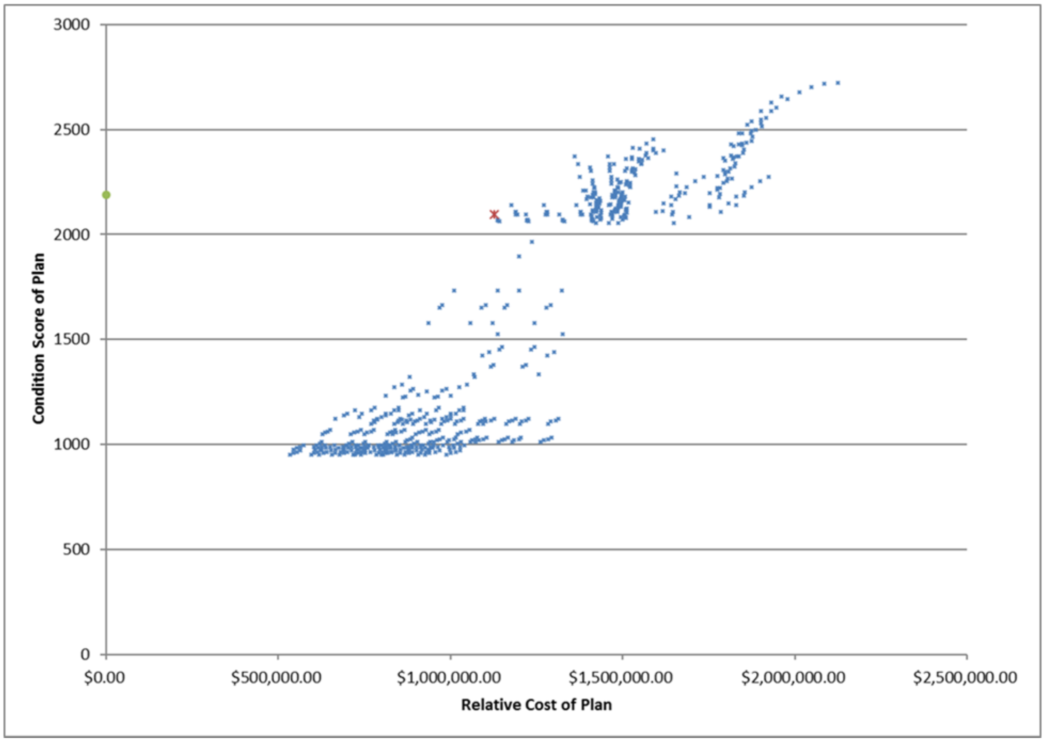

FIGURE 28 Example of trade-off between bridge condition versus cost. (Note: image is not from software; shown only for illustrative purposes.)

Figure 29 is an example of graphs displayed in the program results page in BrM. Notice the Pareto horizon, calculated in the top right, which scales from do nothing to replace all the bridges.

The green triangle shown in the upper right in Figure 29 is the program that has been selected by the user, showing that there are other options that will provide greater benefit, but at a higher cost. This is where the scenario explorer module in BrM really comes into play. Decision-makers and bridge managers are able to create multiple scenarios to see how different levels of funding impact the results. This type of trade-off analysis is incredibly useful for decision-makers who determine funding levels. To do this in BrM, we will create different scenarios for the different funding levels as shown in Figure 27.

Furthermore, the scenario explorer is not limited to just comparing different levels of funding. The user can create scenarios for several items that may impact the results of the optimization. For example, the user can run several scenarios for performance measure targets, such as the percent of bridges that are structurally deficient. Targeting particular performance measures may result in spending funds more or less optimally, but may provide better performance of the network overall.

SUMMARY

Determining the optimal allocation of resources between competing bridges in a transportation network is difficult; particularly when available resources, specifically funding, to address preservation and improvement needs are limited. And, as the nation's infrastructure continues to

FIGURE 29 Program results page in BrM.

age there is an ever-greater demand on this essential and expensive component of the transportation system. Furthermore, recent federal regulations placed an even higher emphasis on distribution of federal funds used for bridge preservation and improvement projects.

Bridge managers, planners, and decision-makers utilize bridge management systems as a tool to assist in meeting the above-mentioned goals and requirements. As demonstrated throughout this paper AASHTOWare BrM 5.2.3 is an excellent bridge management software solution, updated to advance bridge management and to provide a tool that can allow states to meet the requirements of the TAMP and performance measures rule-makings. BrM provides a robust multicriteria approach to bridge management and project selection. Additionally, the software is highly configurable to meet agency specific needs, policies, and practices and can improve the performance and resiliency of bridges in their transportation network.

To learn more about BrM please visit the website at aashtowarebridge.com or send an e-mail to BrM@Bentley.com.

Innovative Technologies and Project Delivery

Blending Science and Engineering

Laguna Creek Bridge Bank Protection

STEVEN OLMSTED
Arizona Department of Transportation

Laguna Creek Bridge Scour Remediation, State Project Number H8913 01C, Federal Aid No. 160-B(205)T, is a scour remediation project on the existing Laguna Creek Bridge located in Arizona Department of Transportation's (DOT) Northeast District. Laguna Creek Bridge is located on U.S. Route 160, Tuba City–Four Corner Monument Highway, MP 420.1, over Laguna Creek in Apache County, Arizona. The bridge is within the boundaries of the Navajo Nation Reservation. This site is approximately 25 mi east of the town of Kayenta, Arizona. This project was administered by Arizona DOT, who maintains the facility. The project scope consisted of installation of riprap gabion spur-dikes upstream and downstream of the existing structure and along the bridge abutments, in an effort to mitigate scour, provide bank protection, and reduce channel meandering at the bridge.

The original bridge structure was constructed in 1961 by the U.S. Department of Interior. In 1984, the original bridge was retrofitted with barriers. During the period from 2004 to 2008, bridge inspections identified excessive scour at both of the pier locations. In 2012, a replacement bridge was constructed just to the south of the previously existing bridge structure. Bridge abutments repairs and countermeasures were recommended, but were found to be insufficient to address the current problem. Since then, inspection efforts have continued to identify and monitor the accelerating meandering of the wash and subsequent undercutting of the abutment fill. The goal of the project was to provide protection of the existing bridge and roadway against the effects of local scour and severe channel meandering just upstream and downstream of the bridge crossing.

In 2015 Arizona DOT set out to develop a statewide stormwater modeling program. The effort was designed to centralize the agency's response to systemwide water issues, introduce next-generation science and engineering modeling techniques (2-D hydrological and 3-D visualization), advance risk-science-technology-engineering development goals, and launch the Arizona DOT Resilience Program. The U.S. Geological Survey (USGS) Arizona Water Science group was key to the new program and supplies Arizona DOT direct (real-time) storm monitoring and data collection, indirect (post-storm event monitoring and data collection), and next-generation hardware/software and surface water flow data collection capabilities. This partnering effort would contribute to expediting and improving Arizona DOT's efforts in planning and responding to incidents of flood, over-topping, system hotspots, hydraulic-related failure, and extreme weather events in connection with (1) National Environmental Protection Act of 1969 jurisdictional and wetland delineation and streamlining; (2) highway stormwater runoff management; (3) evaluating scour potential and countermeasure development at water crossings; (4) drainage structure siting, design, and construction; and (5) response to federal extreme weather regulatory activities.

The first of six pilot efforts to address different types of water exposure on Arizona DOT's highway system were initiated in 2016. The Laguna Creek project was identified as the main pilot to test a suite of USGS next-generation technologies as they relate to transportation infrastructure: lidar, unmanned aerial systems (drones), rapid deployment streamgage, noncontact velocity sensors, video camera and particle tracking data collection, and 3-D land surface models.

BANK PROTECTION PROJECT

US-160 is classified as a rural principal arterial highway in Arizona Department of Transportation's (DOT's) Functional Classification Map. It is located in the FHWA National Highway System. The original Laguna Creek Bridge (#705) was constructed in 1961. This bridge was replaced in 2012 (#20001) with a single-span bridge, however, abutment bank protection was not included in the project. Since completion of the bridge construction, Laguna Creek has meandered resulting in a significant amount of undercutting of the fill slopes adjacent to the bridge abutments. The work under this Scoping Letter is to analyze the existing bridge conditions and provide scour remediation alternatives to prevent further erosion and protect the bridge abutments and approaches.

A listing of the original and subsequent construction projects that incorporated all or part of the project segment are listed in Figure 1.

Site Conditions

Laguna Creek originates at Tsegi Canyon and flows northeast. Soil in the watershed is fine grained and susceptible to sediment transport at relatively low velocities. The US-160 crossing is characterized by a meandering vertical bank channel (unstable). Laguna Wash has the potential to adversely impact the US-160 structure at the following three locations: (1) Abutment no. 1; (2) Abutment no. 2; and (3) the approach roadway west of the structure. The following aerial photograph (Figure 2) shows the existing channel and the historic scarring due to previous migrations of the wash (old oxbows).

Project No.	Begin MP	End MP	Project Description	Year
AU I(28)	420.1	420.1	Laguna Creek Bridge # 705	1961
F-064-1(3)	420.1	420.1	Barrier Retrofit	1984
			Pavement Preservation	2004
U-160-A-202	311.46	470.83	Corridor Feasibility Study	2007
160 AP 420 H7571 01C	420.04	420.24	Laguna Creek Bridge # 20001	2012

FIGURE 1 Project hydraulic final report (Arizona DOT, 2016).

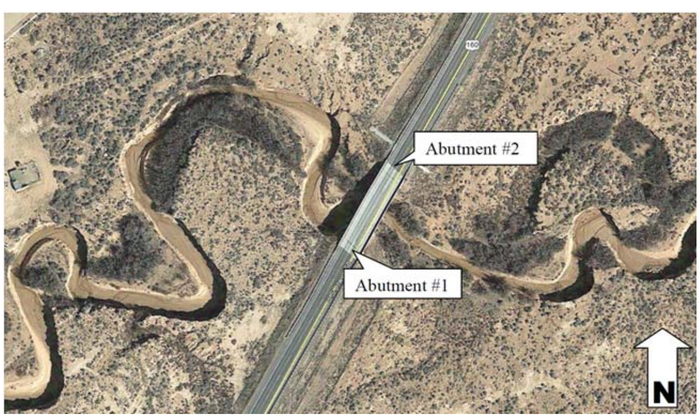

FIGURE 2 Aerial SR-160 Laguna Creek Bridge (Arizona DOT, 2016).

Drainage Conditions

A hydraulics report for Laguna Creek Bridge was completed in May 2011 by Arizona DOT Bridge Group, Bridge Hydraulic Section (TRACS No. 160 NA H 7571 01C) (Figure 3). According to this report, the total watershed area for Laguna Creek Bridge is 848 mi^2. Discharge and water surface elevation are summarized below. The proposed bridge required 1 ft of freeboard for the 50-year storm. The bridge soffit elevation is 4,973.67.

Proposed Improvements

In order to protect the existing bridge it is recommended that guide banks be constructed to direct stormwater through the structure. The guide banks will be constrained by limited rights-of-way (200 ft to the north of US-160 centerline and 100 ft south of US-160 centerline). The area of disturbance (excavation limits) may not extend outside of the rights-of-way and the proposed improvements may not adversely impact the cultural site adjacent to the structure. Team members evaluated several alternatives for the guide banks including cement-stabilized alluvium (CSA), Arizona DOT standard rail bank protection, gabion mattress, gabion basket, grouted rip rap, rip rap, and sheet pile bank protection.

Design Alternatives

Seven design alternatives for Laguna Creek Bridge bank protection were evaluated in this scoping letter. The preferred alternative was determined to be gabion baskets (Alternative 1) due to the smaller area of disturbance, constructability, long-term bank protection, and lower cost relative to the other alternatives. Below is a description of four of the alternatives. The preliminary configuration of Alternative 1, gabion baskets is shown below.

Alternative 1: Gabion Baskets

Description

This alternative consists of 3- x 3- x 6-ft gabion baskets, filled with 4- to 8-in. nominal rock that will be placed around the perimeter of the bridge abutments and approaches to prevent lateral stream migration and bank protection (Figures 4 through 6).

Storm Event	Discharge (cfs)	Water Surface Elevation	Freeboard Height (ft) (Constructed)
Q50	6,895	4969.79	3.88
Q100	10,826	4973.20	0.47
Q500	23,370	4979.67	-6.00

FIGURE 3 Project hydraulic final report (Arizona DOT, 2016).

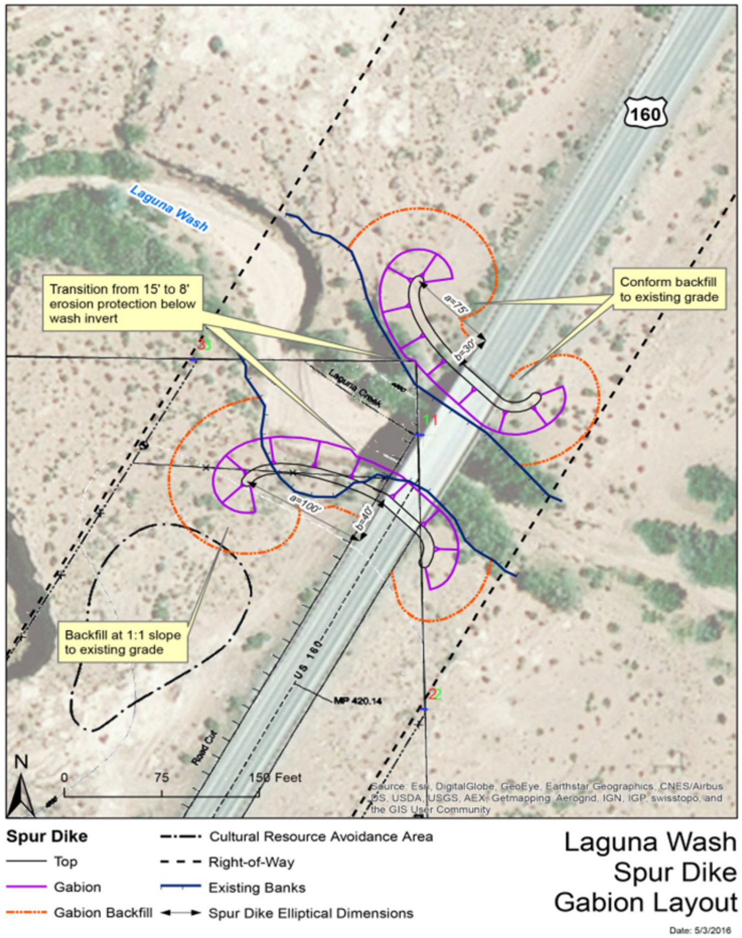

FIGURE 4 Project design overview (Arizona DOT, 2016).

FIGURE 5 Looking downstream approaching the structure (notice the floating fence, upper right) (Photo: USGS, 2016).

FIGURE 6 Looking downstream from the structure (Photo: USGS, 2016).

Conclusion

The preferred alternative was determined to be gabion baskets. The use of gabions baskets reduces the disturbance area, is constructible based on the site constraints, and is the most cost-effective of all of the alternatives considered in this scoping letter. Gabion baskets have the advantage of being constructible beneath the existing bridge, requiring only a small section of concrete infill at the top of the new gabion wall where vertical clearance beneath the existing bridge is less than approximately 5 ft. Rock suppliers have been located in southern Utah and in the Phoenix area.

Alternative 2: Driven Sheet Pile Bank Protection

Description

This alternative consists of driving a continuous wall of sheet piling to the required scour elevation plus the required embedment depth for the piling. Various pile sizes were considered. Due to the height of material required to be retained by the piling, large pile sections such as AZ-24 and larger are required. It is also necessary to construct a three-tiered wall system with the back rows acting as buried tie-back walls. For the segment of protection directly below the bridge there is insufficient vertical clearance to drive piling, so a separate type of wall such as gabion baskets or a CSA wall would need to be constructed and connected to the sheet piles outside of the bridge limits.

Conclusion

Sheet piles were eliminated due to high cost, difficulty of construction, and large area of disturbance.

Alternative 3: Cement-Stabilized Alluvium

Description

This alternative constructs a compacted cementitious soil fill to create a hardened surface layer that will prevent stream lateral migration and erosion due to scour.

Conclusion

CSA was determined not to be practical due to the limited space to operate large grading and compaction equipment beneath the existing bridge. The area of disturbance exceeded the rights-of-way limit due to the flatter side slopes required for typical CSA construction. This alternative is also the highest cost of the three constructible alternatives considered for the site.

Alternative 4: Rail Bank Projection

Description

This alternative uses Arizona DOT standard rail bank protection constructed around the perimeter of the bridge abutments in a configuration similar to the gabion basket bank protection alternative.

Conclusion

Rail bank projection is eliminated because the existing standard is not capable of providing more than 10 ft of vertical projection above existing grade. The vertical face requiring protection adjacent to the bridge abutments exceeds 20 ft in most locations. Rail bank protection is not constructible beneath the bridge due to insufficient vertical clearances.

Integration of USGS Technology for Future Design

Due to the emergency nature of this project the preliminary scoping was based largely on hydraulic and geotechnical information obtained for the design of the replacement bridge (#20001). In large part this is typical for many scour design projects; limited information is available in the preliminary design stage when key decisions are made that impact the project construction costs, durability, and long-term maintenance costs.

The availability of real-time information has many beneficial impacts. The USGS information allows planners and designers to view current channel configurations as well as to look at time lapse information for comparison. On the Laguna Creek project our team was able to compare current stream velocity and path with prior survey data and hydraulic studies for confirmation of our assumptions of the long-term channel movement in the vicinity of the bridge. It will now be possible to monitor the channel behavior with the bank protection in place to improve our understanding of the protections long-term performance and potential refinements for future designs. The velocity vector and flow limits data can be used to verify 1-D hydraulic models as well as calibrate or verify 2-D hydraulic models early in the design process.

This will enable designers to progress more quickly through the design process while enhancing their confidence in the results.

The availability of this information is valuable to owners and agencies in several ways. The topographic and stream flow data can be obtained using drone-mounted photogrammetry. This enables information to be cost effectively obtained over large areas and streamlines or eliminates the permitting process required to obtain traditional field survey and stream flow data outside of existing right-of-way limits. Software is available that allows this data to be transformed into renderings that allow a quick visual interpretation of the site characteristics to facilitate coordination among diversified staff and agencies.

This information also adds value for construction cost control. In a typical design-to-construction cycle the topographic survey data is obtained as early in the design process as possible. The construction project is then bid against the plans, specifications, and estimates which are based on the original survey. It is often the case that stream migration, erosion or infill, and changes in accessibility occur in the time period between the design survey and the construction project award. These items can become change orders during construction as well as result in delays for design changes to be produced. While not currently implemented, it should be possible to utilize this hydraulic data to improve construction cost estimating and to mitigate potential delays and funding shortfalls due to naturally occurring shifts in channel profiles.

ARIZONA DOT RESILIENCE PROGRAM

Transportation infrastructure is a complex system of assets required to deliver a myriad of services and functions. As fiscal constraint for the development and rehabilitation of such structures continues to be cost prohibitive, new and novel approaches to life-cycle costing and long-term planning become paramount. In addition, the management of these infrastructure systems has now evolved from a decentralized, project based focus to one that now encompasses enterprise wide endeavors (*1*). Three areas of concern for state DOTs and the main catalyst for developing an Arizona DOT Resilience Program involved how to

- Centralize to one operating area the unknown, erratic, and abrupt incidents of stormwater and its contributors of flooding (overflow of water that submerges land), overtopping (rise over or above the top), system hotspots (roadway flood prone history), and hydraulic-related failures (structure failure mechanisms);
- Introduce extreme weather adaptation to agency and engineering design processes and establish transportation asset sensitivity to extreme weather; and
- Handle scientifically informed climate data downscaling as it relates to transportation systems and development of an Arizona DOT Climate Engineering Assessment for Transportation Assets.

Flood Event: State of the Practice

Flooding and the effects and impacts of flooding along transportation corridors has caused billions of dollars of damage and countless deaths. Technology currently exists to accurately pinpoint those areas along a transportation corridor that are susceptible to flooding. “Although there are tools…they have not yet been integrated to provide sufficient planning and prediction information required by state DOTs to carry out flood planning, risk management, mitigation, operations and emergency response activities." Further research is needed to translate the available technologies into a suite of tools and methods for use by decision-makers (*2*).

The largest hurdles for state DOTs in connection with flooding and risk assessment tends to be the shortage of an end-to-end framework that addresses planning, risk management, hazard mitigation, maintenance repairs, and life-cycle projecting. This is particularly true when an event or emergency has extensive cascading impacts. The main avenue to finding a solution is to develop an approach that could funnel all these issues to one place for proper analysis within the state DOT utilizing current technology, tools, and partnerships that could benefit the DOTs (*3*).

State DOTs generally utilize some form of flood frequency analysis to evaluate a given asset. Design and response standards may not provide enough flexibility to unusual or extreme weather occurrences. Certain assets may not require any special treatment, as available data and standard design guidelines offer acceptable levels of mitigation. This is particularly true when the asset is either at the largest, most monitored level, or is very small and maintenance oriented. Issues arise when a nonmonitored asset is overwhelmed, and when the best representations of probability distribution of floods equal to or longer than 2 years, no longer applies. The range for the confidence limits of that distribution is relatively tight, because in general the 50 largest floods are used to establish the best fit line for the asset (*4*).

Overtopping and System Hotspots: State of the Practice

NCHRP Synthesis 20-05 (46-16) was completed in late 2016 with the objective to produce "a state of the practice report on how the transportation community is protecting roadways and mitigating damage from inundation and overtopping." The report documented "the mechanics of damage to the embankment and pavement, analysis tools available, and design and maintenance practices for embankment protection." The synthesis considered the inundation-only condition of pavements and subgrades (*5*). The immediate risk of overtopping and inundation (an overwhelming abundance) is an indication of how crucial that particular segment of roadway within the system is to moving traffic. Determining how quickly that route can be up and running again is essential to an efficiently functioning transportation system. In rural parts of Arizona it is common to use vertical curves and to take advantage of natural low flow areas to address the roadway prism drainage needs. To clarify, roadway overtopping generally begins when the headwaters rise to the elevation of the roadway (as seen in Figure 7). The overtopping will usually occur at the low point of a sag vertical curve on the roadway.

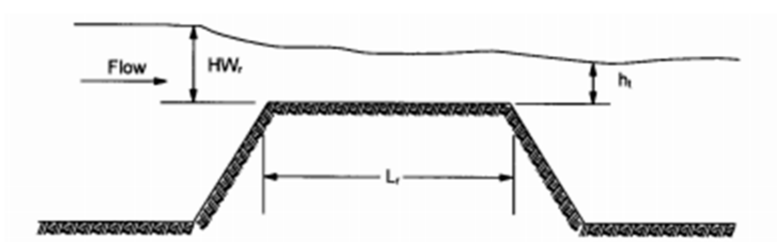

FIGURE 7 Arizona DOT cross section (*Drainage Manual*, 2016).

Hydraulic-Related Failure: State of the Practice

Current industry practice looks to develop a "risk and reliability-based methodology" that can be utilized to better link scour depth estimates to probability at the crossing of rivers, washes, streams, and transportation assets. In addition, state DOTs need an approach for determining a target or range of reliability for the service life of that asset that is consistent and reasonable for the design load and resistance factors (*3*).

Event uncertainty makes identifying probability from a limited number of flood events and linking a range of reliability from those events challenging. The probability of exceeding a design-scour depth over the service life of a bridge additionally has a low likelihood. Hydraulic parameters such as roughness coefficient, channel energy, and critical shear stress also contain uncertainties. Inputs in hydraulic models estimate flood elevations and velocities. Uncertainties in the estimates will translate to uncertainties in resulting calculations.

Changes in the wash structure over time such as upstream and downstream impacts, surface water and groundwater wash–instability magnification effects (as seen in Figure 8) drought, vegetation, bank erosion, and sediment transfer are all factors which inject additional uncertainty. Supplementary data to more efficiently depict activity in the area is needed to limit the amount of uncertainty in the estimates. A way to limit the uncertainty of all these conditions and factors is the utilization of measured data such as high water marks, discharge measurements, broad system sensors, water surface effects, and new usages of historical data. Attempting to establish a repeatable process to address hydraulic-related failure and assessing hydraulic, geomorphic, vegetation, construction, and maintenance impacts from those failures is specific to the FHWA asset management Moving Ahead for Progress in the 21st Century Act guidance.

In addition, FHWA guidance directs federally funded projects to incorporate risk into bridge scour analyses and project development. It dictates that design efforts for new bridge foundations should withstand "the effects of scour caused by hydraulic conditions from floods larger than the design flood" (*3*).

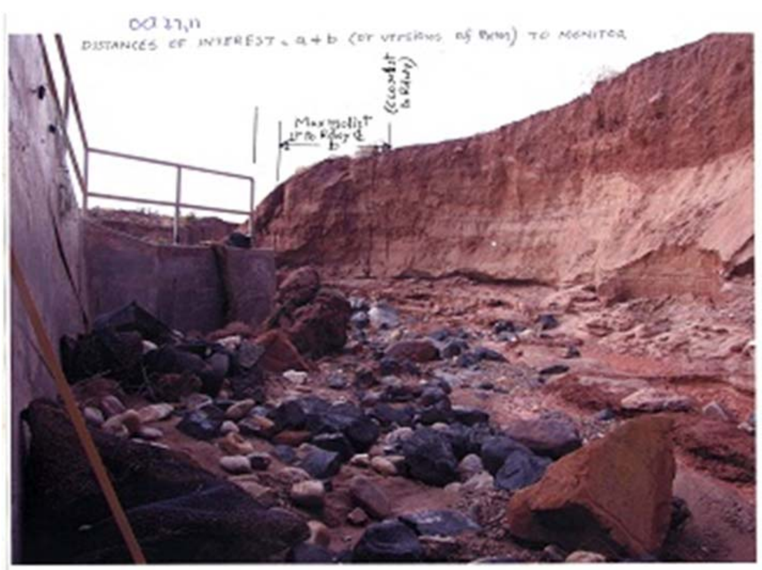

FIGURE 8 Hydraulic-related failure monitoring due to ground water, channel energy, and altered flow. (Photo: Arizona DOT, 2012.)

Extreme Weather: State of the Practice

FHWA's climate change and extreme weather vulnerability assessment pilot projects collected a wide geographic sampling of vulnerabilities in the transportation asset universe. Establishing transportation asset sensitivity to extreme weather can contribute to a systematic approach to programming adaptive capacity strategies and asset life-cycle prioritization. These vulnerability assessments gather data on assets, identify characteristics and sensitivities, analyze historical weather data, determine a usable climate projection model, and through this iterative process develop a vulnerability framework (*6–8*).

Integrating risks based on the severity or consequence of an extreme weather impact and determining the probability or likelihood it would occur to a specific asset at some point now or many decades forward is challenging. But, even low-likelihood risk assessment modeling using historic, current, or 2050/2100 climate projection can allow state DOTs to categorize assets by a low, moderate, or high rating. "The integrated risk is often represented by a two-dimensional matrix that classifies risks into three categories (low, moderate, high) based on the combined effects of their likelihood and consequence." An example matrix risk rating matrix used by the San Francisco pilot is provided in Figure 9 (*7*). Risk application within the context of this paper refers to the additional analysis undertaken to put a variety of determined flows into a context of likelihood or future recurrence interval in relation to historic peak-flow discharge.

Resilience Building: Arizona DOT State of the Practice

The reality of a changing climate means that transportation and planning agencies need to understand the potential effects of changes in temperature, storm activity, and precipitation patterns on the transportation infrastructure and services they manage. These changes can result in increased heat waves, droughts, storm activity, early snowmelt, wildfires, and other impacts

		Consequence				
		1	2	3	4	5
Likelihood	1	2	3	4	5	6
	2	3	4	5	6	7
	3	4	5	6	7	8
	4	5	6	7	8	9
	5	6	7	8	9	10
Risk	Low		Moderate		High	

Unacceptable, major disruption likely; priority management attention required.

Moderate Risk (Orange)

Some disruption; additional management attention may be needed.

Low Risk (Green)

Minimum impact; minimum oversight needed to ensure risk remains low.

FIGURE 9 Risk rating matrix; preliminary study of climate adaptation for the statewide transportation system in Arizona (Report No. FHWA-AZ-13-696, Arizona DOT, 2014).

that could pose new challenges for Arizona DOT. The goal of the 2014 *Preliminary Study of Climate Adaptation for the Statewide Transportation System in Arizona* study was to establish a path for Arizona DOT to continue working toward being more resilient, flexible, and responsive to the effects of global climate change. The study identified, key individuals within Arizona DOT with decision-making authority relevant in incorporating climate change adaptation in planning, design, and operations, framed relevant literature and best practices for climate change adaptation as relevant to the desert southwest, developed a research agenda for Arizona DOT to further understand the impacts of climate change on the agency, and identified key areas for further research on climate change adaptation for Arizona DOT's statewide transportation system beyond the scope of the initial study (*8*, p. 5).

The 2014 effort led Arizona DOT to participate in the FHWA Climate Change Resilience Pilot program—the pilot effort assessed the vulnerability of Arizona DOT-managed transportation infrastructure to Arizona-specific extreme weather. In the long term, Arizona DOT sought to develop a multistakeholder decision-making framework—including planning, asset management, design, construction, maintenance, and operations—to cost-effectively enhance the resilience of Arizona's transportation system to extreme weather risks. Arizona DOT elected to focus on the Interstate corridor connecting Nogales, Tucson, Phoenix, and Flagstaff (I-19, I-10, and I-17). This corridor includes a variety of urban areas, landscapes, biotic communities, and climate zones which present a wide range of weather conditions applicable to much of Arizona. The project team examined climate-related stressors including extreme heat, freeze–thaw, extreme precipitation, wildfire, and considered the potential change in these risk factors as the century progresses.

As part of the pilot program, the study leveraged the FHWA Vulnerability Assessment Framework customizing it to fit the study's needs. The project team gathered information on potential extreme weather impacts, collected datasets for transportation facilities and land cover characteristics (e.g., watersheds, vegetation), and also integrated these datasets to perform a high-level assessment of potential infrastructure vulnerabilities. Each step of the process drew heavily on internal and external stakeholder input and feedback. This assessment qualitatively addresses the complex, often uncertain, interactions between climate and extreme weather, land cover types, and transportation facilities—with an ultimate focus on potential risks to infrastructure by Arizona DOT district. Preliminary results were presented in focus groups where Arizona DOT regional staff provided feedback on the risk hypotheses developed through the desktop assessment. The results of the assessment were, organized first by district, then by stressor, and then further delineated by land cover types (e.g., desert) which are considered qualitative potential factors that could either alleviate or aggravate the impacts of extreme weather phenomena (*9*, p. ES-1).

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This effort is in response to hydrological and hydraulic engineers need to address climate change and engineering dynamics. In addition, the research problem statement goes on to explain, in order to provide "hydraulics engineers with the tools needed to amend practice to account for climate change, output from climate models must be downscaled and modified to provide recommended changes to regional precipitation data for design events used by hydraulics engineers. Collaborative efforts between climate scientists, hydrologists, hydraulic engineers, and coastal engineers, are essential to producing these design inputs that are needed to amend hydraulic designs. Incorporating the results of climate models will have very large cost implications for future infrastructure. Overestimating the magnitude of peak flows suggested by climate models can result in costly over sizing of drainage infrastructure, while underestimating may leave infrastructure vulnerable and their resultant flooding impacts on surrounding lands and structures inadequately addressed" (*10*).

USGS PARTNERSHIP AND LAGUNA CREEK PILOT PROJECT

Infrastructure in or near dry-land river channels are susceptible to a variety of geomorphologic and hydrologic hazards caused by floodwaters. Historically, many dry-land channels in northeastern Arizona were broad, shallow, and mainly unvegetated. As a result, floodwaters in the past were conveyed slowly and gently through stream channels and surrounding floodplains at relatively low velocities and shallow flood depth. Today, many dry-land channels have changed dramatically and have become largely incised into the floodplain, while the carved banks are being stabilized by vegetation, in many cases by nonnative tamarisk (*Tamarix ssp*). The increase in bank stability may cause channels to incise deeper into floodplains, leading to narrower, less sinuous stream beds that can potentially convey floods at higher velocities. Additionally, channels in this region have the ability to convey and deposit large amounts of sediment, and sediment volumes may become larger as flood velocities increase. Ultimately, larger floods at higher velocities can erode the outside of channel bends where velocities are typically high, and deposit sediment on the inside of bends where velocities are naturally lower. This commonly causes channel migration, meander cut-off, and avulsion (*11*). Prior studies have found that river channel instability in arid and semiarid regions and erosion caused by channel migration resulted in economic losses that were potentially five times greater than potential flood inundation losses (*12*). However, potential channel migration is rarely accounted for in flood risk assessment (*11*).

Channel erosion and deposition have occurred at Laguna Creek at US-160 bridge site. The channel is incised into the floodplain, the banks on the outside of bends are eroding, and clear evidence of past channel migration and meander cutoff can be seen just 250 ft downstream from the bridge structure. Additionally, any further erosion that occurs near the roadway may impact the transportation infrastructure. This erosion caused the Arizona DOT and the USGS Arizona Water Science Center to deploy multiple sensors and use new technologies to collect many different types of data to help better understand the dynamic hydrologic conditions at the bridge site prior to construction efforts.

The effort started with the deployment of real-time hydrologic data collection equipment to help form a baseline reference for the river conditions at the bridge. First, a rapid deployment stream gage was installed on the bridge structure. This gage contains equipment that measures river stage and surface velocity, which is telemetered to provide users with real-time stream flow information on the web. The gage was also outfitted with two video cameras that are triggered to capture video once the river stage exceeds a predefined threshold. These cameras provide video of underneath the bridge and also the bend upstream of the bridge, which is the focus of this effort, and can provide insight to engineers by capturing video evidence of flows and recording potential erosional events on the banks. The video can also be analyzed with Large-Scale Particle Image Velocimetry (LSPIV) software to calculate and map the surface velocity of the flows upstream and downstream of the bridge. The LSPIV software is used to help calibrate and confirm the gage's velocity sensor along with computing the river discharge at the gage. All of the sensors were deployed over a year in advance of planned construction and provided a snapshot of the potential flow conditions at the Laguna Creek site.

The next objective included conducting an indirect measurement of discharge using evidence from the flow that increased the bank erosion near the upstream bridge abutment prior to gage installation. This indirect measurement resulted in a peak flow value of 1,300 ft^3/s. This discharge was compared with the USGS program StreamStats, which uses information from gages in the region to predict the flood flow frequency at ungaged watersheds using basin characteristics, in this case, watershed area. This method predicts the 2-year event to be 1,600 ft^3/s (*13*). In other words, a statistically common flow event of 1,300 ft^3/s caused the bank erosion on the bank upstream from the bridge. After the gage's installation, an additional flow event occurred on September 30, 2016, which provided another opportunity to conduct an indirect measurement of discharge. This event peaked at 1,270 ft^3/s with a peak measured velocity under the bridge structure of 6 ft/s. Again, this event was less than the predicted 2-year flow event using StreamStats and continued to erode the bank next to the upstream bridge abutment.

Another interesting approach that the USGS was able to utilize at the Laguna Creek site was the use of ground-based lidar scans in conjunction with photogrammetric surveys collected via small unmanned aerial systems (UAS or drones). These data collection techniques are used to collect high-resolution point clouds, often under 2-cm resolution, to create 3-D digital elevation models and high-resolution orthoimagery. The use of the terrestrial lidar system is beneficial under structures and when very high-resolution models are sought; the small UAS-based collection is best for efficiently collecting topographic data over large areas (miles) and areas that are difficult to survey using ground-based systems. These models can be collected before and after events to both visualize and measure land surface changes and can be especially important when trying to quantify erosional changes in stream channels. The land surface models can also be used in 2-D hydraulic modeling software as well as LSPIV software to both confirm and predict different flow scenarios in the channel and around the bridge structure. Lastly, the high-resolution orthoimagery can be used to visualize current conditions at the site and can be used to help inform decision-making for the project. These technologies allow engineers to bring the site into the office and can be very useful in planning and scoping scenarios.

This collaborative effort conducted by the Arizona DOT and the USGS was designed to provide snapshots of the potential hydrologic conditions at Laguna Creek at the Highway 160 bridge site. Collecting data during the year prior to construction provided a comprehensive data set including stream flow, river stage, surface velocity, video capture of flows, and high resolution digital elevation models and orthoimages. These data provide complete hydrologic monitoring that can be used by engineers and scientists alike to better understand the hydrologic and hydraulic conditions at the Laguna Creek site, and these data can be used to inform decision making for future construction efforts. These instruments will be left in place post-construction for continued data collection, and the data can be used to provide insight into the effectiveness of bank stability operations around the bridge structure.

ACKNOWLEDGMENTS

The authors wish to thank the people that supported this effort: Arizona DOT State Engineer's Office and the Arizona DOT Environmental Planning Group. The agencies that funded this effort: Arizona DOT and the FHWA Sustainable Transport and Resilience Team.

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Inspection and Assessment for Bridge Structure Management Decision-Making

Development of Limit State–Based Structural Health Monitoring Thresholds for Efficient Reporting and Alerting

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Intelligent Infrastructure Systems

A common criticism of structural health monitoring (SHM) systems is the inability of the system (or installers/consultants) to provide the translation of raw sensor measurements into actionable information that owners and operators (referred herein as end-users) can use for management decision-making. Reasons why this shortcoming exists include poor anticipation during the system design of how the SHM system was to be used, complexity of the measurements made, and the lack of a comprehensive input–output characterization of the structural system. To address the challenges associated with these limitations, it is important to recognize that the location and specification of sensors does not alone constitute a SHM system design. Instead, the system design must include the definition of performance-based allowable thresholds that directly correlate to structural safety, traffic safety, or operations limit states. This paper presents a framework for performance-based design of SHM systems, from the specification of instrumentation type and location through the requisite analysis needed to properly specify allowable thresholds. The paper will also discuss how such an SHM system design could be integrated into a bridge management system.

An illustrative example will be presented based upon an actual SHM system design and computation of allowable substructure movement thresholds. For this case study, the thresholds were computed based upon superstructure live-load strength-limit states, substructure serviceability-limit states, and a kinematic-limit state associated with allowable movement at the expansion joints. The thresholds were then used within a live visualization during a critical construction event where the end-user was quickly able to establish the performance of the bridge in real time. The proposed framework is a step forward in addressing the challenge of understanding how SHM systems can be used not only to understand how a structural system responds to a given input, but more importantly to present how that response affects the structure's ability to withstand performance limit states within a bridge management system.

A common criticism of structural health monitoring (SHM) systems is the inability of the system (or rather the installers–consultants) to provide the translation of raw sensor measurements into actionable information that end-users (owners and operators) can use for management decision-making. There are many reasons why this shortcoming exists including but not limited to poor anticipation during the system design of how the SHM system was to be used, complexity of the measurements made, and the lack of a comprehensive input-output characterization of the structural system. However, one of the largest reasons why this shortcoming might exist is the lack of broad design and specification guidelines. The lack of guidance manifests in two types of SHM system failures: (1) a system that was specified, designed, and installed well but lacks integration into the bridge operator's decision-making procedures and (2) a system that was improperly specified or designed leaving the end-users with unrealized expectations.

A design framework for SHM systems was developed to address the shortcomings discussed above. In deriving the framework, emphasis was placed on the end-use of the system and how it integrates into existing decision-making processes. In this light, the framework can be considered a performance-based design approach for SHM.

SHM PERFORMANCE-BASED DESIGN FRAMEWORK

The SHM performance-based design framework was developed after years of experience (personal and industrywide) in both successful and unsuccessful applications of SHM. All too often, it was heard that SHM requirements were recommended by vendors to the end-users often with little consideration of the end-user's personnel workflow, operations, and experience with SHM. Additionally, vendors did not properly educate the end-user on managing a system that generates large amount of data that does not directly correlate with their metrics of interest (you cannot directly measure remaining service life). This was seen largely after the collapse of I-35 in Minnesota where sensor and SHM vendors flooded the market with potential monitoring systems for avoiding a similar disaster for their customers. The lack of success of the systems installed in this era created a backlash from which the industry is still recovering.

A common theme in the discussion above is the lack of communication with the end-user of the SHM system. It is critically important to understand the needs of the end-user while understanding the limitations with which they operate. These two points will drive how the system is designed and eventually how the system integrates into existing bridge management workflows with minimal or only positive disruptions. Thus, when the framework was developed (shown in Figure 1), engagement with the end-user was appointed both the first and last steps to highlight the fact that the system is borne out of end-user based requirements and ultimately is a tool that the end-user must accept and use. Each of the five steps in the framework will be discussed in further detail in the following sections.

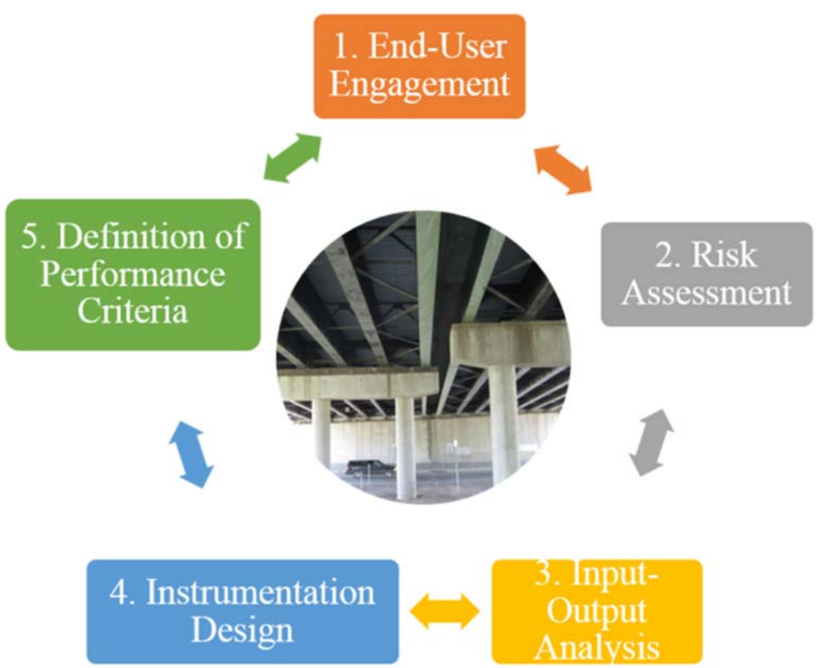

FIGURE 1 SHM design framework.

Step 1: Engage the End-User

The first step of any SHM design project is engaging the end-user to understand the motivation of the project, to establish the level of exposure the end-user has had to SHM requirements (installation methods, power and communication, computer hardware, maintenance and operations, software, etc.) and to identify the specific metrics that the end-user wants to monitor. At the end of the initial end-user engagement phase, the feasibility and applicability of SHM to the specific case should be established. Given that SHM is applicable, the end-user should have clear understanding of what the project entails, from installation through long-term operation. The goal of the designer is to have clear performance metrics defined to which the SHM system shall be designed. An example of performance metrics to guide SHM design include but are not limited to expansion bearing performance, critical load path distribution, traffic operations, traffic safety, and movable span operations.

The designer should also have clearly defined threshold parameters from the end-user at this stage as well. In some cases, the thresholds could be directly defined at this stage ("I want a text message when 3-minute sustained winds exceed 50 mph"). However, most of the thresholds will be defined with respect to other parameters that the end-user is familiar with or could be very high level. For example, the end-user might be interested to know when their movable span is not seated properly. This is a challenging requirement because what is the definition of "proper"? Is it simply traveling a certain distance, or are there other parameters that define a proper seating? Usually, there is no direct measure of proper seating, and the engineer must devise an instrumentation program to indirectly assess that measure. Such a threshold would then be defined through engineering analysis as part of the project. However, it is the SHM designer's role to translate measured responses (and the thresholds defined as a function of their values) into metrics that the end-user is familiar with. To continue the example of the movable bridge seating, perhaps an instrumentation approach of measuring distance and strains allows for an engineering calculation of total travel and the seated imbalance of the movable span. Together with the end-user, the SHM designer can then define threshold bounds on these metrics that align with the existing bridge maintenance and operations guidelines for that bridge.

As seen in many new SHM projects, the end-user might be interested in having an SHM system provide support for remaining service life calculations. In this case, the monitoring metrics are very high level and require further analysis to extract monitoring requirements and threshold criteria. Since there is no "remaining life" sensor that we can simply apply, SHM designers are responsible to work directly with the bridge designers or responsible engineers to identify if, and how, a monitoring system can be used as a tool to aid in their engineering assessment of service life.

An additional component to consider in this initial stage is to have understanding of how the SHM system is envisioned to integrate within the existing bridge management platforms. Will the alerts generated out of the SHM system be issued through an existing intelligent transportation systems? Similarly, will visualization be created for the SHM to be displayed within traffic management centers? These are important questions to consider, as the goal is to minimally disrupt the end-user's management structure and complement the existing maintenance and operations components with the measured performance metrics. Again, the more informed the end-user is at the conception of the project, the more productive and successful the end product will be.

Step 2: Risk Assessment

The second step of the SHM design process is closely married to the first step. The SHM designer carries out a formal risk assessment of the structure where the hazards and vulnerabilities are clearly defined. Risk can be defined in many different ways depending on the field of application, however in this context the author defines risk as a combination of three components:

- Hazard: the likelihood of an event to occur which could potentially induce ill effects,
- Vulnerability: the likelihood that, given the occurrence of a hazard, a system will fail, and
- Exposure: given the failure of the component, what are the consequences that arise (financial, human life, quality of life, etc.).

By looking at the product of these three components of risk, one is able to then prioritize a set of risks. The benefit of doing such an analysis is that at the end of the prioritization task the analyst is able to develop ways in which to mitigate the specific components so as to reduce the overall risk, relatively compared to the others. For example, a hazard could be the impact to a movable bridge from a ship. The corresponding vulnerability could be the failure of the operating machinery, resulting the bridge not being able to open. The exposure would potentially be the loss of the bridge plus damages to the ship and any human life that might be lost. This risk can be mitigated by reducing the likelihood of the ship impact to occur (decrease likelihood of hazard) or strengthening the bridge against such an impact (reducing vulnerability), or a combination of the two. Note that this could be done procedurally or legislatively and would not require monitoring.

For the interested reader, please refer to Moon et al. (*1*) on a comprehensive study of major risk components faced by bridges and how that can be used in prioritization efforts for a network of bridges. While that paper discusses the applicability of risk assessment to bridge networks, the approach is applicable for one bridge since the interest is to prioritize the set of risks defined for one bridge.

It is most likely required that the SHM designer will need input from the end-user to populate the list of hazards and vulnerabilities, however it is important for the designer to have structural engineering experience to help supplement the list with appropriate items. A role where the end-user plays a crucial role in this step is the prioritization of risks (combination of hazards and vulnerabilities). While in formal risk assessments a third component of risk, exposure, is also computed, in this case it is not required since the owner and engineer will work to prioritize the risks based on heuristics rather than other common exposure metrics (mostly value-driven). The result of the second step is a list of prioritized hazards and vulnerabilities that will drive the design of the SHM system.

Step 3: Input–Output Analysis

The third step in the SHM design framework is the conversion of each of the prioritized risks into a series of measurable inputs and outputs. Inputs are defined as those measured parameters which are independent of the bridge structure (e.g., wind speed, vessel impact, overloads, temperature gradients) while outputs are defined as the response of the structure as a function of material or structural properties (e.g., displacement, strain, surface temperature). As the designer carries out this analysis, the sensing approach to meet the SHM design objectives begins to take shape. Some input–output measurements are fairly straightforward (wind speed) while others still require a degree of indirect measurement to ascertain whether a hazard occurred (ship impact).

Step 4: Instrumentation Design

With a list of specific measurement requirements defined from an end-user driven risk assessment the SHM designer can properly locate and specify the sensors needed to meet the design objectives. This stage may require refined analysis to aid in the process of locating sensors through sensitivity studies as well as computing the magnitudes of response to the desired inputs so that sensor ranges can be properly specified. The end result of this step is the development of SHM contract drawings and specifications which may be bid for construction or used by an SHM integrator for procurement and installation.

Step 5: Definition of Performance Criteria and Thresholds

As part of the SHM design process, each of the prioritized risks is used to specify performance criteria and alerting thresholds to be commissioned upon installation of the system. SHM performance criteria are defined as the metrics used to establish acceptable levels of hazards or vulnerabilities. Some performance criteria are based on institutional or code requirements, such as maximum wind speeds for bridges to remain open to traffic, and often include various levels of satisfactory performance. However, most of the requirements used in SHM system design are based on structural safety performance metrics and require detailed engineering analysis to establish what the acceptable levels of performance, or thresholds, are. It is this step of the SHM design process where it is important to have structural engineering expertise on the SHM design team. A common complaint of SHM systems is that thresholds proposed by SHM vendors are not founded on engineering design metrics of the bridge and are instead focused on anomaly detection or machine learning. It is important for the designer to understand how the response thresholds they are presenting translate to strength and serviceability limit states of the structure.

When the SHM system is fully designed and specified and the performance metrics and corresponding thresholds have been computed, it is critical to receive acceptance of each component of the SHM system design. The end-user must agree on the performance criteria used and on the threshold values presented, for it is their staff that will be receiving the alerts when thresholds are exceeded and it is important for them to have full understanding of what is entailed in an alert. This stage is critical for the end-user to visualize how the SHM system is going to integrate into their bridge management system.

At this stage, it is important to consider that bridge condition, hazards, and vulnerabilities can be time-dependent. That is, it is recommended to revisit this process periodically to assess not only how the initially defined thresholds are performing, but to reassess the state of the bridge and its environment to ensure that the monitoring system and the alerting protocols are still appropriate. This suggestion is not uncommon to bridge managers as it is analogous to the protocols followed for updating of live load ratings. When the condition or loading of a structure changes, the load ratings must be updated.

APPLICATION: CASE STUDY ON A STEEL MULTIGIRDER BRIDGE

The SHM design framework discussed above was carried out on a steel multigirder bridge in the United States. A brief background of the structure will be presented followed by a discussion of how the framework was implemented for this structure over the next sections. The project is of a confidential nature, and specific details cannot be shared in this paper. However, the general application of the framework is still discussed.

Background

A steel multigirder bridge in the United States was located on a site where significant construction was occurring nearby as an industrial facility was being built. As part of the construction, heavy loads (large prefabricated components moved on self-propelled modular transporters) were proposed to be hauled underneath the bridge due to limitations in transportation logistics. The owner of the structure was concerned about what impacts the heavy loads would have on the performance of the structure and required the site developer to establish requirements for and install an SHM system for the multigirder span.

The structural system consists of a two-span continuous steel multigirder structure supported by expansion bearings on an abutment, fixed bearings on a 70-ft-tall reinforced concrete pier and a steel plate girder floor beam which is pin connected to both the steel girders and the supporting 60-ft-tall reinforced concrete columns below. The two reinforced concrete columns supporting the multi-girder span floorbeam also support the expansion bearing of a multi-span continuous steel truss over 1,000 ft in length. The steel superstructure is composite with a reinforced concrete deck and supports four lanes of traffic. Further details about the bridge are presented in Warren and Dubbs (*2*) for the interested reader, however the aim of this paper is focused on the SHM design process, particularly the definition of performance criteria.

SHM Design

End-User Engagement

The SHM project was initiated by a meeting with the SHM design experts, site developers, and the bridge owners. At this meeting, the performance requirements of the SHM system were specified by the bridge owner as the following:

- Monitor for permanent rigid body translations of the two piers in all three directions;
- Monitor for permanent rigid body rotations of the two piers in all three dimensions;
- Ensure that any measured rigid body movements do not impact the load rating of the steel multigirder span with respect to strength and serviceability limit states;
- Ensure that any measured rigid body movements do not generate cracking in the reinforced concrete piers; and
- Ensure that any measured rigid body movements do not bottom out any of the movement systems.

While the owner laid out the requirements above, it was still required to ensure that additional hazards were not relevant on the site which might also affect structural performance during one of the moves.

Risk Assessment

The project presented herein included an SHM system with a very specific set of performance requirements and a rather narrow set of risks. However, there is a vast amount of literature where risk assessments are used for bridge prioritization (*1*) and the interested reader is referred to those resources for more general examples. The main risk for this case study included the hazard of the heavy load passing between the two reinforced concrete piers. The vulnerabilities included overstressing of superstructure components, cracking of the concrete piers, bottoming out of movement systems, and differential settlement of the piers. The main risk defined as the combination of these hazard and vulnerabilities is thus the reduction of load carrying capacity of the existing bridge system due to the heavy load.

Input–Output Analysis

The input–output analysis was carried out to not only determine what sensors were needed to monitor the objectives laid out above, but also to identify what other factors contribute to normal movements of the structural system which would need to be characterized as part of the baseline evaluation of the bridge system. The two main monitoring objectives were focused on tracking rigid body movements (translations and rotations) of the two reinforced concrete piers. Thus, it was important to consider all possible contributing factors to those movements and all possible ways that the piers might deform under service loads. One would not expect rigid body deformation of any kind under service loads, so it was required to provide enough measurements that elastic flexural deformations could be decoupled from any potential rigid body deformation. When considering movement of the substructures, there were two major inputs defined that were of interest to this project—thermal gradients and the heavy moves. The corresponding outputs are of course either the translation or rotation of the piers. The translation of these inputs and outputs to monitoring measured is shown in Table 1.

Instrumentation Design

The measures outlined in Table 1 were used to drive the instrumentation design. The benefit of this systematic framework is that by the time the designer reaches the instrumentation design stage, the efforts can mostly be placed on sensor location and specification instead of attempting to conceptualize the entire design in one step. As mentioned above, Warren and Dubbs (*2*) present additional information on the background of the bridge. The paper also presents the final SHM instrumentation plan in detail. Generally, the SHM system utilized 18 tiltmeters, 15 strain gages, 12 displacement gages, six piezometers, 10 inclinometers, six extensometers, and a weather station to meet the measurement requirements. The instrumentation plan is shown below for clarity (Figure 2).

TABLE 1 Input–Output Analysis

Category	Performance Metric	Measurement Types	Sensing Approach
Input	Thermal movement	Ambient temperature Local temperature Superstructure strain Superstructure expansion Pier rotation	Weather station Thermistor VW strain gage VW displacement gage VW tiltmeter
Input	Input from heavy moves	Pore water pressure Soil inclination Soil strain	VW piezometer VW inclinometer VW extensometer
Output	Global rotation of piers	Rotation (differential or uniform)	VW tiltmeter
Output	Global movement of piers	Rotation Superstructure strain Superstructure expansion	VW tiltmeter VW strain gage VW displacement gage

NOTE: VW = vibrating wire type sensing.

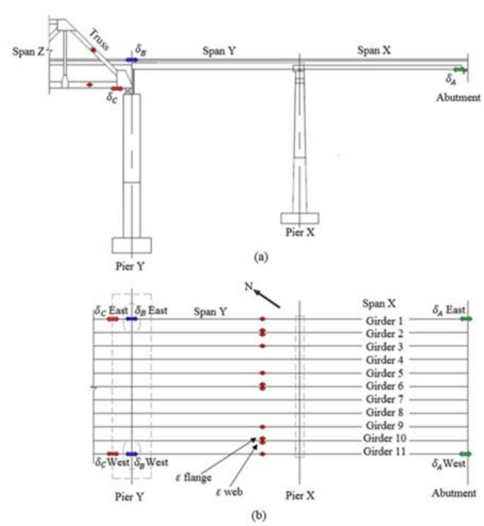

FIGURE 2 Instrumentation plan for the SHM system (*2*).

Definition of Performance Criteria

The challenging part of this project was the definition of acceptable performance criteria. Once the system design was prepared, the bridge owner was re-engaged to present the design and to discuss performance criteria and alerting thresholds. The last three SHM performance requirements listed above heavily drove what analysis was needed to support the computation of monitoring thresholds, which include:

- Ensuring that any measured rigid body movements do not impact the load rating of the steel multigirder span;

- Ensuring that any measured rigid body movements do not impact the load rating of the steel multigirder span; and
- Ensuring that any measured rigid body movements do not bottom-out any of the movement systems.

Note that none of these performance requirements are readily measured by a single sensor. There is no measure for remaining live-load capacity or allowable additional stress until onset of onset of cracking is initiated. Sure, one could measure strain in the piers, but what magnitude of response is of concern? In order to develop quantitative thresholds associated with these performance limit states, the following analyses were carried out:

1. Superstructure live load rating. A 3-D finite element (FE) model was used to develop the refined rating of the superstructure in its current configuration. The preliminary live-load rating factors were all well above 1.0, suggesting that there was sufficient capacity to accommodate demands from substructure movements. The extent to which the substructures could move before unsatisfactory live-load rating factors were observed was computed by incrementally applying rigid body movements to the substructure elements in the FE model and regenerating the superstructure live-load rating factors until a value of 1.0 was reached. The amount of movement needed to generate this break-even rating factor was then defined as the threshold for this performance limit state.
2. Substructure cracking. It was hypothesized that the piers would potentially crack under tensile stresses due to *p*-delta effects associated with their rotation and the dead load of the multigirder span above, resulting in a failure of a serviceability performance state. A geometric nonlinear analysis of the same FE model described in the first analysis above was used to establish at what extent of rotation tensile stresses in the extreme fiber of the piers reached cracking magnitudes associated with the material properties of concrete.
3. Kinematic analysis. The final performance limit state analyzed was a kinematic assessment of the allowable movement of the expansion mechanisms. If either pier rotated or moved longitudinally, the movement systems could potentially either be closed or opened too far, either case presenting a performance failure for the bridge owner. The kinematics of all movement mechanisms were analyzed and the net allowable movement at each joint (factoring in the space needed for normal expansion and contraction of the structure) was computed. The corresponding translations and rotations of the substructure that generated these movements in the mechanisms was then computed and reported as the allowable thresholds for this analysis case.

Following the three analyses described above, the governing limit state per each of the response metrics of interest of the bridge was computed are shown in Table 2. A schematic of the bridge showing the various directions of movement for the two piers is also shown in Figure 3.

Implementation

A report was prepared documenting the allowable movements for the piers during a heavy move operation and accepted by the bridge owner. It should be noted that the analysis carried out herein focused on the superstructure performance alone and did not consider the geotechnical capacities or forces generated on piles. Those analyses were completed by a separate consultant.

TABLE 2 Computation of Allowable SHM Thresholds with Governing Performance Case

	Movement	Allowable	Governing Case
Pier 1	+ Ry (°)	0.15	Pier serviceability–differential rotation
	– Ry (°)	0.06	Kinematic–Abutment 1, expansion joint
	+ Dx (in.)	2.2	Pier serviceability–differential longitudinal movement
	– Dx (in.)	0.91	Kinematic–Abutment 1, expansion joint
	+ Dy (in.)	0.35	Pier serviceability–differential transverse movement
	– Dy (in.)	0.55	Pier serviceability–differential transverse movement
	+ Dz (in.)	0.2	Pier serviceability–differential settlement
	– Dz (in.)	0.3	Pier serviceability–differential settlement
Pier 2	+ Ry (°)	0.3	Kinematic–Span 3, rocker bearing
	– Ry (°)	0.09	Kinematic–Span 3, rocker bearing
	+ Dx (in.)	4.34	Kinematic–Span 3, rocker bearing
	– Dx (in.)	1.32	Kinematic–Span 3, rocker bearing
	+ Dy (in.)	0.75	Pier serviceability–transverse movement
	– Dy (in.)	0.55	Pier serviceability–transverse movement
	+ Dz (in.)	6.2	Superstructure rating–negative bending interior girder
	– Dz (in.)	6.2	Superstructure rating–negative bending interior girder

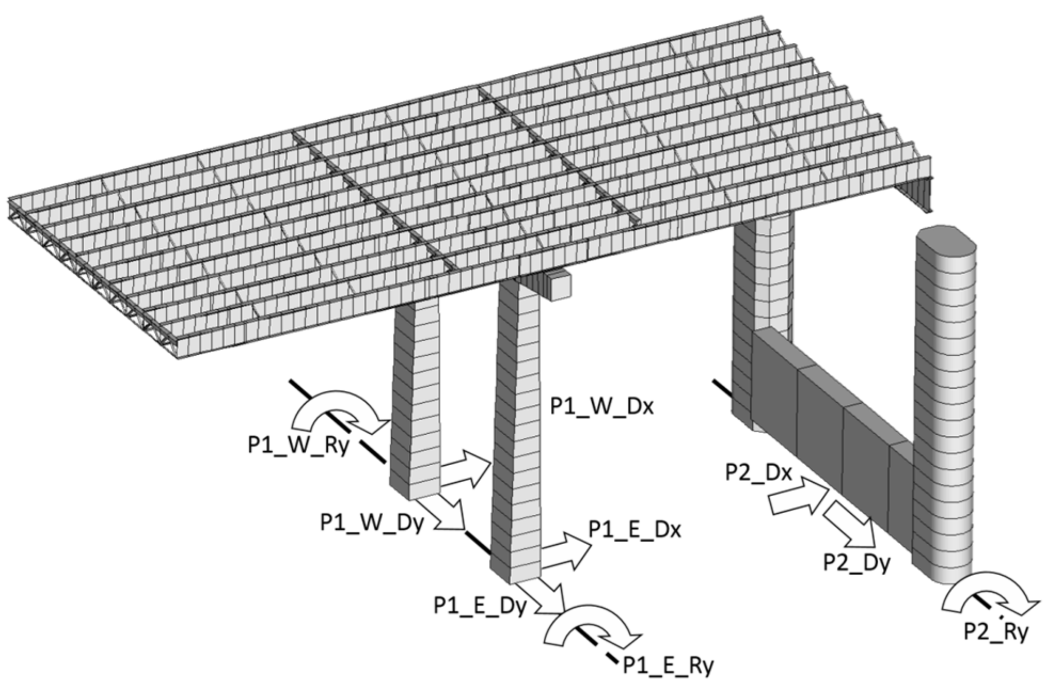

FIGURE 3 Schematic showing movement sign convention for the two piers.

The SHM system was installed by agency contractors in October of 2015 and is in operation as of publication of this paper. The planned monitoring period is 5 years in total, which was necessary to capture all the planned heavy moves on site. For this project, the main monitoring effort focuses on system performance during the heavy moves. As such, detailed reports will be prepared after those moves documenting not only the SHM results but also findings of pre- and post-move bridge visual inspections.

As part of the implementation process, the bridge owner requested a load test to verify SHM system performance and to understand the performance of the geotechnical instrumentation installed as part of the geotechnical consultant's scope of work. The load test utilized a 200-ton Caterpillar 777D truck fully loaded with stone. The vehicle made several passes under the multigirder span at varying spacing between the two piers. It was noted that the superstructure sensing did not deviate at all from their normal performance and were instead responding to the live loads passing on top of the bridge. The geotechnical sensors did, however, respond to the load and the data was used by the geotechnical engineers to validate subsurface assumptions and soil parameter recovery times.

INTEGRATION OF SHM DATA INTO A BRIDGE MANAGEMENT FRAMEWORK

The specific case study was not incorporated into a bridge management framework since it was a single application of a fixed duration construction monitoring project. However, the framework followed and the products that were developed from the project do lend themselves to a bridge management application. As previously mentioned, a major challenge in the SHM industry has been the translation of raw sensor measurements to actionable information that end-users can readily understand. The case study presented herein was able to translate raw measures of sensor data into a single dashboard that indicated the performance of the bridge based on a series of engineering calculations and analyses. At any point in time, but most likely during a heavy move, the bridge owner can open the dashboard and immediately see what effect, if any, the operation is having on their bridge and what specific performance limit state is most vulnerable.

For a general bridge management application, the author does not envision that a bridge owner needs active involvement with the SHM system. Conversely, a properly designed SHM system should take on the active role of analyzing and interpreting the data and alerting key personnel of performance issues (including SHM system performance) immediately as they occur and then providing a means of quickly and effectively disseminating the system measurements so that the personnel can either confirm or deny the relevance and importance of the issued alert. It is envisioned that integration within bridge management software would be straightforward given properly designed alerting thresholds. The bridge manager would need to identify ways in which third-party applications can interface directly for automated integration, or at the very least could set up manual integration process where SHM-issued alerts are first received, reviewed, and manually entered into bridge management applications.

CONCLUSION

The study presents a framework that can be used to approach the development of performance-based thresholds for SHM systems. The acceptance of such a framework provides the opportunity for direct integration with bridge owners' current bridge management frameworks by translating raw sensor measurements into information that the owner can readily understand and, most importantly, that the owner can act upon. The framework is systematic in that it forces the designer to begin the design process with high level requirements laid out by the end-user of the system and then end the design process by translating the raw sensor measurements into actionable information by deriving response thresholds founded on structural engineering analysis. The benefits of such an approach lie in the need of the SHM design to explicitly plan how the responses from each sensor are going to be used in a management framework by informing the end-users of a specific performance limit state exceedance.

A case study was presented where the framework was utilized to design an SHM system that not only used targeted instrumentation but also used structural engineering analysis to compute quantitative thresholds founded on end-user prescribed acceptable performance requirements. Since the end-user did not have experience with SHM, it was the responsibility of the SHM designer to provide a system that can translate raw sensor output into a quantitative indicator of structural performance.

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Robust Registration Algorithm for Performing Change Detection of Highway Bridges Using 3-D Laser Scanning Data

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United States transportation infrastructure facilities, such as bridges, currently have a grade of C+ according to the ASCE Annual Report Card. Long-term spatial changes of bridges can be important precursors of serious structural accidents. Visual inspection methods rely on the experience of engineers for assessing the spatial changes of bridges, but subjective manual six-change inspection introduces several uncertainties due to the lack of detailed spatial data and comprehensive change analysis methods. Three-dimensional (3-D) laser scanning technology enables change analyses of structures by comparing 3-D imageries collected at different times. The challenge of using 3-D laser scanning in bridge change analysis is that existing algorithms for point cloud registration are for aligning data sets collected in a scene where no changes occur. Applying these algorithms for aligning data sets that contain long-term changes of bridges using data collected from different times require engineers to manually select parts of environments that do not change before the algorithm can reliably assess changes. Some studies tried to setup control network in the field to overcome this challenge, but maintaining a control network that would not change between data collection sessions could be time-consuming and difficult for outdoor bridge jobsites. This paper presents a robust point cloud data registration algorithm that accurately registers two sets of 3-D laser scanning data sets collected at different years and contains changes. The results indicate that this new 3-D data registration approach can accurately register the 3-D laser scanning data sets collected from different years for effective bridge change analysis.

Transportation infrastructure facilities such as bridges are deteriorating at an alarming rate due to continuous spatial changes such as deformation or deflection of the bridge elements (*1*). Uncertainties in predicting the exact deterioration rates for bridges can lead to loss of life and property (*2*). Transportation Research Board (TRB) utilizes a transportation asset management (AM) framework for strategic maintaining, managing, and upgrading physical assets such as civil infrastructures through their life cycle (*3*, *4*). Migliaccio et al. conducted a study on the data quality assessment and improvement framework for improving the quality of data collection activity on transportation assets (*5*). Samali et al. highlighted the importance of gathering and analyzing bridge condition data for the bridge management system for predicting the condition of bridges using a data-driven decision-making and plan for maintenance funding (*6*). Several studies also stated the need for reliable sensor data-driven decision-making in the bridge management system for accurately assessing the health of a bridge structure and for performing risk-based AM studies (*7*). In general, traditional surveying technologies collect three-dimensional (3-D) measurements at manually selected 3-D surveying locations to aid engineers in identifying geometric changes of structures. Unfortunately, such methods (e.g., total stations) could hardly collect dense geometric measurements and often requires experienced professional to interpret the data and accurately identify the damages on a bridge (*8*). The 3-D laser scanning technology provides detailed geometric data that facilitate in detecting geometric changes of the changes of the bridge during its service period. However, periodic investigation of the bridge structure using 3-D laser scanning data requires manually aligning two sets of point cloud data collected at different times. Such aligned process is termed as registering two point cloud data sets into one single coordinate system. However, such manual alignment process may significantly affect the analysis results.

Unreliable or inaccurate registration of 3-D laser scanning datasets of a bridge collected at different times (e.g., from year to year or from month to month) can lead to improper detections of spatial changes and eventually leading to unreliable condition assessment of bridge structures. Failure to accurately detect spatial changes may lead to incorrect decision-making and wastage of maintenance resources. Traditionally 3-D laser scanning data processing software utilize common feature points between several scans of a bridge structure to perform the automatic registration process (*9*). Based on this principle, several previous studies developed automated algorithms based on robust feature point registration for aligning two sets of 3-D laser scanning data (*10*, *11*). Such algorithms identify common feature points between two data sets and align them using an iterative closest point (ICP) registration method that minimizes the difference between the two point cloud data sets (*12*). However, these algorithms were developed for aligning 3-D data sets collected within a short time (e.g., within the same day) and need the collected data sets share a significant number of unchanged features (e.g., within the same day; most parts of a job site remain unchanged). On the other hand, the authors found that the long-term change analysis of bridges requires registration of data sets collected from data collection sessions that are months or even years apart from each other, which can contain large amounts of gradual changes of bridges and environments. Therefore, utilizing conventional feature-based algorithms for registering 3-D laser scanning data sets collected from different times can lead to significant registration errors and eventually leads to detecting geometric changes reflected by such registration error. In the next section, the authors provide the details about the steps taken to implement the registration using manual feature point selection and limitations of using traditional registration approach.

LIMITATIONS OF TRADITIONAL REGISTRATION APPROACH

This section presents a motivating case to highlight the necessity and contribution of the study described in this paper. Figure 1 shows the 3-D laser scanning data of a two-lane pre-stressed concrete bridge located in Mesa, Arizona, collected in 2015 and 2016. As per the 2-D drawings, the bridge is 396.25 m long and 13.5 m wide and consists of 18 spans. Each span is 19.8 m long that is supported by four 32-m long columns. The authors first remove the unwanted data in both the 3-D laser scanning data sets. Such unwanted data are mostly from objects in the environments, such as trees, hills, traffic noise (moving cars), water under the bridge, etc. Performing the registration with these unwanted data will significantly affect the registration results, as these objects can change significantly compared with bridge structures. The authors manually remove all unwanted data points in both the two 3-D laser scanning data sets to be compared using the interactive segmentation tool found in CloudCompare (*13*). The 3-D laser scanning data collected in 2015 consists of around 657 million points whereas the data collected in 2016 consists of about 335 million points. However, both data sets have the same number of scans. Such data collection process shows that the point cloud data collected in 2015 have scans having higher data densities (spatial resolutions), which eventually leads to parts of data having

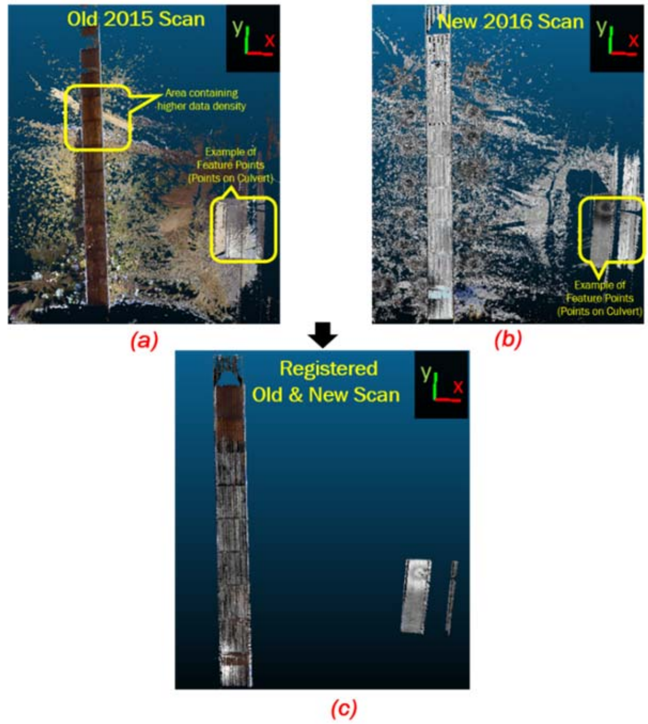

FIGURE 1 Registered 3-D laser scanning data collected in 2015 and 2016 using traditional approach.

denser and more number of points. During the registration, denser parts of the point clouds provide more data points for matching data from 2 years, and the algorithm will tend to bias towards those parts having denser point clouds. Automatic registration methods such as ICP (*14*) or registration methods would generate results biased towards denser data parts and high errors in parts of the scene that have sparser or missing data. Figure 1*a* highlights the denser parts of data collected in 2015. This figure shows that the registration will be biased towards the highlighted areas and produce registration errors in parts that have fewer data points. Primarily, such registration errors will affect the change analysis of the bridge structure and lead to improper decision-making. Therefore, a subsampling method that can generate 3-D laser scanning data sets which have similarly distributed points around the point cloud data is thus necessary for overcome this issue (similarly distributed data density between the point cloud data sets).

Another way to overcome the bias issues caused by varying data densities is to perform registration by manually selecting common feature points between both the 3-D laser scanning data sets. Such features include railing ends, signs on bridges, etc. Varying data densities of the point cloud data generally do not affect the traditional registration approach that relies on common feature points because those algorithms only use selected feature points not all the points in the point cloud. Figures 1*a* and 1*b* highlight few common feature points that can be utilized for performing the registration between the 2015 and 2016 3-D laser scanning data sets using manual feature point selection (Figure 1*c*). This manual approach can be utilized for change analysis of the bridge structure but has few limitations. First, the amount of time invested in manually selecting common features is high. Another major limitation of this approach is the assumption that the manually selected feature points would not change significantly when compared with changes of the bridge structure. Selecting feature points that have large spatial changes than the bridge structure's changes will mislead the change analysis as well. A novel registration approach that performs reliable registration between two 3-D laser scanning data sets containing spatial changes is in need.

Several researchers combined the use of total station data, and the data collected the 3-D laser scanners to establish a control network of points that would not change. This process involves scanning the bridge structure along with the use of a total station to establish a control network that will not change significantly between the data collection sessions. This process of scanning the bridge structure along with the established control points helps in aligning 3-D laser scanning data collected at different times. However, the process of establishing the control network is tedious and becomes impractical when a bridge submerged in water (*15*). Additionally, checking and ensuring that at least three control points are visible from any pair of registered laser scans is also tedious and could hardly be practical for complex outdoor jobsites. For instance, scanning a control point that has been setup far away from the bridge structure requires high-resolution scans that generate a large amount of raw data for preprocessing.

This paper presents a novel robust registration approach that automatically registers two sets of 3-D laser scanning data collected at different times that are 1 year apart from each other. First, the approach extract bridge features from two 3-D laser scanning point clouds and roughly register two bridge data sets by matching salient bridge features. Next, the algorithm extracts feature points from both the surroundings and on the bridge structure and then use a new robust 3-D data registration algorithm that automatically identifies changed features between two data sets through a robust fitting method. Finally, the algorithm utilizes the robustly registered feature points to perform accurate registration of the point clouds and label changed parts between two point clouds. The authors tested this robust registration approach using 3-D laser scanning data of a highway bridge collected in 2015 and 2016, respectively. The following section briefly reviews previous literature on conventional 3-D data registration methods. The authors then describe the developed methodology in detail and then present registration results of the new method on the data collected on a highway bridge. The authors then validate the new approach by comparing it with conventional 3-D data registration method that uses manually selected feature points for aligning 3-D data sets from different data collection sessions. Finally, the paper concludes by summarizing the results and discussing the limitations.

LITERATURE REVIEW

Bridges undergo several spatial changes during its service period. These changes have to be periodically detected and analyzed to identify the abnormal changes affecting the bridges loading behaviors. Recent developments in the field of computer vision (2-D and 3-D imagery data) applications in civil engineering enable spatiotemporal information retrieval from imagery data for engineering decision support on construction sites (*16*). Spatiotemporal changes observed in point cloud data sets collected at different times provides detailed visual information for monitoring changes and analyzing structural deformations (*15, 17*). Lindenbergh and Pfeifer utilized terrestrial laser data of a lock (sea entrance of a harbor) for statistical deformation analysis (*18*). The statistical analysis consists of calculating the deformation of the lock detected between two point clouds scanned at the exact same position. Such analysis concluded that terrestrial laser scanners could achieve deformation detection in the order of 9 mm. However, the major limitation of the statistical analysis study for deformation monitoring is that the researchers conducted the experiment by fixing the scanner's position. This is a limitation in cases having to detect deformation of civil structures at larger time gaps and unable to access previous scan position for the next data collection. Numerous studies conducted change detection studies using two sets of point cloud data scanned within 24 h. Girardeau-Montaut et al. detected changes between two sets of point cloud data collected every day (*17*). The change detection study utilized the point cloud data to monitor applications on a building site by registering two 3-D laser scanning data sets having shared points nearly not moved. Such registration process consists of using a minimum threshold value for the shared points and then utilizing the ICP approach to perfectly align them. The major disadvantage of using such approach is to detect changes in structures that undergo significant spatial changes over the time period such as a bridge structure.

Researchers also conducted studies to monitor complex deformation of objects having complicated shapes (*19*–*21*). Antova (*19*) discussed several registration processes that can perform deformation monitoring using laser scan data in the field containing objects having complicated shapes. These registration processes automatically generate targets using planes in overlapped scanned for performing the registration. However, the accuracy of the registration results is dependent on the percentage of overlapping between the scans. Other studies involved combining terrestrial laser scanning technology with static global navigation satellite system positioning and tacheometry point-wise surveying techniques. Vezocnik et al. conducted long-term high-precision deformation monitoring of underground pipelines subjected to high-pressure conditions and concluded that the combined use of laser scanning and point surveying techniques is a valid solution for monitoring deformation in a 3-D space (*20*). The limitation of using such techniques is the amount of time invested in the data acquisition and processing and in assuming that the selected surveying point do not change over a few months. Therefore, the authors developed a novel robust registration approach to reduce the amount of time needed in data acquisition and to accurately register 3-D laser scanning data collected at different times. The following section presents the developed approach in detail.

METHODOLOGY

The developed robust registration algorithm automatically registers two sets of 3-D laser scanning data collected in different years (Figure 2). It utilizes points that are common and are less likely to change between two 3-D laser scanning data sets of the bridges and registers them into one global coordinate system. The major advantage of this robust registration algorithm is that it automatically identifies such common points that do not have significant changes between 2 years' data. These automatically identified points aid in performing reliable registration of the two 3-D laser scanning point clouds in order to accurately detect the geometric changes of bridges from year to year. The first step in the robust registration approach is to perform rough registration of the two 3-D laser scanning data sets. This rough registration can be either performed manually or using commercially available registration software tools (e.g., Leica Cyclone). Next, the authors manually remove redundant data found in 3-D laser scanning data. Inaccurate segmentation of such redundant data may cause unreliable registration. The following section details the data preprocessing and 3-D point cloud subsampling process.

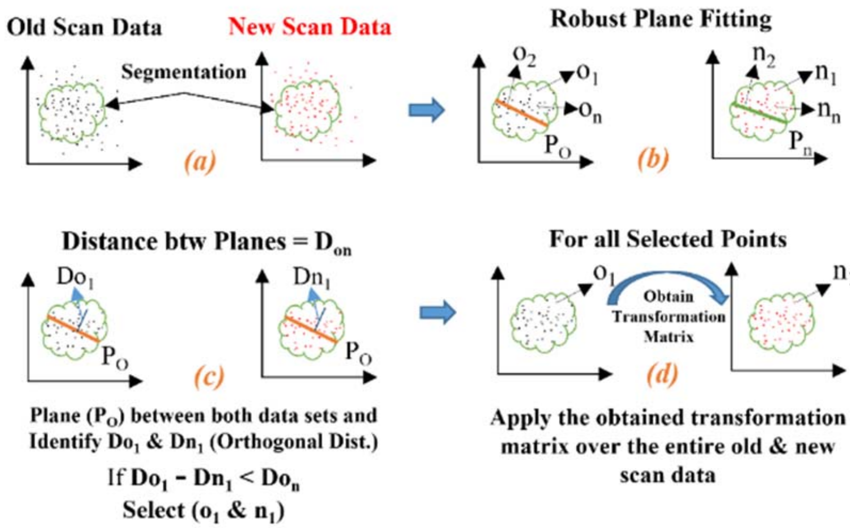

FIGURE 2 Robust registration approach to register old and new scan data.

Data Preprocessing and Subsampling

The process of segmentation removes all unwanted data, but it is very important that both the data sets have similar data densities to avoid biases of the registration towards denser parts of data. Hence, the authors use a two-step process to subsample both the 3-D laser scanning data sets to maintain similar data densities across the point clouds. The two-step process first subsamples both the 3-D laser scanning data sets to maintain uniform spacing between points. This process will subsample the 3-D laser scanning data sets by maintaining a similar number of neighbors around a point in denser areas and not altering points in parts having sparser data points. The next step is to interpolate the sparser parts of the point cloud data and increase its density to the same level as other parts keeping similar densities across point clouds. The authors conducted these two steps using the subsample tool available in CloudCompare (*13*). Figure 3 shows an example of a subsampled 3-D laser scanning data sets collected in 2015 and 2016 having uniformly distributed points. After the segmentation and subsampling process, the robust registration approach detailed in the following section will align 3-D data sets from different years for change detection.

Robust Registration Algorithm

3-D laser scanning data collected at different times enable spatial change detection of the bridge structure. Examples of these spatial changes include overall deviation of the bridge structure (rigid body motion), deviations of individual bridge elements, and deformation of the individual bridge elements. However, the first step is to identify the rigid body motion of the bridge structure, which can help in identifying the other spatial changes. Such rigid body motion of the bridge can be identified by accurately registering 3-D laser scanning data collected at different times. The collected 3-D laser scanning data sets contain several common features and other additionally captured features of objects around the bridge structure. There may be cases that one

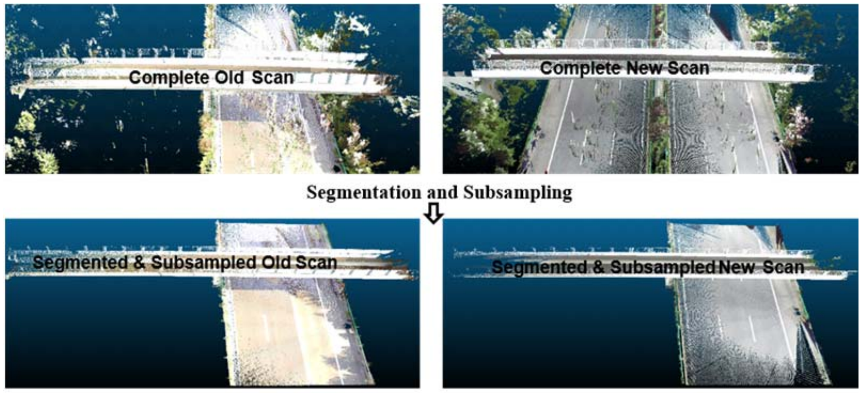

FIGURE 3 Segmentation and subsampling process of 3d laser scanning data for robust registration.

point cloud data may contain features that might be missing in other point cloud data set. If a registration process is implemented during such case, the registration result will be biased toward the additional features, which is missing in one of the captured point cloud data. Hence, the reliable registration approach must segment both the point cloud data sets so that both contain exact same environment and bridge features that improve the quality of the registration results. The following paragraph details the process of segmenting both the point cloud data sets to contain exact same environment and bridge features that utilize a robust plane fitting approach to identify unchanged data points between the collected data sets. Failure to accurately segment the point cloud data sets will affect the plane fitting step that eventually affects the overall robust registration approach.

The segmented and subsampled 3-D laser scanning data sets contain several common points between them. Manually identifying unchanged points between two data sets is tedious. Hence, the authors developed an automatic method that utilizes all the points in the point clouds to automatically and accurately identify unchanged parts between the two compared 3-D laser scanning data sets (e.g., data collected in 2015 and 2016). First, the algorithm utilizes a robust plane fitting approach to fit a plane between all the points found in both the old (points o_1, o_2, $o_3 \ldots o_n$) and new (points n_1, n_2, $n_3 \ldots n_n$) 3-D laser scanning data. The robust plane fitting approach utilizes the principle component analysis, which minimizes the perpendicular distances between the points and the fitted plane (*22*). Using such plane fitting approach, the authors robustly fit one plane between the points from the old (P_O) data collected in 2015 and an another plane between the points from the new (P_n) data collected in 2016. The output of such plane fitting process is the center of the plane and the orthogonal distances between the fitted plane and all the points. However, if either of the point clouds contains data points that capture objects in one of the point cloud data and is not captured in the other point cloud, the robust plane fitting approach may generate a plane biased towards such additionally captured data parts that are missing in one of the compared point clouds. That plane would not well represent the overall trends of data points in the data set that have parts of data missing, making the comparison of two point clouds not on the same basis. In order to avoid such issues, the authors only keep data points that are visible in both of the compared point clouds. That process segments both point clouds such that they share the exact same boundary, which contains the captured bridge and environmental features. Such segmentation is important so that a robustly fitted plane from one point cloud can be a good basis to assess the changes of the other data set. These two data sets capturing similar parts of the scene should have similar trends represented by a robustly fitted plane for analyzing differences between 2015 and 2016 point clouds which contain several spatial changes. The authors utilize the cross-section segmentation tool found in CouldCompare (*13*), which utilizes a bounding box to edit and segment 3-D laser scanning data sets. The cross-section segmentation process consists of maintaining the exact same size of the bounding box, which eventually helps in maintaining similar features between the two 3-D laser scanning data sets. This step will aid in improving the overall quality of the robust registration algorithm. Figure 3 shows an example of a segmented 3-D laser scanning data of a bridge structure collected in 2015 and 2016 respectively. The authors performed the segmentation process such that both the 3-D laser scanning data sets contain the similar parts of the scene.

Since both the 3-D laser scanning data sets are roughly registered and in the same global coordinate system, the algorithm then calculates the orthogonal distances between the data points in the old point cloud collected in 2015 and the old plane that is derived from old point cloud (Do_1, Do_2, $Do_3 \ldots Do_n$, hereafter). Similarly, the algorithm calculates the distances between the data points in the new point cloud collected in 2016 and the old plane that is derived from old point cloud (Dn_1, Dn_2, $Dn_3 \ldots Dn_n$, hereafter). Such process of calculating the orthogonal distance between the old and new points with the same old plane derived from old point cloud will help to identify unchanged points among the old and new point clouds. The authors now calculate the distance between the two fitted planes P_O and P_n, say D_{on}. The next step in the robust registration algorithm is to associate every point in the old point cloud (2015 point cloud) to each point in the new point cloud (2016 point cloud) using the nearest-neighbor approach. The nearest-neighbor approach associates each individual old points to each new points based on the smallest distance between them. The rough registration approach brings both the data sets into a single global coordinate and the nearest neighbor approach associates each point in the old point cloud (2015 point cloud) to its corresponding closest point in the new point cloud (2016 point cloud). Assuming that o_1 is the nearest neighbor to n_1, o_2 is the nearest neighbor to n_2 and so on for all other points.

Now, the algorithm calculates the difference between orthogonal distances of the all the associated nearest neighbors such as $D_{O1} - D_{n1}$, $D_{O2} - D_{n2}$, etc. If one of the calculated orthogonal difference is smaller than D_{on}, then the algorithm identifies those corresponding points as unchanged. For instance, if $D_{O1} - D_{n1} < D_{on}$, the algorithm identifies that the corresponding point D_{O1} and D_{n1} remain unchanged between old and new point cloud data. Hence, the algorithm identifies all corresponding old and new points that have the difference in the orthogonal distances smaller than D_{on}. This process now eliminates all the changed points and extracts only those unchanged points that are utilized for automatic registration between both the collected 3-D laser scanning data sets. The algorithm now utilizes an ICP registration (*14*) to register unchanged old and new points and determine its corresponding transformation matrix. This transformation matrix provides the translation and rotation values required to accurately align the new points to their corresponding old points and eventually to register the entire old and new 3-D laser scanning data from which those points were extracted. Therefore, this process determines the transformation matrix between the unchanged old and new points and algorithm uses this transformation matrix to register both the collected 3-D laser scanning data sets required for reliable geometric change detection of bridges.

VALIDATION

To validate the developed robust registration approach, the authors compared its registration results with the traditional registration approach, which relies on matching features points between two sets of 3-D laser scanning data. The comparison process relies on comparing the transformation matrix generated by the robust registration approach with that of the transformation matrix generated by the traditional registration approach. A transformation matrix consists of translation parameters that consist of displacement along x, y, and z coordinates and rotation parameters that consists of rotation along α (rotation around the x-axis), β (rotation around the y-axis), and γ (rotation around the z-axis) that helps to register the 2015 3-D laser scanning data with the 2016 3-D laser scanning data (*23*). The final output of the robust registration approach is the transformation matrix, which is compared with the registration results of the traditional registration approach. The following section provides details about generating the transformation matrix using the traditional registration approach.

The authors executed a registration approach that iteratively selects unchanged feature points between the two data sets. The improved manual feature point selection approach utilizes manually selected feature points on the bridge and its surrounding common in the 3-D laser scanning data collected in 2015 (old data) and 2016 (new data), respectively. Specifically, the authors selected several feature points on a nearby culvert and few feature points on the part of the bridge structure. The process of manually selecting feature points involves selecting few common feature points between the old and the new 3-D laser scanning data. For instance, the authors have selected 11 common feature points (bridge and environment) between the two data sets. Then the authors select three points each from the previously selected set of 11 common feature points such that the triangle formed by connecting the three feature points in the old data is similar to the triangle formed by the feature points in the new data. Here, the similarity between the two triangles can be obtained by maintaining the equal length of the sides of the triangle. Now the authors perform the registration between the old and the new 3-D laser scanning data using these three selected feature points to obtain the transformation matrix. After this registration step, the authors calculate the change in the distance between the remaining eight feature points from the old 3-D laser scanning data with their corresponding eight feature points from the new 3-D laser scanning data. Such calculation will provide information about those features points that have undergone significant changes after the first registration step.

Next, the authors identify the least changing common feature point between the old and the new 3-D laser scanning data. After identifying the least changing feature point, the authors again perform the registration between the original old and new 3-D laser scanning data using the previously identified three common feature point and the least-changing common feature point. This registration step generates another transformation matrix. The authors calculate the difference in the new transformation matrix (four feature point registration) and the old transformation matrix (three feature point registration) and identify if any of the translation (translation along x, y, or z coordinate directions) value difference is above a certain threshold. The authors set 30 cm as value for the threshold. Here, the authors ignored the rotation values from the transformation matrix, as these rotation values are significantly smaller. If the difference between both the transformation matrices is above the threshold, then the authors continue the registration process by calculating the change in the distance of the remaining seven feature points from the old scan with their corresponding seven feature points from the new scan to identify the least changed feature point. In the next step, the authors again perform another registration between the original 3-D laser scanning data sets using the four previously selected feature points and the new identified least changed feature point to obtain another transformation matrix. If the difference between the new transformation matrix and the previous transformation matrix is below the threshold value, then the authors end this registration process and treat the new transformation matrix as final. If the difference between the new transformation matrix and the previous transformation matrix is above the threshold value, then the authors continue the registration process by again identifying another least changed feature point among the remaining common feature points. The above described registration using manual feature point selection approach iteratively identifies least changing feature points by gradually registering both the old and the new 3-D laser scanning data. This iterative registration approach can be utilized in cases of a bridge data having no similar environmental feature points to perform the robust registration approach. The authors validated the developed robust registration approach using a case study of a highway bridge structure detailed in the following section.

CASE STUDY

First, the authors segmented, subsampled, and roughly aligned both the 2015 and 2016 3-D laser scanning data sets (Figure 4). Now the authors applied the robust registration algorithm to accurately register both the 2015 and 2016 3-D laser scanning data (Figure 4*c*). Figure 4 shows the obtained transformation matrix (Table 1), which contain the translation and rotation parameters to robustly register both the 3-D laser scanning data sets. These robustly registered 3-D laser scanning data sets to aid in reliable geometric change detection of bridges for performing accurate condition diagnosis. Therefore, the changes detected from such robustly registered 3-D laser scanning data sets reflect the actual geometric changes of a bridge structure rather reflecting changes due to registration errors between the two data sets. Now the authors implement the improved feature point registration approach to manually register both the 2015 and 2016 3-D laser scanning data. To implement the improved traditional registration approach, the authors initially selected 11 feature points and then identified that there is no significant change in the obtained transformation matrix when using six least changed commonly identified feature points. Table 1 shows the final transformation matrix using the six identified feature points and its comparison with the transformation matrix generate using robust registration approach.

The comparison results show that the developed robust registration approach is qualitatively same but slight vary quantitatively from the registration results using manual feature point selection. This means that both the registration approaches output results that have the same direction of translation and the direction of rotation along all the coordinate axes. Additionally, the quantitative difference between all the registration results is very small and does not significantly affect the results of the geometric changes detected between the collected 3-D laser scanning data sets. This comparison study validates the robust nature of the developed robust registration approach and its substantial advantage for performing automatic and reliable geometric change detection of the bridges using 3-D laser scanning data over other traditional approaches.

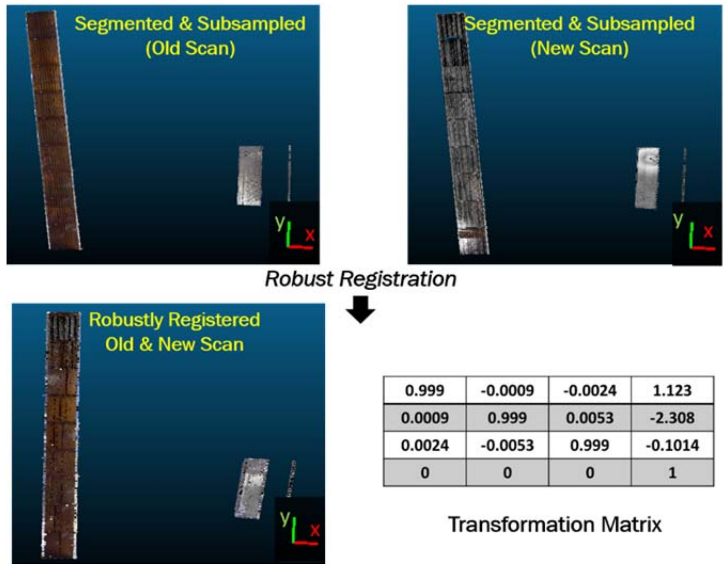

FIGURE 4 Segmented, subsampled, and robustly registered 3-D laser scanning data of the highway bridge (*z*-axis along elevation).

TABLE 1 Comparison of the Registration Results (Robust Registration Versus Manual Registration)

Registration Type	Translation Values			Rotation Values		
	x	*y*	*z*	α	β	γ
Robust registration approach	1.123	–2.308	–0.1014	0.0053	0.0024	–0.0009
Registration using improved manual feature point selection	1.208	–2.743	–0.0812	0.0078	0.0026	–0.00018

CONCLUSION AND FUTURE RESEARCH

This paper presented a novel robust registration approach that automatically detects unchanged common points between two sets of 3-D laser scanning data and accurately registers them into one global coordinate. The developed approach first segmented redundant data and subsampled both the 3-D laser scanning data sets. Then a robust registration algorithm automatically extracted unchanged points on both the bridge and its surrounding environment to perform a point-to-point registration. Such process does not require any manual intervention or the tedious process of manually selecting unchanged points. The authors applied the developed registration approach on highway prestressed concrete bridge and validated the registration results by comparing it with the traditional manual feature point selection registration approach. The developed robust registration algorithm utilizes several environment feature points that surround the bridge structure. However, in some cases, these environment feature points undergo higher spatial changes than the bridge structure.

In the future, the authors plan to study the effect of spatial changed environmental feature points on the registration results. The authors plan to use the surveying data collected using a total station sensor to establish several control point network using the environmental features around the bridge structure. These ground control points can aid in understanding the spatial changes of these environmental features that can be incorporated in registering two sets of 3-D laser scanning data collected at different times. Hence, using both the data generated by the 3-D laser scanners and the total station sensor can help in developing more robust registration approach that is not affected by the spatial changes of the environment surrounding a bridge structure.

In addition to developing reliable registration techniques, the author plans to develop a 3-D imagery data-driven bridge deterioration monitoring and decision-making framework that evaluates the health of a bridge structure becoming an integrated part of the bridge management system for conducting reliable risk asset management. Several researchers developed a bridge data management system that manages the sensor and bridge metadata for damage detection and long-term monitoring of bridge structures (*24*). Therefore, the author plans to develop a 3-D imagery data management system that collects and manages timely imagery data of several bridge structures for aiding detailed geometric analysis, condition assessment tracking, and spatiotemporal change monitoring.

ACKNOWLEDGMENTS

This material is based upon work supported by National Science Foundation (NSF). NSF's support is gratefully acknowledged. The authors also thank Jimmy Camp and Ryan Jones from Maricopa Department of Transportation for giving us permission to collect laser scanning data and providing the design drawings of the bridge.

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Visual Inspections and Key Performance Indicators
Bridging the Gap

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The service quality of a bridge—measured by key performance indicators (KPIs)—is considered insufficient if they do not meet performance goals. Widely recognized KPIs are safety and serviceability. A bridge is regarded as structurally safe if the probability of failure during its service life is not expected to exceed some nominal value. Similar approach applies to serviceability.

Whereas in the design phase the safety and serviceability concerns are addressed directly in quantitative manner, in the service phase, the condition state is determined based on inspection results, which is a qualitative performance indicator. The condition state is actually a vague measure for the deviation of inspected bridge from the "as new" condition. The direct assessment of safety and serviceability is regarded as not practicable. In this paper, the approach to determine KPIs based on inspection results is proposed.

The KPIs are subject to observations obtained by visual examination and simple non-destructive testing. Typically, they are related to bridge components and indicate existing or expected bridge dysfunctionality that can result in insufficient KPIs. The challenge is to establish procedures that connect observations with KPIs.

The papers suggest Bayesian networks to this end. The à priori values of KPIs, i.e., the ones of the intact bridge are assessed as a baseline for the assessment during the service life. The à posteriori assessment of KPIs based on inspection results is performed using Bayesian networks that model observations and its uncertainties. The proposed approach is illustrated in a simple example.

There is a broad consensus that the benefits of road infrastructure for the society cannot be overestimated. The investments in road infrastructure raise the growth potential of a national economy, which is realized by efficient utilization of the road infrastructure. The road infrastructure enables road users to be involved in various productive activities that yield private, public, and social goods. Maintaining these benefits on the long run in economically efficient, environmentally responsible, and socially reconcilable manner is the fundamental task of road authorities. Bridges are critical components of the road infrastructure as they ensure fast safe passages over otherwise hardly surmountable obstacles. From the users' perspective, it is irrelevant whether a road is carried by the bridge or being in tunnel or merely resting on the soil, so long it provides the safe and fast travel from origin to destination. In this context, it is necessary to define what is meant by fast and safe.

KEY PERFORMANCE INDICATORS AND PERFORMANCE GOALS

There are standards that apply to the design of a road infrastructure—which affect road users—and they are related to clearance, speed, and weight allowance. The design travel speed defines the minimum travel time on an arbitrary road link. In reality, this minimum travel time can be achieved only in the case of unrestricted traffic flow, i.e., if road capacity is sufficiently higher than traffic volume. Based on the real or predicted traffic one can choose some value of travel time—larger than minimum travel time—to define the "fast travel." This value can be regarded as performance goal.

The safe travel however is somewhat difficult to define since it doesn't imply the accident-free traveling. There is always the basic accident rate, which is a function of traffic volume but not related to the condition or to the design of infrastructure. This accident rate defines "safe travel" and can be regarded as performance goal. Correspondently, the measured or predicted travel time and the accident rate are performance indicators that are compared with performance goals. Both travel time and accident rate belong to user-related performance indicators.

Additional standards are used in road design aim to protect general public and abutter from negative effect of road traffic. These include but are not limited to noise and pollution. These can be measured as well and comprise the society-related performance indicators. For these indicators performance goals can be also defined, but this is out of the scope of this paper.

In context of bridges the above-mentioned performance indicators are not used directly. Instead, as in bridge design, the primary concern is safety and serviceability. Safety includes traffic or user safety as well as structural safety. A bridge is regarded as structurally safe if the probability of failure during its service life doesn't exceed some nominal value. Similar approach applies to serviceability in which the exceedance probability of some service limits has to be sufficiently low. In addition to it the bridge riding surface also fulfills the performance goals that apply for the pavement, one can regard that the bridge meets performance goals for road users i.e. sufficient quality of service. It appears that serviceability and safety can be chosen as adequate key performance indicators (KPI) for bridges.

In Brown et al. (*1*) performance issues are suggested that may be interpreted as performance indicators as represented in Figure 1. Herein, the serviceability is combined with durability in performance issue "structural condition" whereas safety is combined with stability to form the performance issue "structural integrity." The performance issue "costs" includes both agency and user costs. It should be noted that the user costs include delay, detour, and accident costs. Finally, the performance issue "functionality" includes clearance, ride quality, and load ratings and restriction on use.

The most indicators relevant to the bridge performance are included in Brown et al. (*1*), but the classification merits some further consideration. For instance, the structural integrity is related only to sudden events, mostly natural hazards such as earthquake, hurricane, and fire. The observable deterioration processes, although they may compromise structural integrity affect only durability and serviceability. The "durability" seem to be understood as a span of time in which neither safety nor serviceability is likely to be compromised. The costs include also user costs that may be also included in user safety and serviceability.

The consideration of agency costs is clearly reasonable if the broader definition of performance applies. The bridge performance is considered better if the same safety and serviceability level can be achieved with lower costs. According to the narrower definition of performance only user and societal perspectives are considered. The agency costs are to be minimized for the given performance goals. Indeed, the agency goal to minimize its costs may be regarded as their optimizing performance goal.

FIGURE 1 Bridge performance according to Brown et al. (*1*).

MAINTENANCE PLANNING

It can be assumed that at the time of commissioning the bridge meets the performance goals. Based on the design data that can be both performance indicators, e.g., material properties as well as other relevant data, the KPIs for a bridge can be evaluated at the commissioning. This evaluation on virgin bridge, even if performed at some later point in time, is reference information needed to evaluate KPIs at some later time instance. In course of time, the road infrastructure is subject to damage processes in addition to increasing traffic volume. Both damaging processes and increasing traffic volume can result in performance indicators that fail to meet performance goals. These performance indicators can be assessed based on observations obtained by visual examination, nondestructive testing, or permanent monitoring systems. Typically, they are related to bridge components, e.g., girders, abutments, cross beams, and indicate existing or expected bridge dysfunctionality that may result in one or more insufficient performance indicators. At some point in time the safety or serviceability goals are not met anymore as presented in Figure 2.

It is assumed that an inspection is performed today as presented in Figure 2. The results from the inspection revealed some damages that in conjunction with the actual loads lead to worsening of the KPIs. They however, still meet the performance goals. Based on inspection results the serviceability and safety are forecast, yielding that serviceability criterion will be not fulfilled at the time instance marked “Tul.” This means that the intervention needs to be executed at the latest at this point in time in order to comply with performance goals. It should be noted however that even bridges with no deterioration or dysfunctionalities may fail to meet performance goals. The reason for this can be found in an increase of traffic volume, the obsolete code of practice used for bridge design, new insights with regard to detailing, natural hazard, climate change, etc.

The owner’s–operator’s performance goal is to minimize long-term costs. From the owner perspective, the road users’ performance goals can be regarded as optimization constraints. The general approach is presented in Figure 3. The intervention is planned prior to the instance in time at which road users’ performance goals are not fulfilled, i.e., Tul. At the time instance “Top” the long-term costs are at their minimum and therefore the maintenance intervention should be planned at the time instance Top. It is a matter of operator’s methodology whether user cost should be added to agency costs in order to minimize them. In this case, the performance goal with regard to travel time and accidents costs should be revisited.

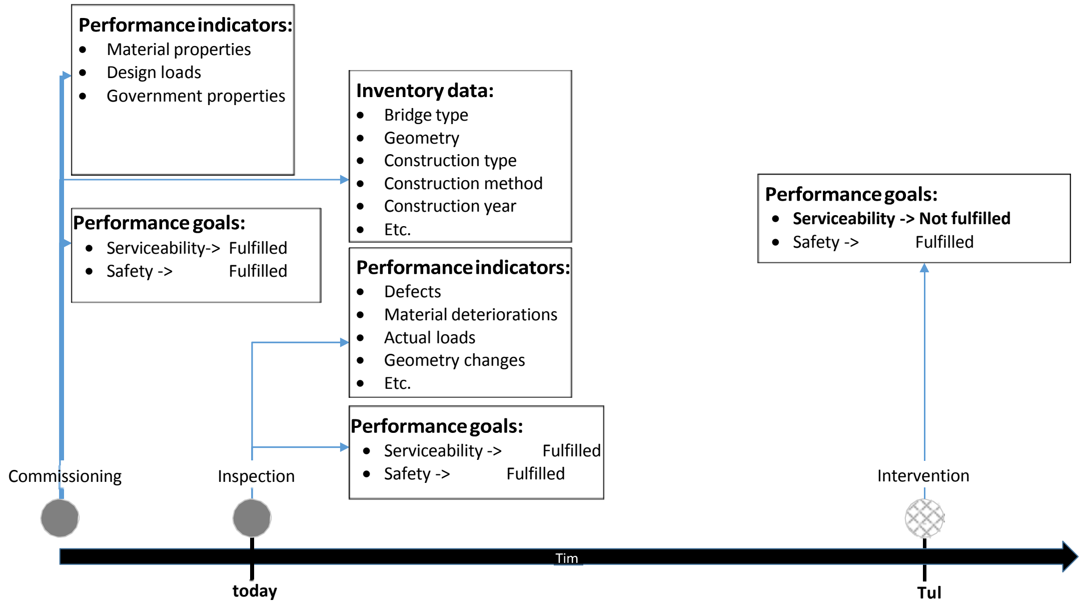

FIGURE 2 Road users' related performance over time—principle.

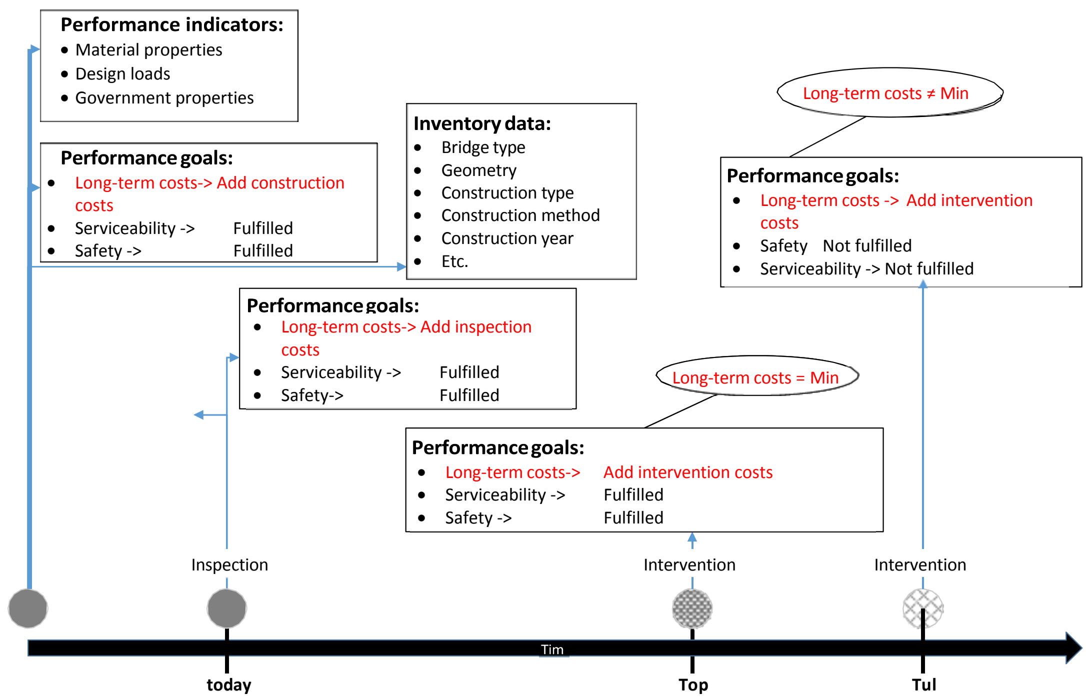

FIGURE 3 Road users' and owner's and operator's related performance over time.

In principle, the same approach can be applied to societal and environmental performance goals. Depending on adopted modeling of related performance indicators these can be treated as optimizing or satisfying goals. In practice, there is a serious obstacle to the approach presented above. Whereas in the design phase the primary concern is to meet safety and serviceability goals in quantitative manner, in the service phase condition state is determined, which is a qualitative performance indicator. The condition state is actually a vague measure for the deviation of a deteriorated bridge from the "as new" condition. The quantitative assessment of KPIs based on inspection results is regarded as not practicable. In this paper, the approach to determine performance indicators based on inspection results is shown.

GLIMPSE INTO THE FUTURE

It is foreseeable that in not-so-distant future building information models (BIM) of both newly built and existing bridges will be available [see Chipman et al. (*2*) and Federal Ministry of Transport and Digital Infrastructure (*3*)]. These models will be included into the bridge management system (BMS) and will significantly enhance the quantity of useful information in BMSs. A BIM can embed the structural system of the bridge as well as the relevant load cases. The evaluation of the KPIs would be therefore possible quasi on-the-fly within the BMS.

The inspection results can be directly captured in the BIM using photogrammetry or some other procedure. Cracks, spalling, deformation, and other defects will be a part of BIM, which in most cases alter the BIM geometry.

Figure 4 shows a column (far left) as a part of a BIM. On this column, a spalling area is observed (second from left). This spalling area is captured as 3-D model and merged with the BIM of an intact column. The result is shown in Figure 4. The embedded structural system is also updated: The resistance of the column can be reduced due to spalling and the internal forces, i.e., bending moment (M in Figure 4) and axial force (N in Figure 4) can be compared with the reduced resistance (e.g., interaction diagram). In this manner, the safety factor can be updated.

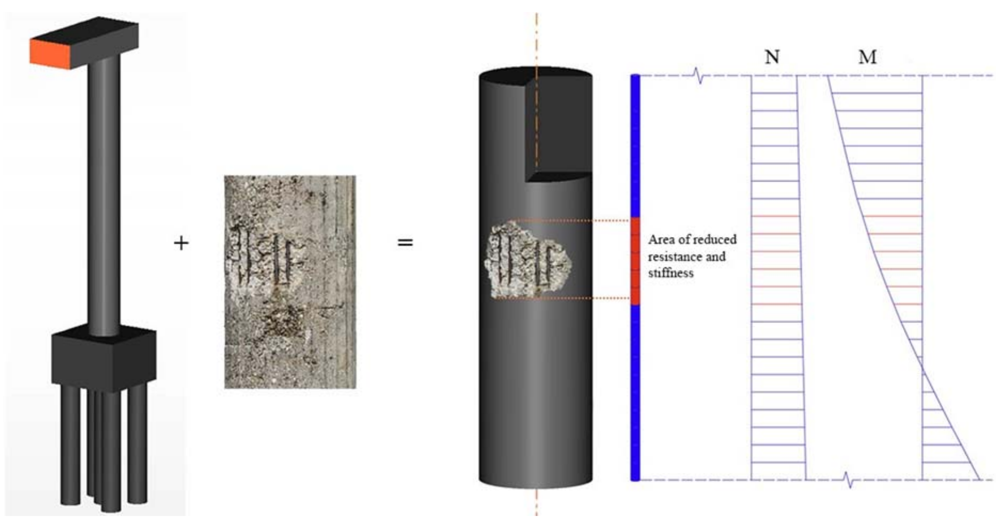

FIGURE 4 Integration of damage area into BIM.

Clearly, also the uncertainty of the material properties and spalling effect on resistance can be taken into account, so that the probability of failure can be determined.

The damaged BIM can be used as the basis for the deterioration simulation, which can provide significantly more accurate forecast that the current methods. The reasons for this are include:

- The resistance and loads, i.e., probability of failure of an intact structure is duly taken into account and
- The exact location of a defect is known so that its effect on the safety and serviceability can be assessed in more accurate manner. A defect in a so called "hot area"—i.e., highly stressed area—would have a different impact on safety and serviceability.

Even if one has to wait for quite some time for the universal availability of BIMs, the above arguments can be seen as guidance for the more-advanced maintenance planning. In particular, data from original design can be used more extensively in bridge management. In this manner, the original weaknesses of the bygone design codes and design practices can be taken into account. Furthermore, apart from dividing bridges into their constitutive elements one can also identify hot areas—i.e., areas in which damages can be particularly dangerous.

ENHANCEMENT TO CURRENT PRACTICE

The above reasoning can enhance both the qualitative and quantitative estimation of KPIs. The corresponding suggestion will be described in following chapters.

À Priori Assessment of KPIs

The safety and serviceability of an intact bridge under current traffic loading or some other action is the result of à priori assessment. This assessment can be—disregarding whether quantitative or qualitative—performed by means of Bayesian networks. The Bayesian networks for qualitative assessment of KPIs can differ from the one for quantitative assessment.

The Bayesian network for qualitative à priori assessment is presented in Figure 5. The effect of the construction year, type of structure (e.g., simply supported beams, continuous beam, frame, arch), and code of practice used in the original design influence the resistance and serviceability limits based upon the experience and known deficiency related to the combination of these parameters. The actual traffic loads that are perhaps more aggressive, are also captured and conveyed into a load effect. The basis for the computation of load effects is a structural system, i.e., the type of structure. The result is a safety and serviceability rating that is in a general stochastic variable. Clearly, it can be transformed in a single value by taking the expected value of its distribution.

For practical purposes this diagram has to be enriched with additional information on elements, construction flaws, etc. The safety and serviceability rating is qualitative and can be chosen based on the existing scale of condition states.

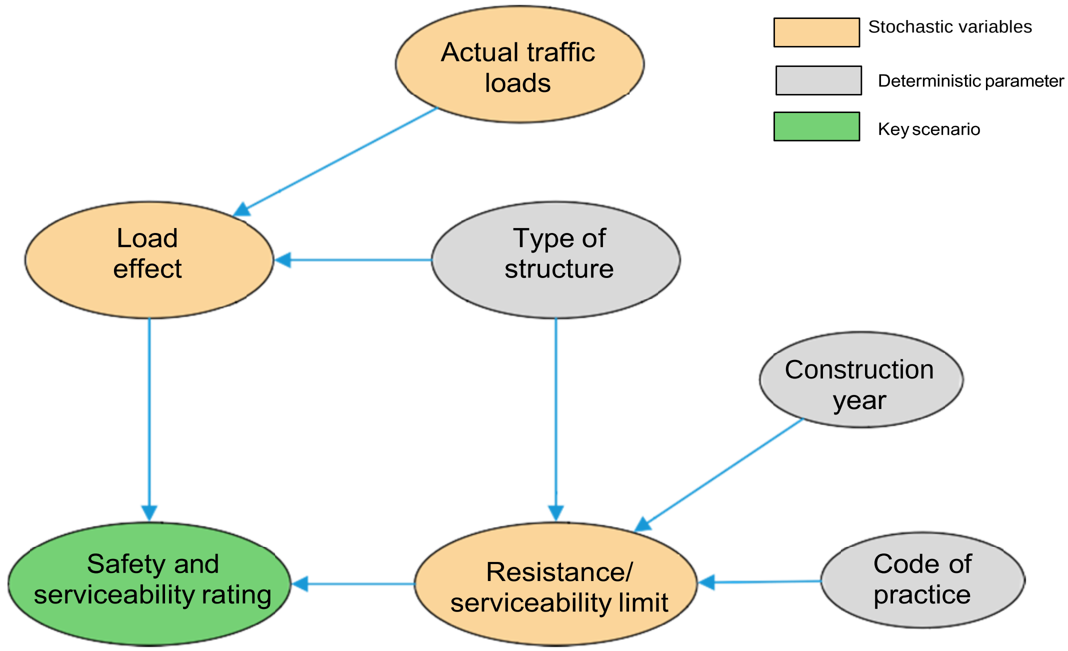

FIGURE 5 Simplified Bayesian network for qualitative KPI assessment.

In the quantitative approach one has also to include a structural system. The structural system can be simplified following the basic rules for preliminary design of bridges. For instance, for vertical loads the majority of "normal" bridges can be sufficiently well modeled with simply supported beams if the spans and the effective width are chosen properly. In Hajdin and Despot (*4*) the possible simplification, albeit in a different context is explained in detail. Bayesian network for quantitative à priori assessment is presented in Figure 6. The simplified structural system is assumed to comprise a series of simply supported beams. The resistance and the serviceability limits can be approximated by the load effect of the original design code on simplified structural system. The construction year can reveal some inherent weaknesses of original design or the deficiency in the code of practice. The ratio of permanent to traffic load is necessary to correctly estimate the probability of failure or probability of exceeding the serviceability limit.

In order to illustrate the presented approach, the safety of an existing bridge that is modeled as simply supported beam (node "simplified structural system" in Figure 7) is assessed. It is assumed that the safety is given if the probability of failure doesn't exceed $5.0 \cdot 10\text{-}4$.

The simply supported beam is presented in Figure 7 and the traffic load (node "actual traffic load" in Figure 6) is modeled as a normally distributed point load with a mean value of 100 kN and standard deviation of 15 kN. Permanent loads are neglected in this example. It is assumed that the bending resistance (node "resistance" in Figure 6) is constant along the beam and is normally distributed with a mean value of 500 kNm and standard deviation of 50 kNm. The span (node "span" in Figure 6) is 10 m.

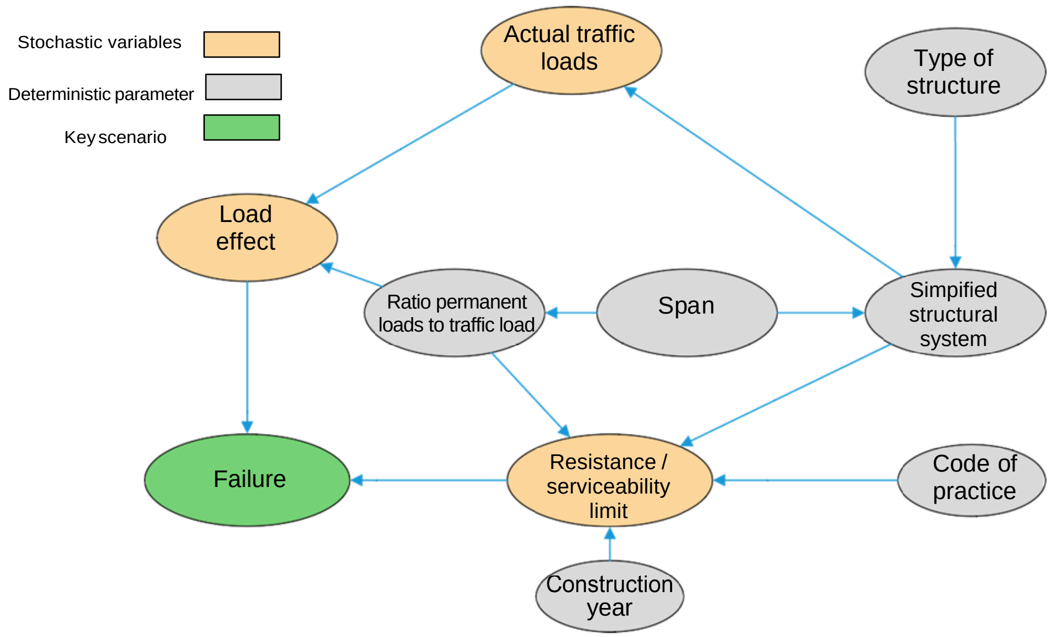

FIGURE 6 Example of a Bayesian network for quantitative KPI assessment.

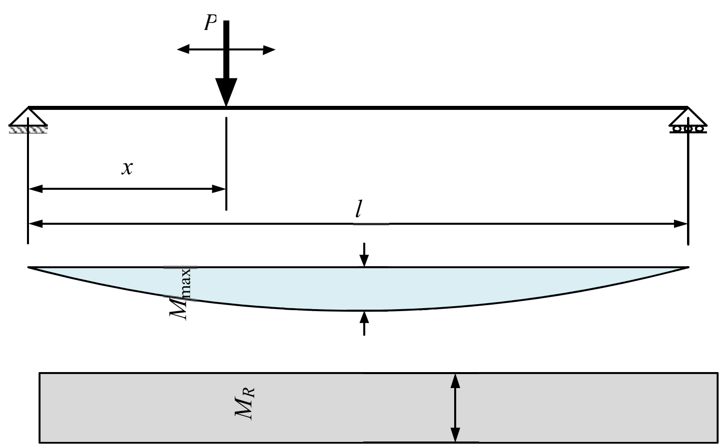

FIGURE 7 Simplified structural system and traffic loading.

Due to the fact that both load effect and resistance are normally distributed, the safety index, which means also the probability of failure, can be computed as follows:

$$\beta = \frac{\mu_{M_R} - \dfrac{\mu_P \cdot l}{4}}{\sqrt{\sigma_{M_R}^2 + \left(\dfrac{\sigma_P \cdot l}{4}\right)^2}} = \frac{500-250}{\sqrt{50^2 + \left(\dfrac{150}{4}\right)^2}} = \frac{250}{50 \cdot \sqrt{\dfrac{25}{16}}} = \frac{1000}{250} = 4.0 \qquad (1)$$

$$P_f = 3.17 \cdot 10^{-5}$$

According to this assessment the virgin bridge meets the safety criteria.

À Posteriori Assessment of KPIs

The à posteriori assessment of KPIs is performed after an inspection or a detail investigation. The à priori values, either qualitative of quantitative are updated based on the observations and the actual traffic load. Similar to à priori assessment, the Bayesian networks for qualitative, à posteriori assessment can differ from the one for quantitative assessment.

Visual inspections are considered to be subjective and uncertain allowing only qualitative outcome such as condition rating. The connection to KPIs is therefore lost and it is sometimes reestablished after a detailed investigation or structural reanalysis. Although it is undeniable that observations made during visual inspection are often fuzzy, they can be useful if their inherent uncertainty is properly modeled. The observation “reinforcement corrosion” is indeed not very informative and this fact has to be modeled adequately. The reinforcement corrosion can be anywhere on the bridge or its elements, i.e., it has to be uniformly distributed. Likewise, a spalling area and a section loss can also be modeled with noninformative or slightly informative distributions. If, however additional information is available such as that the reinforcement corrosion is located in highly stressed, i.e., “hot” areas, the uncertainty with regard to its influence on KPIs can be significantly reduced. The spalling area and section loss can further reduce uncertainty. This reasoning applies both for qualitative as well as quantitative assessment of KPIs.

The Bayesian network for qualitative à posteriori assessment is presented in Figure 8. The node “load effect” is taken over from the à priori assessment. The force reduction in posttensioning, the reinforcement corrosion in the hot area of superstructure and the dysfunctionality of bearings is observed during an inspection. These observations influence the resistance and serviceability limit resulting in updating of the safety and serviceability rating evaluated in à priori assessment. The observations can also include forecast and to this end the parameter “time” is included in Bayesian network. The forecast can be deterministic or stochastic using, e.g., Markov chains.

The observations can be uncertain as well. For instance, the reliability of the force measurement is often far from ideal, so that some false positives or some false negatives may occur. This can be also modeled with likelihood functions.

Bayesian network for quantitative à posteriori assessment is presented in Figure 9. The only difference to the Bayesian network for qualitative à posteriori assessment is the replacement of the node “hot areas” with the node “location.” Furthermore, the joint distributions of the nodes in Figure 9 are not discrete but continuous and therefore quantitative. This also means that some observations e.g. dysfunctionality of bearings have to be transformed into quantitative distribution, which is indeed tedious and require expert knowledge.

If the location of the reinforcement corrosion is not known then one has to assume noninformative distribution of location variable over the whole superstructure.

In order to illustrate the à posteriori assessment of KPIs, the same example as for the à priori assessment is analyzed. The visual inspection revealed a spalling area with the reinforcement corrosion (node “reinforcement corrosion” in Figure 9) with a section loss of 10%. This is a typical entry in numerous BMSs, in which defects are classified in so called knowledge catalogues. The location of the defect is not known and there is inherent uncertainty with regard to section loss. The experience has shown the likelihood of section loss can be expressed as in Table 1.

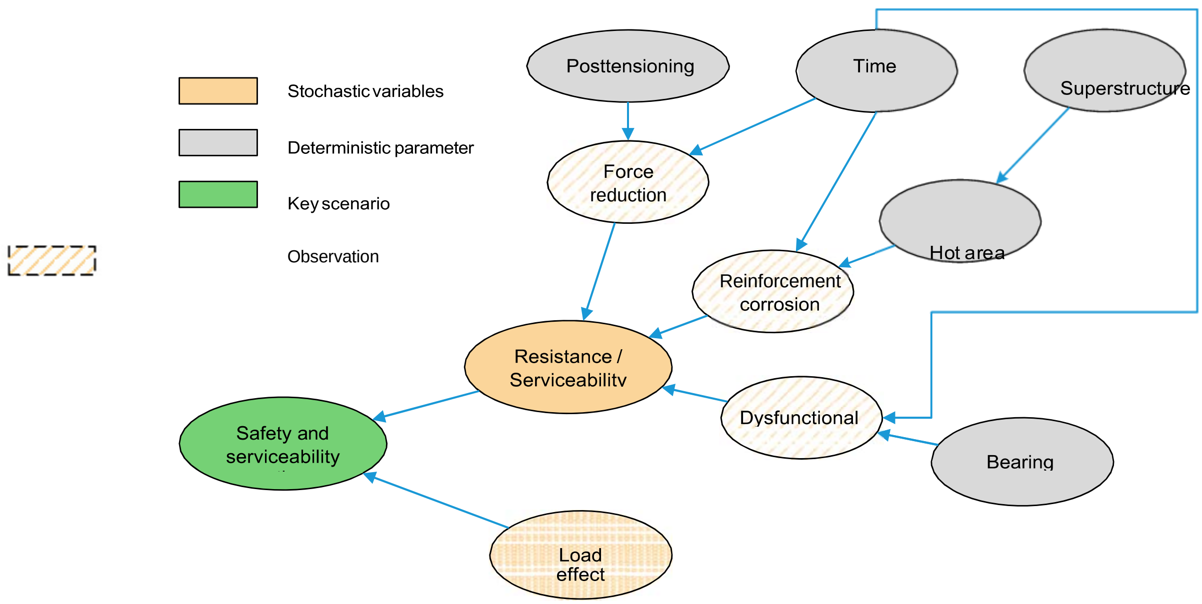

FIGURE 8 Simplified Bayesian network for qualitative à posteriori assessment of KPIs.

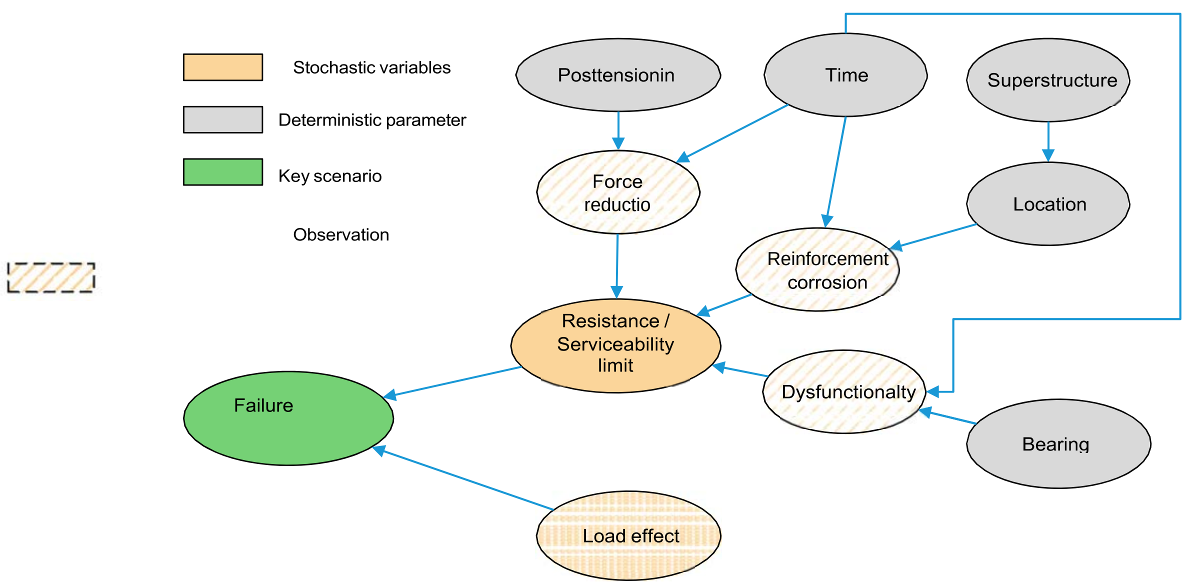

FIGURE 9 Example of a Bayesian network for quantitative à posteriori assessment of KPIs.

TABLE 1 Likelihood of Indicating Specific Section Loss

Section loss	5%	10%	15%	20%
Probability	60%	20%	10%	10%

In the first step one has to evaluate the safety index for the deterministic section loss with no information on its location. This means the safety index has to be computed for every possible location of a corroded reinforcement. The location of the corroded reinforcement is given with the distance *y,* as in Figure 10.

The safety index cannot be larger than the one obtained in à priori assessment. It can be however lower if the section loss is in the area of high bending moments. This reasoning results in the following expression for safety index.

$$\beta = Min\left(\frac{\nu \cdot \mu_{M_R} - \mu_P \cdot \xi \cdot (1-\xi) \cdot l}{\sqrt{\sigma_{M_R}^2 + (\sigma_P \cdot \xi \cdot (1-\xi) \cdot l)^2}}, 4\right) \tag{2}$$

$$P_f = \Phi(-\beta)$$

The safety index can be plotted as a function of location of corroded reinforcement. It can be seen in Figure 11 that for certain locations, the section loss has no influence on safety index. In the middle part, there is however a clear reduction of safety index indicating a "hot area".

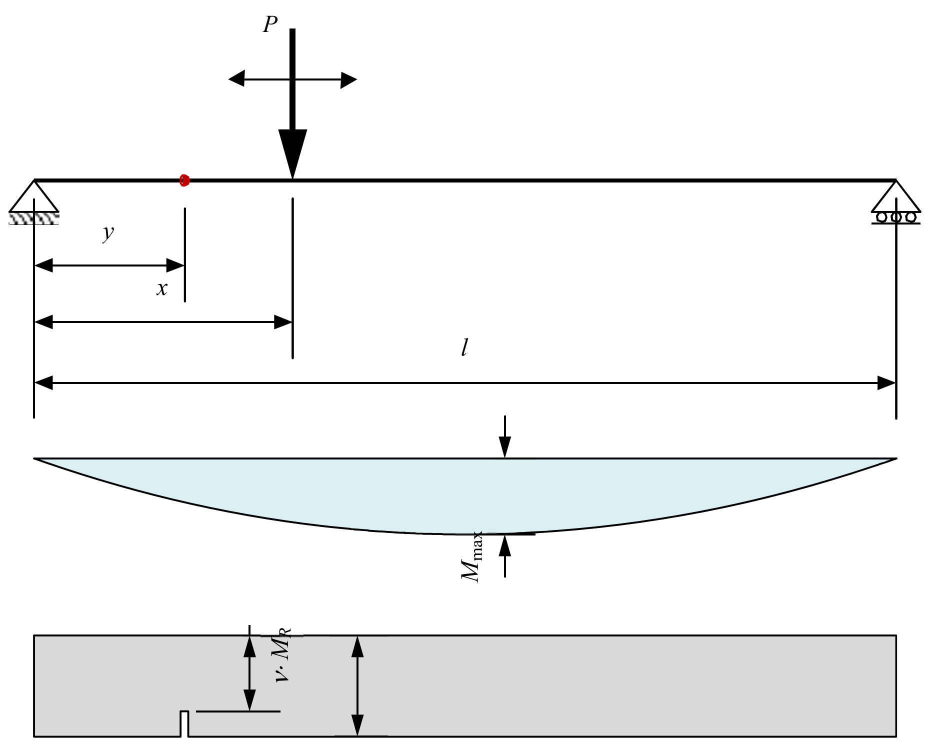

FIGURE 10 Resistance of the damaged structural system.

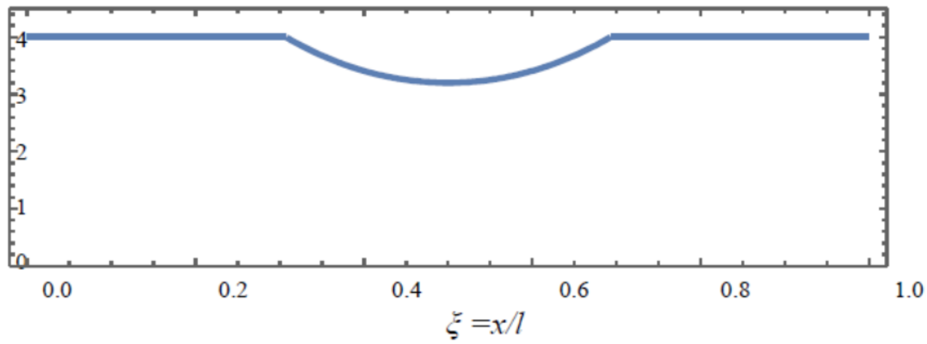

FIGURE 11 Safety index in function damage location.

Assuming that there is an equal probability of corroded reinforcement being anywhere on the beam one has to integrate the probability of failure for all locations as in following expression:

$$
\begin{aligned}
P_f &= \int_0^1 \Phi\left(-Min\left(\frac{\nu \cdot \mu_{M_R} - \mu_P \cdot \xi \cdot (1-\xi) \cdot l}{\sqrt{\sigma_{M_R}^2 + \left(\sigma_P \cdot \xi \cdot (1-\xi) \cdot l\right)^2}}, 4\right)\right) \cdot d\xi \\
&= \int_0^1 \Phi\left(-Min\left(\frac{0.9 \cdot \mu_{M_R} - 100 \cdot \xi \cdot (1-\xi) \cdot 10}{\sqrt{20^2 + \left(15 \cdot \xi \cdot (1-\xi) \cdot 10\right)^2}}, 4\right)\right) \cdot d\xi = 15.69 \cdot 10^{-5} \qquad (3) \\
\beta &= -\Phi^{-1}\left(P_f\right) = 3.6
\end{aligned}
$$

The probability of failure increased fivefold although the location of the crack is still not considered. In the second step the uncertainty of the observation can be also considered. Considering the likelihood values from Table 1 the probability of failure can be computed as follows:

$$
\begin{aligned}
P_f &= 0.6 \cdot \int_0^1 \Phi\left(-Min\left(\frac{0.95 \cdot \mu_{M_R} - 100 \cdot \xi \cdot (1-\xi) \cdot 10}{\sqrt{20^2 + \left(15 \cdot \xi \cdot (1-\xi) \cdot 10\right)^2}}, 4\right)\right) \cdot d\xi + \\
&+0.2 \cdot \int_0^1 \Phi\left(-Min\left(\frac{0.9 \cdot \mu_{M_R} - 100 \cdot \xi \cdot (1-\xi) \cdot 10}{\sqrt{20^2 + \left(15 \cdot \xi \cdot (1-\xi) \cdot 10\right)^2}}, 4\right)\right) \cdot d\xi + \\
&+0.1 \cdot \int_0^1 \Phi\left(-Min\left(\frac{0.85 \cdot \mu_{M_R} - 100 \cdot \xi \cdot (1-\xi) \cdot 10}{\sqrt{20^2 + \left(15 \cdot \xi \cdot (1-\xi) \cdot 10\right)^2}}, 4\right)\right) \cdot d\xi + \qquad (4) \\
&+0.1 \cdot \int_0^1 \Phi\left(-Min\left(\frac{0.8 \cdot \mu_{M_R} - 100 \cdot \xi \cdot (1-\xi) \cdot 10}{\sqrt{20^2 + \left(15 \cdot \xi \cdot (1-\xi) \cdot 10\right)^2}}, 4\right)\right) \cdot d\xi \\
&= 0.6 \cdot 5.134 \cdot 10^{-5} + 0.2 \cdot 15.69 \cdot 10^{-5} + 0.1 \cdot 56.62 \cdot 10^{-5} + 0.1 \cdot 190.55 \cdot 10^{-5} = 30.94 \cdot 10^{-5}
\end{aligned}
$$

The probability of failure is doubled again and the safety index β dropped to 3.42. This means that the safety is not given any more as the probability of failure exceeds *5.0·10-4*.

CONCLUSION

In this paper, an approach is proposed to include results of visual inspections, which are often fuzzy, into assessment of KPIs for bridges, i.e., safety and serviceability. It makes extensive use of information from design phase, which needs to be merely updated based upon the results of visual inspections. The approach closes a gap characteristic for today's practice, in which different performance indicators are used during the service life of a bridge.

The proposed approach relies heavily on Bayesian networks that can yield both qualitative and quantitative results. The qualitative approach seems to be the next logical step given the current inspection practice worldwide. To this end the Bayesian networks has to be adapted to accommodate relevant types of observations that are common in different countries. In further steps the quantitative approach can be gradually adopted, perhaps together with the introduction of BIM.

Finally, it should be noted that the visual appearance of a bridge is not addressed in this paper, although it may play important role in decision-making process. Spalling concrete, dripping joints, and corrosion traces are not very appealing and the owner or operator is inclined to remedy them in order to protect its reputation. The commonly used condition rating is often strongly influenced by visual appearance and in fact it can be used to evaluate it. A decent visual appearance can therefore be regarded as a performance goal as well. It's up to an owner or operator and the social environment to set up criteria for the decent visual appearance.

AKNOWLEDGMENTS

This paper is partly based upon work within the COST Action TU1406 supported by COST (European Cooperation in Science and Technology).

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Plenary Session

PLENARY SESSION

Highlights from the 11th International Bridge and Structures Management Conference

Putting Bridge Management into Practice

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Applied Pavement Technology, Inc.

This presentation summarizes several highlights during the conference and observations about putting bridge management into practice. The author's perspective is influenced by her background in asset management and pavement management, so she admitted that she may not know some of the bridge specifics. However, she provided her insights from an asset management perspective. She also pointed out that she couldn't attend every session, so the presentation is based on what she heard or saw, as well as comments provided during interactions with attendees.

For those who attended the conference there was general agreement that they found the following:

- A strong and diverse program;
- An exceptional support team assisting attendees;
- Many opportunities to learn from others (several people mentioned this as the greatest benefit to attending the conference); and
- An attentive and engaged audience that asked questions of the speakers and took the opportunity to exchange ideas.

The presentation is organized around the six theme areas listed below:

- Responses to legislation,
- Use of technology,
- Inadequate funding,
- Increased emphasis on preservation,
- Increased focus on risk, and
- Staffing and workforce development needs.

For each area, Zimmerman highlighted the practices that were discussed at the conference and the challenges that still need to be addressed.

TOPIC 1. RESPONSES TO LEGISLATION

There were two sessions specifically targeted towards the transportation asset management (TAM) requirements under the legislation: one from a federal perspective and another from the states' perspective. The FHWA indicated that the legislation requires states to develop

Transportation Asset Management Plans (TAMPs) that describe how the National Highway System (NHS) will be managed to achieve system performance effectiveness and state departments of transportation (DOT) targets for asset condition, while managing the risks, in a financially responsible manner, at a minimum practicable cost over the life cycle of its assets. The FHWA conveyed TAM as a combined focus on system preservation, sustainability, and increased accountability. Participants indicated that these sessions were very informative and contained more information than has been available in the past. FHWA also indicated that additional guidance would be issued in the near future.

One of the requirements relates to the minimum standards for bridge and pavement management systems (BMS and PMS). These systems are expected to be able to:

- Store inventory and condition data;
- Forecast deterioration;
- Determine the benefit–cost over life cycle to evaluate alternate actions;
- Identify short- and long-term needs;
- Identify projects to maximize benefits within financial constraints; and
- Recommend programs and schedules.

Based on the presentations at the conference, agencies are strong in terms of using bridge management systems for storing inventory and condition information, but are less experienced in the other areas. As a result, there were a number of presentations about developing performance models and customizing software.

There were several challenges related to the legislation that emerged during the conference, including those listed below.

- Differences between state and federal performance measures. One presenter discussed differences between existing state and federal performance measures. The speaker indicated that in that state, the state measures are more meaningful to the way the agency manages bridges. He considers the federal measures to be too simplistic for managing the system.
- Stovepipes. Several participants commented that the TAMP is leading agencies toward more coordinated decision making that focuses on the system rather than individual projects. The Utah DOT suggested that there has to be coordination between pavements and bridges before a project moves forward.
- Balancing flexibility with need for standardization. There were some interesting discussions among participants about balance state data needs with federal data requirements. The state participants indicated that they want flexibility in how the manage the system, but they noted that standardization allows for more shared efforts.
- Validating the accuracy of performance models. There was some discussion about how performance models are becoming increasingly important when conducting life-cycle planning. There was a presentation on validating the accuracy of models so agencies can determine whether system performance matches assumptions that are being made.
- Implementation of TAMP. The FHWA will be conducting consistency checks to ensure that states are managing in accordance with their TAMPs. This means that State Transportation Improvement Programs (STIPs) should reflect the investment expenditures outlined in the TAMP.

- Involvement with metropolitan planning organizations (MPOs). While most states have well-established planning procedures, the new requirements are forcing more coordination with MPOs and other agencies managing portions of the NHS. This will require states to think about new ways to ensure the coordination takes place.

TOPIC 2. USE OF TECHNOLOGY

There were a lot of presentations related to the use of technology. Presentations generally addressed one of the topics listed below.

- 3-D bridge modeling is advancing. It appears that a number of agencies have been working to advance 3-D modeling and it appears to be taking off.
- BMS capabilities are evolving beyond Pontis. There were numerous sessions on BMS capabilities and the enhancements that are now available. There were also several presentations from states describing how they are using these tools.
- Analytics. The participants also heard that technology has made possible some more sophisticated forms of analyses, including:
  - Risk assessment. These assessments were being used to help set agency investment priorities.
  - Life-cycle assessment. Presentations on this topic illustrated how life-cycle assessments help identify cost-effective strategies to manage bridges.
- Validating common collapse assumptions. A representative from New Mexico DOT talked about common assumptions about bridge collapse (e.g., failure due to deterioration). He indicated that deterioration is not the only factor. His study found that overloading has been more significant in his experience.
- Performance model validation. This topic was mentioned earlier.
- Earthquake preparedness and response. Technology is being used at the Oklahoma DOT to reduce bridge inspection requirements after an earthquake. In one example, the original model required inspection of 772 bridges within a 50-mi radius of the epicenter. The new protocol suggests only 189 bridges need to be inspected within a 30-mi radius. There was also a presentation about ShakeCast, which includes real-time motion models that further reduce the inspection requirements to 32 bridges. These are examples of how technology is leading to significant reductions in staffing demands, which leads to agency savings.
- There were also some challenges associated with the use of technology that were discussed at the conference, as noted below.
- Bridge management is still primarily used for storing data—not to influence investments. As noted earlier, bridge management is still primarily used for storing inventory and condition information. To fully take advantage of the new capabilities that are available, guidance and training will be needed. One European presentation indicated that agencies are "still approaching bridge management."
- Management systems are not integrated into existing planning and programming processes. Several presenters indicated that BMS recommendations aren't influencing project- and treatment-selection processes. If these activities are not coordinated, it may make it difficult for state DOTs to make and achieve their performance targets.

- TAM policies before tools—have to invest time up front before implementing tools. The phrase "TAM policies before tools" was mentioned several times. A number of agencies indicated that they tend to jump at the glamor of tools without thinking about the policies and processes that have to be set ahead of the implementation to ensure success. On individual indicated that if you don't put the time in up front, you'll get nothing useful out of the system because you won't trust the results.
- Still have a long way to go in analyzing trade-offs. Several state DOT representatives indicated that they are interested in being able to analyze trade-offs between assets (e.g., pavements and bridges) and programs (e.g., preservation, capacity, and safety); however, they recognized that there's still a long way to go in terms of being able to do that effectively.

TOPIC 3. INADEQUATE FUNDING

Throughout the conference, many people indicated that funding was not adequate to address their needs. However, there were also several state DOTs who indicated that they were seeing more funding being allocated to bridges in recent years. These examples included the following:

- Nationally, there is evidence that 3 years of high-profile bridge failures have led to accelerated replacement programs in several agencies.
- To protect bridges, Pennsylvania DOT increased load limit postings to justify funding needs. The presenter indicated that he believes the increased load limit postings had an impact on a measure that increased funding levels.
- Several state DOTs mentioned in presentations that they have general guidance available for project and treatment selection, but a number of states indicated that they're moving towards having a plan for each bridge to demonstrate funding needs.

Many of these same state agencies that indicated that they had developed a plan for each bridge also reported that they can't fund the plan. They reported that they'll have to develop more realistic strategies that consider funding constraints for their TAMPs. One agency indicated that a "tidal wave is coming" and the agency is not prepared to handle it. Some state agencies indicated that setting targets is difficult because several large bridges, some of which are not on the state-maintained system, can have a huge influence on the percent of Poor surface area and these agencies are having a hard time trying to predict when that will happen. State agencies also discussed challenges associated with the shifting emphasis that takes place between the availability of funding for pavements and bridges. These agencies indicated that they would prefer a more balanced approach so funding is more consistent over time. A few states indicated that they are moving in the direction of having more-balanced programs.

TOPIC 4. INCREASED EMPHASIS ON PRESERVATION

There was a lot of discussion at the conference about the increased emphasis on preservation strategies for bridges. Agencies indicated that life-cycle strategies tend to encourage the use of preservation, which enables agencies to sustain conditions and asset value over the long term. Agencies also indicated that a BMS could be used to identify good candidates for preservation, but several state agencies indicated that they had developed a "health index" to guide investment decisions for bridges. One of the messages that was conveyed is that agencies can't afford to let their bridges fail, so preservation strategies have become critically important to keep those bridges operation. With pavements, agencies can let some roads deteriorate to the point that they become gravel roads, but that isn't an option with bridges (although a presentation by New York City DOT indicated that they had developed a benefit–cost prioritization process that enabled them to identify which bridges are expendable). With bridges, the primary strategies to date have involved posting a bridge or decommissioning it if conditions get really bad. However, some state DOTs are coming up with other solutions. For instance, the Arizona DOT presented an option in which they turned a bridge into a pedestrian bridge rather than let traffic continue to use the bridge.

There are several challenges associated with the increased emphasis on preservation. For instance, many state DOTs continue to rely on a worst-first strategy in which funding goes to address the worst bridges and there is little investment in preservation activities. Other challenges that were identified during the conference include the following.

- Bridge preservation work is often deferred since impacts aren't realized immediately. Several speakers talked about the fact that when districts are given flexibility to use maintenance funding for either pavements or bridges, pavements are often selected over bridges because roads deteriorate faster than bridges. These decisions have long-term impacts on bridges from a life-cycle perspective that are not recognized.
- Data-driven investment decisions often lead to changes in business processes that challenge well-established ways of doing business. Several state DOT employees talked about the fact that moving towards data-driven decisions are changing the way transportation agencies do business. This is forcing agencies to set up new business process that better link field practices with analysis results. This is becoming increasingly important since state DOTs are not setting performance targets and reporting progress towards that progress. This may require more oversight and coordination between central office and districts.
- "Governor rents bridges for 4 years." Another topic that was raised was the fact that it can be a political challenge to get funding for bridge preservation because many elected officials don't have a long-term perspective. One person commented that the governor of his state doesn't think of bridges as a long-term investment. He acts as if he just rents them for the 4 years he is in office.

TOPIC 5. INCREASED FOCUS ON RISK

There was a lot of discussion about using risk to prioritize bridge investments. Zimmerman observed that the risk analysis being done for bridges appears to be more advanced than on the pavement side since it has been a part of the analysis for so long. She observed that the risk analysis goes well beyond what is required under the federal requirements and the approaches used have become more scientific and consider multiple objectives (such as safety, mobility, and sustainability). Paul Thompson introduced a framework for conducting this type of analysis.

Even though the analysis of risk is fairly advanced, several challenges exist, including those listed below.

- Elected officials do not understand risk. Since risks are not well understood; agencies have tended to scare them with respect to disasters. Thompson suggests that by focusing on resilience rather than vulnerability, agencies have an objective process that emphasizes what is being bought for each dollar of investment.
- There is a persistent gap between risk management needs and available funding. The funding gaps that were raised earlier also related to bridge risks. Agencies indicated that funding is not sufficient to mitigate their risks.
- Need to move to a prioritized, multiyear risk mitigation plan with a common basis for trade-off decisions. One of the earlier points in the summary concerned agencies' interest in analyzing trade-offs between different assets and programs. It was discussed that risk provides a possible basis for conducting that analysis.

STAFFING AND WORKFORCE DEVELOPMENT CHALLENGES

Throughout the conference there were also many comments about staffing and workforce development needs in at least the two areas listed below.

- Using BMS programs effectively. Conference participants identified several factors that influence this need. They indicated that the available systems can be too complex and staffs don't have the years of experience to know whether the results make sense. Additionally, staff reductions have limited the time available to run the necessary analyses. This becomes a bigger issue because those same agencies don't have money available to hire consultants to help them. While some states outsource work, several state DOTs indicated this option is not available to them.
- Changing business processes to ensure alignment. As mentioned earlier, state DOTs are now setting long-term and short-term performance goals and targets, so it is increasingly important that agency decisions are aligned. There may need to be training to help agency personnel understand why the agency is changing the way it has done business for so many years.

MOVING FORWARD

Zimmerman identified several potential areas of research based on her observations during the conference. These opportunities include the development of guidance, methodologies, and analysis tools to address the challenges that were identified. She noted there are also opportunities to help agencies think about how they will adapt their organizations to the "new culture" that supports data-driven, integrated decisions.

Zimmerman appreciated that opportunity to participate in the conference and thanked the audience for their attention.

Appendix

APPENDIX

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