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International Bridge and Structure Management

Tenth International Conference on Bridge and Structure Management

October 20–22, 2008
Buffalo, New York

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International
Bridge and Structure Management

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Preface

The 10th International Bridge and Structure Management Conference was held on October 20–22, 2008, in Buffalo, New York. This conference was conducted by the Transportation Research Board (TRB) Bridge Management Committee (AHD35) and the Structures Maintenance Committee (AHD30) in cooperation with the Federal Highway Administration, New York State Department of Transportation, New York State Thruway Authority, New York State Bridge Authority, Multidisciplinary Center for Earthquake Engineering Research, and University at Buffalo: The State University of New York.

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* in April 2003. The 10th 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 aid bridge practitioners, managers, and researchers in taking advantage of the characteristics of existing systems from around the world, and identifying new and anticipated enhancements. The papers were not subjected to the TRB peer review process.

Contents

STRUCTURAL PERFORMANCE, MONITORING, AND REMAINING LIFE

ACCELERATED CONSTRUCTION, FIBER-REINFORCED POLYMERS, AND CORROSION EVALUATION

BRIDGE MODELING AND NBI TRANSLATOR

Application of Bridge Management in Transportation Agencies

The German Approach to Bridge Management
Current Status and Future Development

PETER HAARDT
RALPH HOLST
Federal Highway Research Institute

Germany's road infrastructure grew over centuries to become the arteries and lifelines of our society. The present safety of the infrastructure has to be ensured under consideration of environmental aspects. At the same time the owner has to make sure that the maintenance activities are carried out in the most efficient way. Considering the fact that financial resources are restricted, maintenance costs have to be spent in a way to obtain the greatest possible benefit. In the case of bridges, which are one of the most important parts of the road infrastructure in Germany, this task is supported by the application of a bridge management system (BMS). The existing German BMS contains assessment and optimization procedures on object and network level and is the basis for advancements to meet future demands. Developments concern life cycle and quality-oriented, holistically optimized procedures. Reasonable infrastructure management will contribute to meeting efficiency and sustainability objectives and to achieving interoperability. Here holistic network infrastructure management methods are required. There is a strong need for management solutions during the whole service life of a structure. The definition of criteria for evaluation of the relevance of failure mechanisms, including acceptance thresholds, requires the availability of relevant data for management procedure. Tools for innovative investigation methods and an effective data management will help in meeting the requirements. Relevant fields of research are improved maintenance strategies to meet future demands concerning heavy goods traffic, application and further development of nondestructive testing methods for efficient and sustainable management structures, and the improvement of analytic management tools to meet future demands.

It can be said today with certainty that a high-quality road infrastructure is a fundamental precondition for an industrial society, insofar as it creates prosperity and provides citizens with a commensurate quality of life. Social, political, ecological, and economic conditions and innovations all have an effect on the necessary continuing development, and a successful road infrastructure policy must adapt to these conditions. Road infrastructure is not exclusively technical by nature; it also has social, political, and ecological dimensions.

As Europe grows economically and culturally closer together, the road infrastructure will come under even greater pressure. It must be adapted to meet new demands, while at the same time funding for repairs and maintenance must be optimized. Failure to provide specific investment to fulfill this need would subsequently result in a considerable burden of costs on the economy and the prosperity of our society would be substantially impaired.

Conditions in Germany are changing as well, for example, as a result of European harmonization, demographic developments, and the emergence of new markets. Thus, in addition to preserving what we have, the flexible operation, reliable availability, and demand-oriented adaptation of road networks will collectively constitute an important task in future.

Already low disturbances in the road network by traffic restrictions or by the failure of single structures lead to strong obstructions of traffic with considerable subsequent economic costs as well as to negative effects on the environment. Life-cycle-oriented road infrastructure management is indispensable.

In Germany this is particularly true for the federal road network, which carries the main load of the transit traffic by reason of its central position in Europe, and it will have to take increasing traffic loads in future due to the further development of the European market. With assets of currently approximately €170 billion, this road network represents considerable fixed assets. About €50 billion fall to bridges and engineering structures whereas bridges represent the most important part. Among other structures there are currently 37,817 bridges with a total length of 1,963 km (1 km = 0.62 mi) and a bridge deck area of 28.59 Mio m^2 (1 m^2 = 10,75 ft^2). Most of them are prestressed concrete bridges (around 70%).

The main part of the bridges was built between the 1960s and the 1980s (see Figure 1). At that time our current knowledge about the durability of structures was not available. Despite the planned lifespan of more than 70 years, scores of bridges show major damages after a time period of 30 to 40 years. Beside common corrosion damages older prestressed concrete bridges show some—until that time unknown—severe weak points (*1*):

- Fatigue effects of tendons in coupling joints,
- Low shear force capacity due to marginal shear reinforcement,
- Rupture of tendons due to stress corrosion associated by imperfectly grouted ducts.

Construction and design principles are subject to continuous improvement toward avoiding faults. Over the years specific problems were considered and a very high quality of construction methods was achieved. Nevertheless maintenance needs today are in the range of 1% of the fixed assets, which represents about €500 million per year.

The maintenance programs prepared for this purpose not only require a high budget, but also influence the traffic infrastructure and, thus, the economy and society as a whole. The present safety of the structures has to be ensured under consideration of environmental aspects. At the same time the structure owner has to make sure that the maintenance activities are carried

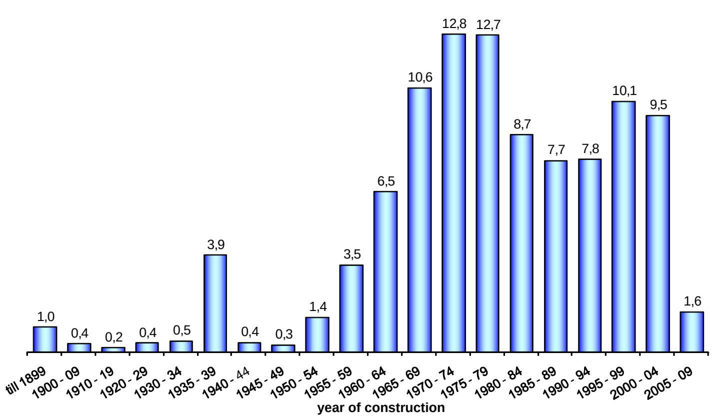

FIGURE 1 Age distribution of federal road bridges in Germany (January 1, 2007).

out in the most efficient way. Considering the fact that financial resources are restricted, the maintenance costs have to be spent in a way to obtain the greatest possible benefit. This task is supported by the application of a management system that is described in the following.

Current Status of Bridge Management in Germany

As a tool to support a cost-efficient and sustainable maintenance management a bridge management system (BMS) is developed with the aim to implement a systematic maintenance approach according to nationwide uniform criteria (*2*). In this context instruments and methods that enable the stakeholders involved in the maintenance process are being developed to plan, steer, control, and check the complex process on time. A simplified flow chart of the tasks to be carried out in the context of systematic bridge maintenance is shown in Figure 2.

Systematic bridge maintenance should be understood as a process. Starting from bridge stock and condition data resulting from regular bridge inspections damage analyses and deterioration forecasts can be made. After checking of possible maintenance strategies a priority ranking as well as a program formation and provision of the required financial means are carried out. Documentation of the achieved results as well as balancing of accounts follow the project planning and the execution of construction work.

Bridge Inspection

To manage the infrastructure in an efficient and sustainable way, it is essential to get sound information about the bridge stock and about the condition of bridges and engineering structures in the course of trunk roads. Therefore all structures have to be inspected in certain intervals. According to the German Standard DIN 1076 (*3*) distinctions are made between main inspections, simple inspections, inspections on special occasions, inspections according to

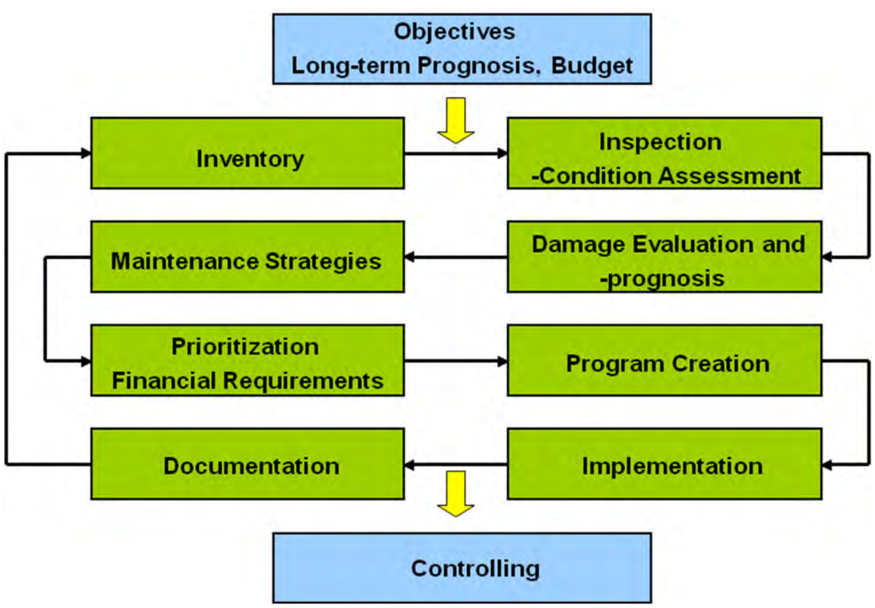

FIGURE 2 Systematic road maintenance in Germany.

special regulations and regular observations. Main inspections are regularly carried out every 6 years; simple inspections are to be carried out 3 years after a main inspection. Besides the regular checks in accordance with DIN 1076 special checks are required after special events or after a claim. The main inspections are executed as visual inspections with a hand near examination of the complete structure. Some field tests for deeper examination are frequently undertaken using special nondestructive testing (NDT) equipment, e.g., Schmidt-hammer to detect concrete strength, simple hammer to detect delaminations, and cover meter to determine concrete cover.

Database

The results of the inspection are collected in the database "SIB-Bauwerke" (Road Information Database–Structures), which contains technical data concerning construction type and characteristics according to the guideline ASB (2004) (*4*) as well as information on damages and suggested maintenance measures for each bridge in the network according to the guideline RI-EBW-PRÜF (2007) (*5*). Every individual damage noticed during the inspection is assessed using a four-stage scale for the valuation criteria stability, traffic safety, and durability. From this information a condition index is derived in the range of 1.0 (very good condition) to 4.0 (insufficient condition). The procedure is described by Haardt (*6*). Figure 3 shows the current distribution of condition index for structures in the federal road network.

Besides construction data current information about the condition of the bridges forms the basis of current systematic maintenance planning. The databases set up according RI-EBW-PRÜF contain extensive information about the different structures. Additional information, e.g., the geographical position, clearance, signposting, and traffic volumes, is included. All data are recorded by the local road administrations, updated every half year, and handed out to the Federal Ministry of Transport, Building and Urban Affairs (BMVBS) for networkwide evaluation.

Object-Related Damage Analysis

In complex, severe, or unspecific cases as described above the common inspection procedures are not sufficient. In those cases special investigations are necessary—on the one hand to come

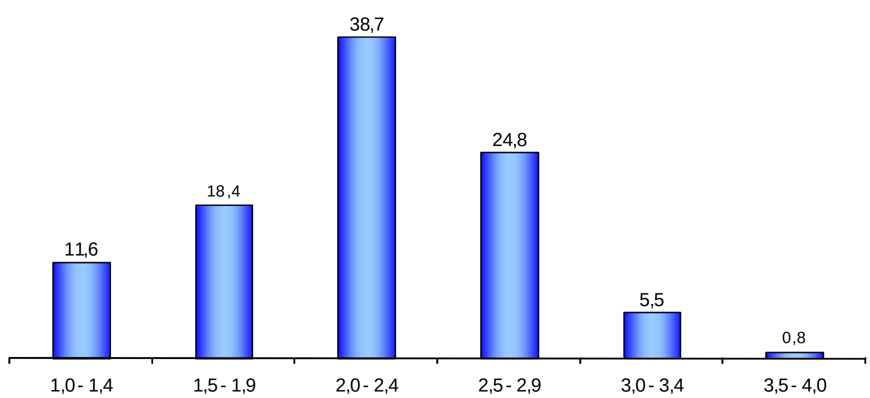

FIGURE 3 Condition index for federal road bridges (January 1, 2007).

to a more precise assessment of size and cause of damage and on the other hand to identify appropriate maintenance measures. Because of compatibility demands on the German BMS, a standardized procedure for special investigations and a guideline of object-related damage analysis were developed (*7*).

Today in the field of inspection and analyses of structures sophisticated NDT methods are applied in the frame of special investigations, if common inspection procedures are not effectual and if severe damage is assumed or already visible. This covers the application of sonic and radar devices to localize prestressing reinforcement and to detect damaged areas, electric methods to detect corrosion, magnetic methods to detect ruptures in steel elements, and the use of high-speed laser scanning devices for inspection of road tunnels. However usual NDT investigations on structures are time consuming and may lead to considerable disturbances of the traffic (*8*).

Monitoring is a promising method to achieve additional information about the time-dependent structural behavior of bridges. Currently it is applied if common inspection procedures are not effectual. Fields of application are measurement of stresses, observation of condition effects, and determination of variable loads and load effects. The progress in development of sensors and other equipment enables monitoring applications to obtain high-quality results.

Our experience is that monitoring cannot be an alternative to inspections according to DIN 1076; however it can be used to deepen and to confirm inspection results. Precondition for a successful application is a profound knowledge of the structural behavior and measurement techniques. Results of regular inspection have to be considered and relevant structural problems have to be investigated by sensitivity analysis. An insensitive monitoring system is required because unexpected problems have to be anticipated (e.g., vandalism, power failure).

Bridge Management System

The BMS is a comprehensive management system for structural maintenance. It is developed as a tool for all stakeholders of road administrations and federal institutions. It provides state authorities with draft maintenance plans required to obtain improvements at project level, to maintain structures in an acceptable condition, and to meet network-level strategies, long-term objectives, and budgetary restrictions. The Federal Ministry is supported by comprehensive information on the current condition of structures, by estimation of future funding requirements, and by developing strategies for achieving long-term objectives (*9, 10*).

The BMS for state authorities falls under the bottom-up type. Object-related analysis and assessment procedures take place based on the results of inspection according to DIN 1076. Subsequently the results are optimized on the network level and integrated in networkwide maintenance programs. Coordinated computer programs provide the subsequent programs with first results. Transparency is guaranteed and the inclusion of additional data as well as direct intervention, e.g., fixed maintenance measures into the calculation processes, is possible. The existing database SIB-Bauwerke and the common road databases (e.g., TT-SIB, NW-SIB) are integrated to provide data for the subsequent modules (see Figure 4):

- BMS-MV (measure variants) for the supply of all the information needed by subsequent computer programs. The module proposes technically reasonable measures for every

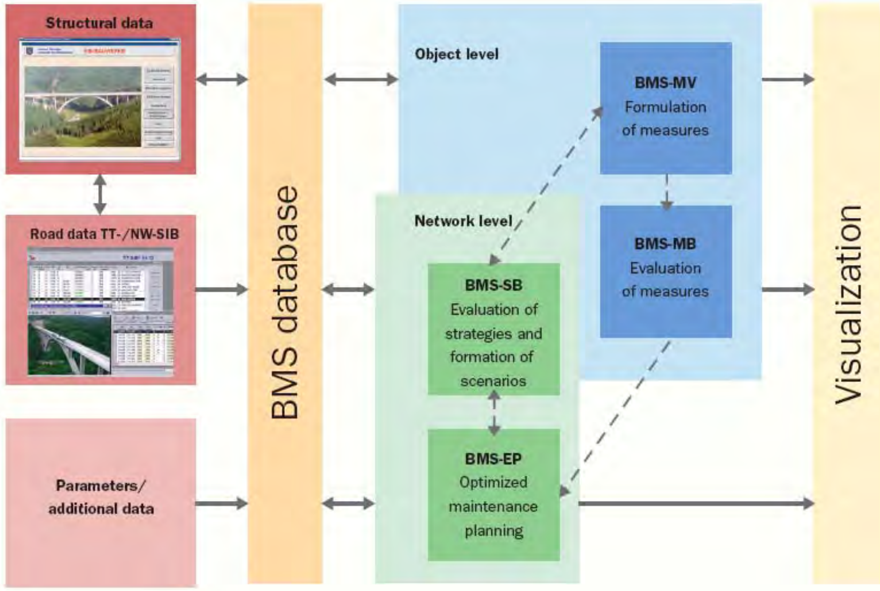

FIGURE 4 BMS in Germany.

damage observed on the structures at hand. In addition, combinations of measures are proposed to minimize overall costs. Measures and measure combinations are considered for a period of 6 years. BMS-MV also includes a comprehensive BMS database.

- BMS-MB (measure evaluation) for evaluation of maintenance alternatives on the object level. This module computes construction costs and economic benefits for each combination proposed by BMS-MV. So far a full-cost approach is realized. This evaluation is performed for each possible year of application and results in an optimized maintenance strategy on object level in an economically sense. Because of boundary restrictions in general this optimal strategy cannot be realized. The output of the analysis results on the object level—measures and measure combinations with associated costs, benefits, and effects on the state of structures—is passed to the network level for further optimization.
- BMS-EP (maintenance program) for optimization of maintenance planning on the network level and presentation of maintenance programs. This module is responsible for choosing the best combination of measures (maintenance strategies) for each year of the planning period of 6 years (short-term optimization). The main objective consists of achieving the best overall state of all structures at the end of the planning period given a limited budget for each year (finance scenario). The module may also be used to determine the necessary yearly budget to achieve a desired quality level in the network (quality scenario).
- BMS-SB (scenario building) for evaluation of object-related maintenance strategies on the network level. This module is designed for medium- and long-term prediction of maintenance costs and condition states for given different maintenance strategies.

At the federal level information from the BISStra (Federal Road Information System) database, which contains amongst others all structural data and the results from the state-level planning process is considered. The BMS for federal authorities falls under the top-down type. Currently long-term expenditure forecasts are prepared, analyzed, and updated, the draft maintenance programs, drawn up at state level, are analyzed and rated, and annual statements of performed maintenance measures are evaluated. The investigations result in the available budget, direct interventions into the maintenance practice, and updating of technical rules. The above described procedures are continuously refined.

Future Development

The road infrastructure network is of vital importance for the society and economy as a whole. Passenger transport and the exchange of goods call for an efficient, safe, and environmentally appropriate infrastructure. However, the present status of the land-based transport infrastructure shows many deficiencies:

- Technical progress also results in changing functional demands on the existing transport infrastructure. If designers, contractors, and road authorities are not responding to them, the level of service will diminish, which will result in various inefficiencies. They affect both the users individually and the economy as a whole.
- Growing traffic demand is linked to more standardized technical and legal requirements. The transport systems in Europe will be more and more harmonized, which is still not the case for the underlying "hardware." Therefore, the management of the transport infrastructure cannot be commissioned exclusively to local entities. There is need for a more holistic approach integrating the best practices across Europe.
- Most developed countries are facing a continuously growing maintenance backlog. In addition to this, an existing infrastructure has to be maintained that consists of many inappropriately designed or constructed structures. Not all negative impacts on safety and security can be mastered or are already identified. So far as they are known, efficient risk management is not in place everywhere due to missing management knowledge.

In the face of European mega trends, certain economic, social, and environmental aspects of German road infrastructure management have to be adapted. In the following some relevant areas for future development are described.

Maintenance Strategies to Meet Future Traffic Demands

Current maintenance strategies aim at remediation of existing structural damages and securing traffic safety. Future effects are only covered with respect to the prediction of condition development and its effect on the prospective maintenance budget in a deterministic mode. So far future developments and demands are barely considered, although today the course is set for an effective and efficient future road infrastructure. The current road infrastructure has to be converted to be fit for the future.

The initial point of substantial investigations at the Federal Highway Research Institute (BASt) is the fact that according to current forecasts heavy goods traffic will increase by about 84% up to the year 2025 based on the conditions of 2004 (*11*). Additionally the quantity of heavy goods vehicles increases dramatically every year, and there is pressure from the transport industry to introduce "modular vehicle concepts" that could reach a maximum weight of 60 tons. The existing bridges are not designed for this purpose. Especially the older bridges designed

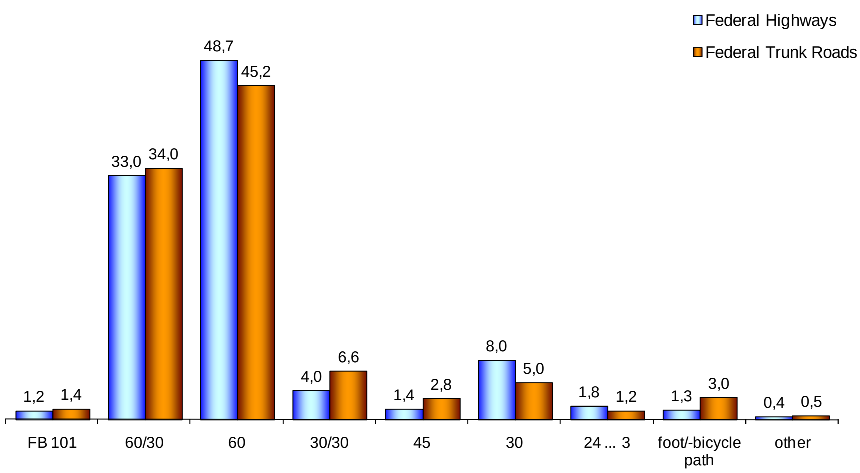

FIGURE 5 Bridge classes (%) in the federal road network (January 1, 2007).

prior to 1987 show remarkable deficits (*12*). Figure 5 shows the distribution of bridge classes according to the outdated DIN 1072 for the federal road network. The bridge class "FB1" is designed according to the current German standards, which correspond to the latest Eurocodes. Analytical investigations by traffic simulation, which in fact consider only the actual heavy goods traffic, showed that most of the older bridges have structural deficiencies and should be strengthened. This would result in costs of more than €1 billion. Indeed in the case of future traffic scenarios some newer structures do not meet design requirements and therefore a great proportion of these bridges have to be strengthened likewise.

The objective is to develop an efficient and comprehensive maintenance strategy that covers both the demands from existing and predicted conditions, and the demands from future traffic. Life-cycle aspects also have to be considered. The replacement of older bridges by sufficiently designed ones could be an efficient strategy in particular cases.

Strengthening of bridges is expected to be an outstanding task of the near future. A multiplicity of different methods exist, e.g., prestressed or slack carbon fiber-reinforced polymer (CFRP) sheets, external prestressing, additional reinforcement, injection of cracks and voids, and extension of concrete cross section by shotcrete. Although these strengthening methods are performed frequently the technological and economic limits are not defined. Questions arise according to the shortcomings and main problems. Which measures are effective in which cases? How should alternatives be evaluated?

To support the responsible road administrations, BASt is developing an expert system for strengthening of concrete bridges with respect to the improvement of technological and economic effectivity and efficiency. The expert system aims at:

- Improved information on loads and resistance of existing concrete structures and implementable strengthening methods,
- Transparent and structured procedure for identification of the most economic and promising alternative, and
- Improved information as a basis for planning and design of strengthening methods.

NDT for Efficient and Sustainable Management of Structures

As mentioned above, maintenance programs are currently prepared on the results of inspections that are carried out every 6 years as a visual inspection, which implies that damages are typically only identified when deterioration becomes visible. This indicates that there is ample demand for methods to establish the condition of structures before severe damage has occurred. For regular inspections, NDT methods should be able to provide a relative quick and inexpensive means to establish whether a structure is still in a serviceable condition or not.

However current NDT investigations on structures are still time consuming and may lead to considerable disturbances of the traffic. To speed up inspections automatic and scanning NDT methods are badly needed. The results of investigations on high-speed laser scanning devices for inspection of road tunnels and on scanning radar and sonic devices for large-scale concrete surfaces showed that these methods could bring real benefit for the inspection of structures (see Figure 6) (*13*).

It also appears feasible to construct a vehicle-based measurement apparatus with radar and magnetic measurement head for nondestructive estimation of moisture and chloride contamination of the reinforced concrete of paved bridge decks (*13*).

Against the background of the aging bridge stock and decreasing maintenance funds, optimized maintenance planning gains in importance. Today a primarily reactive management system is applied, which uses empiric or deterministic deterioration models in connection with visual inspection as described above. In the future probabilistic deterioration models are intended. By NDT measurements the uncertainty of models and input parameters could be reduced successively and predicted values could be improved. Compared with traditional visual inspection NDT methods have the advantage to detect potential damages at an early stage. Then adequate measures could be induced before severe damages occur.

Closely connected with these ideas are considerations of future life-cycle management. NDT measurements could be included here for contribution of relevant parameters and their expected development. This could also form a robust basis for infrastructure planning

(a) **(b)**

FIGURE 6 Ultrasonic scanner for application on (*a*) top of bridges and (*b*) inside a box girder (*13*).

in the frame of public–private partnership (PPP) projects. The added value of NDT in PPP models differs if a new structure or the transfer of an existing one is intended:

- New structures: application of NDT in the frame of quality assurance and control, "birth certificate," initial point of life-cycle monitoring, early detection of deficits.
- Existing structures: NDT for condition assessment, estimation of lifetime, extrapolation of condition changes.

Improvement of Management Tools to Meet Future Demands

The experiences from current practice of maintenance management in Germany show that in terms of being "fit for the future" there are still open questions in the field of maintenance planning dealing with:

- Enhancement of data quality–quality assurance,
- Further development of deterioration models,
- Development of models to describe the extent of damage and deterioration,
- Enhancement of the server-based BMS to a web-based management system,
- Coordinated maintenance planning, and
- Enhancement of the BMS for other engineering structures belonging to roads, e.g., tunnel, noise barriers, retaining walls, and signpost bridges.

Enhancement of Data Quality–Quality Assurance Our experiences are that one of the most important features for an efficient and sustainable BMS is high-quality input data. In the past the results of the bridge inspection have been evaluated "by hand" and normally by the inspection team itself to find out and to correct missing information or wrong data. Often this procedure is not successful which results in irreproducible results of BMS algorithms.

To increase data quality, BASt developed SQL queries, which serve as tools for increasing the quality of the SIB-Bauwerke database. These queries will help to find out data inconsistencies. Further steps will be the enhancement of data quality as part of the training course for bridge inspection engineers and additional detailed queries of the SIB-Bauwerke database in reference to the data required for the BMS.

Enhancement of Deterioration Models The damage and deterioration models currently used in the frame of the German BMS are based on deterministic material laws that were adapted on the basis of experiences of the road administration. The models depend on personal experiences and in fact do not always produce comprehensible results. Especially experiences diverge with regard to the lifespan of bearings and expansion joints. In future the "based on experience" type of models will be replaced by probabilistic approaches. For this attempt a suitable database is to be created, which will be a challenging task because of the variety of structural elements and their behavior.

Development of Models to Describe the Extent of Increased Damage Experiences with the German BMS showed that the issue of damage extent and its forecast is an important feature for plausible and accepted evaluation results. Therefore it is necessary to examine the phenomenon of how the damage extent develops for certain structural elements more exactly than it has been done before.

Enhancement of the Server-Based BMS to a Web-Based Management System The present state of the BMS allows for an application at a personnel computer or server. Future versions are to be complemented for an application on the web. This implies the possibilities of the web and, by combination with geographic information systems, interactive application will become possible. Furthermore it will be easier to exchange and discuss results with other stakeholders.

Coordination of Maintenance Planning The application of pavement management system (PMS) and BMS in practice is an important step forward to a sustainable maintenance of roads and bridges. To improve efficiency it becomes necessary to develop a combined approach. This includes the stakeholders performing maintenance management as a method of coordinated maintenance planning (roads and structures).

At present there is no unique model to maintain the road in total with one management system and it will be very difficult to find such a system in the near future. There are still open questions:

- What could be the purpose of coordinated maintenance planning?
- Is it possible to compare measures on roads to measures on bridges regarding "evaluation criteria?"
- How is it possible to deal with complementary criteria like financial resources and results of cost-effectiveness analysis?
- How can life-cycle analysis be implemented in existing and future infrastructures projects?

The result of a coordinated maintenance planning could reduce traffic hindrance and costs, and increase life cycles and maintenance intervals.

Enhancement of the BMS for Other Engineering Structures Belonging to Roads Currently the German BMS takes bridges into consideration as the most important part of the engineering structures of the federal highways. But there are even other engineering structures that have to be maintained and that show a high economical value (tunnel, noise barriers, signpost bridges, and retaining walls). These structures also are inspected regularly and the achieved results are recorded in SIB-Bauwerke. Indeed, for consideration in the frame of the BMS new or adapted deterioration models and other components still have to be developed.

Besides, the aspect of risk management will play an increasing role for road tunnels as fire disasters in tunnels have shown in past decades. This point has consequences, because in the case of tunnels damages and their removal have to be seen under the aspect of reducing the danger potential for road users.

CONCLUSION

The road infrastructure network is of vital importance for the society and economy as a whole. Passenger transport and the exchange of goods call for an efficient and sustainable infrastructure.

Future challenges have to be anticipated today. There is a strong need for road infrastructure to be “fit for the future”:

- The requirement to manage maintenance effectively is becoming more pressing.
- The requirement to minimize the impact of infrastructure construction, maintenance, and operation on the environment is current and likely to increase in the future.
- Difficulties in planning for future functional demands for infrastructure are greatly increased.

In the face of European mega trends certain economic, social, and environmental aspects of German road infrastructure management have to be adapted. Relevant areas are

- Maintenance strategies to meet future demands concerning heavy goods traffic. The relevant criterion for bridge management in Germany is not only the condition index but also load-bearing capacity.
- NDT for efficient and sustainable management structures. There is a need for sophisticated methods to find structural problems before severe damages occur without hindrance of traffic flow. NDT measurements should also serve as a basis for life-cycle management procedures especially in the frame of PPP projects.
- Improvement of management tools to meet future demands. There are still open questions in the field of maintenance planning that have to be solved concerning reliable probabilistic approaches, solutions for coordinated asset management, and reliable data management.

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Progress Report on Oregon's Efforts to Integrate Its State Transportation Improvement Program Project-Selection Process with Pontis

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Since the initial release of Pontis software in the early 1990s, Pontis has received wide acceptance by state departments of transportation (DOTs). According to surveys conducted by FHWA, 39 states have subscribed to the software. Although the current implementation licensing status is well documented, application of the software by individual states varies considerably. Some states are using Pontis simply as a place to store inventory data, while others are using the modeling capability to support decisions. At the 8th Annual International Bridge Management Conference (IBMC) held in 1999, the FHWA reported that seven states indicated that they were using Pontis as part of the bridge management process. None indicated exclusive use of Pontis in the development of bridge projects for the Statewide Transportation Improvement Program (STIP). Continued search of the literature since 1999 indicates a great deal of work by states in implementing Pontis, but limited success in using Pontis as the primary project selection tool for development of the STIP. Oregon has maintained bridge inspection condition information at the element level in Pontis since 1993. In 1999, Oregon's project selection method integrating inspection data from Pontis with other bridge condition data—specifically nondeterioration-based needs, including, as examples, seismic, scour, and functional deficiencies—was described in a paper given at the 8th IBMC. At that time, Oregon DOT (ODOT) linked various data collections to identify projects in 12 categories. Data primarily from Pontis were used to select problem bridges in the substructure, superstructure, and deck condition categories. Data outside of Pontis were used to select problem bridges in the seismic, scour, bridge rail, deck width, load capacity, vertical clearance, paint, coastal bridge (cathodic protection), and movable bridge categories. Outside of its use as a repository for inspection data, ODOT's efforts to implement Pontis through development of the deterioration and cost models was derailed for a period of about 5 years due to reaction to shear cracking in reinforced concrete deck girder bridges and a major reorganization of the Highway Division, including the Bridge Section. As a result of this period, greater emphasis has been given to load capacity on freight routes and route continuity, moving away from a strictly "worst first" project selection process. Increases in the costs of traffic mobility and project staging have also influenced the popularity of targeting route segments for repair and replacement projects. Beginning in 2006, ODOT resumed efforts to implement Pontis. This report outlines practical aspects of progress and challenges in implementing Pontis while attempting to maintain the comprehensive nature of our 12-category bridge management system in the STIP development process and implement a corridor-based approach to project selection. We discuss the apparent difficulty in implementing Pontis simultaneously for cross-asset management resource allocation and project-level decision making.

The Pontis bridge management software has been available to state departments of transportation (DOTs) since the early 1990s. The software has promising analytical and project decision support capabilities. In addition, a large number of states have held Pontis licenses for a long time. Yet, according to recent (2006) surveys, very few, if any, states are using Pontis in a way that matches up with early expectations for the tool (P. D. Thompson, unpublished data). Why is this? This paper reports on the Oregon experience with Pontis implementation and some of the challenges we continue to face as we go forward.

INITIAL DEVELOPMENT OF A SYSTEMATIC PROCESS

At the 8th International Bridge Management Conference, Bruce Johnson and Frank Nelson reported on Oregon's Project Selection Method Integrating Bridge Management System (BMS) Data and Nondeterioration Based Needs. Oregon DOT (ODOT) had obtained a license for Pontis soon after beta versions of the software became available in the early 1990s. In 1994, Oregon began to use Pontis to record bridge inspection data based on the Commonly Recognized (CoRe) structural elements that would later be published by AASHTO in December 1997. By 1995, ODOT, with the support of the FHWA Oregon Division, initiated a bridge project selection process for the Statewide Transportation Improvement Program (STIP) that integrated inspection data from Pontis with other data collection systems (Table 1). Element-level data from Pontis, along with National Bridge Inventory (NBI) data were used to identify needs in the deck, substructure, and superstructure categories. Data from other sources were used to identify deficiencies regarding bridge width, vertical clearance, seismic vulnerability, scour susceptibility, bridge rail, painting, cathodic protection, and electrical–mechanical needs. The result was a comprehensive process, data driven and geographically neutral, that could be used systematically to develop statewide bridge program priorities (*1*).

This project selection process was used by ODOT to develop the 1998–2001, the 2000–2003 (Table 2) and the 2004–2007 STIP. The same data and analysis methods were used to develop a 20-year needs study for bridges used in the 1999 Oregon Highway Plan (*2*).

TABLE 1 1995 ODOT Categories for Bridge Projects (*1*)

Category	Data Collections Involved	Selection Criteria
Seismic	Inventory, seismic	Major river crossing, seismic rank
Scour	Inventory, scour	Spread footing, erodable material
Substructure	Inventory, inspection	NBI, Pontis element condition rating
Superstructure	Inventory, inspection	NBI, Pontis element condition rating
Deck	Inventory, inspection	NBI, Pontis element condition rating, ADT
Railing	Inventory, rail, inspection	Site risk, element rating
Deck width	Inventory, accidents	Width, lanes, accidents, ADT
Load capacity	Inventory, inspection, load rating	Temp. structures, load rating
Underclearance	Inventory, inspection	15 ft or less vertical, impact damage
Paint	Inventory, paint	Lead paint, paint rating 3 or worse
Coastal bridge	Inventory, coast, inspection	Spalling, chlorides, element(s) rating
Movable bridge	Inventory, movable	Electrical, mechanical equipment rating

TABLE 2 2003 ODOT Categories for Bridge Projects

Category	Data Collections Involved	Selection Criteria
Level of service	Inventory, load rating	Weight capacity, legal, and permit loads
Structural safety	Inventory, bridge drawings, underwater inspections	Nonredundant elements, active foundation scour
Structural condition	Inventory, inspection	NBI deck, substructure, and superstructure rating
Seismic	Inventory, seismic	Seismic lifeline routes
Traffic volumes	Traffic volume	ADT
Route importance	District-level local knowledge	Route based
OTIA III connectivity	Inventory, mapping	Route based
Detour issues	District-level local knowledge	Travel time, feasibility

EFFECTS OF THE OREGON CRACKED GIRDER BRIDGE CRISIS ON THE PROJECT SELECTION PROCESS

Although ODOT had created a rational system for making bridge investment decisions, concern over some of the process's inherent limitations along with a volatile combination of external factors resulted in an entirely different approach to the bridge problem beginning in 2001. In March 2001, a bridge on Interstate 5 was closed for emergency repairs, resulting in a detour through the main streets of two small southern Oregon towns ill-equipped to handle the traffic for 3 weeks. Concerns were raised regarding safety, infrastructure damage to local facilities, and a negative effect on commerce in the region.

The problem was seen as a symptom of a much larger problem than the one bridge and two communities. ODOT's limited annual construction funding budget; the large magnitude of unmet needs; the ascendance of awareness of the importance of freight mobility to the national and regional economy; concern over girder cracking in reinforced concrete deck girder (RCDG) bridges designed in the late 1940s through the 1960s; and a coalition of other political and economic forces resulted in an entirely different approach to the bridge problem. The "old" project selection process, data driven and geographically blind, became known as the "worst first" approach. ODOT created a Bridge Strategy Task Force. The Task Force sought to explain the emergence of the cracked concrete bridges and identified strategies to address them. In the process, the Task Force condemned the use of a "worst-first" approach to choosing bridge repair and replacement projects and recommended that a "corridor-based" strategy be used instead. The final recommendation of the Task Force focused on Oregon's two major Interstate routes (I-5 and I-84) in the interest of returning full load carrying capacity to the nearly 675 mi of Oregon's Strategic Highway Network (STRAHNET).

ODOT completed an Economic and Bridge Options Report in 2003. This report identified 365 cracked girder bridges in need of repair or replacement (*3*). This same year, the Oregon Legislature passed a bill known as the Oregon Transportation Investment Act III (OTIA III), authorizing Highway User Tax Bonds in the amount of $1.3 billion to be issued to address the problem of Oregon's cracked girder bridges and the associated load restrictions on major freight routes. Since it was anticipated that the implementation of OTIA III would reduce the amount of bridge work that the State Bridge Program would need to accomplish, the planned repayment of the bonds at the rate of $31 million a year for 25 years has been deducted from the future annual Highway Bridge Program funding stream to be allocated to the State Bridge Program during the STIP process.

For one STIP development cycle (2006–2009), the STIP was overshadowed and heavily influenced by these new developments. ODOT modified its BMS to include a project selection process that used NBI and other data (but no element-level condition data from Pontis) to develop project ranking based on categories of structural condition and operational need. The project selection process for both OTIA and the 2006–2009 STIP was corridor based, but focusing primarily on load capacity it was not comprehensive, as the earlier project selection process had been.

During the time frame of the development of the 2006–2009 STIP, ODOT reorganized the structure of the department. Bridge design functions were moved from the headquarters unit to the regional offices, and a separate organization was created to deliver the OTIA III projects. The experienced structural managers who had been instrumental in the development of previous State Bridge Program STIPs left the department for jobs with consulting firms.

In the wake of the reorganization, the Bridge Section began the selection process for the 2008–2011 STIP by modifying the systematic process used in earlier STIP cycles to incorporate the lessons learned regarding the importance of a corridor-based approach that emphasized freight mobility. The result was a project selection process that was both corridor based and comprehensive. At this same time, the Bridge Section returned its focus to Pontis implementation efforts.

ODOT ADOPTS ASSET MANAGEMENT PRINCIPLES

Not unlike the rest of the nation, Oregon is facing an aging transportation infrastructure; anticipated population and traffic growth; and state and federal revenues supporting highway programs that have failed to keep pace with needs. In September 2006, coincidentally, Oregon both adopted a new transportation plan and hosted the U.S. Domestic Scan Program's "Best Practices in Transportation Asset Management." Six priorities that became key initiatives emerged during the planning process (*4*):

- Maintaining and maximizing the assets in place;
- Optimizing the performance of the existing system through technology;
- Integrating transportation, land use, economic development, and the environment;
- Integrating the transportation system across jurisdictions, ownerships, and modes;
- Creating sustainable funding; and
- Investing in strategic capacity enhancements.

ODOT is in the early stages of implementing a comprehensive asset management program. Asset management is a key focus area at present for many transportation agencies in the United States and abroad. Asset management focuses on using quality asset data and well-defined objectives to improve a transportation department's processes for resource allocation and utilization.

Once the asset management system becomes more robust, its data and information will be key in framing legislative dialogues regarding the nature and extent of transportation issues and challenges (*5*). It is anticipated to also provide clearer depictions for the public as to what transportation conditions they can expect at various levels of investment. As it currently stands, the existing decision-making structure in ODOT is largely integrated vertically and horizontally. All program areas are included and represented in executive, steering, and technical committees. The overall process for agencywide priority setting, funding allocations (including but not limited to the STIP), performance measures, legislative budgeting, and operational planning and budgeting identifies asset management as one of the key inputs, at both the statewide policy level and the operational level. Additionally, the transportation plan identifies a policy for Triage in the Event of Insufficient Revenue, currently in effect at ODOT, which specifies that “In the event of inadequate revenue to meet system needs, support Oregonians’ most critical transportation needs, broadly considering return on investment and asset management” (*4*).

The Oregon Transportation Commission (OTC) is currently engaged in a discussion of “multimodal tradeoff analysis,” defined as “How can the State invest transportation dollars to obtain the best combination of immediate and longer-term benefits to users, regardless of modal system?” (Cambridge Systematics, unpublished paper, 2008).

Although this issue is very much larger than the relative investment levels in pavements and bridges, it brings this and other internal, highway-related issues to the foreground. In a discussion paper developed for Oregon by Cambridge Systematics, they cite results from an FHWA-sponsored workshop held October 2005, in which representatives from 15 different agencies and organizations came together for a day to exchange information and experience on the topic of multimodal trade-offs. Cambridge reports that “A number of agencies report that they do conduct comparative analysis within modes or programs to inform project selection and funding decisions.” These processes often involve tools such as economic analysis, pavement and BMS, and traditional travel demand models. In the majority of applications, the comparisons remain within a mode or funding program. Of the 12 public agencies present at the aforementioned FHWA workshop on trade-off analysis, for example, none identified any specific tools or procedures in use to conduct trade-off analysis between modes. The Cambridge report goes on to say that “On the other hand, ODOT is clearly a leader among states in terms of its investment in and refinement of the types of analytical tools, processes, and staff resources that would be required to more fully implement a multimodal tradeoff process.” Where there is a will, there is a way. Figure 1 illustrates ODOT’s asset management overview.

ODOT’s adopted vision for a fully integrated asset management system is defined as “ODOT’s assets are managed strategically by utilizing integrated and systematic data collection, storage, analysis, and reporting standards on a broad range of transportation system assets, optimizing funding and life-cycle decisions for operations, maintenance and construction business functions” (*5*).

Oregon has been evaluating Asset Manager NT and PT software. These products rely entirely on results from scenarios run in other programs, such as Pontis. The ability of software such as Asset Manager NT to produce valuable results depends on the quality of the results generated from the input systems.

Pontis Software

According to the BRIDGEWare Strategic Plan of February 2005, “Pontis is a software product that stores bridge inventory and inspection data as required by federal regulations; formulates network-wide preservation and improvement policies for use in evaluating the needs of each bridge in a network; and makes recommendations for what projects to include in an agency's capital plan for deriving the maximum benefit from limited funds" (*6*).

A 1999 paper by Edgar Small, Terry Philbin, Michael Fraher, and George Romack of FHWA recounts the history of the development of BMS. They indicate that by the late 1980s, the increasing differential between funds available and needs became a greater concern. To document and examine the current practices and tools available for more efficient bridge planning and programming, projects were initiated by FHWA and through NCHRP. These initial research projects provided the basis for further BMS development. The FHWA project continued with the development of Pontis. The NCHRP research continued with the development of BRIDGIT. The authors report that as of May 1998, there were 45 subscribers to Pontis, including 38 state DOTs (*7*).

While the Pontis and BRIDGIT development projects were under way, the U.S. Congress passed the Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA) (Public Law 102-240). This legislation mandated the development and implementation of six intermodal management systems by metropolitan planning organizations and individual states, including BMS. BMS development efforts were accentuated by the legislative requirements.

Initial developments followed different philosophies. Pontis employed a "top-down" approach while BRIDGIT followed a "bottom-up" approach. In the top-down approach, budgets and standards are used to develop optimal policies that are then used to plan projects. Feedback is provided to refine the models. Budgets and standards may be modified to perform

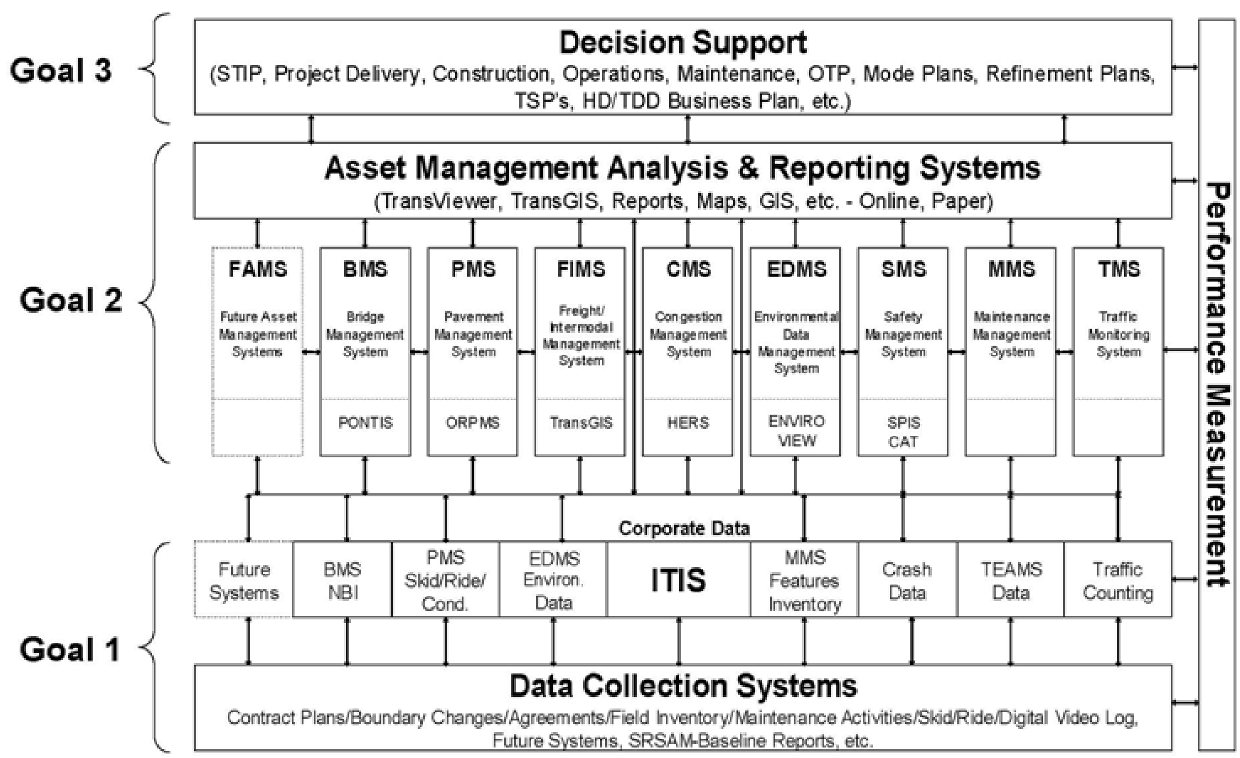

FIGURE 1 ODOT asset management overview (*5*).

what-if analyses. In the bottom-up approach, standards assist in planning projects. Planned projects are totaled to generate costs that are then compared to budgets. This is used to adjust the standards and modify the plan (*7*). Figure 2 shows a comparison of the two approaches.

In 1995, the National Highway System Designation Act (Public Law 104-59) officially repealed the legislative requirements for BMS implementation. Small et al. reported that all but three states intended to implement the systems anyway. Summarizing a 2006 survey, researcher Paul D. Thompson reported that 26 states are currently using Pontis for decision support for one or more business processes. An additional five are actively developing deterioration models, cost models, or other inputs so that they will be able to use the decision support features in the future (Paul D. Thompson, unpublished data).

Thompson indicates that "Forty states currently consider themselves to be Pontis states, and four more are evaluating Pontis." That said, the survey results summarized indicate that less than 10 states are using Pontis for project scheduling, priority setting, and budgeting. Why is it taking so long to fully implement Pontis? While reasons vary by state and by the level of desire, Oregon has been earnestly working on fully implementing the analytical capabilities of Pontis since filling a full-time engineering position for this purpose in 2005.

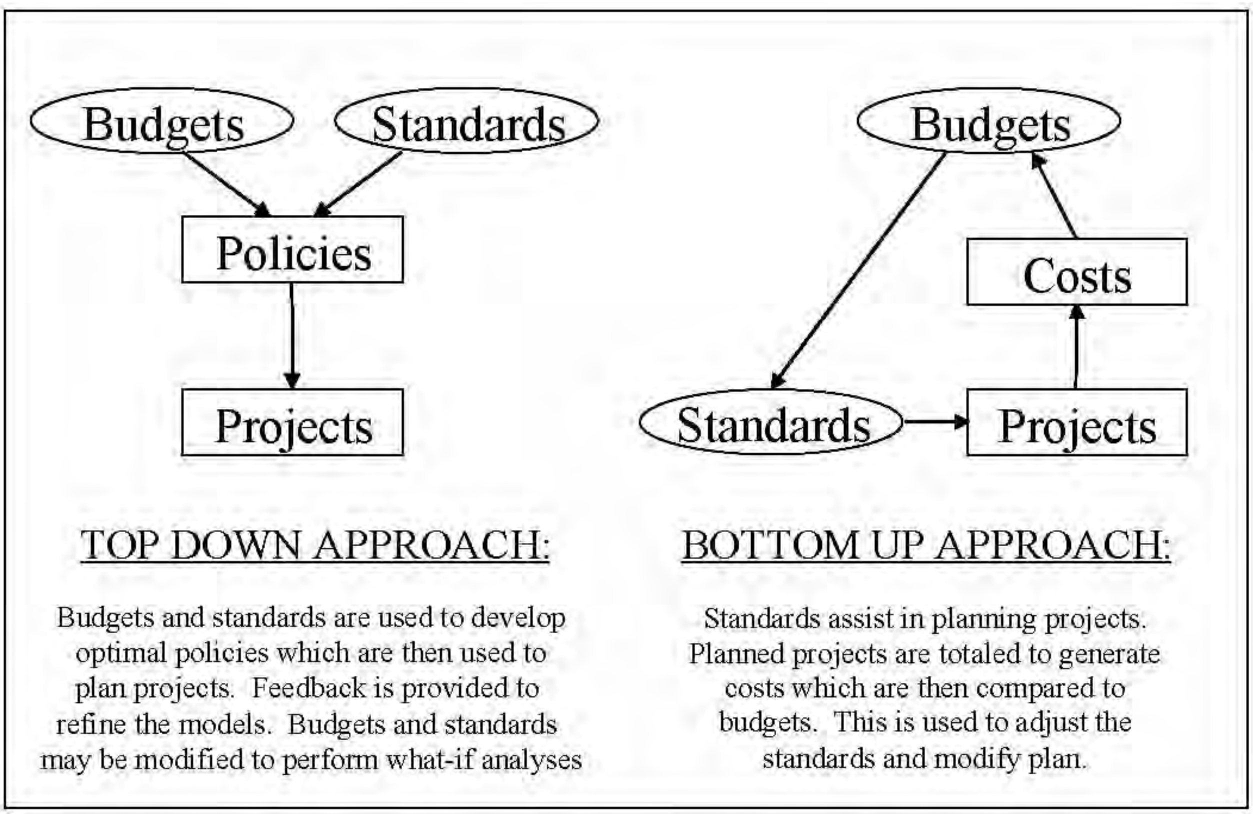

FIGURE 2 Alternative BMS development philosophies (*7*).

Pontis Implementation: Recent Oregon Experience

Following the guidance of the Pontis User Guide, ODOT conducted a deterioration expert elicitation and attempted to conduct one for costs. Several difficulties arose. We recount our experience here as a possible advisory to others that may yet be considering Pontis or have not reached this stage of implementation.

General

The nature of the elicitation process is "best guess" based on experience. The elements included in the elicitation process are items that have a repair frequency ranging from years to decades. Ideally, the person most able to perform an elicitation would have many decades of experience with a singular set of bridges. Results are as good as your guesses, and without such a knowledgeable resource, the guesses may not be very good. Even if ODOT had not undergone a major reorganization and lost many of its knowledgeable bridge experts, it is unlikely that resources available would be able to provide the needed information. With the loss of many experts, ODOT's institutional knowledge was impaired. In spite of the limitations, for the reasons listed below, we believe expert elicitations are the preferred alternative.

Deterioration

Insufficient past data on actual deterioration rates have been recorded in Pontis to be a useful aid. Pontis does have an option to use historical data to create deterioration models. However, to be really useful, a long set of inspection records from many years are needed for each element in each environment and condition state. Our experience has been that it is a rare element–environment–condition state combination for which we have enough observations to be useful, and for no element do we have enough data for a complete set of element–environment–condition state observations.

Many of the bridges in Oregon's inventory are relatively old and as such can be considered the survivors, atypical as opposed to average bridges. Developing deterioration rates based on atypical bridges will result in a less than useful model for average bridges.

Insufficient bridge repair history is available in ODOT in Pontis. We have not begun using Pontis to track repairs through work candidates and projects, so Pontis cannot tell when repairs have been done and when the "clock" should be reset on a particular element. "Invisible to Pontis" repairs result in unrealistically long element lives.

The absence of good repair records provides an additional complication. Without knowing what repair has been done to a bridge and when the repair was completed, there is no way of knowing if an element in good condition on an old bridge is due to a low deterioration rate or the result of a past repair.

Costs

1. Our initial cost expert elicitations were conducted in 2000, prior to the reorganization. In 2006, we conducted a second expert elicitation intended to review and evaluate inflation on the first results. Unfortunately, the first cost expert elicitation was not well documented and

could not be defended by members of the second group. In order to proceed we found it necessary to start over by addressing the following:

a. Define "typical" elements. As there is considerable variation in element size, and in some cases, configuration, we decided to adopt a particular size and configuration for each element that would be used to establish costs for that element. We are currently evaluating our inventory for each element and selecting that closest to average.

b. Define actions. Pontis uses feasible actions as the repair actions it can perform in a simulation. ODOT uses the feasible actions set out in the AASHTO Guide for Commonly Recognized (CoRe) Structural Elements. These feasible actions are short phrases generally describing a repair and are not detailed. To set up a cost, one has to intrepret what "Rehabilitate the Deck" means, for example. Such an action could refer to a variety of repairs. A "typical" repair is selected to represent the Rehabilitate the Deck action. Once that is done, the repair must be broken down into subactions for which costs can be determined. We are currently selecting "typical repairs" and detailing sub-actions for maintenance, rehabilitation, and repair (MR&R) actions. As there really is no typical repair, there are additional questions to answer regarding the inclusion of fixed costs associated with repair actions such as work platforms and temporary structures or bents. The result will be a thorough documentation of the assumptions of the expert elicitation.

c. Define costs. Once the subactions are defined, it is our plan to use R.S. Means data to determine costs. We do not anticipate that costs for our sub-actions will be readily available, so we plan to build them using items included in R.S. Means. We have tested this, and although difficult, it does appear to be possible. We are resorting to the use of R.S. Means data for the reasons outlined below.

2. Cost data for the element-level MR&R actions was nonexistent at ODOT.

3. The Bridge Section has collected construction cost data, based on bid items and quantities included in construction contracts. These bid items have no direct linkage to MR&R action costs, and an additional complication is the nature of the competitive bid process itself. Contractors can underbid the costs of one item by burying these costs in another, if they feel that this will give them an advantage. They are not consistent in this, so bid costs for any one item can vary by large amounts while there may not be much difference in the bid cost for the overall project. With a relatively small number of bridge projects bid each year, the possibility of skewing of average cost data is sizable.

4. Lastly, the tracking of bridge maintenance expenditures in the ODOT accounting system is both unrelated and unfriendly to extracting the actual data needed on an ongoing basis. Future planned replacement of the ODOT accounting system should provide an opportunity to correct this situation.

By January 2007, ODOT was seeking outside assistance with Pontis implementation. The services of Cambridge Systematics were procured to provide guidance to ODOT through a series of work sessions that would

- Finalize the preservation models,
- Update the improvement models and develop Pontis scenarios, and
- Develop procedures for incorporating Pontis into ODOT's bridge management practices.

Prior to the first working session, ODOT had completed the Pontis MR&R models. Unit cost information was believed to be suitable for modeling and wasn't changed. However, the cost data had not yet met the "red-face" test of whether it reflects something close enough to reality to be used to develop high-level needs cost data. It became obvious that our costs were not well documented and while most met the "that looks about right" test, those that did not could not be defended. We did not know what exactly we meant by any given feasible action and we really couldn't say just what the size of the element was that we used as a model to determine the costs. In other words, our documentation was practically nonexistent. We were shown methods to modify and test transition probabilities so that the MR&R models and simulations would return results more in line with ODOT practice. Although we knew we needed a lot of work on our costs, we elected to schedule the second working session.

Most of work session 2 was devoted to developing and testing initial simulation scenarios. We were successful in testing several simulation rules and running simulations to identify improvement recommendations. We had discovered a serious data problem that prevented the system from processing improvement recommendations for all but a few bridges. We brought this up in the session and part of the session was dedicated to tracking it down. Following the work session, the source of the problem was identified and corrected. ODOT has since been able to run successful improvement simulations. At this point, we can get results, but it is still too soon to know if they are results we can trust. We decided that it was important to get our cost and deterioration models set up the way we wanted prior to scheduling the third session.

Currently we are engaged in revising our cost models and, as described above, discovering just how big a job that can be.

Simulation

In our trial run scenarios performed prior to the work sessions, we noted that very little work was recommended by Pontis. Partly this was a data problem that made improvements impossible and partly it was the Pontis MR&R model. Pontis runs two models, the MR&R model and a simulation model. The MR&R model is run independently from the simulation model. The MR&R model allowed our bridges to deteriorate to a degree with which we were uncomfortable. This is a complaint heard from other states, but no written discussion on the subject has been found. AASHTO has modified Pontis by adding agency rules to address this issue. Agency rules can be defined to override the MR&R model if it is necessary to control the relationship between condition states and repairs.

The MR&R model uses the cost and deterioration elicitations to determine a least cost solution as to which action (including do nothing) should be performed in each condition state for each element and environment. It then selects that least cost solution as the action that will be used in a given condition state for a given element every time the element enters that condition state. This will be applied to every occurrence of the element on every bridge in the inventory. It is run prior to a simulation and so is independent of the simulation.

The simulation model uses the deterioration elicitations to determine the condition state of each element in each year of the simulation and uses the results of the MR&R model to determine what action to recommend for each element depending on its condition state. For each year of the simulation Pontis calculates cost–benefit ratios for all improvement actions it recommends, ranks all projects according to these ratios and assigns work based on the rank and the budget for that year.

To achieve a Pontis simulation that models agency policy, the MR&R model has to select the action the agency prefers in each condition state and the simulation must recommend a repair after an interval that agrees with the agency's experience. Therefore, the modeler has two tasks. First, the MR&R model must be adjusted until it reflects agency practice. This is done by assuming the cost elicitations are correct and adjusting the deterioration elicitations for each element until the MR&R model mirrors agency practice. Second, the simulation model must be adjusted until it reflects agency experience. To accomplish this, a simulation is run for each element, one at a time. A single bridge is selected on which the target element exists and is 100% in condition state 1. The simulation is set up so that only that bridge and that element are modeled. The simulation is run with sufficient budget so all recommended repairs are made and the time to the first repair is noted. If the time to the first repair is not consistent with agency experience, the time period to the repair is adjusted by changing the deterioration elicitation.

There will be instances when, while performing the iterative process to adjust the MR&R model and simulation model to conform with agency policy and experience, it will not be possible to arrive at deterioration probabilities for an element where both the MR&R model is selecting the desired action and the simulation is predicting the time period to repair consistent with agency experience. In this case, agency policy rules will be needed to cause the desired action to be recommended in the target condition state after the expected time interval.

We remain optimistic that after the deterioration elicitation for all (nearly 150) elements has been adjusted and any necessary rules written, Pontis will model MR&R repairs correctly. A similar process to adjust the improvement model (strengthen, widen, raise) will also be needed. However, we do not anticipate that adjusting the improvement model will be as complex or time consuming.

An additional limitation of the model is bridge replacement. In general, Pontis will only replace a bridge for functional reasons, although it will allow a bridge replacement if the combined cost of recommended MR&R actions is over an agency set percentage of bridge replacement costs. It does not replace bridges as a direct result of the age and condition of a bridge (in relation to its age). In other words, the model implies that bridges can be maintained indefinitely and do not have a life expectancy that should only be exceeded cautiously in the interest of public safety. This is an open question among structural engineers and is unlikely to be resolved in the near future. Currently in Oregon, most bridges are replaced for a combination of condition, age, and functional reasons.

Creative methods of addressing other bridge needs such as scour, seismic retrofitting, substandard bridge rail, insufficient permit load capacity, or collision damage will need to be developed. Likewise, establishing project timing in selected corridors to reduce traffic control, detour, and mobilization costs will require some experimentation with selective screening, scenario simulation, post-simulation processing or other means to obtain the desired results. Ensuring that the user costs and benefits calculations in Pontis make use of the best information available will also be as critical as good project cost information in ensuring that Pontis recommends expenditures correctly.

CONCLUSION

Pontis is precisely the kind of software that can assist ODOT in its cross-asset allocation aspirations. System health over time is a network-level issue that Pontis excels at. What are important for these analytical purposes are expenditures versus improvement in performance measures. It is necessary that those who want to use Pontis results understand what the results mean. This brings up important issues. First, we must have the right measures and use data to measure them that are accurate and quantifiable—if we want to use measures to drive investment. The second is the implication that exactly which bridge is fixed is not as important as changes in the overall health of the system.

Asset management and selection of projects for the STIP are not the same thing. Asset management can be looked upon as strategy and STIP project selection as tactics. In using Pontis for asset management, we want to answer questions of how the system will change in certain time periods at a given funding level or how much would need to be budgeted to achieve a target performance level. For such an application, it is more important to know the health of the system than which bridge will need a particular action in a future year.

On the other hand, project selection for the STIP is very much concerned with exactly what work is needed on which bridges, although typically within a shorter time frame. Once we have the models set up to mimic what we actually do, Pontis should be able to do a good job of predicting the condition of specific bridges over the STIP time interval. Pontis (once we have it working right and with any necessary postsimulation processing) ought to give us a list of MR&R and improvement projects that mimic our policies and rank them by cost–benefit ratio.

In answering the question as to why Pontis is apparently not being used to its full potential by most users, we conclude the following:

- The models are difficult and time-consuming to set up.
- Information needed to set up the models can be difficult to find.
- There is no detailed guidance on methods available to perform expert elicitations.
- Pontis is complex. Staffing commitment must be full-time, consistent, and long-term.
- The Pontis MR&R model is inflexible in the sense that it cannot easily be set up to select a particular repair when an element reaches a given condition state. As a consequence it requires more work and some creativity to be adapted to specific agency needs.
- On the other hand, Pontis is, if anything, too flexible. There are a lot of decisions and wide latitude in choices to be made. Those looking to use the software right “out of the box” will find Pontis frustrating. Even agencies that believe they have well-articulated policies and good historical data may find the Pontis software challenging to fully implement.
- Using Pontis to achieve the goals of both asset management and STIP project selection may lead to unexpected and challenging conflicts in the absence of agreement on performance measurement and the suitability of the measurement data for its intended purpose.

As for the Oregon-specific answer, it appears that:

- We have underestimated the amount of work and time it would take to obtain usable results from Pontis.
- We have suffered from the absence of a full-time, consistent, and long-term staffing and management commitment to implementation.
- We lack agreement on performance measures and the data needed to measure them. Specifically, we have been unable to identify and agree to one or more measures to capture the comprehensive set of reasons that we undertake to repair or replace bridges. Without such

agreement, we will find ourselves using Pontis to budget for bridge repair and replacement based on one performance measure and selecting bridge projects based on others.

- We find the best use of funding based on engineering judgment may not directly influence performance measures. One example of this is replacing deteriorated timber bridges that are not deficient. Also, there are condition-based needs that are not based on deterioration that must be addressed. An example of this is concrete decks on new bridges that are severely cracked due to construction problems. Addressing these needs will not help performance measures, but sealing the decks to prevent corrosion is an excellent use of funds.
- We are unwilling to trust long-range planning or budgeting results at the network level until we can verify that we can use Pontis to correctly identify the highest priority needs from the back log for programming of projects in the 5- to 6-year time frame. We remain hopeful that given enough time and persistence we can prove this to ourselves.

ACKNOWLEDGMENTS

The authors acknowledge all Pontis users attempting to improve both the quality of the quantitative data and the analytical software in service of systematic decision-making processes to improve the condition of the nation's bridge inventory.

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Use of Bridge Management for Agency Decisions in Planning, Programming, and Performance Tracking

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Most state departments of transportation employ a bridge management system (BMS) to assist in monitoring bridge condition and performance and identifying needs for work. Some agencies also use a BMS for more advanced or specialized applications. The objective of this study has been to gather information on how bridge management information is used for agency decision making on bridge investments and resource allocation, considering processes such as planning, programming, budgeting, and setting and tracking performance objectives and targets. While bridge management information is used in resource allocation decisions in all transportation agencies that were included in this study, the specific practices, analytic capabilities, and performance measures that are used differ, in some cases significantly. Some agencies apply bridge management techniques that are at the state of the art, but the majority appear to use only a subset of existing BMS capabilities. Nonetheless, the agencies that were studied appeared to have tailored their bridge-related practices and information to their agency's philosophy of bridge management, as well as to the financial, technical, and managerial environment in which planning, programming, and resource allocation take place. A number of agencies are working to advance their bridge management practices through BMS customization, e.g., in the definition of state-specific measures of condition and performance. Two trends that may influence bridge management practice in the future are the growing application of asset management principles and methods, and a U.S. Department of Transportation review of the National Bridge Inspection Standards that is now under way.

Bridges are critical and highly visible components of a transportation system. Designing, building, maintaining, repairing, and replacing bridges involve significant investment decisions for agencies due to the high cost of these investments, the need to sustain an appropriate level of investment throughout the considerable life cycle of a bridge, and the important structural, functional, and safety implications of the selected investments. Agencies therefore try to get these investments right, both to minimize life-cycle cost and to provide safe and efficient mobility to transportation system users. As subsequent sections of this paper will show, however, agencies differ in how they go about these investment decisions and on what information they are based. NCHRP Synthesis Topic 37-07, Use of Bridge Management for Transportation Agency Decision-Making, had as its objective to document how bridge management—its processes, analytic tools, and information—meets the needs of upper management in their planning, programming, budgeting, and resource allocation decisions. This paper summarizes the key findings to date of this study.

BRIDGE MANAGEMENT PROCESS DEVELOPMENT

Bridge management in the United States has advanced considerably in the past 40 years, with significant accomplishments at the federal and state levels. The National Bridge Inspection Standards (NBIS) which were implemented in the 1970s, established a single, unified method of collecting data on the nation's public highway bridges (*1*). The NBIS enabled the FHWA and state departments of transportation (DOTs) to monitor bridge condition and performance nationally on a consistent basis, identify bridge needs, define criteria of project eligibility for federal bridge funding, and thereby promote the public safety through better stewardship of bridge assets. Bridge structural deficiency (SD) and functional obsolescence (FO), two ratings that the NBIS defined, became key performance measures that agencies continue to monitor today. Similarly, the bridge sufficiency rating (SR) is embodied in the eligibility formula for federal bridge funding. While some revisions to NBIS have occurred, the definition and application of these bridge ratings have remained essentially unchanged for more than 30 years.

While the NBI database and the computed SD, FO, and SR ratings have provided current and comprehensive data on bridge status and investment needs during the past 35 years, the deficiency and sufficiency ratings are recognized to have several shortcomings:

- The SD and FO ratings are coarse, i.e., while they signal a potential problem, they do not distinguish between single versus multiple causes or their possible impacts.
- The weights and formulas used to compute SR are fixed and may be arbitrary as bridge designs, use of different construction materials, vehicle loads, preferred bridge investment strategies, and other factors continue to evolve.
- The SD and SR ratings are somewhat inconsistent with respect to bridge decks. While SD directly reflects a deck condition that is poor or worse, the SR is much less sensitive. Since SR is one of the criteria of project eligibility for federal highway bridge program funding, this inconsistency can create problems in financing needed deck repairs, even when decks are rated SD.
- The SD, FO, and SR ratings are reactive: i.e., they do not signal a bridge problem until it has already occurred. Moreover, they do not show an improved bridge condition unless corrective or remedial work is done. They are therefore unsuited to more economical preventive maintenance strategies that could prevent or forestall bridge damage before it occurs.
- There is no generally accepted and used set of predictive models for SD, FO, and SR. (The models that have been developed to date are for specific agencies or purposes as discussed below and are not in general use.) Lacking such deterioration or performance models, agencies cannot forecast trends in deterioration of SD, FO, and SR. Such trends, if they were available, could be used to predict needs, quantify the benefits of bridge investments, analyze different scenarios regarding infrastructure policy, performance, and cost, and assess trade-offs.

In addition to their value in recording data from biennial bridge inspections and reflecting bridge condition and appraisal, the NBI ratings play a significant role in apportioning federal bridge program funds to states:

> Utilization of the NBI as the primary data source for the disbursement of funds through HBRRP [Highway Bridge Replacement and Rehabilitation Program, now the Highway Bridge Program] and the Special Bridge Program has been the basis for bridge management decision making since the early 1970s. (*2*)

The NBI measures are also general, which is both their strength and their weakness as a management tool. Because the NBI condition/performance ratings are aggregate measures, they are sufficiently general to apply to the many combinations of bridge designs, materials, traffic loadings, and geographic locations throughout the country. Their weakness, however, is the difficulty of developing a general set of predictive models that could apply to the many different bridge configurations nationwide.

> This form of bridge management [based on NBI data] utilizes aggregated information and thus has limited applicability for analytical decision making. While the formula is convenient for funds allocation, it is not necessarily sufficient for analysis and needs prediction.… A new form of bridge management decision support to facilitate budgeting, policy analysis and project-programming [came to be] desired. (*2*)

A new generation of bridge management tools, which began to be developed in the 1980s and early 1990s, overcame these limitations. They represented significant advances in bridge management at the national and state levels since the implementation of the NBIS. Today all state DOTs have a bridge management process. Most also employ some type of automated BMS with an associated database of bridge-related information. This database typically includes NBIS data and ratings, but often incorporates additional, more detailed, or customized data.

The characteristics and capabilities of the bridge management systems that have been implemented to date may vary considerably. Some bridge management systems focus solely on database management, e.g., input, quality checking, and processing of bridge data, and production of reports. For example, the Alabama Bridge Information Management System (ABIMS) provides a series of bridge inventory and inspection menus by which users may input bridge descriptive information and inspection data. The ABIMS database is a repository of descriptive information on bridge structural characteristics, traffic loads, geographic and route location, functional class, and age, as well as current and historical records of inspection data. NBI data are included for annual reporting to FHWA, and additional, custom data defined by the agency are likewise included. Another set of menus allows users to specify reports on, for example, bridges due for inspection (for inspection planning), status of maintenance, bridge posting status, the rating history of a bridge structure, a variety of inventory listings, and priority ranking. The criterion for priority ranking may be specified as either the FHWA SR or the state's unique deficiency point calculation (*3*). This example illustrates one measure of value of a BMS: the ability to address unique aspects of bridge operation and customized features of bridge management.

The second example is Pontis, a full-featured BMS in use in more than 40 state DOTs and several other city, county, and international agencies. Pontis was developed for FHWA in 1989 and is now supported through AASHTOWare as a product in AASHTO's BRIDGEWare suite. It is a full-featured BMS that provides a number of capabilities useful in supporting bridge program management and resource allocation (*4*):

- Bridge inventory: establish and maintain an inventory of bridge and culvert information, and exchange data with other agency information systems.
- Managing inspections: schedule bridge inspections, enter or import inspection data, produce Structure, Inventory, and Appraisal (SI&A) and other inspection reports, and produce the NBI files that are required to be submitted to FHWA annually.

- Needs assessment and strategy development: estimate and update bridge element deterioration and treatment cost models based on individual agency experience; develop long-range, network-level policies for both structure preservation and bridge improvement based upon agency standards and guidelines and economic factors including agency and road user costs; assess current and future preservation and improvement needs; and evaluate alternative bridge program investment scenarios based upon predicted structure condition and performance, accounting for the technical, economic, and policy-related factors above.
- Project and program development: develop projects to respond to inspector work recommendations and agency policies and standards; evaluate the impacts of project alternatives on structure performance; rank projects; develop programs of projects subject to budget constraints; and track project status and completion.

Pontis' analytic processes are extensive and address several aspects of bridge management. Their descriptions are contained in current documentation (*4–6*). These advanced capabilities involve prediction models that are used to forecast bridge network condition and performance, actions to repair or restore condition, and costs and benefits to the agency and bridge users. These procedures can be applied in functions such as needs estimates, identification of recommended programs subject to budget constraints, scenario analyses to test options and assumptions, and trade-offs between competing network-level attributes over various time periods. Pontis also has features dedicated to incorporating user-defined additions or revisions within its analytic framework and graphical user display. A number of items may be customized, including the definition and classification of bridge elements, the definition and classification of bridge actions, the cost index that is used, internal formulas for data processing, and organization of the desktop and assignment of user privileges. Features such as data input forms and reports may also be customized to accommodate the analytic revisions.

Pontis also provides a broader selection of standard reports than the first BMS example, reflecting its more extensive features and functionality. The reports are organized by system modules including Bridge Inspection, Bridge Preservation Needs and Projected Work, and Bridge Projects. If a report pertains to a given structure (as opposed, for example, to summaries for a bridge network), the bridges may be selected by district, county, owner agency, custodian agency, function class, National Highway System (NHS) or non-NHS, defined administrative area, defined bridge grouping, or inspector responsibility.

APPLICATIONS OF BRIDGE MANAGEMENT TO DECISION MAKING

Past Studies

Several past studies have looked at topics similar to the subject of this paper: the use of bridge management processes and systems to support agency business processes and decisions. The following points summarize these previous studies to help establish context for the current effort. More detailed discussions of these studies will be presented in the study final report.

- NCHRP Topic 27-09, the results of which were published in 1997 as *NCHRP Synthesis of Highway Practice 243*, investigated the uses of DOT management systems for planning and programming in the post-ISTEA (Intermodal Surface Transportation Efficiency Act of 1991) era. With respect to BMS, the study showed that

- Quantifiable measures of program objectives or performance emphasized technical measures of condition or performance, rather than economic or value-based measures;
- BMS were used by about 70% of responding agencies for developing program goals, identifying projects, and setting priorities, but less frequently for establishing program funding levels and capital-maintenance allocations; and
- Prioritization methods for bridge programs used technical measures of sufficiency or deficiency much more than economic or other types of measures (*7*).

• A 1999 FHWA survey of Pontis BMS users investigated ways in which the BMS was used to support planning and programming processes. Among the findings were the following:

- Of those agencies that had a strategic planning process, roughly two-thirds expressed program goals quantitatively but applied different measures, while the remaining agencies relied on generally understood priorities such as reducing the number of deficient or posted bridges;
- Performance measures that were used concerned primarily bridge condition and structural or functional performance;
- Only four of the 26 surveyed agencies used their BMS for statewide transportation improvement program (STIP) development; and
- Agencies varied in their project-level planning and programming practices, with methods ranging from direct use of the STIP to reliance on the SR, agency-specific prioritization procedures, engineering judgment, and inspectors' recommendations (*2*).

• A review of the status of Pontis implementation in 2002 indicated that

- Half of the Pontis users surveyed applied the system solely to input and manage bridge inspection data, and did not apply the more advanced functions that were available;
- Those that did use the higher-level planning and programming tools in Pontis typically took advantage of only a subset of the available features;
- Approximately 12% of participating agencies reported using the full range of system features; and
- More than 80% of the agencies reported completing moderate to extensive customization of the system (*8*).

• NCHRP Project 20-57 reviewed analytic tools to support asset management. Interviews with 10 agencies, most of which had bridge management systems, indicated the following as of 2002:

- Many of these agencies used their BMS (albeit to varying degrees) to support project prioritization and budget–performance trade-offs within individual program categories;
- Only one agency had attempted cross-program trade-off analyses;
- While many of the agencies used life-cycle cost analyses, most of these applications were for pavement projects or major projects above a certain cost threshold; only one agency reported using economic analyses explicitly for bridges (*9*).

Synthesis Topic 37-07

Current information on how bridge management processes and systems relate to agencies' decision making has been gathered in the Synthesis Topic 37-07 study. Data-gathering methods included structured telephone interviews with chief engineers and bridge managers from 15 state DOTs, and state DOT and Canadian provincial agency responses to a comprehensive survey. The results showed that bridge management information is used in program-level decision making in all transportation agencies that reported in this study. However, the specific management practices, analytic models, and performance measures differ among agencies, in some cases significantly. While a common element among virtually all bridge management processes is information on bridge inventory and condition appraisal (including that required by NBIS among U.S. agencies), BMS approaches vary widely in other management functions, e.g., prediction of future needs, use of economic analyses, and consideration of project or funding options and trade-offs.

Nonetheless, agencies that were addressed in this study appear to have integrated their bridge management procedures and systems well within their individual planning, resource allocation, programming, and budgeting processes. Philosophies of bridge management may contrast across agencies (e.g., centralized versus decentralized decision making; use versus nonuse of prediction models). In each example that was studied, the agency has configured its bridge program management to fit within its organizational, financial, managerial, and technical modes of operation. It also has tailored its internal communications of information, as well as its institutional relationships with other agencies, accordingly.

The role of bridge management in agency functions such as planning, programming, and resource allocation can be better understood when the characteristics of different BMS are considered. BMS vary in analytic capabilities and sophistication, as observed earlier. Full-featured systems operate both at the program level and at the level of individual bridges or projects. Those agencies that have a full-featured BMS thus have the ability to apply higher-end analyses such as project planning, network-level optimization of recommended investment strategies, budget versus performance scenarios, trade-off analyses, and economic analyses of agency and user costs and benefits. However, the actual use of these capabilities is by no means certain. For example, while there are individual agencies that use virtually the full set of Pontis features and might therefore be viewed at the leading edge of bridge management system practitioners, more generally the characteristic use of bridge management for state DOT decision making is as follows:

- BMS results are technical (focusing on bridge condition and performance) rather than economic (e.g., benefit-cost) or social (e.g., impacts on different categories of road users on affected transportation corridors).
- BMS results are for near-term rather than long-term analysis horizons.
- Recommended actions are reactive to current conditions rather than proactive or anticipatory of future conditions.
- Recommended actions focus on a single strategy rather than a comparative analysis of several options or scenarios.
- Calculated costs are solely those attributed to the agency rather than including as well the costs borne by road users.
- Costs are calculated for near-term budgets rather than for the bridge life cycle.

- The BMS functionality that is used entails well-defined, basic management procedures (e.g., data management) rather than higher-level procedures such as predictive models, scenario analyses, trade-off analyses, and economic analyses.

Again, these are general findings across the population of state DOTs that participated in the Topic 37-07 study. They do not necessarily reflect the characteristics and practices of any single agency.

The gap that exists between the state of the art versus the general state of practice of bridge management systems has persisted for more than 10 years. The cumulative effect of studies of BMS implementation and use shows that the capabilities of BMS products are under-utilized. These systems are applied most frequently for tasks such as database management and standard types of analyses. Higher-end applications, such as to evaluate the costs and benefits of different network investment strategies, to evaluate long-term as well as near-term needs, or to apply BMS outputs in budgeting and STIP development, are used by only a relatively small subset of DOTs.

Despite this underutilization, the ability to tailor bridge management practices and system outputs to individual agency needs, and to compensate for gaps and constraints in existing practice, helps to strengthen the relevance of bridge-related information to agency decision making. Several findings that have been developed in the Topic 37-07 research illustrate this point.

- While resource allocation decisions with budget constraints are ideally policy-driven and performance-based, agencies that are faced with rigid funding mechanisms and arbitrary eligibility rules work around these handicaps using methods such as off-the-top bridge funding, program set-asides, or transfers of funding. While these methods are not ideal, they are a practical way of gaining greater immediate flexibility in meeting bridge needs. Long-term, however, they may not send correct signals regarding needed updating of bridge priorities or funding needs.
- Agencies differ in their reliance on bridge management systems for decision making. Those that believe in making full use of the available features of a BMS to inform decisions use tools such as scenario analyses, trade-off analyses, and economic analyses. Their top management fully supports using these methods, and is comfortable in receiving program recommendations on that basis. Other agencies, in which these analytic capabilities either are lacking or are not applied, tend to rely instead on subjective professional judgments and assessments regarding, for example, bridge needs, priorities, and resource allocations. Top-level decision makers use these inputs and may also be involved themselves in making subjective judgments and assessments.
- Regardless of whether their BMS is simple or sophisticated, many agencies have customized their own data and analytic procedures to reflect the particular characteristics of their road network, bridge structures, and traffic volume and composition, as well as their philosophy of bridge management. Among agencies that were interviewed in this study, these customizations are important to ensuring that bridge management information remains relevant to agency decisions across all affected organizational units and levels.
- In particular, customized performance measures such as deficiency-point calculations and custom bridge condition and health indexes in several cases were felt to be critical to advancing state-specific practices technically, managerially, and procedurally. These indicators

were entirely acceptable to upper management and served the bridge office as well as executive informational needs for investment planning, resource allocation, and budgeting. Customized bridge rating indexes were also seen by some agencies as a way to get better guidance on bridge investment needs and benefits, to compensate for what they felt were shortcomings in the SR as a criterion for bridge replacement and rehabilitation.

- Some senior bridge managers have asked their personnel to think beyond BMS outputs and consider broader implications of different bridge investments, e.g., operational impacts and criticality of needs, or political and social impacts.

Examples of how bridge management information is reportedly now applied to particular types of agency processes and decisions are described in the following sections.

Quantifying Bridge Policy Objectives and Performance Targets

The policy objectives and performance targets that agencies use to guide bridge program development include several types of measures, but underlying them are themes that cut across the various practices among agencies, e.g., the widespread use of NBI deficiency ratings, but countered by a desire in many agencies to overcome the limitations of these ratings; the development of customized measures of bridge condition and performance; the preference of many agencies to track progress toward objectives and targets somewhat informally, especially by looking at general trends rather than firm thresholds and schedules of accomplishment; and, where explicit policy objectives and performance targets are not available for strategic guidance, the use of other mechanisms to guide resource allocation (e.g., advisory panel guidance and executive–professional judgment). Moreover, the field is in flux: a number of agencies that were interviewed in the study described new, improved measures of system condition and performance that were under development and could be used to express better their program objectives and performance targets. These new quantities, they felt, would help them to understand better the condition of their bridge inventory, the implied bridge investment needs, and the potential benefits of funding these bridge needs. A caveat noted by even those states that had well-developed approaches to policy guidance and performance measurement, however, was that meeting transportation objectives and performance targets in a consistent manner required a stable, sustained, long-term trend in their program funding.

The use of bridge management systems to quantify performance measures and targets is shown in Figure 1. Almost all respondents indicated that their BMS is used to calculate current bridge condition or performance directly, with relatively little need for additional input from supplementary analyses or professional judgment (first response in Figure 1). Eighty percent of respondents reported obtaining corresponding condition-performance information for particular subsets of the bridge network. For the other options in Figure 1, the BMS was reportedly used by a smaller share of respondents (roughly 60% in each case).

Bridge Management Support of Planning Process

The use of an agency's BMS to support the planning process is shown in Figure 2. More than half of the respondents reported using their BMS for planning-related information in the following areas:

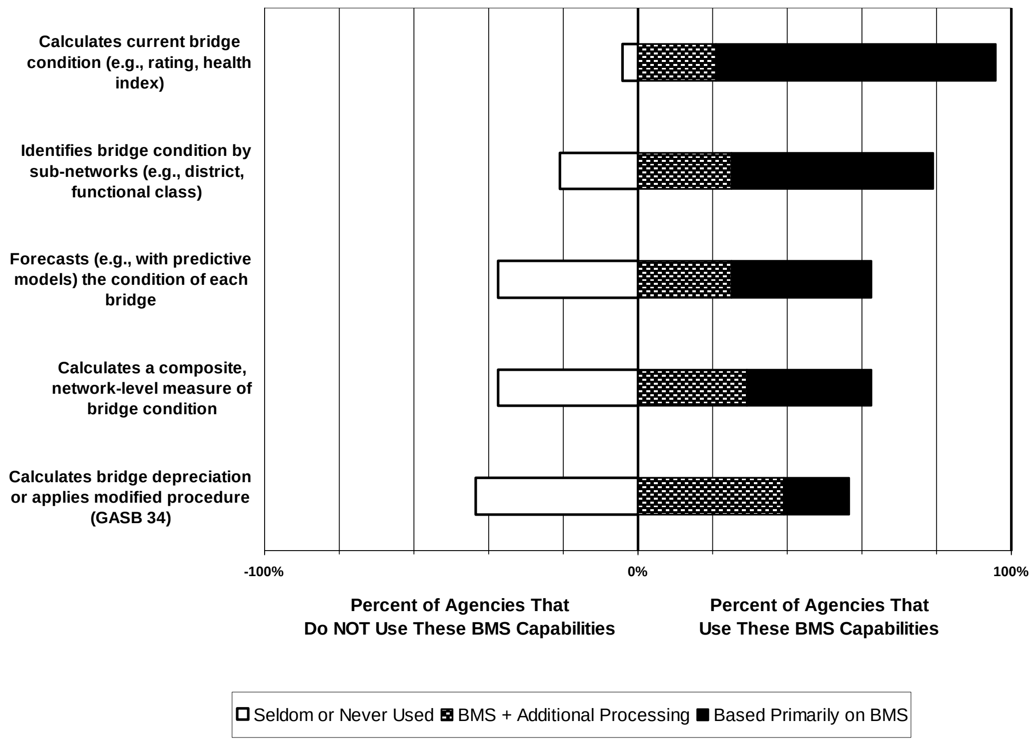

FIGURE 1 Agencies' use of BMS to quantify performance measures.

- The bridge inventory and condition and performance in several categories: structural and functional deficiency; susceptibility to catastrophic damage from scour, fracture critical elements that require attention, and seismic events; other safety problems; measures of statewide and district condition or health; and comparison of performance measures to targets.
- Past and planned work by organizational or geographic unit.
- Reporting in accordance with Governmental Accounting Standards Board (GASB) Statement 34.

About 30% to 40% of respondents reported using their BMS for higher-level management functions in budgeting, scenario testing, trade-off analyses, and generation of quantifiable parameters that could provide guidance in project selection, as well as documenting past and planned bridge projects by political jurisdiction. Fewer than 10 percent of the respondents used their BMS for economic analyses: life-cycle costing or computation of avoidable user costs as a function of alternative budget scenarios.

These results have implications similar to those found in previous studies: a strong use of bridge and other asset management systems to track inventory and asset condition and performance, but lesser use for more advanced tasks in management, budgeting, and predictive analyses. While economic methods are recognized as important techniques in good asset management practice, they reportedly receive little attention in BMS applications to planning.

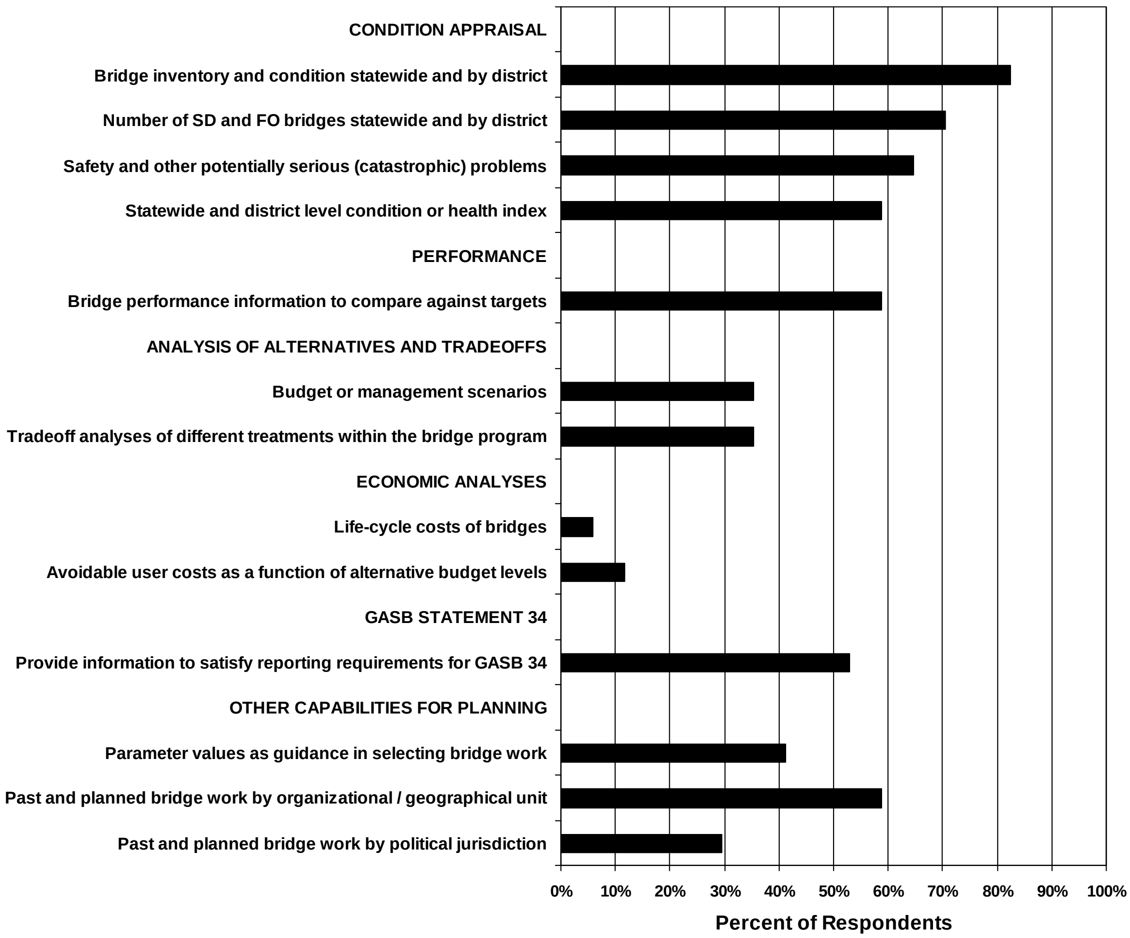

FIGURE 2 BMS support of agencies' planning processes.

Bridge Management Support of Programming and Budgeting

Senior management use of various categories of programming and budgeting information is shown in Figure 3. The greatest reported use is for items that are of immediate interest and most direct and unambiguous in their scope, e.g., a single recommended bridge program budget, estimates of short-term needs for different funding scenarios, and information on major bridge projects. Use of BMS results declines as the focus of this information extends to longer planning horizons, more predictive types of analyses such as trade-offs and impacts of different resource allocations, and various ways of breaking down the information. (It is possible that agencies organize their information differently from the ways suggested in the survey.) Many responses, including those for widely used budgeting capabilities, indicated that additional processing is needed beyond that provided by the BMS before the information is in a form useable by agency executives. Supplementary comments suggested that additional information and analyses may relate to things like district and local priorities, more comprehensive project information, socio-economic and political considerations, and information for other, non-bridge programs such as roadway pavements, safety, and operations.

Bridge Management Support of Resource Allocation and Trade-Offs

BMS applications to resource allocation and trade-off analyses are reported by 60% to 70% of respondents, as shown in Figure 4. In many of these cases the BMS information is supplemented by additional analytic processing or subjective judgments—something that occurs often regarding decisions on funding allocation, according to project interviews. Use of BMS information to produce project-level or network-level summaries of the impacts of different proposed budgets, as might be used by bridge personnel and upper management to justify particular levels of investment, was reported much less frequently and, where it is performed, is rarely accomplished using the BMS alone. The reasons for these results may include one or more of the following: (*a*) preferences by different managers vary on what categories or formats of information to display; (*b*) models and data that are needed to compute these impacts are not now part of the agency's BMS; (*c*) data or analytic models that are needed to calculate the desired impacts may not be available or credible in the opinion of potential users; and (*d*) agency personnel do not feel that predictions of the impacts of different budget levels are needed or useful.

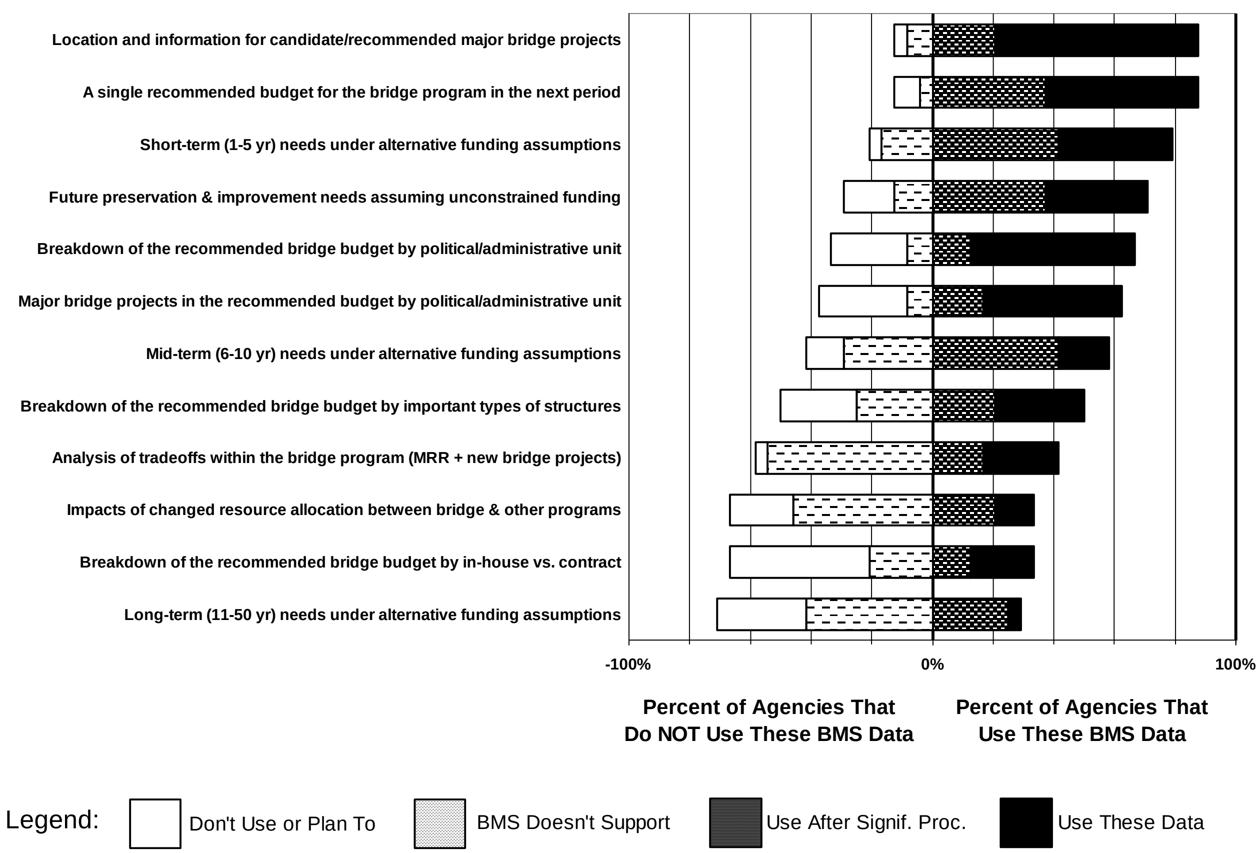

FIGURE 3 Senior manager use of information for programming and budgeting.

POTENTIAL INFLUENCING TRENDS

Two trends that are now under way may influence the way in which bridge management information is applied to agency decision making in the future: (*a*) the growing application of asset management principles and methods; and (*b*) the current review of the NBIS by the U.S. DOT Inspector General.

Growing Application of Asset Management Principles

Asset management encourages a policy-driven and performance-based management approach. It has moved rapidly from its conceptual beginnings to practical implementation. Asset management has already enjoyed successes in agencies' abilities to improve notably transportation system condition, to develop information on infrastructure investment needs and the consequences of different budget scenarios that is reported to legislators and other stakeholders, to thereby increase agency credibility and accountability, and to sharpen the management acumen of agency personnel and the capabilities of supporting information systems. Successful asset management processes have enabled agencies to transition from a worst-first approach to one based upon long-term cost effectiveness, employing life-cycle-cost principles.

Current bridge management practices reflect several characteristics of good asset management practice, e.g., a reliance on a suite of both standard and custom performance measures, a well-defined data structure founded in the NBI database, standardized and

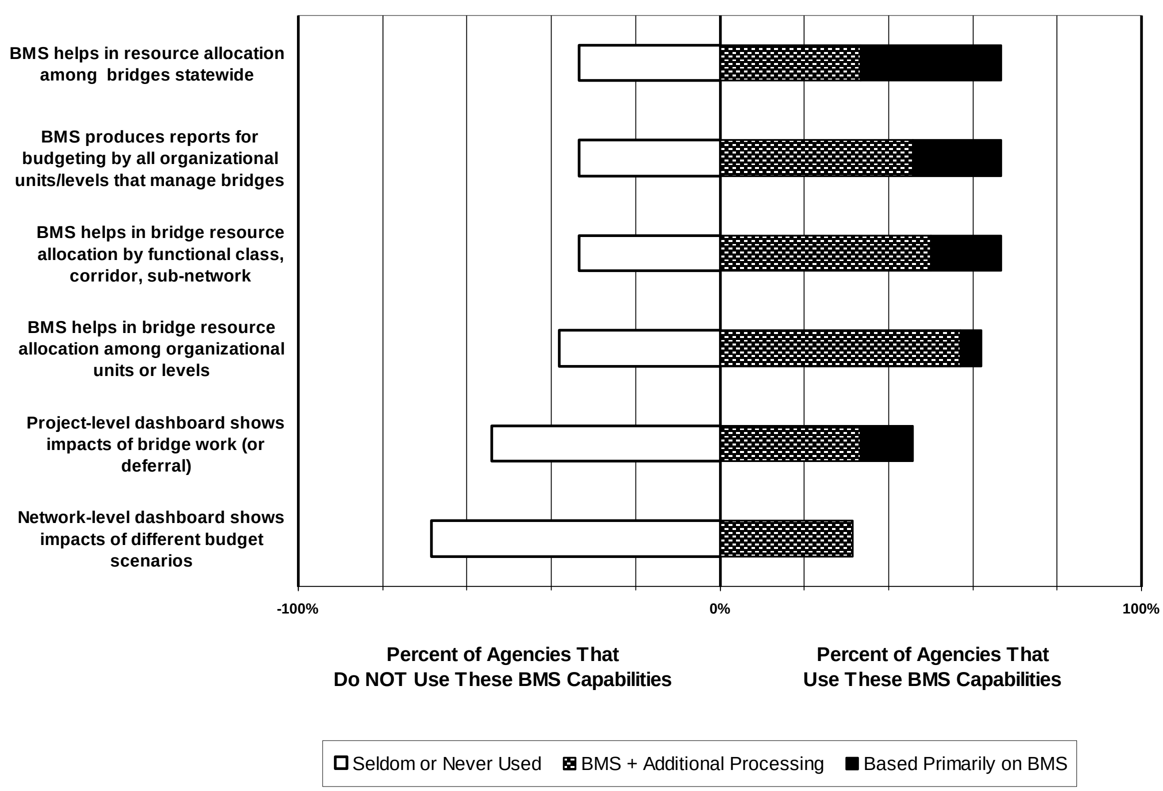

FIGURE 4 Agencies' use of BMS for resource allocation and trade-off analyses.

customized element-level data in many agencies, and a number of management systems and other analytic tools, again with custom features in many cases. Those agencies that apply more advanced features of BMS also are able to take advantage of economic as well as technical data and analyses, scenario and trade-off analyses, and decision-support procedures. These analytic capabilities, and the bridge management business processes that rely on them, almost always typify good asset management practice.

Within this framework, however, some characteristics of current bridge management that were described in Chapters 2 and 3 and do not conform to asset management best practice could be strengthened to improve agency decision making:

- Current policy direction and objectives of the bridge program are often informally stated and loosely tracked. Monitoring of accomplishments is often based on looking at general trends. The process lacks specific time frames and stated levels of accomplishment. Stronger policy statements could provide additional benefits besides greater clarity of an agency's commitment to bridge infrastructure, e.g., they would establish more clearly high-level priorities among competing bridge needs; they could guide definition and use of more precise performance measures; and they could help ensure consistency in the measures and criteria used in planning, programming, resource allocation, and budgeting.
- Several elements of current bridge program management are inconsistent with one another, and detract from more effective agency decisions and uses of available funding. These inconsistencies derive primarily from the different ways of thinking that underlie the formulation of the NBIS versus more recent bridge management concepts and techniques. Specific examples will be given in further conclusions below regarding the current NBIS review.
- The perceptions of bridge management practitioners regarding current BMS models are, when taken together, somewhat contradictory and present a confused picture as to how to advance this aspect of the state of practice. While some survey respondents noted the lack of certain BMS capabilities and suggested research to develop additional types of analyses, other respondents reported using these same features, which are readily available today in BMS products, while still others voiced concern about the "black box syndrome" and the usurping of managers' decision-making prerogatives by such high-level BMS operations.

Implications of the Current NBIS Review

After more than 30 years in service, the NBIS inventory data, ratings, and appraisals continue to be an important influence on perceptions of bridge condition and performance, determination of bridge eligibility for federal HBP funding, and project priority. This situation may change with the complete review of the NBIS by the U.S. DOT Inspector General that was ordered by the Secretary of Transportation immediately following the I-35W bridge collapse. While the causes of the collapse and the ongoing bridge replacement project are outside the scope of this study, selected reactions to the catastrophe and related congressional testimony by state DOT CEOs and other senior managers (some representing AASHTO) are highly relevant to the subject of this paper (*10–13*). The discussion of the NBIS and NBI ratings earlier in the paper provides relevant background information.

- The NBI database serves several important functions. It is unequaled as the most comprehensive, up-to-date, unified source of bridge information nationwide. It has amassed an almost 40-year history of bridge characteristics, condition, and performance. NBIS data are the basis of the bridge portion of the biennial conditions and performance report submitted to Congress, and tabulations of deficiency and sufficiency ratings are widely known and consulted.

- The NBIS was originally established to protect public safety by preserving bridge structural and operational integrity. It was not conceived as a management tool, although it exerts a major influence on bridge investments, federal apportionments, and project funding eligibility.
- Key aspects of the NBIS are inconsistent with the current state of bridge management. State DOTs have recognized this shortcoming of NBIS for some time. While they cannot change statutory and regulatory provisions of how NBI data are collected and applied, agency managers have adapted their bridge management processes and systems to compensate for certain weaknesses, using a variety of approaches, e.g., by increasing the detail and frequency of bridge inspections, defining custom bridge condition and performance measures and indexes, developing custom models to estimate the near-term and longer-term impacts of bridge investments, and applying funds from different sources (e.g., through transfers and resource allocations) essentially to impose more flexible and economically efficient investment criteria than those provided by the SR.
- Announcement of the NBIS review that followed the I-35W bridge collapse has catalyzed proposals to revise ways in which NBI data are used in federal bridge program funding and investment criteria and in public communication. State DOT executives, appearing before U.S. House and Senate committees, outlined several issues with the current NBIS, e.g., inconsistencies between SD and SR that tended to reduce funding priority for deck repairs, arbitrary and nonoptimal funding criteria attached to the SR, and the 10-year rule that inhibits more economically efficient preventive maintenance strategies.
- The recommendation of DOT executives to Congress was to remove (or at least relax) these impediments to better management from federal bridge program administration and allow state DOTs to take advantage of modern bridge management approaches, which could guide bridge investments according to data-driven analyses of long-term performance and costs. Several agencies have developed these newer types of approaches as part of their asset management implementation.
- More general public reaction regarding the NBIS has raised questions about the clarity of the term “structural deficiency” and the difficulty nontechnical audiences have in understanding what this means for bridge condition and public safety (*14*).

The review of the NBIS was just begun as the material for this paper was being prepared, so no definitive information or results were available to include in this paper.

CONCLUSIONS

Most state DOTs employ some type of bridge management system to assist, at a minimum, in monitoring bridge condition and performance, and identifying needs for work. Many agencies also use BMS information for more advanced or specialized management functions. The extent, however, to which BMS processes and information are used in higher-level decision making—e.g., network-level planning and programming, economic evaluation of project alternatives, budgeting and resource allocation, and tracking system performance against targets—varies from one agency to another.

The objective of NCHRP Synthesis Topic 37-07, Use of Bridge Management in Transportation Agency Decision-Making, has been to gather information on current practices that agency CEOs and senior decision makers use to make network-level funding decisions for their bridges, and how they apply their agency's bridge management capabilities to support these decisions. This paper summarizes the key findings of this study. Among the findings are the following conclusions.

Bridge management information is used in resource allocation decision making in all transportation agencies reporting in this study. However, the specific management practices, analytic models, and performance measures differ, in some cases significantly. While a common element among virtually all DOT BMS processes is information on bridge inventory and condition appraisal, BMS approaches vary widely in how they support other management functions, e.g., prediction of future needs, use of economic analyses, and consideration of project or funding options and trade-offs. While some agencies apply these advanced BMS features and capabilities, the overall thrust of BMS applications nationwide is toward relatively straightforward analyses of short-term bridge needs based on current condition and performance, and use of bridge age as a surrogate for longer-term needs estimates. More comprehensive or longer-term analyses are reserved for major bridge projects, or are the purview of those agencies that routinely employ more advanced BMS features involving predictive models, optimization or other decision-support procedures, economic analyses, and trade-off analyses.

While resource allocation decisions with budget constraints are ideally policy-driven and performance-based, agencies with limited BMS capabilities employ other mechanisms such as off-the-top funding (or set-asides) for certain bridge needs. In cases involving simple as well as sophisticated BMS, agencies have elected to customize their data and analytic procedures. These tailored approaches are felt to better reflect the particular characteristics of statewide road networks, bridge structures, and legal-load vehicles, as well as the agency's philosophy and culture of bridge management.

Notwithstanding the variability in practices, agencies reportedly have adapted their bridge management approaches to both internal agency operating practices and external funding and institutional relationships. While bridge management practices reportedly serve upper management acceptably given current procedures for federal bridge program funding and administration, two emerging trends may influence how bridge information and management techniques may inform agency decisions in the future. These trends are (*a*) the growing use of asset management techniques, and (*b*) the review of the NBIS that has been set in motion by the U.S. DOT following the I-35W tragedy in August 2007.

ACKNOWLEDGMENT

The results reported in this paper draw upon material developed in the NCHRP Synthesis Topic 37-07 study. The author wishes to acknowledge the helpful advice and support provided by Jon Williams, NCHRP Manager of Synthesis Studies, the topic panel members, and the agencies that participated in the study through survey responses and interviews. Sincere thanks to all. While the study's factual findings have been summarized as accurately as possible for this paper, the author takes responsibility for the statements that assess their implications and the resulting conclusions.

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Design and Implementation of Bridge Management Systems

KUBA 4.0

The Swiss Road Structure Management System

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KUBA is a comprehensive road structure management system developed for the Swiss Federal Roads Authority. KUBA is similar to other state-of-the-art management systems but has numerous distinctive characteristics. It consists of four components, a road structure inventory (KUBA-DB), a preservation planning tool (KUBA-MS), a reporting tool (KUBA-RP), and a heavyweight transport evaluation tool (KUBA-ST). This paper discusses the two first component components in detail and gives a brief outline of the latter two. Following a presentation of road structures on the Swiss National Road System the paper discusses the overall preservation planning framework including the collection of inspection and performed preservation intervention data, and the application of the deterioration history assessment module for assigning elemental preservation interventions. Finally the paper outlines further development steps and related research.

The annual spending on road infrastructure drains substantial portions of limited public funds in both developed and developing countries. The aging infrastructure on one hand, and increasing traffic on the other, require ever more funds to maintain current levels of service to road users. Under permanent pressure to cut spending, the road authorities must develop ways to increase the efficiency of available resources. Higher efficiency means not only lower agency costs (i.e., taxpayers' money) but also realizing an optimum balance between costs and benefits from available road infrastructure.

In order to cope with these decision-making challenges, the Swiss Federal Roads Office is engaged in the development of a computer-aided system that will enable consistent, unified, and comparable decision processes for road structures on the national highway system. The result of this development is the computer-aided system KUBA (from the German "KUnstBAuten" or road structures).

ROAD STRUCTURES ON NATIONAL HIGHWAY SYSTEM

Due to the Swiss mountainous topography, the road structures form a crucial part of the road network. Unfortunately, there are no reliable data on the number of road structures on the cantonal and community roads. On the national roads there are currently around 6,700 road structures, including ca. 3,500 bridges. The total bridge surface area amounts to ca. 4 million m^2 and to a total length of 263 km. In its final stage the national road network will comprise about 4,000 bridges and around 10,000 other road structures. The total construction costs of these structures will reach CHF 20 billion. The major part of the current road structures has already reached an age of 30 years.

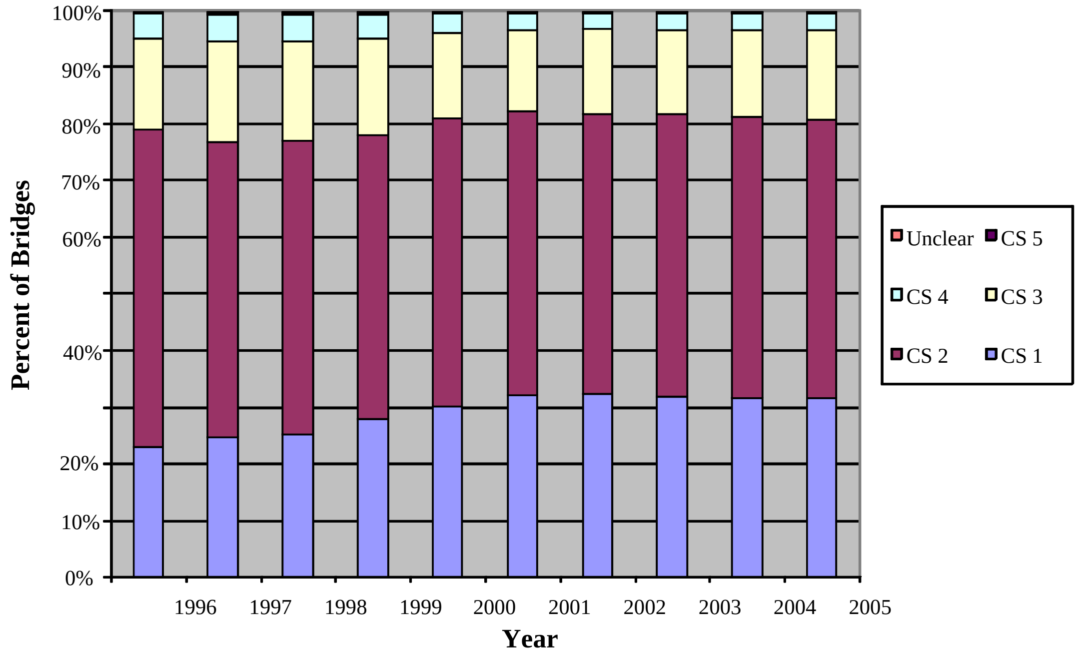

FIGURE 1 Condition states (CS) of road structures on national road network (CS 1 best, CS 5 worst).

The condition state (CS) of road structures on the national road network can be regarded as good. The condition development of bridges based on regular inspections is presented in Figure 1, with CS 1 being the best, and CS 5 the worst state.

Figure 1 only considers bridges that were put in service before December 31, 2006. As such, the beneficiary effect of newly constructed bridges on the overall condition profile has been avoided. The number of bridges in each CS stays practically constant over time and the number of bridges in the worst CS is negligible. This indicates that the current maintenance strategy is sufficiently funded.

The preservation of road structures (excluding tunnels) comes to roughly 43% of overall preservation cost based on a detailed cost analysis performed in 1993. Based on this percentage, one can approximate the development of preservation expenditures for road structures (see Figure 2).

Given that the current replacement value of road structures is estimated at CHF 18 billion, the yearly expenditures are about 1.5% of the replacement value. This percentage is significantly higher than the generally recognized threshold below which no sustainable condition can be achieved (*1*). Therefore, the condition profile of road structures on the national road network is likely to remain stable.

In conclusion, it is safe to say that the road structures on the Swiss national road network are in good condition, and that funding is sufficient. This can change in the near future, as there is an increasing pressure to reduce public funds for transportation infrastructure in general.

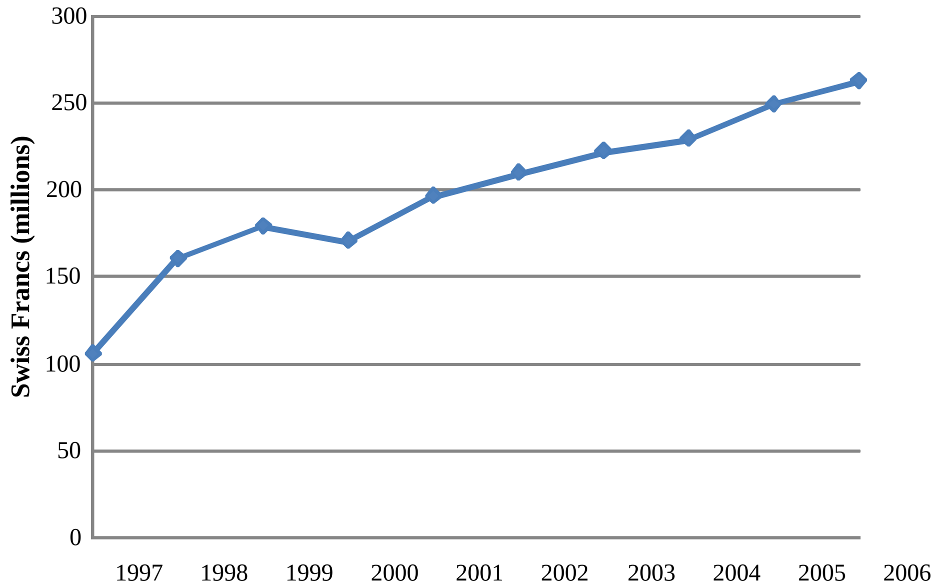

FIGURE 2 Development of preservation costs for road structure (approximate).

ROAD STRUCTURE MANAGEMENT

Objective

The purpose of road infrastructure is to ensure fast and safe passage between any two points on the road network. This is exactly the service that users (consumers) demand from a road and that the road operator should supply. These two entities can be regarded as stakeholders of the road infrastructure, together with other entities affected by roads and traffic. The first two stakeholders are internal stakeholders and the other stakeholders are external stakeholders. Each stakeholder has its legitimate goals, which have to be respected in planning, operating, and maintaining road infrastructure.

The goals of users are already stated; they are interested in fast, safe, and reliable passage between origin and destination. In other words, they are interested in obtaining safer roads and in increasing the average travel speed. The goals of operators are to minimize their expenditures and still maintain traffic capacity. In an ideal market environment the price of these road passages would determine the equilibrium between the users' and operators' goals. However, there is no ideal market for road passages since there are no competing road networks and therefore these opposing goals have to be reconciled in public interest. The external stakeholders also have their objectives, which may oppose each other. Environmentalist groups aim for a reduction of pollution and of the consumption of nonrenewable resources related to traffic, whereas business-friendly groups advocate better connections to their businesses.

The objective of road management is to consider these goals objectively in a decision-making process and identify the optimum course of action.

Goal Quantification and Value System

In the decision-making process the goals mentioned above are compared in order to find the optimum decision. To that end, the achievement level of stakeholders' goals has to be measured, preferably in monetary terms. The value of a fast and safe passage is to be expressed in monetary terms, and the same holds for the expenditures by the operators to supply fast and safe passages. In order to obtain the value of a fast and safe passage one has to look into the public benefit induced by road networks.

An efficient road network has multiple positive effects on the economy, including reducing transportation costs, enabling the concentration of production facilities, fostering spending, inducing new real estate development projects, etc. It is well known from various statistical indicators that new roads induce an increase in gross domestic product (GDP) (*2*). In a recent study commissioned by the Federal Office for Spatial Development and the Federal Roads Office (*3*), the value generated by road traffic is estimated at CHF 46 billion or about 10% of the GDP. This does not mean that by shutting down the road network a 10% decrease of the GDP is to be expected. Although the consequences of this hardly imaginable scenario are rather difficult if not impossible to evaluate, they would in any case be significantly larger than CHF 46 billion per year.

In general, the absolute increase in GDP due to new or improved road links, or correspondingly the GDP decrease due to their closure, is very difficult to quantify. In infrastructure management, marginal changes in GDP are therefore estimated by monetarily quantifying changes in traffic conditions. These changes are usually called user costs (if negative) or user benefits (if positive). Longer travel times, for instance, due to physical road conditions or congestions, are quantified using an hourly rate. Correspondingly, an increase in accident rates is monetarily assessed including both material damage and loss of life and limb. Assessment models for the user costs or benefits with various levels of sophistication are currently available.

Unfavorable changes in traffic conditions can occur either on the supply side, due to natural deterioration, catastrophic events, etc., or on the demand side, e.g., due to traffic increase and load increase.

The goals of external stakeholders are difficult to quantify monetarily although there are some recent efforts in this direction (*4*). It is therefore common to replace an explicit monetary quantification with regulatory constraints that have to be met. Within these constraints one has to find the optimum course of actions, which minimizes the overall costs consisting of operators' expenditures and user costs.

Adopted Approach

The road structure management on the Swiss national road network is based on the fundamentals explained above. However, the following additional requirements were defined:

- The deterioration of a road structure will not be allowed beyond the level at which user costs can be expected due to lower speeds and higher accident rates. The deterioration level is expressed by a condition state, the worst condition state being associated with potential users costs and requiring urgent interventions.

- The user costs during the intervention due to a restricted traffic regime are to be considered.
- Preservation and improvement interventions are to be treated separately. Preservation interventions do not induce any user benefit since the level of service before and after the intervention remains unchanged. The purpose of improvement interventions is to reduce user costs, i.e., induce long-term user benefit (less congestion, higher speeds, heavier trucks).

These requirements define a framework for the development of a methodology for road structure management. The emphasis is set on the decision process for preservation interventions, since these can be algorithmically generated from the CS of the structural elements. This does not apply to improvement interventions, which have to be defined by a qualified engineer.

Condition States

To assess the condition of a structure means to record damages in terms of their extent and severity, and to attribute a predefined condition state to the structure as a whole, or to its structural elements. For road structures on Swiss national roads, the assessment units are structural elements. It is, therefore, necessary to define the elements for each structure prior to first inspection. Elements are primarily classified using a catalogue of element types and a catalogue of so-called "construction types." For example, with a column of reinforced concrete, the column is the element type and reinforced concrete is the construction type. For inspection purposes the elements may be further divided into segments. An element can contain one or more segments, depending on its geometry, size, structural role, and the prevailing environmental conditions. Elements are classified by their element type and construction type, while segments are characterized by their extent and by their exposure to environmental influences. Figure 3 shows the hierarchic levels used in the decomposition of structures.

A condition rating describes the type, severity, and extent of damage. Type of damage is, in this context, a synonym for the deterioration process, which characterizes the appearance and the development of the particular damage. A scale of five CS was adopted, ranging from 1 (good condition, i.e., no damages) to 5 (requiring urgent attention). It is assumed that in CS 1 to 4 the safety and serviceability of the element and hence of the structure are ensured. In CS 5 this may not be the case as previously stated.

During the inspection the inspector has to identify damages and group them based on their visual appearance in damage zones as shown in Figure 4.

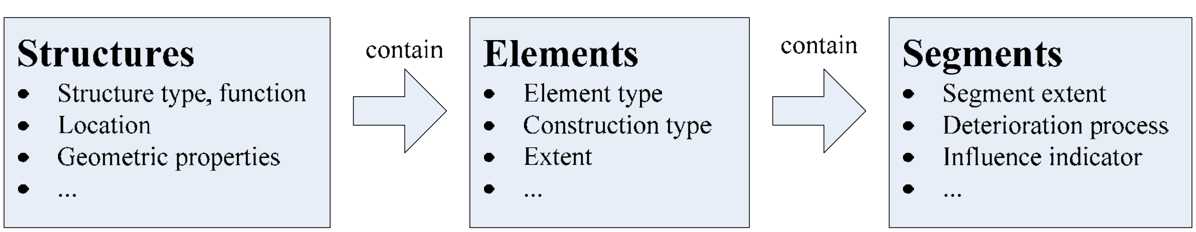

FIGURE 3 Decomposition of a structure for inspection purposes.

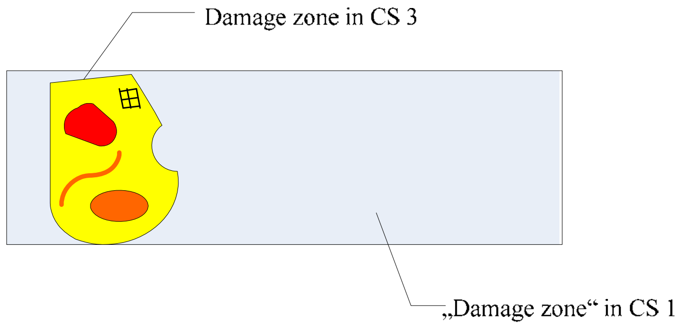

FIGURE 4 Damage zones.

The inspector also has to identify the deterioration process acting in these damage zones and group them into segments. Damage free zones, e.g., ones being in CS 1, are also added to segments if there is a clear indication of the governing deterioration process. Finally the severity data, i.e., CS and extent of each damage zone, are collected. These data are the basis to estimate the CS of the segment, element, and the whole structure.

The inspections are performed roughly every 5 years, although in certain cases more frequent inspection may be required. For newly constructed structures the interval between the inspection "zero" (commissioning) and the first inspection may be set to 10 years.

Condition Forecast

The condition forecast allows defining preservation projects for any given period in the future. The forecast requires a model that simulates the future condition of an element (or segment), given the current condition and the likely environmental influences. For the Swiss national roads Markov chains are used for the condition forecast. Each deterioration process is modeled by its own, characteristic Markov chain. The transition matrices of the Markov chains are calculated and updated by a statistical analysis of the condition data collected during inspections. This is a self-learning feature that results in an increased accuracy of the condition forecasts as time goes by (*5*).

Portions of the same element subjected to the same deterioration process may behave differently. In order to consider these differences, each segment is attributed a so-called "influence indicator." Three influence indicators are used: favorable, average, and unfavorable, which are correlated to the segment having slow, moderate, or fast deterioration. Segments are likely to have a favorable environmental influence if they are not directly exposed to weathering and not contaminated with chemically aggressive substances, i.e., chlorides. An average influence corresponds to normal exposure to weathering and chlorides, and an unfavorable influence to a more aggressive than normal exposure. An unfavorable influence may also correspond to the presence of multiple deterioration processes or to defects due to poor construction or mechanical damages. The effect of the influence indicators is implemented by adding so-called "influence matrices" to the transition matrices of the Markov chains.

Feasible Preservation Interventions

A key part of the management system is its catalog of feasible preservation interventions for each condition state of a given element type. These interventions are characterized by their unit costs and effectiveness. The cost predictions generated by the system depend on these data. The catalog is the result of a statistical analysis of executed preservation projects and is subject to changes reflecting developments in construction technology. The unit costs refer to a specific measurement unit. The unit for preservation work on steel elements, for example, is the square meter (m^2), i.e. the surface area. This unit has to be the same unit used for the condition rating.

A preservation intervention results in a CS improvement, which is represented by transition probabilities reflective of the effectiveness of the intervention. This approach incorporates the empirical knowledge that a preservation intervention often does not restore a segment to CS 1. The transition probabilities representing intervention effectiveness are calculated and updated by a statistical analysis of the condition data collected during inspections before and after interventions (*5*).

Network-Level Planning

Given the condition of an element and the related technically feasible interventions and their costs, the optimum intervention can be identified for the element, without consideration of the structure to which it belongs. In this manner, a single optimum intervention for a given combination of element type, condition state, and influence indicator can be derived.

Optimization on the element level has to be performed for a large number of combinations of element types, deterioration processes, and influence indicators. The result for each combination is an intervention set containing the optimum intervention for each condition state (optimum policy), and the annuity pro element unit to be invested if this policy is pursued (Figure 5). The results of the optimization reduce the number of element interventions that are to be considered for project generation.

In order to explain the optimization procedure, an optimum policy will be assumed according to which, for instance, interventions will be performed in condition state 3. Two possible realizations of this policy are presented in Figure 6. Since the deterioration process is a stochastic process, the interval between interventions is a stochastic variable with the probability density function as presented in Figure 6. The unit cost of intervention is given in Figure 6 as well. The annuities can be calculated when the unit cost is divided with the expected interval between interventions.

Probability density functions of the intervention intervals for the four intervention policies I2 to I5 and the corresponding expected intervention intervals are presented in Figure 7. The annuities for these policies are hyperbolic functions of intervention intervals as presented in Figure 7. The I2 policy assumes a particular intervention type in condition state 2, the I3 policy some other intervention type in condition class 3, etc. For a given intervention policy the corresponding expected intervention interval yields the expected annuities for this intervention policy as shown in Figure 7. From the diagram it is clear that the optimum preservation policy is I4 since it minimizes the expected annuities.

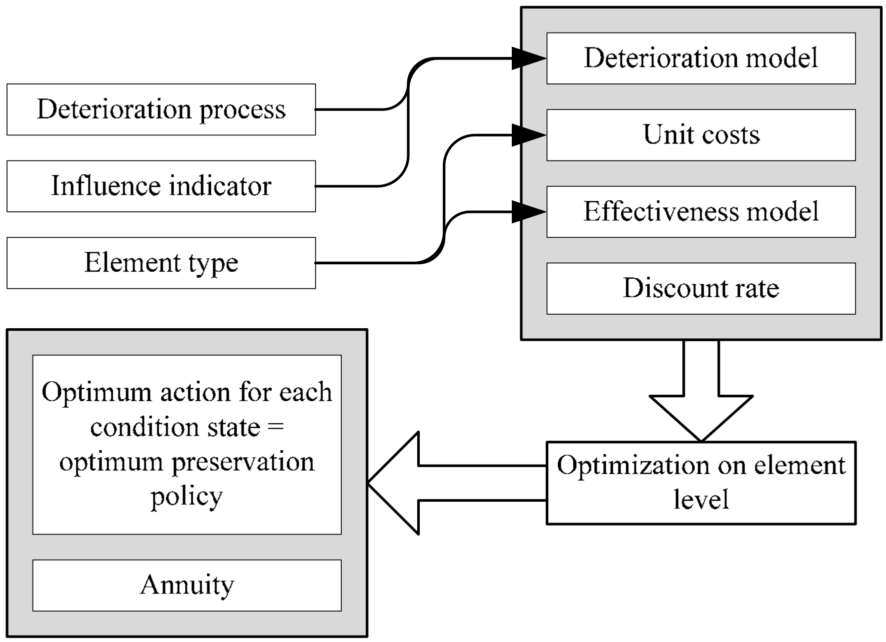

FIGURE 5 Optimization on network level.

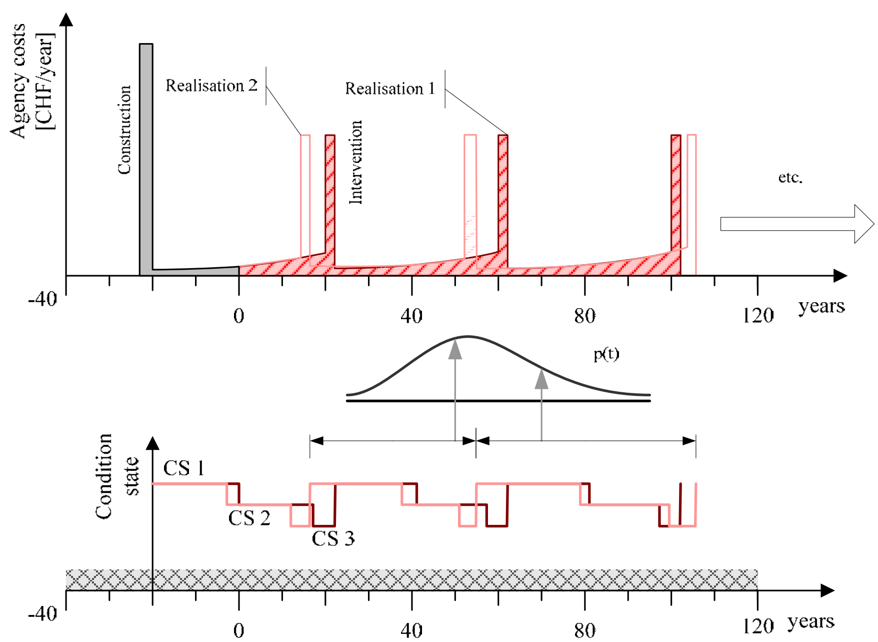

FIGURE 6 Policy realizations.

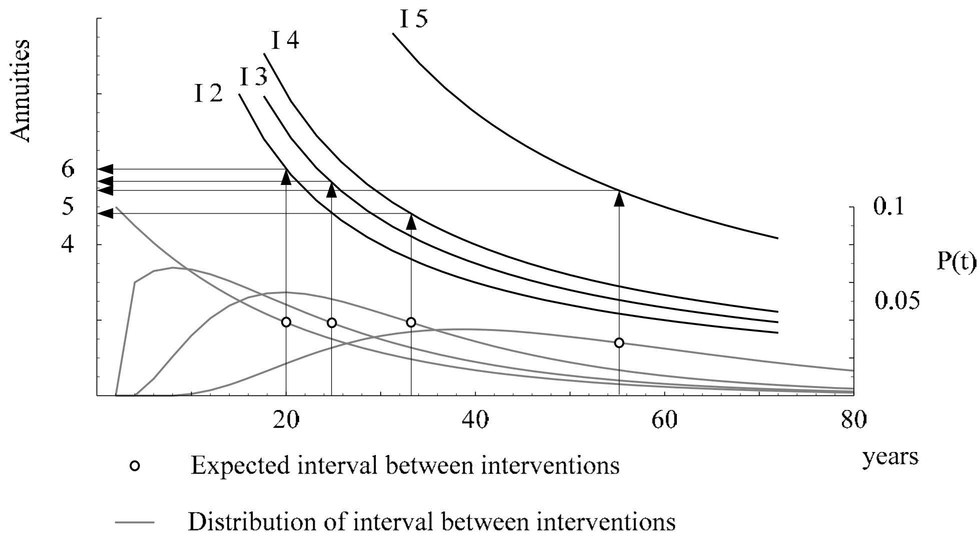

FIGURE 7 Optimum policy.

It can be seen that the optimization procedure is in principle independent of the deterioration function, provided that the probability density functions of the intervention intervals can be derived for each preservation policy.

Project-Level Planning

The generated preservation projects consist of element interventions. A project is generated for a structure as soon as one of its elements requires an intervention according to the chosen element preservation policy. This policy and the condition forecast determine which intervention should be performed for an element, including “do nothing” as an option. The extent of an intervention to be performed and its unit cost are used to calculate the intervention cost. The CS after the intervention is given by the intervention effectiveness. The preservation project generated using the optimum policies for all elements is of particular interest, since it consists of optimum element interventions.

The generation of preservation projects is subject to a difficulty stemming from the stochastic deterioration model. The deterioration model merely yields a probability of an element being in an intervention-triggering CS. It is therefore necessary to establish threshold probabilities to decide which elements must undergo interventions. In practice, however, when deterioration is modeled using Markov chains a significant number of elements affected by a common deterioration process and influence exhibit the same probability of being in the intervention-triggering CS at some point in time. For such cases, a model has been developed that takes into account the former performance of particular elements to decide whether or not they should undergo interventions. This model will be discussed in more detail in the next chapter.

The interventions for all elements (segments) of a structure form a project. The project is characterized by the sum of the costs of interventions. In order to perform a cost–benefit analysis for all projects, annuities have to be computed as the sum of annuities obtained in the optimization on the element level. The benefit of a project is the cost saving compared to the project with the highest annuity. Interventions on certain elements may require traffic restrictions. Examples include the repair or rehabilitation of pavement, a bridge deck, joints, or retaining the walls along a road. The restriction of the traffic results in speed reductions or congestion delays, vehicle detours, and increased accident rates. Based on the duration of the intervention and the hourly rates published (*4*), one can make a rough estimate of the user costs during the intervention. These costs can be transformed into annuities using a discount rate and build a weighted sum with the agency annuities. The weighted sum of the user costs changes the benefit of each project.

Optimization on the project level aims to support the agency in deciding which projects to perform in a planning period. The horizon of the projects is 3 to 10 years. The same procedure applies for both preservation and improvement interventions. The cost data for improvement interventions have to be provided by an experienced engineer.

The optimization on the project level is not needed if sufficient funds are available. This means that for each structure its project with highest benefit (i.e., minimum annuity) enters the intervention program. If, however, the available funds are not sufficient, an optimum set of projects is estimated that fits into the available budget. This problem is known as the "knapsack problem" and is essentially a binary program. For the Swiss national road network, a simple but approximate solution procedure for this binary program called incremental cost–benefit analysis is implemented. The incremental cost–benefit analysis may, strictly taken, yield suboptimal solutions. This happens when a single project consumes a large portion (>30%) of the available budget. This is, however, rarely the case and for practical purposes the incremental cost–benefit analysis yields an optimum project list.

The list of projects selected by the incremental cost–benefit analysis can be used to establish working programs, and allows the optimization of resources. Clearly, some decisions are not motivated by economic criteria, but rather by political reasons. In these cases working programs can be changed manually and the corresponding costs can be estimated.

GENERATION OF PRESERVATION PROJECTS

The Markov transition probabilities can be regarded as relative frequencies of a large number of observed condition state transitions. For instance, the coefficient p_{23} expresses the relative frequency of observed transitions from CS 2 in CS 3. This also means that by the law of large numbers the same relative frequency of transitions can be expected in the future. This means that for a sufficiently large number of elements and their extents one can assume that the total damage extent ${}_iD^t$ of all elements in condition state *i* at time *t* will, in time *t* + 1, be divided in two extents as follows:

$$\begin{aligned} {}_iD^{t+1} &= p_{ii} \bullet {}_iD^t \\ {}_{i+1}D^{t+1} &= p_{ii+1} \bullet {}_iD^t \end{aligned} \tag{1}$$

It is assumed that there are only two neighboring nonzero coefficients in each row of the Markov matrix, i.e., CS cannot be skipped. The splitting given by Equation 1 also applies for a single segment if damage zones are in CS 1. It does not apply, however, for damage zones which are not in CS 1. There is only a probability p_{ii+1} for the whole damage zone to deteriorate further from CS i into CS $i+1$. Consequently, if the chosen preservation policy requires intervention in condition state $i+1$ this intervention will be performed only with a probability of p_{ii+1}. This is clearly not desirable since in reality this intervention is either going to be performed or not be performed. For the project-level planning a deterministic trigger is therefore required that would solve this dilemma. For this purpose an average deterioration rate for each segment is used, which is based on historical data. Consider N damage zones ${}_i d_l^t$ in CS $i \neq 1$, which are ordered by increasing deterioration rate α_l:

$$ {}_i d_1^t, {}_i d_2^t, {}_i d_3^t, \ldots, {}_i d_l^t, \ldots, {}_i d_N^t \qquad \alpha_l \leq \alpha_{l+1} \tag{2} $$

The damage zone is sought that minimizes the following objective function:

$$ \left| \sum_{l=1}^{m} {}_i d_l^t - {}_i D^{t+1} \right| = \min! \tag{3} $$

The value m that minimizes this objective function defines which damage zones stay in CS i in time t+1 and which damage zones are entering condition state i+1:

$$ {}_i d_l^t = {}_i d_l^{t+1} \qquad \forall l \leq m $$
$$ {}_i d_l^t = {}_{i+1} d_l^{t+1} \qquad \forall l > m $$

The above procedure ensures the best fit of the total damage extent ${}_iD^{t+1}$ on the network level by discrete damage extents on the project level. The difference can, however, be significant if an individual damage extent takes a large portion of the overall damage extent. Practical experience with this simple procedure has been quite satisfactory.

CONCEPTUAL ARCHITECTURE

The KUBA system conceptually consists of four components: KUBA-DB, KUBA-MS, KUBA-RP, and KUBA-ST. These components will be explained shortly in this chapter.

KUBA-DB is a data collection system and serves as a basis for the other components in KUBA. It has full sovereignty over the inventory, inspection, and intervention data. It enables user-friendly collection of inventory, preservation, and inspection (see Figure 8) data. The screen shot in Figure 8 shows data collection related to damage zones as explained in the previous section.

KUBA-DB also stores information on people and organizations involved with construction, inspection, and maintenance, and documents related to construction, inspection, and maintenance of structures.

The component KUBA-MS (Management System) is the kernel of KUBA. Although KUBA-DB has significant value by itself by providing information on existing structures, the main benefit for authorities stems from KUBA-MS. The purpose of KUBA-MS is to enable a

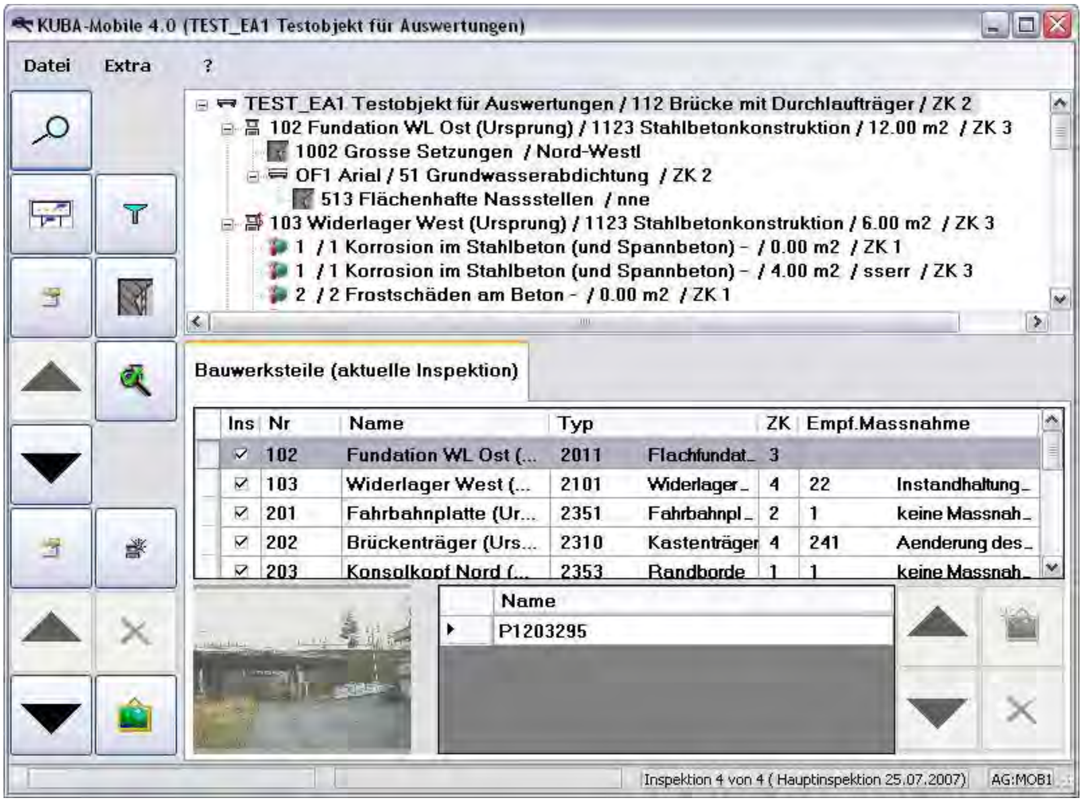

FIGURE 8 Collection of inspection data in KUBA.

consistent, unified, and objective decision processes for road structures on the Swiss national road system. The approach explained in the previous section is implemented in KUBA-MS.

The component KUBA-RP (Reporting) enables extensive querying and reporting of all data in KUBA.

The component KUBA-ST (Special Transport) assists authorities in the decision whether to clear special transports to pass over a single bridge or over several bridges along a predefined route. It is based on the comparative analysis of internal forces (bending moments and shear forces) resulting from code of practice loads and special transport loads. For this purpose the bridge structure has to be simplified to a sequence of simple supported beams.

CURRENT STATE OF DEVELOPMENT AND FORTHCOMING STEPS

The system KUBA 4.0 was officially released on December 4, 2007. Its predecessor KUBA 3.1 has already been in use in all Swiss cantons since 2001.

The concept for the next version KUBA 5.0 is already finished. Its realization is to start in April 2008, and will be finished in April 2010. It will include two new features: geographic information system (GIS) presentation with linear referencing of road structures, and the capability to collect tunnel data. It will also have an interface to the core system of the overarching Swiss Road Management and Information System MISTRA, which will allow a regular data exchange between KUBA and the core system.

A feature that is not planned for KUBA 5.0 but could be implemented in KUBA 5.x is the representation of road structures in 3-D.

GIS Representation and Linear Referencing

KUBA 4.0 has only rudimentary GIS support and this feature will be significantly enhanced. Within KUBA 5.0 it will be possible to

- Draft or import an outline of a road structure and
- Superimpose an outline in an aerial photo, raster, or vector cartographic material (see Figure 9).

The outline of the road structure can be defined in planar coordinates (latitude and longitude) or in linear road axis-related coordinates.

Tunnels

In KUBA 5.0, comprehensive support for tunnels will be implemented. This will allow the collection of tunnel data on both the structural and the element level. Furthermore the localization of elements, damages, and extents of interventions can be performed in local linear coordinates related to the tunnel axis. In such a manner the collection of inspection data by modern laser equipment will be possible.

In Figure 10 a photo obtained during a tunnel inspection is shown. The photo has been superimposed onto the unrolled, developed view of the tunnel surface. The photo is calibrated in tunnel coordinates so that the collected data obtain geographical reference automatically.

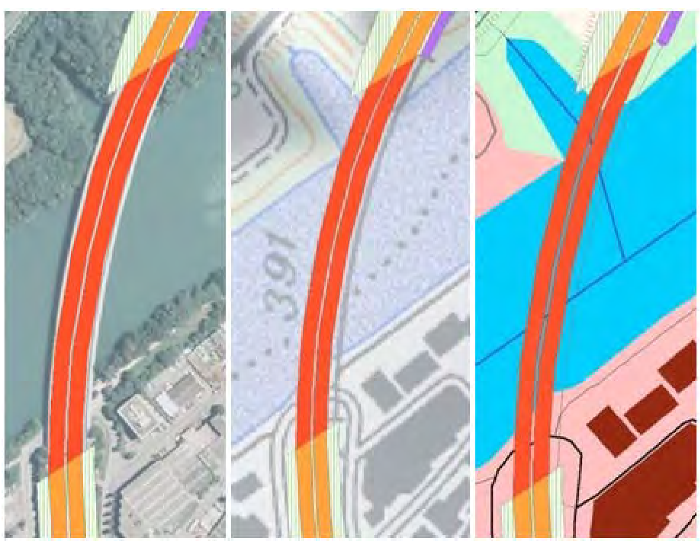

FIGURE 9 Outline of a bridge on different maps.

FIGURE 10 Tunnel inspection photo.

3-D Representation

The potential of 3-D representation for collecting inspection data and defining the extents of preservation intervention is widely recognized. The feasibility of 3-D representation has been studied during the conceptual phase of KUBA 5.0. The focus has been set on

- The effort to create a 3-D model from available drawings in different levels of sophistication and
- The available technologies to include a 3-D module in KUBA, which would allow not only viewing of 3-D models but also collecting damages such as cracks and corrosion areas directly on the screen.

The conclusion of the study is that the prime time for 3-D models in road structure management systems is yet to come. There are both technical and administrative obstacles in integrating components of major computer-assisted drafting and GIS vendors in road structure management systems. However, the fast development on the market may open an opportunity for this exciting feature, illustrated in Figure 11, in the near future.

RESEARCH RELATED TO KUBA

Apart from the intensive software development of KUBA, there are also vivid research activities going on in bridge management. As mentioned earlier, the implemented approach in KUBA assumes that the consequences for road users due to deterioration can be neglected. This assumption reduces the complexity of the decision process significantly. If one only concentrates on gradual deterioration, this is indeed a sound concept since it is unlikely that the condition of a structure causing even minimal disruption in functionality would be tolerated. However, this

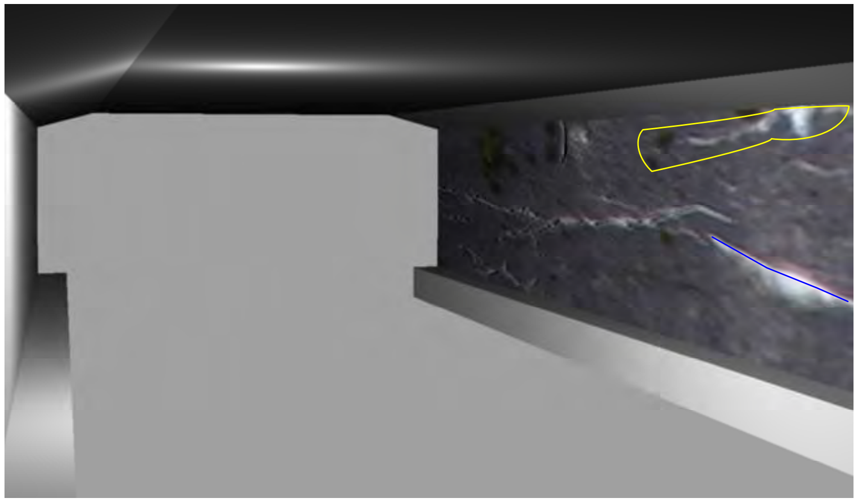

FIGURE 11 Direct collection of damage data using stylo (yellow and blue lines).

concept has particularly significant drawbacks when the sudden failure of the road structure cannot be neglected. In the current approach the benefit of preservation projects stems from expected savings of agency costs. This means that two identical road structures in the same CS could be treated equally even if the consequences of their failure were vastly different. This can be justified only if there is zero probability of failure, which is clearly not the case with natural hazards. This problem is addressed by several research projects currently in progress (*6*) with the goal to provide sound groundwork allowing risk-related decision support in KUBA.

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A Bridge Management System for the Western Cape Provincial Government, South Africa
Implementation and Utilization

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This paper describes the implementation and utilization of the bridge management system (BMS) of the Department of Transport and Public Works of the Western Cape Provincial Government. The implementation of the BMS as well as the visual assessment of all the structures on its road network were completed in 2003. The system consists of inventory, inspection, condition, budget, and maintenance modules and is capable of utilizing visual assessment data to prioritize structure maintenance projects. The BMS database is integrated with the Department's Road Network Information System. The system's visual assessment methodology is based on a 4-point DERU (Degree, Extent, Relevancy and Urgency) system for rating observed defects. The relevancy rating forces the bridge inspector to evaluate the consequences the defect in terms of the structure's serviceability and safety. Each of these parameters is combined in the condition module to determine a priority ranking of structures requiring repair. During 2006 a bridge and culvert rehabilitation project was identified in the Eden District Municipality, utilizing the BMS for the first time in the validation of assessments and prioritizing of structures in terms of their maintenance needs in that particular region. The project, which included the rehabilitation of 65 structures, was awarded in September 2006 and completed in a period of 15 months. The paper discusses the implementation of the pilot project, lessons learned and proposed enhancements in terms of the BMS, structure visual assessments, and the implementation of contracts.

The Western Cape is one of the nine provinces in South Africa, and the Roads Infrastructure Branch of the Provincial Government of the Western Cape (PGWC) Department of Transport & Public Works is currently responsible for the management of 6,000 km of paved and 10,000 km of unpaved roads. These are basically all rural roads in the province that are not national routes, and include approximately 2,300 bridges and major culverts. Prior to 2000, a bridge database with limited inventory information on each bridge and a plan database consisting only of a listing of as-built drawings of each bridge, and condition-based bridge inspection forms were used to manage the structures on provincial roads. This system did not produce meaningful results and thus there was no real management of bridge maintenance and rehabilitation.

The PGWC identified the need to acquire a management system to motivate for and allocate limited available funds to rehabilitation projects where most needed and to projects where the long-term benefit would be the most cost effective, i.e., to have a bridge management system (BMS) in place in order to be able to identify projects in order of importance and also to maintain long-term bridge rehabilitation at an optimum level. It was imperative that the system generate credible information in the eyes of the decision makers, thereby building confidence in the identification, prioritization, and planning processes in order to prevent regress to traditional ad hoc and political decision-making processes. As in the case of most road authorities, bridge and road maintenance and rehabilitation are funded from the same budget and have to compete for funds.

In order to effectively integrate bridge rehabilitation with road rehabilitation (which normally occurs more frequently), it was important that the BMS be sufficiently reliable and effective for future integration with other management systems, such as the Pavement Management System and Road Maintenance Management System. A further requirement was that the system should be able to cater for other road structures such as culverts and retaining walls. A culvert module was thus developed and incorporated into the system. The BMS was required to make provision for structure type for the purpose of visual assessments and structure classification, in accordance with the PGWC definitions based on minimum span length and total structure length. Modules to accommodate retaining walls, sign gantries, and minor culverts are in the process of being included in the system.

In 2000 the Provincial Government Western Cape adopted the STRUMAN Bridge Management System (*1*) developed by the Roads and Transport Division of the Council for Industrial and Scientific Research (CSIR Built Environment) together with Stewart Scott International. During the following 3 years, all 2,300 bridges and major culverts on the provincial road network were inspected and the data captured into the BMS.

During the past few decades, little attention has been given to the overall condition of structures in general, and many of the bridge rehabilitation projects that were commissioned were done on an ad hoc basis. Using the BMS, the Design Directorate now proposes a program of rehabilitation of all bridges and major culverts in the province that are in need of remedial work and safety-related improvements.

BACKGROUND

Description of the BMS and Modules

During the first phase of the project, the inventory and inspection modules were customized to meet the needs of the department, which included the development of a culvert module and the integration of the BMS database with the Road Network Information System (RNIS). A map module front end was also developed and integrated with the other BMS modules for graphical viewing of the structure data. This was based on shape files exported from the department's geographical information system (GIS). The system is currently being updated to include access to bridge and culvert drawings in electronic format. The BMS has also been made accessible to regional offices and other authorized users via the Internet.

As in the case of most bridge management systems, the STRUMAN BMS consists of an Inventory Module, Inspection Module, Condition Module, and Budget Module. Its main distinction is perhaps in the Inspection Module, where the focus is on the observed defects of the various structure elements rather than the overall condition of each element. The PGWC's system therefore basically consists of the following:

Inventory Module

This is the basic module of a BMS and consists of detailed inventory data for bridges and culverts. The original inventory module was customized and expanded to meet the requirements of the PGWC. The main sections are as follows:

- Location details,
- Contract details,
- Structural features,
- Design characteristics,
- Hydraulic data,
- Dimensions and geometry,
- Services details,
- Road configurations and traffic volumes,
- Archive details—electronic linking of drawings for each rehabilitation project,
- Rehabilitation history—information and photo links for each rehabilitation,
- Factors influencing field inspection, and
- Inventory photos—photographic history of structure.

Inspection Module

This module contains the detailed inspection data for each structure. The main sections are

- Inspection heading and summary,
- Ratings,
- Remedial work activities, and
- Inspection photos—photos of all observed defects.

Condition Module

Bridges and culverts are prioritized according to preset parameters. In the Condition Module structures are prioritized in order of the need for repair or rehabilitation. All structure items have adjustable weighting factors built into the prioritization algorithm so that important items such as abutments, piers, and decks (in the case of bridges) that have defects with a high degree (D) rating combined with a high relevancy (R) rating have a greater influence on the Priority Index (PI) of a structure than other minor items such as parapets, deck joints, and bearings. The Condition Index (CI) is used to rank the structures in terms of overall condition as opposed to the need for receiving maintenance. The Functional Index (FI) is combined with the Priority Index to take into account the strategic importance of the structure or route on which it is located. The Overall Priority Index (OPI) is a weighted combination of the PI and FI.

Budget Module

The predefined remedial work activities that are utilized during the visual assessments for identifying required repairs to defects have associated unit costs. These costs are used in the budget module to determine estimated repair costs for individual structures. Optimization is done using the relevancy–cost ratio per defect and budget limits per year. Repairs are allocated to the Current Year, Year 2–3, Year 5–10 ,or Routine categories based on the urgency rating (U). In the case of structures that have been identified for repair, either selected or all repair items for these structures are allocated to the Current Year and the budget is reoptimized.

Structure Inspection Rating Methodology

The BMS utilizes a defects-based rating system (DERU) whereby each defect of a structure element is rated according to its degree (D), extent (E), and relevancy (R). An urgency (U) rating is also given to indicate the perceived urgency of the proposed remedial activity. Only the worst defect (highest relevancy or highest degree for the same relevancy) on each item or sub-item is rated, but each defect is assigned a remedial work activity with an urgency rating.

Each of the DER ratings is rated on a scale of 1 to 4 as follows:

- D = degree of severity of defect (1 = minor to 4 = severe; 0 = no defect);
- E = extent of defect on bridge element (1 = local to 4 = general); and
- R = relevancy of defect to serviceability of bridge element (1 = minimum to 4 = critical).

The relevancy rating forces the bridge inspector to evaluate the consequences of the defect in terms of the bridge serviceability and safety. Each of these parameters is combined in the condition module to determine a priority index for each structure. A remedial worksheet is used during structure inspections to summarize the items requiring repair. In the case of an element that does not exist or is missing (e.g., guardrails and invert slab), both D and E are rated as 4. The bridge inspector is therefore not required to rate the condition of each structure item, but only the defects observed on each item. A visual assessment manual was also developed to improve uniformity of the inspector rating standards.

OUTPUT OF THE BMS ASSESSMENTS

Assessments Carried Out from 2001 to 2003

During a period of approximately 2 years (2001 to 2003), 15 bridge inspectors (most of them based in the Cape Town area) were used to inspect the 2,300 structures (850 bridges and 1,450 major culverts) in the province’s five District Municipality regions and the Cape Town Unicity (excluding structures that fall under the jurisdiction of the City of Cape Town). The locations of all these structures are shown in Figure 1. As many as possible of the locally based bridge engineers were given the opportunity to engage in bridge inspections for the PGWC.

The inspectors were not only required to carry out principal inspections, but also to obtain all the relevant inventory information of each structure—either from as-built design

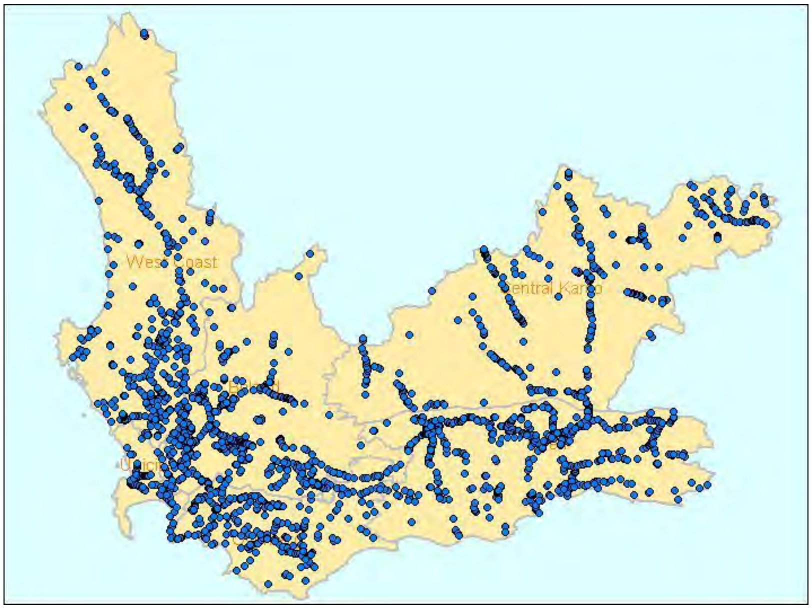

FIGURE 1 Map showing all structures in the BMS database.

drawings, if available, or from measurements on site if drawings were not available. Inspectors were required to record all visual defects—not because it is the intention that all defects will eventually be repaired, but to have a reference base for all the defects. This information, together with the inventory and inspection photographs, was then captured by the inspector into the BMS; each bridge inspector received a copy of the BMS inventory and inspection modules for this purpose. On completion, the electronic data were submitted to the PGWC for incorporation into the main database.

Although significant emphasis was placed on quality and uniformity during the compulsory BMS training course and the briefing sessions, the recorded inventory information and especially inspection ratings were not always of the required consistency necessary to obtain reasonably accurate prioritization of rehabilitation needs. The reason for this inconsistency could be attributed to the fact that not all the inspectors had similar previous bridge design and rehabilitation experience, and 15 bridge inspectors are perhaps too many to achieve a satisfactory degree of consistency

This was the first round of inspections with the STRUMAN system and thus electronic comparisons with previous inspections were not possible. However, for a certain number of structures where the conditions of the structures were well known, the results obtained from the BMS were verified. By observing the defects shown on the inspection photos for these structures and through verification inspections of the structures, the BMS prioritization of these structures (relative to each other) could be assessed. By being able to calibrate various system and weighting factors in the condition module, it was possible to optimize the BMS output to produce results that were considered to be accurate and realistic as far as these structures were concerned. The most important aspect was to verify that the structures at the top of the priority list were in fact those most in need of repair, i.e., to verify the calibration of the prioritization algorithm.

At this stage it is envisaged that principal (lower cost) inspections will be undertaken every 5 to 7 years as well as on completion of the repair and rehabilitation of structures. The inspections are only visual, but they are the BMS's primary data source for determining the structure's condition, and diagnostic testing is generally only used for detailed project-level inspections after identification of repair projects.

Prioritization of Structures

Prioritization of all the inspected structures on all the roads (Trunk, Main, and Divisional Roads) in the province was done. All structures with a Priority Index value below 60 were identified as requiring attention and displayed in the Map Module. The number of structures on the Provincial road network that met this criterion is about 175. Areas (with a radius of approximately 50 km) were identified where the highest concentrations of structures in the above category were situated. Each of these areas was earmarked as a project and all the structures in these areas were identified to be included in the project (*2*).

For sound economic reasons (e.g., cost of site establishment) it is beneficial not only to rehabilitate the high-priority (worst condition) structures on the higher road classes—which were evidently scattered over the whole province—but also to include structures situated on lower road classes and with a lower priority (but with high benefit–cost rehabilitation needs) that are in close proximity to the identified project areas. The aim is therefore to group bridge rehabilitation into projects of suitable size that can be awarded to one construction firm. The final selection of structure maintenance projects also takes into account planned road maintenance projects.

Identified Repair Work

The results of the inspections highlighted a number of common problems throughout the province. However, as expected, a number of these defects were found to be more pronounced in coastal and high rainfall areas. Rehabilitation needs included the following:

- Routine maintenance repairs:
  - Approach embankment and scour protection works,
  - Approach and deck resurfacing,
  - Cleaning of waterways and siltation inside culverts, and
  - Removal of vegetation from sidewalks and deck joints;
- Road safety improvements:
  - Installation, extension, and attachment of guardrails at bridge abutments,
  - Warning signage, and
  - Reconstruction–repair of bridge parapets and handrails;
- General serviceability repairs and protection:
  - Repair of spalled concrete,
  - Replacement of bearings,
  - Replacement of deck joints, and
  - Crack sealing and durability enhancement coatings.

PROJECT IMPLEMENTATION IN SOUTHWESTERN REGION

Overview

One of the regions with a high concentration of structures in a poor condition was the Eden District Municipality area (southwestern region of the province), which had about 50 structures with a priority index less than 60, as well as many other structures requiring lesser rehabilitation and safety improvements.

An area (with a radius of approximately 50 km) within the Eden region was identified where the highest concentration of structures in the above category were situated. The BKS/GOBA Joint Venture was appointed by the Department of Transport and Public Works, Provincial Administration Western Cape: Road Infrastructure Branch in January 2005 to undertake detail design, tender documentation, and site supervision of the rehabilitation of 65 bridges and major culverts in the Calitzdorp, Oudtshoorn, and De Rust area (*3*). The locations of these structures are shown in Figure 2.

The report stage of this appointment was completed in May 2004 and the consultant was instructed to proceed with a Detail Assessment Report based on the recommendations in this report. This process consisted inter alia of further detail site investigations related to concrete condition and asphalt surfacing.

A final scope of work was decided on based on the findings of these investigations as well as further presentations to the branch officials. On completion of the detail assessment a detail cost estimate of the works was submitted (Construction Cost Option A). This cost estimate was significantly higher than that originally anticipated in the report stage. The main factors

FIGURE 2 Map showing all structures included in the rehabilitation project.

contributing to the additional costs related to the following items, which were identified during the detail site investigations:

- Extent of the asphalt on the bridge decks and approaches that was to be replaced (this was mainly due to provision for asphalt on all bridges located on gravel surfaced roads as well as the length of the approaches that were resurfaced);
- Upgrading of drainage elements to bridge approaches;
- Upgrading of approach guardrails and other road safety features; and
- Extent and type of the coatings to concrete surfaces.

Based on this estimate, budgetary provisions and the fact that this contract was mainly intended to address bridge remedial measures only, it was subsequently decided to reassess the scope of work and omit all the road works items that were not considered to be essential for the safe functional operation of the bridges and that could be done during routine maintenance work or other road works contracts at a later stage. The scope of work was also discussed and evaluated in detail on site and at various meetings with the Chief Engineer Structures as well as a representative of the District Roads Engineer (Construction Cost Option B).

After these additional investigations all the findings were consolidated in a Detail Assessment Report to provide final recommendations and costs, which was completed in January 2006.

Based on these recommendations a contract was advertised during 2006 and construction commenced in September 2006. The construction of the above work was completed in January 2008.

Key Conclusions from Detail Assessment Report

The following key conclusions were made in the report:

- The condition of the concrete of the structures was generally good although it was considered advisable in some instances to perform limited repairs and apply protective coatings.
- The structures were generally in a fair to good structural condition except for the specific structural problems, which need to be addressed to ensure the safe functional operation of the structures.
- Although most of the asphalt over the bridge decks appeared old with excessive voids and poor compaction, most surfaces were still serviceable. There were, however, reservations regarding the remaining life expectancy of the asphalt and it was recommended that the asphalt be replaced on all the structures designated for new joints.
- The waterways and approaches were in a reasonable condition requiring attention in some instances, mostly to limit further scour damage and blockage of drainage openings. Drainage also required attention.
- The majority of bridge joints was in a fair to poor condition and did not satisfy the functional requirements for which they were intended. Various remedial options were proposed.
- There were a number of minor defects associated with the kerbs and sidewalks.
- Although there was a general problem with missing or vandalized aluminium handrails, the concrete parapets were in a fair condition with a general problem of reinforcement corrosion. New precast reinforced concrete railings were proposed in instances were vandalism

had occurred.

- Bearings were generally in a good condition.
- Selected road safety elements were also identified for repair.
- No significant impact on road and rail traffic was expected.
- Environmental issues were to be addressed by means of an environmental management plan.
- Preferential procurement targeted procurement goals were identified considering the capacity of the local community and also the nature of the construction.

Recommended Rehabilitation Strategies

Two rehabilitation strategies, as noted above, were provided with the following main provisions:

- Construction Cost Option A: This strategy recommends that, in addition to the proposed bridge remedial activities, all the desirable road works be carried out in the vicinity of the structures. In this option, it is important to note that where asphaltic plug-type joints were proposed, provision was made to replace the asphalt surfacing to ensure that it would have a service life similar to that of the bridge joints as this would ensure an optimum use of resources. Extensive upgrading of approach guardrails and drainage elements was also proposed in this option. The viewpoint was also taken that for the gravel roads, new asphalt surfacing would be provided over the exposed concrete surfaces of the bridge decks.
- Construction Cost Option B: This strategy addresses, in addition to the proposed bridge remedial activities, the items that relate to the bridge and the approaches on the basis that the structures are to be repaired to a safe functional condition; additional interventions that were identified would be done under routine road maintenance contracts at a later stage. In this option asphalt surfacing was only replaced where it was in need of immediate repair and where it could affect the integrity of the structure, or where it was essential for joint replacement activities. Ancillary elements that impacted significantly on road safety were also addressed in this option, but upgrading of road elements was minimized.

The recommendation to proceed with the second option was finally accepted for implementation.

Summary of Planned and Actual Costs

Project cost estimates during the planning stages for the above options are summarized in Table 1 with the final construction costs, which are based on moderated rates as received from the contractors. These amounts include pro rata allowance for preliminary and general cost items. The following observations are made with reference to these costs:

- A major part of the cost related to ancillary items such as traffic accommodation and road works (drainage, asphalt surfacing, signage, and guardrails). The repairs to these elements were however considered an essential functional remedial action.
- A comparison of the estimated to the final cost indicated a reasonable correlation and cost increases were generally attributed to the remoteness of the site, spread-out nature of the work, and oversupply of work in the industry, except for the components as noted below.

TABLE 1 Comparison of Planned and Actual Costs

Ref.	Description	Option 1 Amount (R 1,000s)	Option 2 Amount (R 1,000s)	Final Amount (R 1,000s)	% of Total in Final Amount	% Var. Final vs. Option 2
A1	Traffic accommodation	R 2,809	R 2,812	R 2,473	7.7	–12.1
A2	Road works	R 16,589	R 7,484	R 13,731	42.5	+83.5
	Parapets, handrails	R 6,599	R 5,745	R 5,906	18.3	+2.8
	Joints	R 2,052	R 2,197	R 2,166	6.7	–1.4
	Concrete and structural	R 6,226	R 5,996	R 5,433	16.8	–9.4
	Scour and miscellaneous	R 2,510	R 2,513	R 2,588	8.0	+3.0
A3	Structures subtotal	R 17,387	R 16,451	R 16,093	49.8	+12.5
	Construction cost subtotal (A1 + A2 + A3)	**R 36,785**	**R 26,747**	**R 32,297**	**100.0**	**+20.8**

- The most significant cost increase (estimate versus final) was primarily attributed to the asphalt resurfacing of the bridge decks. This activity was recommended to ensure that the roadway and joints (after repairs) would have a similar life expectancy. The tendered values were however found to be significantly higher than the estimate as provided in Cost Option B above. The main contributing factor related to the oversubscription of construction work in the industry, increased bitumen costs, as well as the premium that has to be paid for working with small quantities on remote sites.
- Another cost increase related to the repairs to the parapets and handrails. In these components it was found that, after cleaning and preparation, the concrete condition was in general worse than anticipated, which resulted in additional costs with respect to repairs and coatings.
- Scour protection works were also more costly due to limited sources of rock in the area.

Key Conclusions from the Repair Contract

In addition to the above the following conclusions were made as a result of the experience attained during the repair contract:

- Access and traffic accommodation requirements have significant cost implications;
- Spall repair measuring and structuring of payment items require careful consideration as quantities could vary significantly once breakouts proceed on site. The location and number of breakouts also have a significant impact on cost and time on a repair contract;
- Geographical location and the number of structures in a contract have significant management and cost implications;
- On completion of a rehabilitation project, the recording and inclusion of details of

specific repair and maintenance interventions on structures in the BMS is essential. This should include a comprehensive record of costs, product specifications, installation dates, and product guarantees as well as requirements for future maintenance interventions;

- The necessity of combining ancillary repairs to structure approaches and road surfacing must be carefully assessed with structural repair contracts and should preferably form part of road maintenance activities. This could also include bridge joint repairs, which are generally performed by specialist subcontractors.

EVALUATION OF THE BMS DATA IN TERMS OF THE PROJECT

Reprioritization

After completion of the repair works, all 65 of the structures were reinspected using the BMS assessment approach and reprioritized as part of the main database. As far as the Priority Rankings are concerned, more than 80% of these structures are now in the lower 60% of the priority list. The structures that were originally in the Priority Index < 60 category required structural repairs; all of these structures are now in the lower 60% of the priority list. As far as the Condition Rankings are concerned, all of the 65 repaired structures are in the lower 50% of the priority list. Table 2 shows the revised Priority and Condition Indices and rankings of 13 structures based on the visual assessment data after the completion of the repair works. Figure 3 shows examples of structures where structural repairs were performed.

TABLE 2 Revised Priority and Condition Indices and Rankings After Repairs

Structure Name	PI	PI Rank	CI	CI Rank	Repairs
Stolsvlakte Road/Rail	100	828	93.4	1805	Provide new RC wingwalls
Cango River	99.5	780	96.7	2050	Provide scour protection works
Le Roux Station Road/Rail	99.1	734	94.5	1901	Repair major spalling and cracking
Meule River	100	829	100	2226	Provide scour protection works
Olifants River (Oudtshoorn)	100	856	99.3	2209	Replace bridge handrails and repair major spalling
Grobbelaars River	100	833	100	2227	Underpin pier foundation and provide scour protection
Vlakteplaas Road/Rail	100	845	100	2230	Provide gabion wall to support approach embankment
Olifants River (Volmoed)	99.6	784	96.7	2051	Reconstruct new RC elements to provide structural capacity
Vlei River	100	820	98.8	2183	Repair major spalling on beams and apply surface protection
Touws River	99.9	809	99.3	2207	Provide gabion and riprap scour protection
Culvert on TR 31/5	100	1815	92.24	1711	Seal inject cracks and construct new pier to strengthen element
Culvert on TR 31/6	100	1861	93.6	1822	Provide epoxy bonded plates to strengthen element
Culvert on TR 33/2	100	1956	100	2304	Provide culvert invert slab as scour protection

(a) **(b)**

(c) **(d)**

FIGURE 3 Examples of (*a*) structural repairs, repairs to culvert wing walls, abutment walls as well as deck slab; (*b*) replacement of existing steel bridge handrails with precast concrete rails; (*c*) repairs to bridge abutment by means of externally reinforced concrete elements; and (*d*) repairs to culvert deck soffit by means of externally bonded steel plates.

BMS Results Compared with Project Outcomes: Evaluation and Recommendations

The use of the 2003 BMS assessment results for defining the scope of the pilot bridge and culvert rehabilitation project in the Eden region, experiences during the contract period, and the reassessment of the structures after the completion of the repair project give rise to the following observations (specific recommendations are noted in italics):

- In a number of instances significant additional repairs were carried out that were not evident from the original inspection data. These included major structural repairs, coatings to concrete elements and joint installation items. This can primarily be attributed to the fact that certain defects were not readily visible by visual inspections and could only be identified by means of a detailed structural assessment, diagnostic investigations (e.g., testing of core samples), or exposure of concrete surfaces by removal of paint or asphalt surfacing during construction. Joint repairs were also considered important to ensure long-term serviceability of the roadway and were included as part of the works but were not always considered a defect by the inspectors. *The recording of these activities cannot be readily tracked from the BMS as some repair items were identified in the detail assessment report stage. Consideration should be given to including a pre-repair principle inspection in the BMS.*

- Major repairs were performed on 20 of the structures. These included severe erosion damage repair by means of concrete underpinning and gabion or riprap protection, concrete deterioration repair by means of patching and coatings, structural repairs using steel plates, new reinforced concrete elements, and the complete replacement of a structure. *Generic BMS repair activities should be updated to allow for major repair options and inspectors should be advised to take a conservative approach. For example, if cracking is suspected to be of a structural deficiency origin, a major structural repair item should rather be noted. A clear distinction should also be made between normal maintenance-type activities (e.g., painting, joint repairs, and bush clearing) and once-off repair activities (e.g., major structural repairs and scour repairs) in the BMS. This would enable asset managers to plan and manage their budgets and program maintenance activities efficiently.*
- In some instances, defects that were not repaired under the repair contract were kept on record. These items were either not serious enough to warrant the associated repair costs or the ratings were amended as a result of the findings in the detail assessment stage to a noncritical status. Examples of this included nonstructural cracking and bearing replacement items. It was however considered important to retain a record of these defects for future reference purposes. *Consideration should be given to including such items in the inventory module as well.*
- Additional defects were noted in isolated instances mainly for monitoring purposes. For example, observation of scour risk at one of the structures was noted. The monitoring of certain aspects is considered important and suitable mechanisms are required to ensure this. These include waterway scour and debris buildup as well as structural cracking in some instances. Related defects have been retained in the inspection listing with suitable monitoring frequencies to ensure this, even if repairs have been done to these elements. *Monitoring of specific items should also be communicated to the local road authorities and where specific actions are required (e.g., cleaning of vegetation in bridge openings), these should be followed up. Ideally this should be done by means of an integrated BMS that is actively managed on a routine basis.*
- The inventory module has been provided with specific data fields for drawings and photos of repairs and retrofitting that have been updated. In addition standard inventory photos, which show the structure and waterway, are updated to provide a historic record and are particularly useful to assess riverbed changes and to provide a record of upgrading of ancillary elements such as repairs to bridge railings. *Key inventory photos such as views of watercourses and safety features should be taken at regular intervals as these may be useful in analyzing flood behavior or accident cases. This will also assist in the monitoring and control of maintenance activities.*
- Road maintenance activities can impact significantly on the intended repair strategy and close interaction with the responsible parties is important. Integration at critical levels of pavement management systems and BMS is important. *Joint replacement and bridge surfacing interventions as well as road safety are examples of this.*
- Substandard geometric elements such as bridge widths are recorded in the inventory but are not flagged clearly. *Geometric upgrading must be flagged to ensure consideration for future upgrades.*

- This contract provides the ideal opportunity to update unit costs provided in the BMS to facilitate more effective usage of the budget module. These should be updated regularly based on the most recent repair contracts. *Budget Module costs should be calibrated to allow for area specific properties, adjustments to isolated repair interventions, and contingency factors.*
- A detail comparison of asset value, maintenance intervention cost, and associated benefits did not form part of this study but would be a useful addition. C*onsideration should be given to the development of a bridge-specific cost–benefit module, which should include accident and vehicle operating costs*.

CONCLUSIONS

The development and implementation of the STRUMAN BMS for the Western Cape Provincial Government has led to a significant improvement in the management of structures on the provincial road network. All 2,300 bridges and major culverts were visually assessed during the period 2001 to 2003 using a defects-based system. The 175 worst structures were identified for inclusion in a bridge repair and rehabilitation program. Other structures with a lower priority will also be included in the rehab projects due to their location in relation to the high-priority structures. A pilot project in the Eden District Municipality has been implemented after which projects for the repair of the remaining high-priority structures will be carried out. The project implementation provided a number of guidelines that should be taken into consideration in the enhancement of the BMS, structure inspection projects, and implementation contracts. These relate mainly to recording and tracking of defects as well as costing and grouping of repair activities in contracts. The pilot project has thus highlighted a number of areas for the improvement of the BMS and for more effective management of the maintenance and monitoring of structures.

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Design and Implementation of a New Bridge Management System for the Québec Ministry of Transport

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In its continuing effort to ensure a safe, efficient transportation infrastructure for the people of Québec, the Ministry of Transport of Québec (MTQ) has been developing a new bridge management system. Known as Système de Gestion des Structures (SGS), the system has a number of novel features designed to meet the specific needs of MTQ. This paper highlights the design of the Système de Gestion des Structures (SGS), presents some of the challenges faced by the Ministry of Transport, and gives an overview of the analytical features of the system's analytical engine, the Strategic Planning Module.

The Ministry of Transport of Québec (MTQ) owns and maintains an asset inventory of over 9,000 structures, of which 4,300 are provincial bridges, 4,400 are municipal bridges, and the remaining are retaining walls and other miscellaneous structures. These structures are managed in a largely decentralized process by the Head Office and 14 regional offices.

As is the case with other agencies, the bridge inventory in Quebec is aging and the average bridge condition constantly challenges the financial and human resources of the Ministry. In its continuing effort to ensure a safe, efficient transportation infrastructure for the people of Québec, MTQ has been developing a new bridge management system. Known as Système de Gestion des Structures (SGS), the system has a number of novel features designed to meet the specific needs of MTQ:

- A decentralized software architecture, using Microsoft's .NET framework, for maximum sharing of inventory and inspection data across the province, with more controlled access to the strategic planning analysis;
- A graphic user interface provided primarily in French with certain bilingual and localizable features;
- Support for a detailed bridge inspection methodology that is span-by-span, but has elements and condition states similar in concept to the Ontario Structure Inspection Manual, with separate inspection of protection systems;

- Strategic planning analysis featuring several levels to fit MTQ business processes: network-level budgeting and performance analysis; priority programming; automated project scoping and treatment selection; and a digital dashboard for interactive design of the scoping and timing of projects; and
- A model framework that handles preservation, functional improvements, and replacement; provides explicit control of the element and project alternatives to be considered; and has features to update deterioration models based on new inspection data.

Begun in Fall 2005, the SGS has an Inventory and Inspection Module, which was delivered in early 2007, and a Strategic Planning Module (MPS), released in early 2008. The SGS combines lessons learned from the Ontario Bridge Management System (OBMS), also developed by Stantec (*1–4*), and recent research and preliminary design projects (*5*, *6*) into a new system that will be highly responsive to the Ministry's needs. The SGS is the first French language BMS of its kind.

OVERVIEW OF SYSTÈME DE GESTION DES STRUCTURES

The SGS is a new state-of-the-art bridge management system (BMS) that replaces an older system that handled inventory and inspection data. The SGS provides a new inventory and inspection system and add analytical capabilities to enable MTQ perform bridge-level and network-level analyses.

Revisions to Inspection

The previous inspection system was based on a numerical rating and inspection manual that was originally developed based on the original Ontario Structure Inspection Manual (*7*). MTQ rated the material condition of each bridge component using a scale from 1 to 6 with 6 being an element in new condition. The numbers on the scale represented a combination of severity of defects and their extent.

In order to modernize the inspection method and to meet the needs of the new SGS, MTQ made significant revisions to their bridge inspection manual. In the new manual, specific bridge elements are defined and the element condition is recorded as severity and extent separately. The inspector is required to record the quantity of defects in each of four condition states for each bridge component. These condition states are Excellent or A; Good or B; Fair or C; and Poor or D.

Each condition state defines defects on the basis of their severity. For example, the Good condition state refers to an element (or part of an element) where the first sign of "light" (minor) defects is visible. Material specific condition state tables are used to describe the severity of defects and assign this to the appropriate condition state. For example, concrete components might have "no observed defects" (Excellent), "light scaling" (Good), "medium scaling," or "severe to very severe scaling" (Poor). This explicit recording of quantities is necessary to allow estimation of types of repairs and their costs by the SGS and it provides the fundamental data needed for deterioration models.

Inventory and Inspection Module

The inventory and inspection module of SGS is deployed to all users of the SGS and is the primary tool for entering, viewing, editing, and reporting structure and inspection information. This module was delivered in early 2007.

Database Organization and Deployment

MTQ maintains an enterprise Oracle database for its structure inventory and inspection data, which also includes structural evaluation results and data on features intersecting the roadway network at bridge sites. This secure database supports the entire inspection workflow process, including acceptance and review of inspection reports received from outside consultants.

Periodically data from the Oracle database are replicated to a Microsoft SQL server database for use in reporting and analytical support modules, including the Strategic Planning Module (MPS). The replication process offers an added measure of security from unintended database updates, and provides a readily configurable interface to major downstream functionality whose development cycle may be independent of the main database.

Decision-Making Process

MTQ is moderately decentralized in its decision-making processes. Bridge-level maintenance and repair decisions are made in regional offices from operational funding. However, most structural expertise is resident in the headquarters office in Quebec City, where major bridge level decisions and all program-level and network-level decisions are made.

With this organizational structure, bridge inventory and inspection data must be shared with regional offices, but strategic planning data and analysis software are used mainly in headquarters.

STRATEGIC PLANNING MODULE

All analysis in the SGS is performed in the Strategic Planning Module or MPS. The MPS is deployed to a limited number of users who perform bridge project planning and network-level budgeting and priority setting.

Desktop Organization

The MPS launches from within SGS and the user is able to select a particular bridge to analyze, or to work with the entire network or a subset of bridges. For a specific bridge the user may examine element-level results or project results. At the network level the user may select the entire network or filter to a subset of bridges that depends on specific inventory, inspection, or even analysis results. For example, an engineer can select all bridges of a particular type with a certain range of conditions, of a certain age, which are load capacity deficient and older than 1970, and have projects costing more than $100,000. This subset can be stored and analyzed in more detail.

Performance Measures

The SGS reports several management indices. These indices are the bridge condition index (BCI) (or in French IES), IFS (a functionality index), IVS (seismic vulnerability–adequacy), and ICS (a combined index of the others). Although each of these indices could be used as a performance measure and analyzed in MPS, only the BCI (IES) is actually used in the analysis. Possible future enhancements to MPS include being able to set budgets based on the target performance measures based on the other indices.

Bridge Condition Index

The MPS uses the BCI (IES) as the principal performance measure index. The MTQ bridge condition index is based on the same concept as the OBMS BCI, which has its roots in the Bridge Health Index used in the United States (*8*). The BCI is a weighted average of the condition state distribution for the various elements that make up a given structure. The element replacement cost is used as the weighting factor so that elements that have a higher replacement cost have a higher weighting in the BCI.

The BCI is calculated as soon as an inspection is saved and is forecast in the MPS analysis for each element treatment. The MPS calculates the resulting BCI for the various strategies for each bridge. The network average BCI is also calculated for each budget scenario so that network performance can be compared for different funding levels.

Life-Cycle Cost Analysis Framework

MPS provides a family of decision support tools to assist in bridge project planning and program planning, as a part of the agency's overall asset management processes. All of the tools work within an integrated engineering–economic framework based on the concept of life-cycle costs.

The analyses are organized into three levels of detail, as shown in Figure 1. The levels are as follows:

- Element level, which focuses on a selected structural element of one bridge. This tool uses a Markovian deterioration model and a set of feasible treatments to produce multiple Element Alternatives, each of which is a possible corrective action to respond to deteriorated conditions. Functional improvements are also included at this level.
- Project level, which combines Element Alternatives into Project Alternatives, each of which represents a possible multiyear strategy to maintain service. The tool uses models of initial costs and life-cycle costs to evaluate the Project Alternatives.
- Network level, which combines the Project Alternatives on multiple bridges into Program Alternatives, each of which is a multiyear plan for work on all or part of a bridge inventory, designed to satisfy budget constraints and performance targets while minimizing life-cycle costs.

Each level of analysis in this framework provides a set of evaluated alternatives to feed the broader-scale tool to its left. All three levels are designed to stay consistent with each other, so if a change is made at one level, the remaining levels will adjust appropriately.

On the other hand, in going from left to right in the figure, network-level analysis provides a context for the project level because the network-level budget constraint affects which

Project Alternatives can be programmed. For example, in a very constrained funding scenario many bridges will have to select the Do-Nothing Alternative.

Similarly, the project-level analysis provides a context for the element level, influencing the cost-effectiveness of individual element treatments, as in the case when element treatments are combined at the same time to minimize traffic control costs.

For each treatment, the MPS generates two Element Alternatives, one for each implementation period, each with a life-cycle activity profile. An additional one is created for the Do-Nothing Alternative. A typical life-cycle profile is shown in Figure 2. This profile consists of zero or one treatment action applied during the 10-year detailed program horizon, followed by an annual average preservation expenditure after the end of the horizon, which continues to the end of the analysis period, usually 50 years.

The treatment is modeled to occur in either year 0 or year 5, which causes an immediate change in condition according to the treatment effectiveness model. Before and after the treatment, conditions change according to the deterioration model. Because bridge elements deteriorate slowly, and because the actual implementation year is uncertain, the fact that treatments may actually occur at any time during the planning period makes very little difference to the results, so it is ignored. The tactical planning tool, described below, is used for evaluating fine-grained changes in scheduling.

For the calculation of life-cycle costs and benefits, the treatment cost is calculated from a benchmark unit cost and discounted to year 0. It is assumed that no other costs are incurred during the 10-year program horizon.

Following the end of the program horizon, all further work is collapsed into a simplified long-term cost model. This model performs a generic life-cycle cost analysis for each condition state over 50 years, to compute the average annual cost. On a given bridge element, the condition state probabilities forecast for the end of the program horizon become the basis for calculating an expected value long-term annual cost. This is discounted to year 0.

The benefit of an Element Alternative is computed by subtracting its total life-cycle cost from that of the Do-Nothing Element Alternative. The Element Alternative with the lowest life cycle cost is selected for the first round of the project-level optimization.

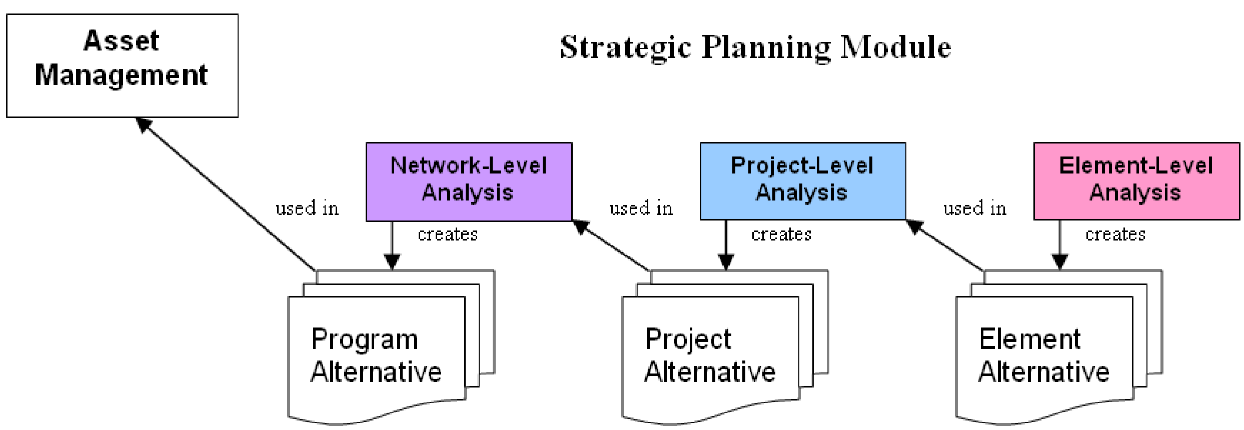

FIGURE 1 Levels of analysis.

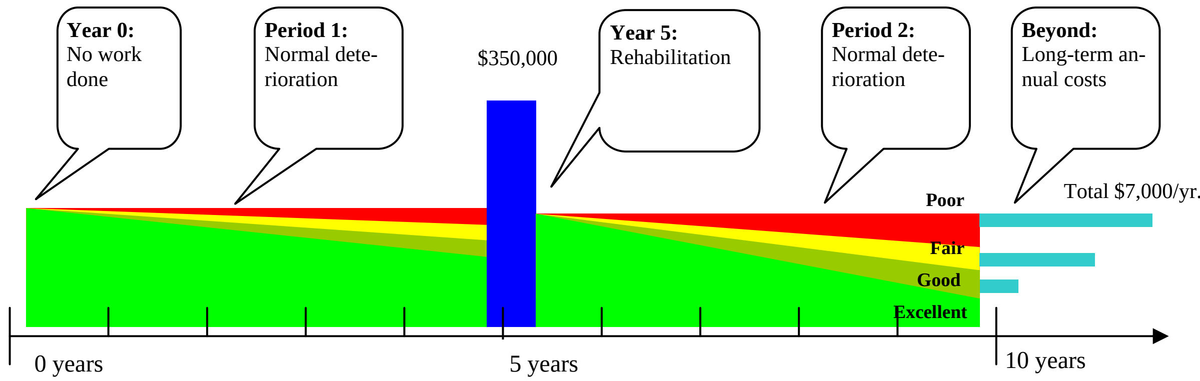

FIGURE 2 Life-cycle profile for typical element or bridge.

Program Planning Tools

The MPS analysis supports three levels of program planning tools, providing a quick user-friendly way for MTQ engineers to evaluate strategic and tactical decisions for any bridge or for an entire inventory. At the Element Level, treatment cost effectiveness is shown with corresponding life-cycle cost and resulting deterioration curve displayed as shown in Figure 3. System recommended or user selected element treatments can be used.

Budgeting and Network-Level Trade-Off Analysis

For a selected network subset, budgets are set in the budgeting screen (Figure 4). This can be done for the inventory as a whole, or a subset of it. Budgeting results are displayed across administrative subsets of the inventory. In addition, target performance measures (BCI) can be set and required budgets determined. The network-level analysis screen shows graphically the trade-off between funding and performance. Performance can also be summarized across geographical or administrative subsets of the inventory.

Priority-Setting Screen

The priority-setting screen (Figure 5) lists all the projects that are identified for possible funding, and indicates how many of them can be funded under the available budget. Additional projects available within a specified margin of budget uncertainty are identified with a different color. The analyst can use the mouse to rearrange priorities and view the effect on performance measures.

Tactical Planning Digital Dashboard

A unique and innovative feature of the MPS is the Tactical Planning Dashboard (Figure 6). The Tactical Planning Dashboard complements the Strategic Planning Module by adopting a purely bridge-level perspective on the planning of future work and enabling detailed study of project analysis. While the analysis is similar to MPS and relies on the same input data, the dashboard fills an important gap in several ways:

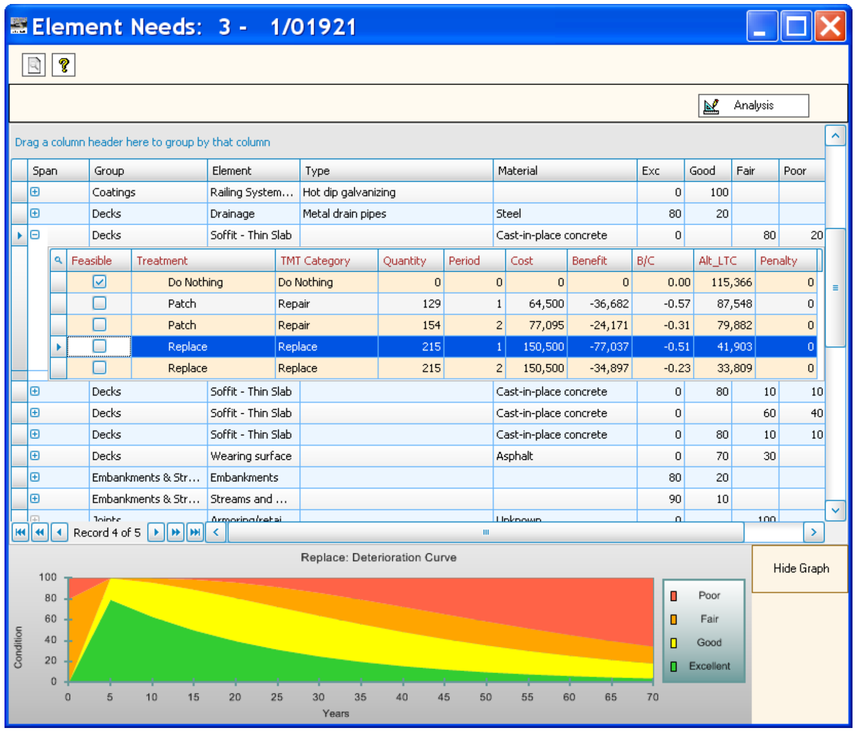

FIGURE 3 Element needs.

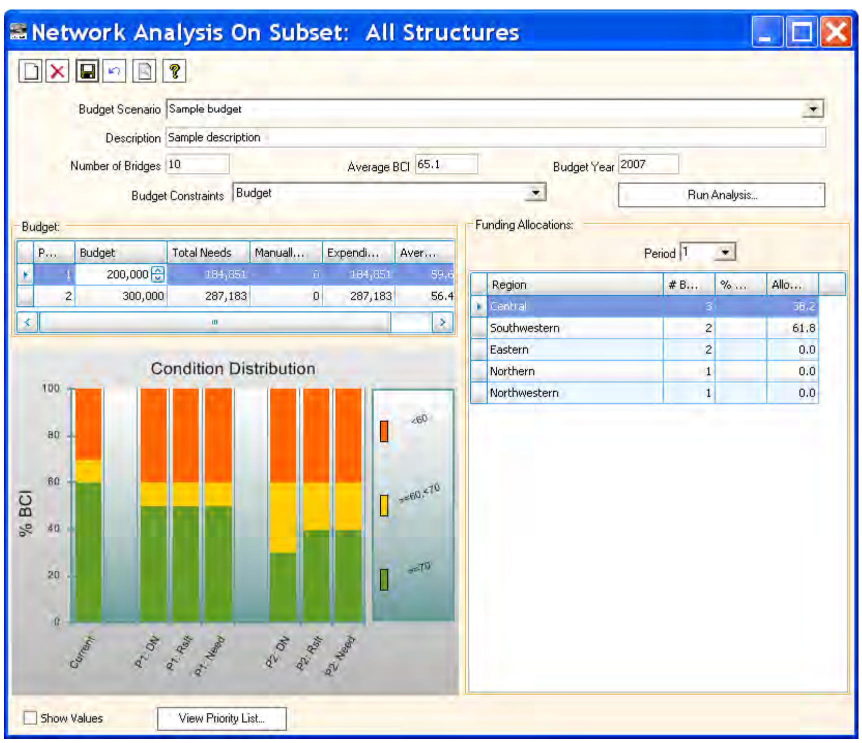

FIGURE 4 Budgeting and setting target performance measures.

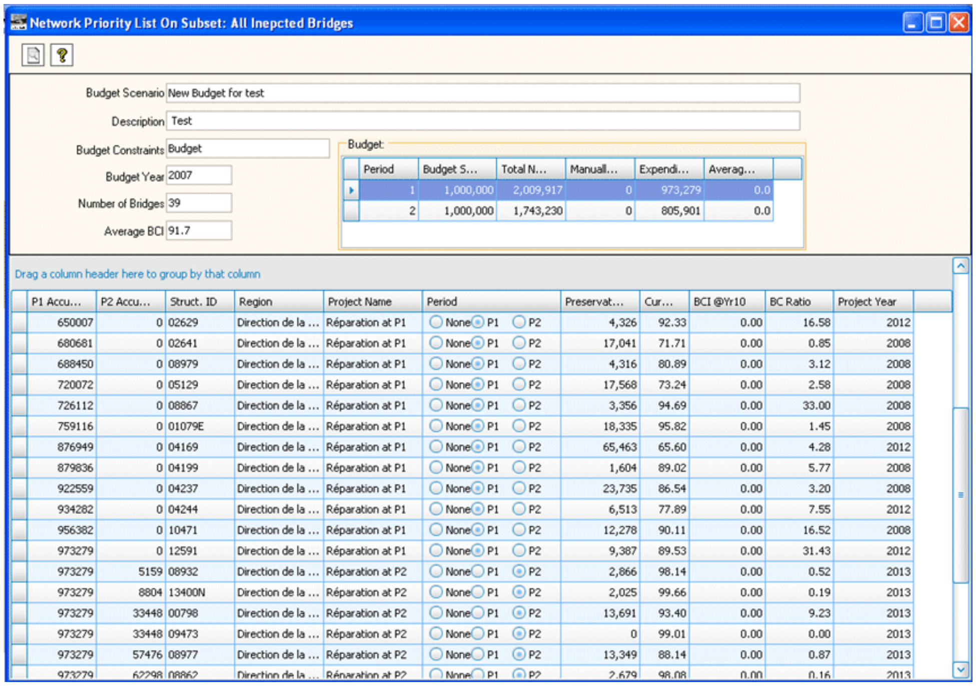

FIGURE 5 Project priorities.

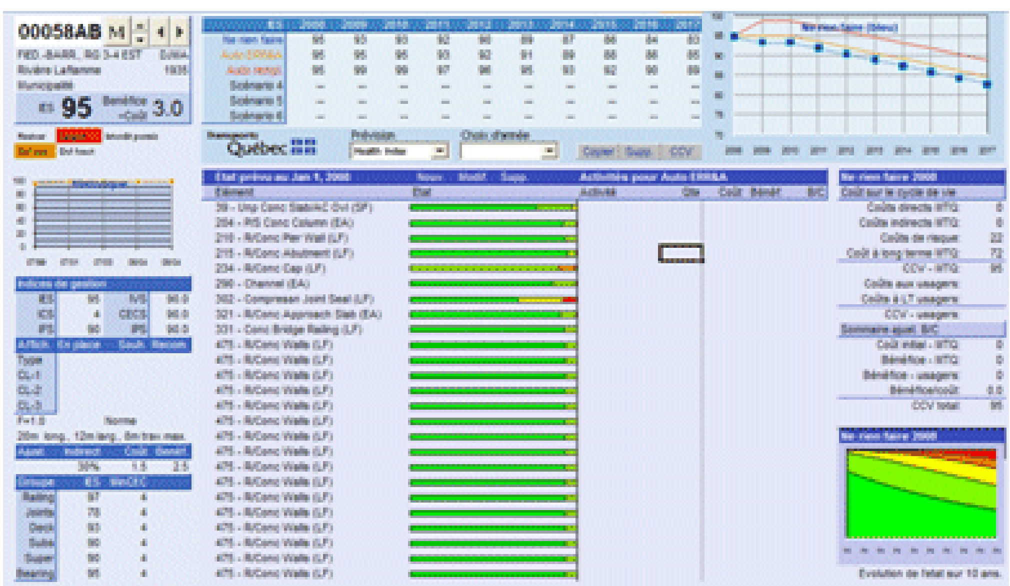

FIGURE 6 Tactical planning dashboard.

- Element- and project-level information as well as bridge inventory and inspection information are brought together in one window, whereas in the BMS this information is displayed in many different screens.
- MPS necessarily focuses on the uniform processing of groups of bridges, maintaining consistency across the inventory so projects can be prioritized and budgeting decisions can be made. The Digital Dashboard is more concerned with near-term considerations of project readiness and local conditions that might require adjustments to the program plan for individual bridges.
- MPS uses decision rules, budget constraints, and benefit–cost analysis to drive the prioritization and scheduling of work. The Digital Dashboard uses the same tools, but in an evaluative way, to show the effect on performance measures when practical considerations modify the circumstances under which a project may have been planned.
- The network-level analysis of MPS is based on a simulation of network-level processes and how every bridge in the inventory responds to these processes, including how bridges affect each other in the context of funding constraints. The Digital Dashboard does not rely on simulation but instead considers a wide variety of options for each bridge by itself.
- The information presented in the digital dashboard about individual bridges is more detailed than what is needed for program planning, and the decision variables are more fine grained.

Project-level decision making using the Digital Dashboard is likely to include a wider variety of information than what is provided in the MPS. Such information includes the engineer's personal knowledge of the site, design plans, various kinds of testing, and engineering judgment. The engineer's own mental processes are what integrate these information sources.

CONCLUSIONS

The Ministry of Transport of Québec's new BMS, SGS, is the first French BMS of its kind. SGS includes an advanced MPS that incorporates a powerful set of three analysis levels designed to stay consistent with each other, so if a change is made at the element level, the project and network levels will adjust appropriately. The budgeting and performance measure setting that is done at the network level has an immediate effect on selected projects and the results of project selection can be viewed in tabular or graphic form. The MPS incorporates an innovative digital Tactical Planning Dashboard that complements the MPS by providing a purely bridge-level perspective on the planning of future work and enabling detailed study of project analysis results and other information only available in several different screens in MPS.

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Bridge Preservation, Maintenance, and Deterioration Rates

Preserving Our Bridge Infrastructure

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The purpose of the presentation is to provide the audience with a summary of bridge preservation accomplishments, ongoing activities, and future objectives by various groups across the country. Partnership opportunities with established or nascent bridge preservation groups from state agencies, academia, industry, and engineering organizations will be included. An overview of ongoing and bridge preservation roadmaps developed by national administrative organizations such as FHWA, AASHTO, and the Transportation Research Board along with technical centers such as the Transportation System Preservation Technical Services Program will be discussed. A similar summary of bridge preservation efforts on the regional level, involving regional working groups, University Transportation Centers, and Local Technical Assistance Programs, will be included. Innovative strategies implemented by various agencies will be highlighted.

Bridging Data Voids

Advanced Statistical Methods for Bridge Management in KUBA

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The Swiss Bridge Management System (KUBA) provides decision support in planning preservation interventions. In order to compare preservation strategies, the system relies on forecasts about the future development of a structural element's condition state. In KUBA, the deterioration of a structural element is modeled by a discrete Markov chain. Estimating the transition probabilities of such a discrete Markov chain is rather straightforward when observational data are available for every period of the chain. Although bridges in Switzerland should ideally be inspected every 5 years, inspection intervals are irregular in practice, varying between 3 and 10 years. To overcome the resulting missing data problem, an Expectation Maximization (EM) algorithm is applied to estimate the transition probabilities. Further, the comparison of preservation strategies requires information about the effectiveness of preservation interventions. KUBA takes intervention effectiveness into account as the probability that a structural element will be in a certain condition after the intervention has been carried out. If historical data are available for the elements' condition states immediately before as well as immediately after the preservation interventions, then estimating these probabilities is straightforward. However, an element's condition is usually only known at the oldest inspection before the intervention, and the youngest inspection afterwards. Therefore, starting from those two inspections, the element's expected condition state immediately before and immediately after the intervention is computed with its deterioration Markov matrix, in order to estimate the expected intervention effectiveness. Finally, using historical data on damage and intervention extents, a piecewise regression approach is employed to estimate the relation between damage and preservation intervention extents. The paper also presents some illustrative computational results to underline the practical applicability of the methods.

The Swiss Bridge Management System (KUBA 4.0) provides decision support in planning preservation interventions, and consists of two principal components: the database system KUBA-DB for information on the bridge inventory and the management system KUBA-MS for computing and simulating long-term optimal preservation strategies at the structural element level (*1*).

In order to compare preservation strategies, the system requires forecasts about the future development of a structural element's condition state. In KUBA-MS, the deterioration of a structural element is modeled by a discrete Markov chain.

Further, the comparison of preservation strategies requires information about the effectiveness of preservation interventions. KUBA-MS takes intervention effectiveness into account as the probability that a structural element will be in a certain condition after the intervention has been carried out.

So far, the main probabilistic parameters of KUBA-MS, being the deterioration and effectiveness probabilities, have been determined by experts. In the near future, KUBA-DB will be used to gather and store element-level inspection data, providing access to a large data set of observed element condition states. This paper describes two algorithms for statistically estimating the deterioration and effectiveness matrices from such inspected condition states.

Further, KUBA-DB will also gather information on the cost and extents of realized preservation projects. Based on these data, the paper proposes a third algorithm to statistically estimate the relation between damage and preservation intervention extents. This function is of particular importance for simulating the effect of a long-term optimal preservation strategy on the short-term condition states of the current bridge inventory and its structural elements in KUBA-MS.

ORGANIZATION OF THE PAPER

The first section briefly describes the Markov chain and the Markov decision process that lie at the core of KUBA-MS. The next section considers the condition state transition probabilities of the Markov chain, and proposes an Expectation Maximization algorithm to estimate maximum likelihood estimators for these probabilities from inspection data. Then the related problem of estimating intervention effectiveness probabilities for the Markov decision process is studied. An algorithm is proposed that uses the condition state probabilities to predict condition states for the points in time immediately before and after a preservation intervention, which then serve as input to estimate intervention effectiveness. Next, the relation between the damage extent and severity on a structural element is described along with the extent of a preservation intervention performed to remedy this damage. To estimate the parameters of this relation, a piecewise regression algorithm is proposed. Finally, conclusions are provided and some directions for future research are discussed.

THE MARKOV MODEL IN KUBA-MS

The deterioration of a structural element through its condition states is modeled by a discrete Markov chain $\{X_t\}_t = 1,2,\ldots$, with $X_t = i$ representing the structural element being in state i at time t. If the structural element is in state i at time t, it transitions into state j at time $t + 1$ with conditional probability $P[X_{t+1} = j|X_t = i] = P_{ij}$, which is independent of the history $X_1, X_2, \ldots, X_{t-1}$. The probabilities P_{ij} are called the deterioration transition probabilities, or just deterioration probabilities, and the matrix P is called the deterioration (probability) matrix.

Based on the Markov chain $\{X_t\}$, optimal preservation interventions are determined using a Markov Decision Process (MDP). Whenever the chain is in state X_t at time t, an action a must be chosen from a set of actions A, which can either be an action corresponding to a preservation intervention, or the do-nothing action, implying that the action simply yields deterioration process given by the Markov chain $\{X_t\}$. The choice for an action a has two consequences. First, it determines the probabilities $P[X_{t+1} = j|X_t = i,a_t = a] = P_{ij}(a)$ of transitioning into each state j after choosing the action $a_t = a$. Second, it causes a cost $C(i,a)$. The probabilities $P_{ij}(a)$ are called the effectiveness transition probabilities of the preservation intervention a, or just the effectiveness probabilities of *a*, and the matrix $P(a)$ is called the effectiveness (probability) matrix of the preservation intervention *a*.

KUBA-MS determines the long-term cost-optimal preservation strategy A^* for each structural element, consisting of an action $a^*(i)$ for each state *i*, such that the discounted sum of costs of infinitely following the strategy $A^* = \{a^*(i)\}$ is minimal. For details on Markov chains and Markov decision processes, refer to Sherlaw-Johnson et al. (*2*).

ESTIMATING DETERIORATION PROBABILITIES

So far, the deterioration transition probabilities that are used in KUBA-MS stem from expert judgments. As of 2008, Swiss bridge inspection data will be gathered in the database system KUBA-DB, providing a continuously growing set of condition deterioration field data. This section describes how such inspection field data can be exploited to estimate the deterioration probabilities P_{ij}.

The Markov deterioration model in KUBA-MS considers 5-year periods, which equals the desired average time span between bridge inspections. In practice, however, inspections do not take place exactly every 5 years, and the time span between subsequent inspections can be in the range of 3 to 10 years. To overcome the resulting a-synchronicity between the inspection data and the deterioration model, the estimation model considers 1-year periods instead of 5-year periods. In a perfect world, where inspection data are available for each structural element for every time period, the relative frequency count is a maximum likelihood estimator for the deterioration probability P_{ij}:

$$\widehat{P}_{ij} = \frac{N_{ij}}{\sum_k N_{ik}}$$

Here, N_{ij} denotes the number of transitions, from one year to the next, from state *i* to state *j* in the data set. Having estimated the 1-year deterioration matrix $\widehat{P}$ the 5-year deterioration matrix is computed as $\widehat{P}^5$.

Unfortunately, the switch from 5-year periods to a 1-year inspection model yields a situation in which most of the yearly inspection data are missing for a given structural element. If, for example, a certain structural element was inspected in the year 1997 and subsequently again in 2004, then its inspection data for the periods 1998 until 2003 will be missing. Note that, on the other hand, it is also unclear how such a situation should be taken care of under a 5-year-period model. To deal with the problem of missing data, an Expectation Maximization (EM) algorithm is applied that iteratively computes an estimate for the deterioration probabilities, uses that estimate to predict the missing inspection data, and then refines its estimate using the computed full data set.

The Expectation Maximization Algorithm

Sherlaw-Johnson et al. (*2*) proposed an explicit EM algorithm for computing maximum likelihood estimates for the transition probabilities of a discrete Markov Chain. This algorithm was implemented and tested for KUBA-MS. As main input, the EM algorithm takes the following inspection matrix I that contains the inspection results for all structural elements of a certain element type for each year in the considered time horizon $T = \{0,\ldots,T_{max}\}$:

$$I_{xt} = \begin{cases} s \in \{1,2,\ldots\} & \text{if element } x \text{ was inspected and classified in condition state } s \text{ in period } t \in T \\ \cdot & \text{if element } x \text{ was not inspected in period } t \in T \end{cases}$$

The matrix below shows an example of an inspection matrix for 4 structural elements under a five state condition classification $s \in \{1,\ldots,5\}$ for a 10-year time horizon $T = \{0,\ldots,9\}$.

$$I = \begin{pmatrix} 1 & \cdot & \cdot & 1 & \cdot & 3 & \cdot & 4 & \cdot & \cdot \\ \cdot & \cdot & 1 & \cdot & \cdot & \cdot & 2 & 3 & \cdot & 4 \\ \cdot & 1 & \cdot & 1 & \cdot & \cdot & \cdot & \cdot & 4 & \cdot \\ 1 & \cdot & 2 & \cdot & \cdot & \cdot & 5 & \cdot & 5 & \cdot \end{pmatrix}$$

Here, the first structural element (first row) was first inspected in period 1 and classified in condition state 1. The element was not inspected in period 2 and 3, and inspected for the second time in period 4, when it was again classified in condition state 1. It was inspected for the third time in period 6 and classified in condition state 3, and for the last time in period 8 and classified in condition state 4.

In fact, the algorithm does not use the inspection matrix I directly, but computes the observed number of transitions O_{ijt} given by I from state i to state j that occurred over the time span t, with $t = \{1,2,\ldots,10\}$. For the above example matrix I, this yields $O_{231} = 1$, since I contains only one observed transition from state 2 to state 3 over one period, namely for the second structural element from period 7 to period 8. Also, the value $O_{342} = 1$ is defined by the transition of the first element from period 6 to period 8, and by the transition of the second element from period 8 to period 10.

The EM algorithm proposed by Sherlaw-Johnson et al. (*2*) operates as described in Algorithm 1. Note that both the input and output matrices of the algorithm are in terms of 1-year periods.

Sherlaw-Johnson et al. (*2*) show that the following computations for the Expectation Step and the Maximization Step yield either a local maximum or a saddle point of the likelihood function.

ALGORITHM 1 Estimating Deterioration Probabilities

Input: starting matrix $P^{(0)}$, termination criterion $\in$, observed number of transitions O_{ijt}
Output: estimate $\hat{P}$ of the Markov chain transition matrix

---

set $t = 0, P^{(-1)}$ as the matrix with all entries $P_{ij}^{(t)} = \in$

while $\left|P_{ij}^{(t)} - P_{ij}^{(t-1)}\right| \geq \in$ for some i,j *do*

Expectation Step: Use $P^{(t)}$ to compute an estimate $\hat{N}_{ij}(P^{(t)})$ of the complete data set

Maximization Step: Compute $P^{(t+1)}$ as the frequency count maximum likelihood estimator of the complete data set $\hat{N}_{ij}(P^{(t)})$

$t = t + 1$

return $P^{(t)}$

Expectation Step

The Expectation Step computes the following estimate of the one-period number of transitions N_{ij}:

$$\widehat{N}_{ij}(P^{(t)}) = \sum_{m,n,t} O_{mnt} \sum_{k=0}^{t-1} Q_{ijk,mnt}$$

where $Q_{ijk,mnt}$ is computed as follows

$$Q_{ijk,mnt} = \frac{[(P]^k)_{mi} P_{ij} [(P]^{t-k-1})_{jn}}{[(P]^t)_{mn}}$$

with P being the current transition matrix estimate $P^{(t)}$.

Maximization Step

The Maximization Step computes the next transition probability estimates $P_{ij}^{(t+1)}$ as the relative frequency counts for the estimated number of transitions $\widehat{N}_{ij}$:

$$P_{ij}^{(t+1)} = \frac{\widehat{N}_{ij}(P^{(t)})}{\Sigma_k \widehat{N}_{ik}(P^{(t)})}$$

For further details, see Sherlow-Johnston et al. (*2*).

The structure of the starting matrix $P^{(0)}$ predetermines the structure of the estimate matrix in the sense that any zero entry $P_{ij}^{(0)}$ will yield that same entry zero in the final estimate matrix $P^{(t)}$ as well as in all intermediary estimates $P^{(1)},\ldots,P^{(t-1)}$. This property should be taken into account when one chooses the starting matrix $P^{(0)}$. In practice, there are several candidates for the starting matrix. One could use a deterioration matrix stemming from expert judgment, such as the ones used in KUBA-MS so far. If the matrix estimation algorithm has been in use for several years already, the most recent estimated deterioration matrix could be used as a starting matrix for a new estimation. Or, a weighted average of an expert judgment matrix and an old estimated matrix could be used.

The termination criterion $\in$ should represent the required precision of the deterioration matrix estimate.

Testing the EM Algorithm

Algorithm 1 was implemented and tested on a data set of 35 structural elements for a time horizon of 26 periods. To test the algorithm, a perfect deterioration data set was constructed first, with condition states for each structural element for every period. About 80% of the entries from this perfect data set were then removed to construct an inspection data set in which every structural element is inspected on average every 5 years. The perfect data set and the inspection data set can be found in Table 1, with bold italic entries referring to inspections, and grey normal entries belonging to the perfect data set only.

The following 1-year deterioration matrix was used as starting matrix for Algorithm 1:

$$P^{(0)} = \begin{pmatrix} 0.8 & 0.2 & 0 & 0 & 0 \\ 0 & 0.6 & 0.4 & 0 & 0 \\ 0 & 0 & 0.3 & 0.7 & 0 \\ 0 & 0 & 0 & 0.5 & 0.5 \\ 0 & 0 & 0 & 0 & 1 \end{pmatrix}$$

With the termination criterion $\in = 0.0001$, the EM algorithm computes the following 1-year deterioration matrix estimate for the inspection data set in Table 1 within about 20 iterations:

$$P^{(t)} = \begin{pmatrix} 0.862 & 0.138 & 0 & 0 & 0 \\ 0 & 0.871 & 0.129 & 0 & 0 \\ 0 & 0 & 0.817 & 0.183 & 0 \\ 0 & 0 & 0 & 0.902 & 0.098 \\ 0 & 0 & 0 & 0 & 1 \end{pmatrix}$$

The relative frequency count maximum likelihood estimator for the perfect data set is:

$$\widehat{P} = \begin{pmatrix} 0.871 & 0.129 & 0 & 0 & 0 \\ 0 & 0.860 & 0.140 & 0 & 0 \\ 0 & 0 & 0.836 & 0.164 & 0 \\ 0 & 0 & 0 & 0.896 & 0.104 \\ 0 & 0 & 0 & 0 & 1 \end{pmatrix}$$

Thus, the maximum absolute difference between the maximum likelihood estimators for the perfect data set and the inspection data set is $\max_{ij} \left| P_{ij}^{(t)} - \widehat{P}_{ij} \right| = 0.019$.

ESTIMATING PRESERVATION INTERVENTION EFFECTIVENESS

Besides the deterioration transition probabilities, the probabilities $P_{ij}(a)$ representing the preservation intervention effectiveness form a second important ingredient for the Markov decision model in KUBA-MS. As with the deterioration transition probabilities, the effectiveness probabilities in KUBA-MS have so far been determined by experts. And as with transition probabilities, the inspection data gathered in KUBA-DB on condition states before and after preservation interventions form a rich data source for estimating the effectiveness probabilities.

TABLE 1 Data Set for Estimating the Deterioration Matrix

Structural element	Period																										No. Insp.
	0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	
1	***1***	1	1	1	***1***	1	1	1	2	2	2	***2***	3	3	3	3	***3***	3	3	3	3	3	***4***	4	4	***4***	5
2	***1***	1	1	2	2	2	***2***	2	2	2	2	***2***	2	2	***2***	2	3	***3***	3	3	3	***3***	3	3	3	***3***	6
3	***1***	1	1	1	1	1	2	***2***	2	2	2	2	***2***	3	3	3	***3***	4	4	4	5	5	***5***	5	5	5	4
4	***1***	1	1	***1***	1	1	2	2	2	2	***3***	3	3	3	4	***4***	4	4	4	4	***4***	4	4	4	4	***4***	5
5	***1***	1	1	1	***1***	1	1	1	1	2	2	***2***	2	2	2	2	***2***	2	2	3	3	4	***4***	5	5	***5***	5
6	***1***	1	1	***1***	2	2	2	2	***2***	2	2	3	3	3	4	***4***	4	5	5	***5***	5	5	5	5	***5***	5	5
7	***1***	1	1	2	2	***2***	3	3	3	***3***	4	4	5	5	***5***	5	5	5	***5***	5	5	5	***5***	5	5	5	5
8	***1***	1	1	***1***	1	1	1	1	***1***	1	2	2	2	***2***	3	3	3	3	3	***3***	3	3	3	***4***	4	4	5
9	***1***	1	1	***1***	1	2	***2***	2	3	3	3	***3***	3	3	4	***4***	4	4	4	4	4	***4***	4	4	4	***4***	6
10	***1***	1	1	1	1	***2***	2	2	2	3	***3***	3	3	3	3	3	***3***	4	4	4	4	5	***5***	5	5	5	4
11	***1***	1	1	2	2	***2***	2	2	3	3	***3***	4	4	4	***4***	4	5	5	5	5	***5***	5	5	***5***	5	5	5
12	***1***	1	1	1	1	1	1	1	1	1	1	***1***	1	1	1	2	***2***	2	2	2	2	***3***	3	3	3	***3***	4
13	***1***	1	1	1	1	1	1	***1***	1	1	1	***1***	1	1	2	***2***	2	2	3	3	3	***3***	3	3	3	***3***	5
14	***1***	1	1	1	1	1	1	***1***	1	2	2	***2***	2	2	2	2	***2***	3	3	3	3	***3***	4	4	4	***4***	5
15	***1***	1	1	1	***1***	1	1	1	2	***2***	2	2	2	2	2	2	2	2	***2***	3	3	***3***	3	3	3	3	4
16	***1***	1	1	2	2	2	2	***2***	2	2	2	2	2	***2***	2	2	***3***	3	3	3	3	***4***	4	4	4	***4***	5
17	***1***	1	1	1	1	1	1	1	1	***1***	1	2	2	2	2	3	***3***	3	3	4	***4***	4	4	5	5	5	3
18	***1***	1	1	1	1	1	1	1	***1***	1	1	1	***1***	1	2	2	2	2	2	***2***	2	2	3	3	***3***	3	4
19	***1***	1	1	1	***1***	1	1	1	1	***2***	2	2	2	2	2	2	2	***2***	2	2	2	***2***	3	3	***3***	3	5
20	***1***	1	1	***1***	1	1	1	1	1	1	1	***1***	1	1	1	1	2	***3***	3	3	4	4	***4***	4	4	***4***	5
21	***1***	1	1	2	***2***	2	3	3	***3***	3	4	4	4	***4***	4	4	4	4	***4***	4	4	4	4	4	***4***	4	5
22	***1***	1	1	1	***1***	1	2	2	2	2	***2***	2	2	2	***3***	4	4	4	***4***	4	4	4	4	4	4	***4***	5
23	***1***	1	1	1	1	***1***	1	1	1	***2***	2	2	2	3	4	***4***	4	4	5	5	5	5	***5***	5	5	***5***	5
24	***1***	1	1	1	***2***	2	2	2	2	***2***	3	3	3	***3***	3	3	4	4	4	4	***4***	5	5	5	5	***5***	5
25	***1***	1	1	1	1	1	1	***1***	1	1	1	2	***2***	2	2	***3***	3	4	***4***	4	4	4	***4***	4	4	***4***	6
26	***1***	1	1	2	2	2	***3***	3	3	***4***	4	4	5	***5***	5	5	5	5	***5***	5	5	5	5	5	5	***5***	5
27	***1***	1	1	1	***1***	1	1	2	2	***2***	2	3	3	3	3	3	***3***	4	4	4	5	***5***	5	5	5	***5***	5
28	***1***	1	1	2	2	***2***	2	2	2	2	***2***	3	3	3	3	4	4	***4***	4	4	***4***	4	4	4	4	***5***	5
29	***1***	1	***1***	1	1	1	***1***	2	2	2	2	***2***	2	2	2	***2***	2	2	3	***3***	3	3	4	4	***4***	4	6
30	***1***	1	1	1	***2***	2	2	2	2	2	2	2	2	***2***	2	2	2	***2***	2	2	2	3	***3***	3	3	***3***	5
31	***1***	1	1	***1***	1	1	1	2	***2***	2	2	3	3	***3***	3	3	3	***3***	3	3	4	***4***	4	5	5	***5***	6
32	***1***	1	1	1	1	1	1	***1***	1	1	1	1	1	1	2	***2***	2	2	2	2	2	***2***	2	2	2	2	3
33	***1***	1	1	1	***1***	1	1	1	1	1	***1***	1	1	1	1	1	1	1	***2***	2	2	2	***2***	2	2	***2***	5
34	***1***	1	1	1	1	1	***1***	1	2	2	***2***	2	2	2	2	***2***	3	3	3	***3***	3	3	3	4	4	***4***	5
35	***1***	1	1	1	1	1	***2***	2	2	2	***2***	2	***3***	3	3	3	***3***	3	4	4	***4***	4	4	4	***5***	5	6
																							total				172

The accurate estimation of intervention effectiveness requires inspection data for both the points in time immediately before and after the preservation intervention is carried out. If, for a given preservation intervention, such data are available for several structural elements of a certain element type, then that intervention's effectiveness for the element type is estimated by the relative frequency counts (similar to $\widehat{P}_{ij}$ in the preceding section).

However, the last inspection before the preservation intervention may, and typically does, take place several years before the time point of the intervention itself. Moreover, the control inspection after a preservation intervention may take place 1 or 2 years after the preservation intervention has been finished. For those reasons, preservation intervention effectiveness is estimated as follows. Using the deterioration matrix P for the considered element type, the expected condition state of each structural element immediately before the intervention is predicted from the oldest inspection before the intervention. Similarly, the expected condition state of an element immediately after the intervention is traced back from the youngest inspection after the intervention. Then, intervention effectiveness is estimated based on the expected condition states for the time points immediately before and after the preservation intervention.

The Preservation Intervention Effectiveness Procedure in Detail

To describe the procedure for estimating preservation intervention effectiveness in detail, define the following parameters:

$t_m(x)$	The period in which structural element x undergoes the preservation intervention
$t_{old}(x)$	The period in which the oldest inspection of structural element x before the intervention took place
$t_{young}(x)$	The period in which the youngest inspection of structural element x after the intervention took place
$s_{old}(x)$	The condition state vector for structural element x in period t_{old}
$s_{young}(x)$	The condition state vector for structural element x in period t_{young}

Note that $s_{old}(x)$ and $s_{young}(x)$ are unit vectors. Additionally, define the following two variables:

$s_{before}(x)$	The expected condition state vector for structural element x immediately before the preservation intervention
$s_{after}(x)$	The expected condition state vector for structural element x immediately after the preservation intervention
$e(x)$	The expected effectiveness matrix of the preservation intervention for structural element x
$E(X)$	The estimated effectiveness matrix of the preservation intervention for the structural element type X

The condition state transition of structural element x from period $t_{old}(x)$ to period $t_m(x)$ is given by the deterioration matrix $P^{t_m(x)-t_{old}(x)}$, and hence $s_{before}(x)$ is computed as

$$s_{before}(x) = (P^{t_m(x)-t} old^{(x)})^T \bullet s_{old}(x)$$

However, $s_{after}(x)$ cannot be computed so easily. Ideally, one would trace back the condition vector $s_{young}(x)$ in period $t_{young}(x)$ to the vector $s_{after}(x)$ for the time point $t_m(x)$ using the reverse process of the Markov chain $\{X_t\}$. Unfortunately, the Markov chain $\{X_t\}$ is not in general time reversible (*2*), due to its absorbing last condition state, as well as the upper-triangular structure of its Markov matrix. Still, the inverse P^{-1} of the Markov matrix might be used as an approximation of the reversed Markov process, in which case it has to be adjusted using some normalization scheme since P^{-1} itself may contain negative entries and row sums larger than one. Instead, KUBA-MS employs a simpler heuristic, and traces the condition vector $s_{young}(x)$ back to the vector $s_{after}(x)$ using the normalized transpose of the Markov matrix. Thus, the Markov matrix entry P_{ji} is interpreted as the (proportional) probability to go back in time from state i to state j. To that end, compute the backtracking transition matrix B as

$$B_{ij} = \frac{P_{ji}}{\sum_k P_{ki}}$$

That is, B is the transpose of P, normalized to a (row-) stochastic matrix.

The vector $s_{after}(x)$ is heuristically computed by interpreting the backtracking matrix B as the transition matrix of the reverse process of the Markov chain, and tracing back the steps of the deterioration process using the following equation:

$$s_{after}(x) = (B^{t_{young}(x)-t_m(x)})^T \bullet s_{young}(x)$$

With these two expressions for the expected condition states immediately before and immediately after the preservation intervention, the expected intervention effectiveness $e(x)$ is estimated as

$$e(x) = s_{before}(x) \bullet s^T_{after}(x)$$

The procedure for estimating the effectiveness of the structural element type X proceeds as described below in Algorithm 2. Essentially, the algorithm calculates the expected intervention effectiveness for each structural element, adds the results of these computations, and normalizes the resulting structural element type effectiveness matrix $E(X)$. For simplicity, the description assumes that every structural element of type X has received exactly one preservation intervention, but the procedure easily applies to structural elements that have received multiple interventions, and any structural element without a preservation intervention should just be omitted.

In KUBA-MS, it is assumed that a structural element can only transition into two conditions states after a preservation intervention, which are typically the condition states 1 and 2. Therefore, any probability mass $E_{ij}(X)$ for entries with $j \geq 3$ is proportionally redistributed to the entries $E_{i1}(X)$ and $E_{i2}(X)$ in a post-processing step.

ALGORITHM 2 Estimating Intervention Effectiveness Probabilities

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Input: for each structural element $x \in X$ the parameters $t_m(x)$, $t_{last}(x)$, $t_{first}(x)$, $s_{last}(x)$, $s_{first}(x)$
deterioration transition matrix P
Output: estimated effectiveness matrix $E(X)$ for the structural element type X

---

set $E(X)$ equal to the all-zero matrix
for each structural element x of the element type X *do*
compute $s_{before}(x)$ and $s_{after}(x)$
compute $e(x)$
update $E(X) := E(X) + e(x)$
normalize $E(X)$ to make it a stochastic matrix
return $E(X)$

Testing the Intervention Effectiveness Estimation

For testing the above estimation procedure, an approach similar to the one for Algorithm 1 was followed. First, a perfect data set was constructed with the condition states for each of 35 structural elements for all 12 periods, 6 of which lie before the starting time point of the preservation intervention in period 5, and 6 after the finishing time point in period 6. Then, for each structural element all but one condition state were removed for the time periods before the intervention to obtain a single inspection in that time window. Similarly, one single inspection remains for each structural element for the time window after the intervention. In this way, the inspection data set in Table 2 was constructed, with bold italic entries referring to the last inspections before and after the preservation intervention, and grey normal entries belong to the perfect data set. Then, Algorithm 2 was applied with the deterioration matrix $P^{(0)}$ from the earlier section.

The following expected effectiveness matrices E were computed:

$$E = \begin{pmatrix} 1 & 0 & 0 & 0 & 0 \\ 1 & 0 & 0 & 0 & 0 \\ 0.646 & 0.354 & 0 & 0 & 0 \\ 0.503 & 0.497 & 0 & 0 & 0 \\ 0.341 & 0.659 & 0 & 0 & 0 \end{pmatrix}$$

The relative frequency counts estimator $\widehat{E}$ for the perfect data set was:

$$\widehat{E} = \begin{pmatrix} 1 & 0 & 0 & 0 & 0 \\ 1 & 0 & 0 & 0 & 0 \\ 0.710 & 0.290 & 0 & 0 & 0 \\ 0.580 & 0.330 & 0.08 & 0 & 0 \\ 0.360 & 0.500 & 0.14 & 0 & 0 \end{pmatrix}$$

TABLE 2 Data Set for Estimating the Effectiveness Matrix

Structural element	**Period**											**Inspection interval**
	0	1	2	3	4	***5***	***6***	7	8	9	10	11	
1	3	3	***4***	4	4	4	1	1	2	2	***2***	2	8
2	3	***3***	3	3	3	3	2	2	***2***	2	2	2	7
3	5	5	5	***5***	5	5	2	2	***2***	2	2	2	5
4	***4***	4	4	4	4	4	2	2	2	***2***	3	3	9
5	3	4	4	***5***	5	5	1	1	1	2	2	***2***	8
6	5	5	***5***	5	5	5	2	***2***	2	2	2	3	5
7	5	5	5	***5***	5	5	3	3	3	***3***	4	4	6
8	3	***3***	3	4	4	4	***1***	1	1	1	2	2	5
9	***4***	4	4	4	4	4	2	***2***	3	3	3	3	7
10	4	5	5	***5***	5	5	2	2	***2***	3	3	3	5
11	5	5	5	5	***5***	5	2	***2***	3	3	3	4	3
12	2	3	3	3	***3***	3	1	1	1	1	***1***	1	6
13	3	3	***3***	3	3	3	1	1	1	1	1	***1***	9
14	3	***3***	4	4	4	4	1	1	1	2	***2***	2	9
15	3	3	3	***3***	3	3	1	1	***2***	2	2	2	5
16	3	4	***4***	4	4	4	2	2	2	***2***	2	2	7
17	***4***	4	4	5	5	5	***1***	1	1	1	1	2	6
18	2	2	3	***3***	3	3	1	1	***1***	1	1	1	5
19	2	2	3	3	***3***	3	1	1	1	2	***2***	2	6
20	4	4	***4***	4	4	4	1	1	***1***	1	1	1	6
21	4	4	4	***4***	4	4	3	3	3	***3***	4	4	6
22	4	4	4	***4***	4	4	***2***	2	2	2	2	2	3
23	***5***	5	5	5	5	5	1	1	***1***	2	2	2	8
24	4	***5***	5	5	5	5	2	2	2	2	***3***	3	9
25	4	***4***	4	4	4	4	1	1	1	1	1	***2***	10
26	5	5	5	***5***	5	5	3	3	3	***4***	4	4	6
27	***5***	5	5	5	5	5	1	2	2	2	***2***	3	10
28	4	4	4	4	4	***5***	2	***2***	2	2	2	3	2
29	3	3	4	4	***4***	4	1	2	2	***2***	2	2	5
30	2	3	3	3	3	***3***	2	2	***2***	2	2	2	3
31	4	4	***4***	5	5	5	***1***	2	2	2	2	3	4
32	2	***2***	2	2	2	2	1	1	***1***	1	1	1	7
33	2	2	***2***	2	2	2	1	1	1	***1***	1	1	7
34	3	3	3	***4***	4	4	1	1	***2***	2	2	2	5
35	4	4	4	4	5	***5***	2	2	2	2	***2***	2	5
												Average	6

RELATION BETWEEN DAMAGE EXTENT AND PRESERVATION INTERVENTION EXTENT

Besides computing the optimal long-term preservation strategies, KUBA-MS also offers the possibility to simulate preservation strategies for a number of periods, thus providing an overview of the shorter-term effects of a preservation strategy for a certain bridge park.

Recall that a deterministic optimal preservation strategy always exists for a Markov decision process, that is, a strategy that assigns a single optimal preservation intervention $a^*(i)$ (or the do-nothing action) to each condition state i (*3*). The optimal strategy a^* thus dictates that whenever a structural element is in state i, the preservation intervention $a^*(i)$ must be carried out. In practice, however, a preservation intervention will only be carried out if the damage extent in condition state i is sufficiently large. Moreover, if the damage extent in condition state i is large enough for preservation intervention $a^*(i)$ to be carried out, it is usually not only applied to the damage extent in condition state i, but also to a part of the surroundings of that damage. In fact, if the damage extent is substantial, the preservation intervention may be applied to the entire structural element.

Keeping the above approach in mind, a damage extent of fraction d of the structural element extent is said to trigger a preservation intervention extent of fraction $f(d)$ that, for simulation purposes, is modeled by the following intervention extent function, and illustrated in Figure 1:

$$f(d) = \begin{cases} 0 & \text{for} \quad 0 \le d < S \\ ad + b & \text{for} \quad S \le d \le L \\ 1 & \text{for} \quad L < d \le 1 \end{cases}$$

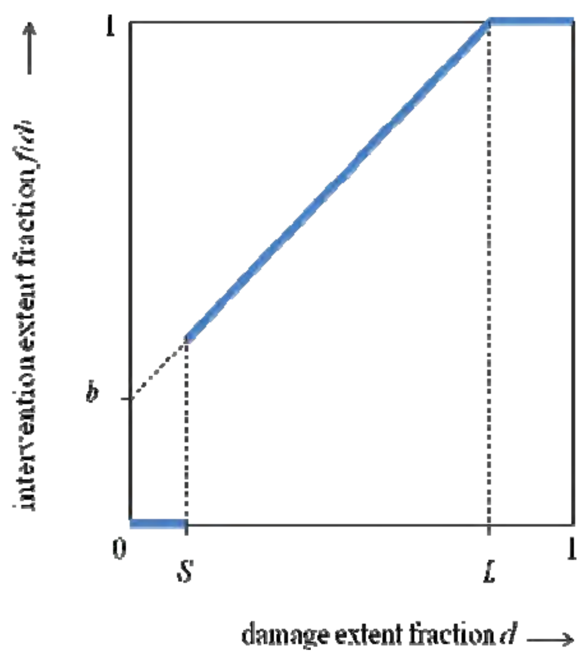

FIGURE 1 Intervention extent function (thick line).

Here, S is the structural element fraction up until which the damage extent is considered too small to trigger an intervention, L is the structural element fraction from which the damage extent is considered so large that the entire element undergoes the intervention, and a and b define the linear function relating damage extent to preservation intervention extent for damages between the fractions S and L.

Estimating the Intervention Extent Function by Piecewise Linear Regression

Based on a data set of n observed preservation interventions with intervention extent fractions $f_1,\ldots,f_n$ and the corresponding damage extent fractions $d_1, \ldots, d_n$ for a given intervention type and structural element type, the function $\hat{f}(d)$ is estimated with a modified regression procedure, which is inspired by the so-called piecewise linear regression (*4*). The modification stems from the fact that the general "hockey stick" shape of the function $f(d)$ is known, and the only parameters to be estimated are the locations of the breakpoints $\hat{S}$ and $\hat{L}$, and the slope $\hat{a}$ and offset $\hat{b}$ of the linear function $ad + b$.

Recall that in classical linear regression, the objective is to find estimated parameters that minimize the sum of squared errors (SSE) between the observed values and the function values computed with the estimated parameters. The proposed piecewise linear regression procedure employs a similar objective function as follows. For given values of S,L,a, and b, the regression objective is to minimize the resulting total SSE defined as

$$SSE(S,L,a,b) = SSE_0(S,L) + SSE_1(S,L) + SSE_R(S,L,a,b)$$

where

$SSE_0(S,L) =$	$\sum_{i:0\le d_i<S} (f_i)^2$	the SSE for the data points with $0 \le d_i < S$
$SSE_1(S,L) =$	$\sum_{i:L<d_i\le 1} (1-f_i)^2$	the SSE for the data points with $L < d_i \le 1$
$SSE_R(S,L,a,b) =$	$\sum_{i:S\le d_i\le L} (ad_i + b - f_i)^2$	the SSE for a classical linear regression on the data points with $S \le d_i \le L$

To find the estimators $\hat{S}, \hat{L}, \hat{a},$ and $\hat{b}$ that minimize the regression function $SSE(S,L,a,b)$ the estimation algorithm enumerates all sensible values of S and L If the observed data points are sorted in increasing order of d_i, that is, $d_1 \le d_2 \le \ldots \le d_n$, one should in principle consider all values $S = \{d_1, \ldots,d_n\}$, and for a value S = d_i, all values $L = \{d_{i+1}, \ldots,d_n,1\}$ to be guaranteed to find the minimum of the function $SSE(S,L,a,b)$.

However, for the level of detail required for the simulations in KUBA-MS, it suffices for the breakpoints S and L to be known up to a precision of two decimals. Thus, the estimation algorithm enumerates the values $S = 0.01k$ and $L = 0.01k'$ for $k = 0, \ldots, 100$, $k' = k + 1, \ldots,100$. For each considered pair of values S,L, it computes the corresponding scores SSE_0 and SSE_1, as well as a classical linear regression for the data points with $S \le d_i \le L$, yielding the score SSE_R.

Finally, it returns those values $\widehat{S}, \widehat{L}, \widehat{a},$ and $\widehat{b}$ for which the total regression score $SSE = SSE_0 + SSE_1 + SSE_R$ is minimal. Algorithm 3 describes the resulting piecewise regression algorithm.

Testing the Piecewise Linear Regression Algorithm

For illustrative purposes, Algorithm 3 was tested on a data set for 45 interventions, with the damage and intervention extent fractions shown in Figure 2. The estimated intervention extent function $\widehat{f}(d)$ is also shown in Figure 2, as a thick line, and has estimated breakpoints $\widehat{S} = 0.14$ and $\widehat{L} = 0.70$, and estimated slope $\widehat{a} = 1.3316$ and offset $\widehat{b} = 0.056$.

In order to test the effectiveness of estimating *S* and *L* up to two decimals only, a second test was performed for a data set of 7,000 interventions. For this data set, the computation time for Algorithm 3 was less than a second. In contrast, the computation time for a variant of the algorithm that considered all sensible values for *S* and *L*, as described above, was in the order of 10 min. The estimated breakpoints of both algorithms did not show a significant difference.

CONCLUSION AND FUTURE WORK

The database component KUBA-DB of the Swiss road structures management system KUBA 4.0 gathers and stores data regarding the inspected condition states and damage extents of structural elements, as well as regarding preservation interventions, intervention extents, and their effects. This wealth of data provides a valuable source for estimating various parameters of the management system component KUBA-MS, as an addition or alternative to expert opinions' parameter estimates. This paper considered three such parameters: the deterioration and effectiveness probabilities of the Markov decision process underlying KUBA-MS, and the shape of the function relating damage extent to preservation intervention extent. The paper proposed statistical algorithms for estimating

ALGORITHM 3 Estimating the Intervention Extent Function *f*(*d*)

Input: observed data sets $d_1, \ldots, d_n$ and $f_1, \ldots, f_n$
Output: parameter estimates $\widehat{S}, \widehat{L}, \widehat{a},$ and $\widehat{b}$ for the function $\widehat{f}(d)$

set $BES_{SSE} = \infty$
for $k = 0, \ldots, 100$ *do*
for $k' = k'+1, \ldots, 100$ *do*
    set $\overline{S} = 0.01k$, $\overline{L} = 0.01k'$
    compute classical linear regression for data points with $\overline{S} \leq d_i \leq \overline{L} \rightarrow$ gives $\overline{a}, \overline{b}$
    if $SSE(\overline{S}, \overline{L}, \overline{a}, \overline{b}) < BEST_{SSE}$ *then*
        update $BEST_{SSE}: = SSE(\overline{S}, \overline{L}, \overline{a}, \overline{b})$
        update $\widehat{S} = \overline{S}, \quad \widehat{L} = \overline{L}, \quad \widehat{a} = \overline{a}, \quad \widehat{b} = \overline{b}$
return $\widehat{S}, \widehat{L}, \widehat{a}, \widehat{b}$

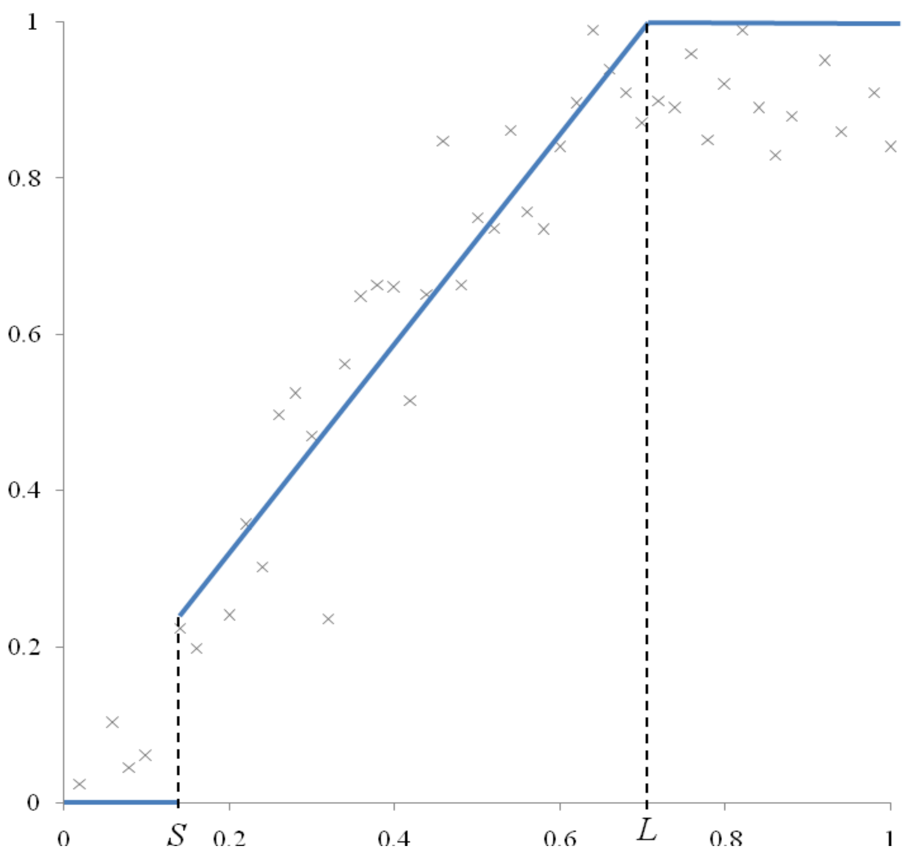

FIGURE 2 Estimated intervention extent function $\hat{f}(d)$ (thick line) for a data set of 45 interventions (x).

each of these three parameters from the data provided by KUBA-DB. Further, the deterioration and effectiveness probability estimation were illustrated with some example data.

For the future, the proposed algorithms will need to be validated using the actual data gathered in KUBA-DB. Based on the validation results, the algorithms may need slight modification, and algorithm input parameters will require fine tuning. Moreover, the validation results may indicate certain flaws in the gathered data and the data collection (structure inspection, preservation project protocols), in which case the data collection process may require a review.

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National Database System for Maintenance Actions on Highway Bridges

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One of the enduring challenges of bridge management has been the need for procedures and technical methods for capturing data on the implementation of bridge maintenance, repair, and rehabilitation work and using this information to improve forecasting models. Often characterized as the feedback loop of bridge management, such procedures and methods, if they can be developed and implemented, would greatly enhance the potential for long-term success of structure management strategies. National Cooperative Highway Research Program Project 14-15 has directly addressed this need. The project viewed the problem as capturing data from routine maintenance management processes and converting them for use in bridge management systems. This conversion required the establishment of an intermediate classification scheme—a bridge maintenance catalog—that is compatible with existing maintenance management systems but also compatible with bridge management systems such as AASHTO's Pontis. Using work accomplishment data classified according to the standard catalog, conventional cost accounting techniques could be applied to convert measurement units and to relate quantitative condition data and economic inputs, to economic outputs and condition outcomes. A software system using the technologies of Microsoft Excel and Access, Visual Basic, eXtensible Markup Language, and Javascript, was developed to demonstrate the conversion procedures. The software was applied to data from several state departments of transportation to compute bridge management system unit costs, action effectiveness measures, and other performance measures useful for bridge management.

Definitions of bridge maintenance were collected from AASHTO (*1*, *2*), from published materials of state transportation departments, and from NCHRP reports. Bridge maintenance can be defined by at least four means:

- Policy: descriptive concepts of the kinds of work and outcomes that are maintenance.
- Action: maintenance denoted by lists of maintenance crew actions and maintenance contract pay items.
- Budgets: maintenance projects identified by the source of funding and by the kind, if any, of federal participation in funding.
- Data: maintenance identified, and perhaps limited, by the capabilities of data systems used for maintenance management.

MAINTENANCE DEFINITIONS

Maintenance Defined by Policy

Policy-level definitions of maintenance were obtained from AASHTO (*1, 2*), from FHWA and from nine state transportation agencies.

According to AASHTO's guide to maintenance management systems (*2*), maintenance is any activity other than new construction. AASHTO's maintenance manual (*1*) more narrowly defines maintenance as routine upkeep and relatively small repairs that keep bridges in good condition. Maintenance actions include routine cleaning and painting as well as repairs and replacements of components. FHWA (*3*) recognizes these same activities as maintenance, but identifies routine maintenance by state departments of transportation (DOTS) as ineligible for Highway Bridge Replacement and Rehabilitation (HBRRP) funds. Indeed, major maintenance can cause a structure that is eligible for federal HBRRP funding to drop off of the eligible list for a period of 10 years.

Jorgensen's NCHRP report on budgeting for highway maintenance (*4*) defines maintenance as actions that preserve assets in their as-constructed condition, a statement that excludes improvements to existing structures as well as new construction. State DOTS often define maintenance in a similar narrow sense: maintenance preserves bridges and can restore bridges to original condition. New construction is excluded. So too are betterments: actions or projects that increase capacity or improve function of bridges.

California DOT (*5*) states that maintenance does not include reconstruction or improvements. Idaho DOT (*6*) considers improvements to be part of maintenance. Michigan DOT (*7*) notes that maintenance projects are of short duration and have little impact on traffic operations. Montana DOT (*8*) states that maintenance preserves the originally intended use and function of bridges. Ohio DOT (*9*) states that maintenance aims to keep bridges in original constructed condition. Oregon DOT (*10*) identifies preserving, repairing, and restoring as maintenance. Texas DOT (*11*) identifies maintenance in three categories: routine, preventive, and major. Major maintenance includes bridge replacement and bridge reconstruction. Washington state DOT (*12*) identifies normal maintenance including cleaning and minor repairs.

Based on stated policies, cleaning and minor repairs are always maintenance. Repairs or replacements of components are often classified as maintenance. Improvements achieved in small projects might be maintenance. Larger projects for improvement, bridge reconstruction, and bridge replacement are not maintenance. New construction is not maintenance.

Maintenance by List of Actions

A review of maintenance actions as presented by AASHTO and by state DOTs reveals seven common operations in bridge maintenance (Table 1).

Most state DOTs identify maintenance actions in all operations shown in Table 1, though terms vary among DOTs. Some DOTs identify minor repair and major repair rather than Repair and Replace. Some DOTS describe betterments, instead of Modify actions. Some DOTs have separate categories for maintenance of movable spans, of motion equipment, of tunnels, and of other structural assets. In general, bridge replacement and reconstruction are excluded from maintenance. Improvements to bridges may be maintenance, if projects are small. Improvements

TABLE 1 Common Operations in Bridge Maintenance

Actions	Description
Clean, Clear	Include sweeping, flushing, removal of incompressibles, removal of vegetation, removal of material in channels, and all similar operations
Seal, Paint, Coat	Provide spot, partial, or complete application of fluid sealers, paints, coatings, or preservatives
Reset	Include repositioning, lubrication, tightening (of bolts and rods), and other minor corrective actions
Repair	Return elements to better condition, and perhaps to as-built condition. Patching is a repair action
Replace	Include replacement in kind of all or part of elements
Modify	Consist of repairs or replacements that alter elements
Emergency	Take place in response to sudden acute problems that must be corrected to restore or continue traffic operations

are not maintenance if projects are large. Modify actions, within maintenance programs, can include replacement of obsolete railings, extension to drain outlets, and relocation of bracing in truss portal frames.

Maintenance Defined by Budget

Budgets in transportation departments identify funds for the maintenance division, for contract maintenance, and for equipment and materials used in maintenance tasks. In a simple sense, the actions and projects funded by DOTs as maintenance are strictly maintenance. The federal HBRRP program has an impact here. Bridge replacement or major rehabilitation projects that are eligible for HBRRP funds are not maintenance. At the same time, projects that extend the life of bridges are maintenance and can be HBRRP eligible. These projects usually entail repair, (element) replacement, or minor modification.

From the DOT budget perspective, cleaning and other routine upkeep are always maintenance. Repairs, component replacements, and minor modifications are usually maintenance and may be eligible for HBRRP funds. Bridge replacement and major rehabilitations are not maintenance. Any project that affects the HBRRP eligibility of a structure is not maintenance.

Maintenance Defined by Database

The capabilities of data systems can impose limits on the work that is tracked as maintenance. Maintenance data are the history of maintenance actions executed on individual bridges. Each bridge is presented to the maintenance database as an entity, as a complete set of descriptive and defining data. The bridge is presented as its National Bridge Inventory (NBI) record, its element-level model, its element-level condition data, etc. Maintenance actions are tied to individual bridges. The existence, and essential immutability, of each bridge and its makeup are necessary attributes. Projects that replace bridges or greatly alter bridges are not maintenance, in this context, because they are not compatible with the basic concept of maintenance data organization.

Maintenance Categories

Maintenance programs consist of two broad categories: cyclic work and singular work. Cyclic work, which includes actions such as deck sweeping, is performed at a set interval. Singular work, such as repair, is performed in response to deficient condition. The categories reveal two distinct origins of maintenance projects. Cyclic work is generated in response to DOT policy. Singular work is generated in response to inputs from bridge inspections and road surveys. A third category, Updating, may be added, though it is not prominent in DOT literature on maintenance. Updating is work to replace obsolete elements such as bridge railings, when the replacement is performed as part of the maintenance program. DOTs use various names for these categories of maintenance. Terminology is addressed in the next section.

Contract Maintenance

For the most part, bridge contract work is let through a standard bidding process. DOTs normally prepare Plans, Specifications, and Estimates (PS&E) for many types of bridge maintenance work. The PS&E, or its equivalent, typically includes pay items for bridge maintenance. The item tabulation for a maintenance contract can include some items that appear only for maintenance work, and other items that appear in new construction as well. Items to patch, to place deck toppings, to perform partial depth demolition are maintenance-related. Items for mobilization, traffic control, and furnishing materials among others could appear in contracts for new construction as well as maintenance. Some DOTs identify maintenance contracts as a separate class, and compute distinct values of average unit costs for pay items in these contracts.

Summary on Bridge Maintenance

Bridge maintenance can be defined in terms of policy statements, lists of actions, budget status and the capabilities of maintenance data systems. U.S. DOTs all recognize maintenance as distinct from construction, where construction includes new structures, replacement of structures, and major rehabilitation of structures. Cleaning, painting, and minor repairs are always maintenance. Replacement or modification of portions of bridges may be maintenance if projects are small and have short duration. Larger projects are construction rather than maintenance. Emergency work, usually in response to accidents or extreme weather, is classified as maintenance, and can enter significant, temporary modifications of bridges.

MAINTENANCE CATALOG

Maintenance actions are identified by item numbers. The numbering system employs as many as three two-digit fields, one field for each among bridge component, maintenance operation, and maintenance action. Numbering for bridge components and maintenance operations is shown in Table 2.

In its most general version, numbering for maintenance actions uses two fields, identifying, in order, a component and an operation (see Figure 1). Actions can be used with maintenance operations to provide more detail on the work performed. Numbering for actions is shown in Table 3. When actions are used, numbering for maintenance items has three fields, identifying in order, a component, an operation and an action (see Figure 2).

TABLE 2 Numbering for Bridge Components and Maintenance Operations

	Component
1	Deck
2	Joints
3	Drains
4	Railings, etc.
5	Bearings
6	Superstructure
7	Substructure
8	Appr., Embk.
9	Channel
10	Culvert
11	Bridge
12	Movable Bridge

	Operation
1	Clean/clear
2	Reset
3	Coat
4	Repair
5	Replace
6	Modify
7	Emergency

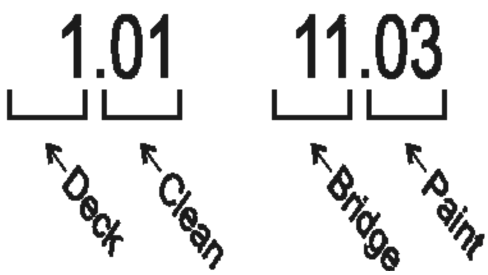

FIGURE 1 Basic numbering for maintenance actions.

TABLE 3 Numbering for Maintenance Actions

	Clean/Clear
1	Wash
2	Zone wash
3	Sweep
4	Flush
5	Unclog/cleanouts
6	Graffiti
7	Vegetation/trees
8	Debris/drift

	Reset
1	Consumable
2	Tighten
3	Caulk
4	Lubricate
5	Reposition
6	Gates/signals
7	Mechanical equip.
8	Electrical equip.

	Coat/Paint
1	Paint
2	Spot paint
3	Seal surface
4	Seal cracks
5	Chemical treatments
6	Surface prep

	Repair
1	Patch
2	Reattach/reanchor
3	Straighten
4	Jack/align
5	Reinforce/strengthen
6	Dredge/grade

	Replace
1	Individual
2	Section
3	Complete
4	Span

	Modify
1	Geometry
2	Protection
3	Vulnerability
4	Strength/capacity
5	Function
6	Assembly

	Emergency
1	Post
2	Shore
3	Closure, full
4	Closure, partial
5	Detour
6	Temporary bridge

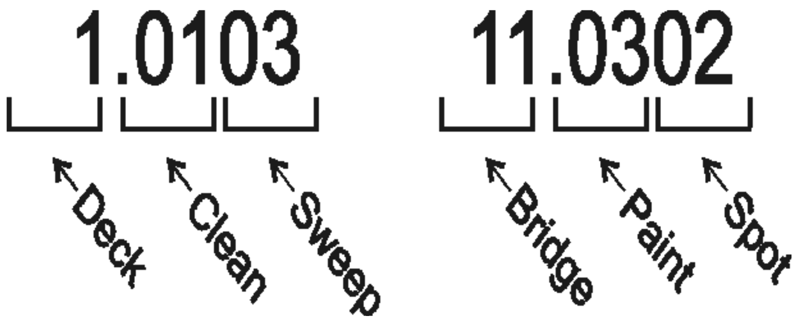

FIGURE 2 Numbering for maintenance items using actions.

The numbering system easily rolls up from its more detailed implementation to more terse. The numbering system allows simple collection of maintenance data by bridge component or by maintenance operation.

ANALYSIS PROCEDURES

Figure 3 shows the conceptual framework for the cost analysis, which comes in four phases:

- Recognition of condition deficiencies, from bridge inspection. When planning future bridge maintenance, the quantity of deterioration is usually the only quantity known with any degree of confidence. Thus, the analysis starts here.
- Economic inputs. These are the resources that are put into a maintenance activity. Such resources typically include labor, materials, equipment, and contract pay items. In most agencies, these inputs can be tracked in maintenance and financial management systems. Usually the inputs that are trackable do not include indirect costs such as engineering, mobilization, and maintenance of traffic.
- Economic outputs. Measured separately from inputs, this is a description of work completed in maintenance activities. For example, if a crew painted 5000 m^2 of steel, that would be considered an output. If the crew used 10 gal of paint and 8 h of time, those are inputs. Outputs typically use different units of measure than inputs. Output costs are often computed in the headquarters office using cost allocation procedures, rather than in the maintenance yard. They typically include indirect costs, especially if the work is done by contractors.
- Outcomes. Typically, the purpose of maintenance activity is to change a deteriorated condition into a better condition. The amount of change is called the outcome. Like the initial condition, outcome is generally measured by bridge inspection, usually the next regular inspection after the work is done.

Both cost and quantity of both inputs and outputs are used in the analysis. If any are missing, the analysis can still work without them. But then, of course, the results will be less complete.

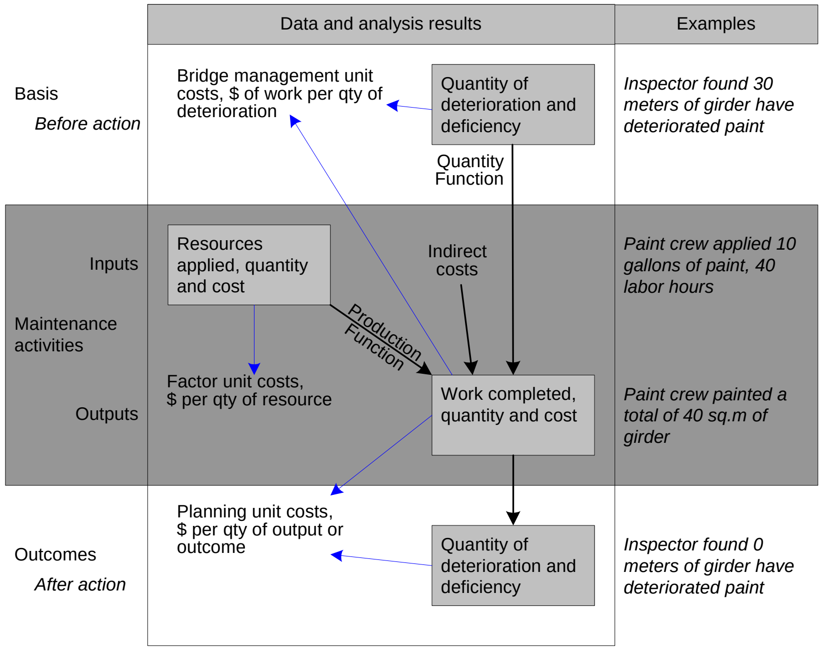

FIGURE 3 Conceptual framework of cost analysis.

Roll-up Procedures

Costs and quantities can be "rolled up" (i.e., summed) to any level of aggregation where it makes sense. Input costs are rolled up to the level of resource definitions, and then to the level of component, operation, and action for the whole database. If several states or time periods are in the same database, they are rolled up together. However, they can be configured to roll up separately as well. Output costs are rolled up to the same levels as inputs, but are also allocated to structures and their elements, and from there to the level of elements and actions for the whole database.

Quantities are not necessarily rolled up to the same levels as cost because of the need for compatibility of measurement units. It's the "apples and oranges" problem that resource quantities measured in gallons shouldn't be added to resource quantities measured in hours. Yet, the database is structured to allow valid roll-ups of the most important quantities.

The analysis computes unit costs and performance measures from this information. Table 4 describes the statistics that are computed.

All of these are descriptive and not normative statistics. That is, they describe what the agency has been doing in the past and not how operations might be improved in the future.

Cost Allocation

All cost data originate with costs gathered in the agency's maintenance management process. Each activity is identified by use of the maintenance catalog with component, operation, and action. This makes it straightforward to roll up total costs for a database as a whole, in the same categories of component, operation, and action.

TABLE 4 Statistics Computed by the Analysis System

Variable Name	Description
qty_def	Quantity of defects, computed as the element quantity in condition state 2 or below for any element.
qty_inp	Quantity of input resources, from the event_resource table.
qty_out	Quantity of output, from the event_activity table.
qty_res	Quantity of resulting defects, an outcome measure determined by qty_def for the following element inspection.
cost_inp	Cost of input resources, from the event_resource table.
cost_out	Cost of output work accomplished, from the event_activity table.
ucost_def	Unit output cost relative to initial defects, computed as cost_out ÷ qty_def. This is the Pontis (*1*) total maintenance, rehabilitation, and replacement (MR&R) action unit cost.
ucost_inp	Unit cost of inputs, computed as cost_inp ÷ qty_inp.
ucost_out	Unit cost of outputs, computed as cost_out ÷ qty_out.
ucost_res	Unit cost relative to resulting condition, computed as cost_out ÷ qty_res.
eff_func	Effectiveness function, computed as qty_res ÷ qty_def, or outcome ÷ initial condition, a measure of the effectiveness of a treatment in correcting the original problem.
qty_func	Quantity function, computed as qty_out ÷ qty_def, or work quantity divided by defect quantity. This is useful in project planning for determining the quantity of work required in response to inspection data.
prod_func	Production function, cost_inp ÷ cost_out, a measure of the amount of a given input required in order to achieve a desired output.
prob1 .. prob5	Pontis action effectiveness probabilities.

A more complicated problem is the association of work accomplishments with the original deterioration that likely motivated the work. The database provides a capability to enter directly, for each activity, the bridge element to which the action was applied. However, as found in *NCHRP Synthesis 227* (*14*), this is not within the capabilities of most agencies at this time.

The inability to directly record which elements are affected by a maintenance event complicates the analysis. Each element definition is uniquely identified with one bridge component, and each bridge maintenance system (BMS) action is identified with one component, operation, and action. The reverse is not true, however: each combination of component, operation, and action may be associated with multiple BMS actions. In fact, nearly every combination of component, operation, and action that corresponds to a bridge management action as defined in the AASHTO CoRe elements (*15*), corresponds to more than one of them. Maintenance action 1.04.01, deck repair/patch, matches 46 different AASHTO CoRe element actions for different elements and condition states.

When a maintenance activity occurs and is recorded in the database, it is required that it be identified with a component, operation, and action. If an element is not identified, then there is a strong possibility that more than one element might be affected. For example, a painting action might affect both the girders and stringers. The cost allocation procedure resolves this issue. Here’s how it works.

Step 1: Definition Preprocessing

A preprocessing algorithm analyzes the bridge management system definitions to index all the correspondences between BMS actions and maintenance management system (MMS) actions. It identifies the cases where a unique match can be made, and keeps track of the measurement units used, since allocation of work quantities must not mix quantities measured in incompatible units.

Step 2: Event Processing, Stage 1

The first processing step on maintenance events is to add up the costs over all activities and resources in an event to get the event's input and output cost totals.

Step 3: Structure Processing

The algorithm completes the analysis of all maintenance events on each structure before proceeding to the next structure. It associates each element inspection on a structure with the next element inspection after it. It asks the question, "If we ended up working on this element, how did it turn out?"

Step 4: Event Processing, Stage 2

Next each event affecting the structure is processed. On the first pass, each event is tagged with the identifier of the last element inspection before the event, and the first one after. It is possible that there might not be an inspection before or after, and it is possible for there to be multiple events between inspections. So these possibilities are all accounted for. At this point it becomes possible to allocate event costs to inspections. This addresses the question, "How much did we spend because of the findings of this inspection?"

Step 5: Activity Processing, Identifying Affected Elements

Now each activity is associated with element inspections, for the inspection that occurred most recently before the activity. Pontis (*13*) allows an element to appear more than once in an inspection, for different structure units or environments. So it is first necessary to aggregate the separate instances of the same element.

A maintenance activity can affect multiple elements, and an element can be affected by multiple activities. We can narrow the list of possible associations in this many-to-many relationship by using the results of step 1 to identify just the relevant elements on the structure, the types of elements to which the activity is generally applicable. Often more than one element qualifies.

When more than one element might have been affected, it is valid to allocate the output costs among elements on the basis of quantity of improved condition, or by quantity of total deterioration if none showed improvement. To allocate costs on the basis of quantity, however, it is necessary to make sure all elements so allocated use the same measurement units of quantity. This provides a basis for further elimination of elements from the allocation process, if measurement units are incompatible with most of the elements that were likely affected.

If any of the elements on the structure improved in condition, which is usually the case after a maintenance event, we can narrow the list even further by eliminating elements that did not improve. If no elements improved, but any were deteriorated before the event, we can eliminate elements that were in perfect condition before the event.

Step 6: Allocation of Cost and Quantity of Output, and Pontis Unit Costs

Finally, after all possible eliminations, we may still have more than one element on the structure that may have been affected by the maintenance activity. When this happens, costs and quantities are allocated between them, by quantity of improvement if possible, or by total quantity of element if necessary. Users of the software developed in the project can easily view all the inspections that likely motivated the maintenance activity.

After this allocation of costs to elements, it becomes possible to roll up allocated output costs by type of element, giving the average unit cost of each element.

For the purpose of Pontis unit costs, it is necessary to take one further step, since Pontis costs are expressed at the level of element, condition state, and action. We allocate costs not just between elements, but also between condition states. This is obviously based on condition. Within a condition state, costs are not allocated between feasible actions because such actions are considered to be mutually exclusive alternatives within the theoretical framework of Pontis. So output costs are fully assigned to each BMS action that was not previously eliminated for each condition state.

Step 7: Quantity Metrics

Since each combination of BMS element, state, and action uniquely identifies a specific combination of MMS component, operation, and action, it is possible to uniquely determine the measurement units of output. Thus, quantities of output, not just costs, can be allocated to BMS actions. This makes it possible to calculate the quantity function, which directly relates quantities of work, in output units, to quantities of deterioration, in bridge inspection units. For example, we can calculate the average square meters of painting required to respond to each linear meter of deteriorated girder. This information will be very helpful in the future for improving bridge management predictive models.

Pontis Action Effectiveness

After completion of the cost allocation procedure on each structure, one further computation yields Pontis action effectiveness probabilities. In Pontis, the condition of an element following an MR&R action is forecast using a vector of transition probabilities, representing the likelihood of each possible condition state after the action These probabilities are memory-less; that is, they don’t depend on the condition before the action.

To perform this computation, first each event is associated with the inspection following the event. This is different from the cost allocation procedure, which relates each event to the inspection before it. An unweighted average is computed, first for all elements of the same type on the structure, then to the level of elements, states, and actions for the whole database. These results are directly usable in Pontis.

Limitations of the Analysis

It is clearly valuable to have a rigorous procedure to estimate bridge management planning metrics. The process described here makes the most of the available data, and is far better than any other automated process developed thus far for this purpose. However, the limitations of the analysis must be recognized:

- Even in a large database, many structural elements might not have received any maintenance work. No results will be generated for such elements.
- The methodology is sensitive to the agency's procedures for estimating the output cost of maintenance activities, which are not always as rigorous.
- There can be great variations from one project to another. More in-depth research using the research products may uncover and quantify the factors that cause variation, making application of the results more precise.
- When data from multiple agencies are combined, there can be differences in element definition and accounting procedures that cause the unit costs and productivity measures to vary. For example, a box girder in California is likely to be a large single-unit box with high costs per linear foot, while in Colorado the same element is more likely to be a small precast box beam placed adjacent to several others, counted separately in length.

In a Florida DOT study of Pontis unit costs (*16*), a rigorous statistical method was followed to estimate the costs from a very complete database of in-house and contract maintenance events. The results were then put before a panel of expert estimators. It was found that 55% of the unit cost values required significant adjustment by expert review.

SOFTWARE SYSTEM

The National Bridge Maintenance Database (NMDB) is a combination of tools that support various parts of an ongoing process, as shown in Figure 4. These tools make the best use of generic technologies for each phase of the data life cycle. The most significant tools are these:

- A generic bridge maintenance database, delivered in XML format. XML is a data standard that is specifically intended to be open, human-readable, generic, and vendor-neutral. The XML database follows a specific structure, called a schema, that determines what kinds of data may be included in the database.
- An Excel workbook file called The Organizer. This file contains procedures for importing, converting, organizing, and editing the XML database. The organizer can import data from an agency maintenance management database provided the database meets certain standards. It can also import from Pontis and from other XML databases using the NMDB schema. The Organizer handles data validation and the cost analysis.
- A website (Figure 5), which gives a very user-friendly and flexible interface for browsing through a database and accessing the most significant analysis results. This site consists of a collection of files in HTML, XML, Javascript, and graphic formats. All executable code on the website operates in the user's browser. This makes the system generic and is most compatible with agency data security standards. The website can be customized on an agency's

web server, or it can be downloaded and run entirely on a desktop computer, local area network, or intranet.

- An output database, which can be used for any analyses that are not provided by the organizer or website. The XML database is designed to be directly importable into common database management systems such as Microsoft Access.

All of these components are supplied with the project deliverables. The content of the databases consists of sample data from 10 agencies that each agency can use in combination with its own data for development of bridge management inputs and metrics. A variety of standard reports (Figure 6) present various stages of the analysis from maintenance management inputs to bridge management results.

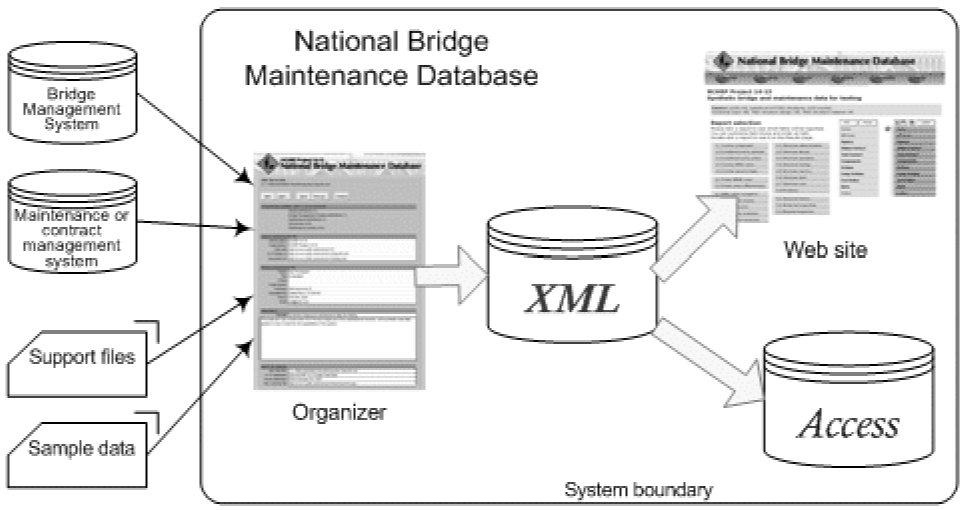

FIGURE 4 Data flows in the NMDB.

MRR Actions (946)

Item	Element	Element	State	Action	Description	Qty-Def	Cost-Out	UCost-Def	BMS-Units	Comp/Action	Activities
1	12	Bare Concrete Deck	1	1	Add a protective sys	0	0	0.00	sq.m	1.0602	Activities
2	12	Bare Concrete Deck	1	2	Miscellaneous Maint	0	0	0.00	sq.m	1.0000	Activities
3	12	Bare Concrete Deck	2	1	Repair spalled/delam	9,997	7,923,922	792.63	sq.m	1.0401	Activities
4	12	Bare Concrete Deck	2	2	Add a protective sys	9,997	7,923,922	792.63	sq.m	1.0602	Activities
5	12	Bare Concrete Deck	3	1	Repair spalled areas	0	0	0.00	sq.m	1.0401	Activities
6	12	Bare Concrete Deck	3	2	Rep spall & add prot	0	0	0.00	sq.m	1.0602	Activities
7	12	Bare Concrete Deck	4	1	Repair spalled areas	0	0	0.00	sq.m	1.0401	Activities
8	12	Bare Concrete Deck	4	2	Rep spall & add prot	0	0	0.00	sq.m	1.0602	Activities
9	12	Bare Concrete Deck	5	1	Repair spalled areas	0	0	0.00	sq.m	1.0401	Activities
10	12	Bare Concrete Deck	5	2	Replace deck	0	0	0.00	sq.m	1.0503	Activities
11	13	Unp Conc Deck/AC Ovl	1	1	Miscellaneous Maint	0	0	0.00	sq.m	1.0000	Activities
12	13	Unp Conc Deck/AC Ovl	2	1	Repair potholes and	0	0	0.00	sq.m	1.0401	Activities
13	13	Unp Conc Deck/AC Ovl	3	1	Repair potholes and	0	0	0.00	sq.m	1.0401	Activities

FIGURE 5 Example report of Pontis unit cost data.

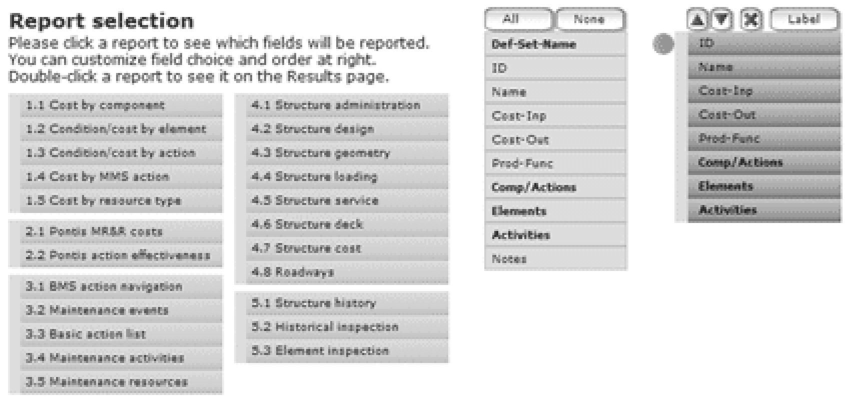

FIGURE 6 Menu of standard reports from the web interface.

ACKNOWLEDGMENTS

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Deterioration Rates of Typical Bridge Elements in New York

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The New York State Department of Transportation (NYSDOT) maintains an inventory of more than 17,000 highway bridges across the state. These bridges are inspected biennially or more often as necessary. Bridge inspectors are required to assign a condition rating for up to 47 structural elements of each bridge, including 25 components of each span of a bridge, in addition to the general components common to all bridges. The bridge condition rating scale ranges from 1 to 7, 7 being new and 1 being in failed condition. These condition ratings may be used to calculate deterioration rates for each bridge element, while considering the effects of key factors, such as bridge material type, on deterioration rates. This paper describes the development of bridge element deterioration rates using the NYSDOT bridge inspection database using Markov chains and Weibull-based approaches. It is observed that a Weibull-based approach is more reliable for developing bridge deterioration curves. Both Markov chains and Weibull-based approaches have been incorporated into a computer program that generates deterioration curves for specific bridge elements, based on historical NYSDOT bridge inspection data going back to 1981. Case studies on deterioration rates of various bridge elements in New York State are presented to demonstrate the two approaches. Case studies show that element deterioration rate information can be used to determine the expected service life of different bridge elements under a variety of external factors. This information is extremely valuable in bridge management decision making.

The condition assessment of the bridge infrastructure is crucial in preparing 5- and 10-year capital programs for the construction and maintenance of bridges in New York State. Currently, more than 31% of bridges in the United States are considered deficient (*1*). In recognition of the state of the national bridge infrastructure, Congress passed the Intermodal Surface Transportation Efficiency Act (ISTEA) in 1991, mandating bridge management systems (BMS) as part of activities of all state departments of transportation (DOTs). Hence, the bridge management program package, Pontis, sponsored by FHWA, was developed in 1992 under the guidance of an AASHTO task force and adopted by a majority of states (*2*). Another software package, BRIDGIT, was developed in 1997 for smaller DOTs having insufficient staffs to maintain a BMS (*3*). Both of these packages have built-in capabilities of calculating bridge deterioration rates using the Markov chains model to predict the probability of transition from one condition state (rating) to another condition state. However, the BMS maintained by the New

York State Department of Transportation (NYSDOT) contains more data than those used by the Pontis and BRIDGIT systems.

The deterioration rates of individual bridge elements are influenced by the combined effects of many complex phenomena (e.g., reinforcement corrosion, concrete degradation, creep, shrinkage, cracking, and fatigue). In the absence of mechanistic-based deterioration models that require the quantitative contribution of these complex phenomena, bridge inspection data may be used to estimate future rehabilitation or replacement needs and prioritize work. In this paper, Markov chain and Weibull distribution approaches, which are both stochastic approaches, are used to investigate the deterioration rates for specific bridge elements. Applications of these two approaches are presented through case studies of deterioration rates for various bridge elements. These deterioration rates can be used for estimating remaining life, comparing alternatives for repair and retrofits, and analyzing effects of external factors affecting the deterioration of bridge elements. Better understanding of these issues is essential for developing and implementing an effective bridge management program.

DATA PREPROCESSING

Data preprocessing is a critical first step in deterioration curve development. It is important to ensure the form of the data being analyzed is consistent with the phenomena being described. Bridge elements deteriorate over time and their conditions do not improve without some type of intervention. In attempts to model deterioration rates that do not include the effects of such interventions, it is important to ensure the data being modeled is properly handled.

The first step was to review elements where the condition rating increased by two points or more. It is likely this indicates significant improvement work. If the rating was a two-point improvement to a 7, it was assumed that the element was likely replaced and therefore for data analysis purposes the element was assumed as new. All data before the improvement were discarded. For those elements where there was a two-point improvement that did not result in a new rating of 7, it was assumed to have undergone significant rehabilitation, but not to the point of being a new element. For analysis purposes these data where not included as we were interested in the development of deterioration rates of nonrehabilitated elements.

A review of those elements that underwent single-point improvements was also done. This could be caused by minor work, or could be caused by subjectivity in ratings from different bridge inspectors. Algorithms were developed to handle the data differently based on how long the element remained at the improved state versus how long the element was at the condition rating before improvement. For example, a bridge element that was at condition rating 5 for 6 years and then went to condition rating 6 for 2 years and then went back to condition rating 5 for 6 additional years was recorded as having been at condition rating 5 for 14 years. Again, the application of these data processing rules is intended to reduce the effects of rater subjectivity and ensure the effects of major work are not included in the development of the deterioration curves.

Because of the fact that inspection information was not available prior to 1981, assumptions were made for data prior to that date. For certain elements, additional criteria were added to improve results. For example weathering steel bridges were started around 1968. Therefore, any weathering steel bridge that had an estimated element year built before 1968 was removed from the data analysis. Additional restrictions were included for different superstructure material types, as well as restrictions for other elements. This pre-analysis filtering was crucial in arriving at the most accurate deterioration information

APPROACHES FOR BRIDGE ELEMENT DETERIORATION RATES

Several deterministic models have been used for analysis of bridge inspection data by Yanev and Chen (*4*). However, bridge deterioration is a stochastic process that depends on many factors, such as average annual daily truck traffic (AADTT), climate, and maintenance level. Hence, stochastic models are best used to characterize the uncertainty and randomness of the bridge deterioration process. These models can be classified either as discrete-time, state-based or discrete-time, time-based models (*5*). In discrete-time, state-based models, such as Markov chains, the deterioration process is modeled through a probability of transition from one condition state to another in a discrete time, given that the deterioration process is dependent on a set of explanatory variables such as AADT, climate, and age. In discrete-time, time-based models, the duration that a bridge element remains at a particular state is modeled as a random variable using Weibull-based probability density functions to characterize the deterioration process, given its dependence on the same set of explanatory variables described above. This paper presents a comparative study of deterioration rates calculated by Markov chains and Weibull-based distributions using NYSDOT bridge inspection data.

Markov Chains Approach

Markovian models are the most common stochastic techniques and have been used extensively in modeling deterioration rates of infrastructure facilities, e.g., pavement prediction model (*6*), storm water pipe deterioration (*7*), bridge types and components (*8*, *9*). These models use the Markov decision process based on the concept of defining states of facility condition transitioning from one state to another during one transition period (*8*, *10*). Statistical appropriateness of the Markovian process for bridge deterioration modeling has been shown by Madanat and Ibrahim (*11*).

Although Markovian models address the uncertainty of the deterioration process and account for the current facility condition in predicting the future, they still suffer from several limitations such as: (*a*) they assume discrete transition time intervals, constant bridge population, and stationary transition probabilities, which are sometimes impractical (*12*); (*b*) Markovian models currently implemented in advanced bridge management systems such as Pontis and BRIDGIT use the first-order Markovian decision process that assumes duration independence for simplicity (*13*), i.e., the future facility condition depends only on the current facility condition and not on the facility condition history, which is unrealistic (*14*); (*c*) transition probabilities assume that the condition of a facility can either stay the same or decline (*11*); (*d*) Markovian models cannot efficiently consider the interactive effects between deterioration mechanisms of different bridge components (*15*); and (*e*) transition probabilities require updates when new data are obtained. Among the limitations described above, stationarity of transition probabilities and duration independence assumptions have been observed to have most significant effects on the reliability of using the Markov chains approach for developing deterioration rates. These assumptions may lead to underestimation in deterioration rates that may not be reasonable for an effective bridge management system. Despite the limitations of the Markov chains process, it is the most widely used approach for deterioration modeling.

For the NYSDOT bridge inspection system, seven bridge condition ratings (from 7 to 1) can be defined as seven Markovian states with each condition rating corresponding to one of the seven states. For example, condition rating 7 is defined as State 1; rating 6 as State 2, and so on. Without repair or rehabilitation, the bridge condition rating should decrease with increase in bridge age. Therefore, there is a probability, p_{ij}, of a bridge element transiting from one condition state, say i, to another state, j, between inspections. The probability of state i not transitioning to state j is then $1 - p_{ij}$. By knowing this probability for each of the states transitioning, e.g., 1 to 2, 2 to 3, 3 to 4, etc., we can obtain the transition matrix P, which is defined as

$$P = \begin{bmatrix} p(1) & q(1) & 0 & 0 & 0 & 0 & 0 \\ 0 & p(2) & q(2) & 0 & 0 & 0 & 0 \\ 0 & 0 & p(3) & q(3) & 0 & 0 & 0 \\ 0 & 0 & 0 & p(4) & q(4) & 0 & 0 \\ 0 & 0 & 0 & 0 & p(5) & q(5) & 0 \\ 0 & 0 & 0 & 0 & 0 & p(6) & q(6) \\ 0 & 0 & 0 & 0 & 0 & 0 & 1 \end{bmatrix} \tag{1}$$

In Equation 1, $q(i) = 1 - p(i)$, $p(i)$ corresponds to $P_{i,i}$, and $q(i)$ corresponds to $P_{i,i+1}$. Hence, $p(1)$ is the probability of transition from condition rating 7 (State 1) to rating 7 (State 1), i.e., staying at rating 7, and $q(1) = 1 - p(1)$ is the probability of transitioning from rating 7 (State 1) to rating 6 (State 2). The lowest rating possible is 1. Hence, the corresponding probability $p(7)$ is 1. Then, by the Markov chains approach, the state vector for any time t, $Q_{(t)}$, can be obtained by the multiplication of the initial state vector Q_0 and the transition probability matrix P raised to the power t, i.e.,

$$Q_{(t)} = Q(0) * P^t \tag{2}$$

where $Q_{(t)}$ is the state vector at any time t and $Q_{(0)}$ is the state vector at time $t = 0$. For example, a newly constructed bridge element at the time of first inspection will have an initial state vector as $Q_{(0)} = [1\ 0\ 0\ 0\ 0\ 0\ 0]$. Similarly, a bridge element with rating 5 at time $t = 0$ will have an initial state vector as $Q_{(0)} = [0\ 0\ 1\ 0\ 0\ 0\ 0]$. Let R be a vector of condition ratings, i.e., $R = [7\ 6\ 5\ 4\ 3\ 2\ 1]$, and R' be the transform of R, then the estimated condition rating $R_{P,t}$ as a function of time by Markov chains is obtained as

$$R_{P,t} = Q_{(t)} * R' \tag{3}$$

The deterioration rate at any age can be defined as the slope of the condition rating curve in Equation 3. The transition matrix P in Equation 1 can be calculated by formulating the nonlinear programming objective function as

$$\min \sum_{J=1}^{N} \left| Y(t) - R_{P,t} \right| \text{ subject to } 0 \le p(i) \le 1, \text{ for } i = 1,2,\ldots,I \tag{4}$$

where N is the number of inspection data used in the minimization problem and I is the number of unknown probabilities, i.e., $I = 6$, p = a vector of length $I = [p(1), p(2),\ldots, p(i)]$, and $Y(t)$ is the average of condition ratings at time t. The objective function in Equation 4 can be minimized by a constrained nonlinear programming approach to obtain values of transition probabilities $p(i)$, $i = 1, 2,\ldots,6$. In the Markov chains approach, nonhomogeneity of the continuous deterioration process (i.e., time dependence of deterioration process) is captured indirectly through segmentation of bridge elements into age groups, i.e., 0–10 years, 10–20 years, etc. Within each group, Markov chains are assumed to be homogeneous. A new bridge element is almost always given a condition rating of 7. In other words, a bridge element at age 0 has a condition rating 7. Thus, the initial state vector $Q_{(0)}$ for a new element is [1 0 0 0 0 0 0], where the numbers are the probabilities of having a condition rating of 7, 6, 5, 4, 3, 2 and 1 at age 0, respectively. The transition matrix P in Equation 1 is determined using nonlinear programming in Equation 4 for each age group. Then, Equations 2 and 3 are used to determine the rating vector RP,t in Equation 3 at different ages. For example, $Q_{(0)}$ = [1 0 0 0 0 0 0] can be used for the age group 1 (i.e., 0–10 years) to iteratively determine $Q(t)$ and RP,t for the 0 to 10 years interval. For the age group 2 (10 to 20 years), $Q(10)$ is used as the starting state vector to determine RP,t for 10 to 20 years interval using P matrix for this age group. This process is followed until RP,t is generated for the entire age range of interest.

One major drawback of this approach is that the nonlinear optimization in Equation 4 will not converge and will essentially give the transition matrix P as a unity matrix or close to it, if condition ratings in a particular age group don't decrease or tend to increase. This issue has been observed by carrying out a second level Markov process. In this process, data generated by Markov chains processes using 10-year age groups are used as original data $Y(t)$ in Equation 4 to derive one transition matrix for the age group from 0 to 40 years. Markov chains data generated by this transition matrix will follow the dominant deteriorating behavior during the first 40 years. For example, Figure 1 shows the comparison between first and second level Markov chains plots for structural decks. It is observed from Figure 1 that while the deterioration curve using first level Markov chains stops decreasing beyond 30 years, the curve by the second level Markov chains follows the original data during first 40 years and continues a decreasing trend beyond that age.

Weibull Distribution-Based Approach

Assuming that the variable T_i ,$i = 7, 6,\ldots,1$, which represents the duration in number of years that an element stays in a particular condition rating, is a random variable modeled by a Weibull distribution, the probability $P(T_i > t)$ that T_i exceeds t years is called survival function of T_i and is denoted by

$$S_i(t) = e^{-(t/\eta i)^{\beta i}},\ t > 0, \beta i > 0, \eta i > 0, i = 7,6,5,\ldots,1 \tag{5}$$

For each condition rating, T_i (durations) is assumed to comprise a random sample from the Weibull distribution described by Equation 5. Each such sample is then analyzed with the objective of estimating (βi, ηi) pair. The parameters βi and ηi in Equation 5 are called shape and scale parameters, respectively. The shape parameter determines whether the so-called failure or hazard rate is

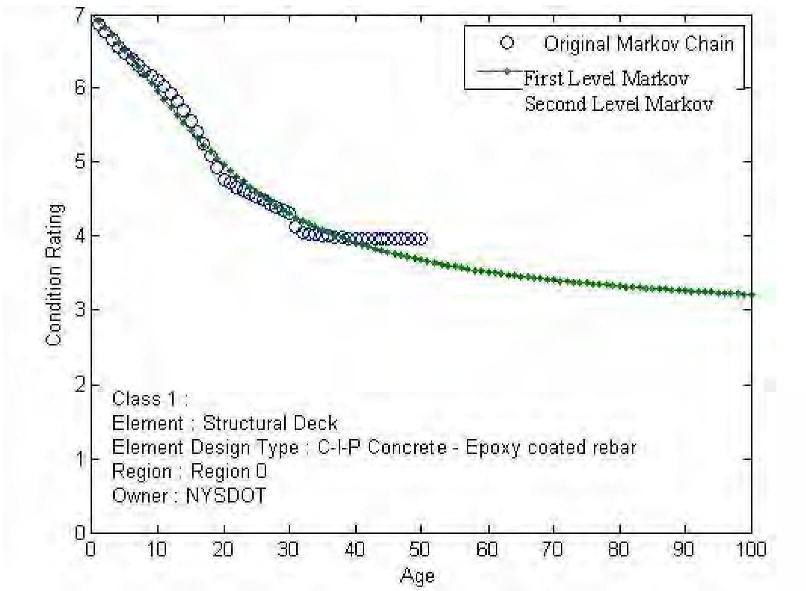


FIGURE 1 Comparison between deterioration rate curves using first- and second-level Markov chains processes.

decreasing ($\beta i < 1$), constant ($\beta i = 1$), or increasing ($\beta i > 1$). An increasing failure rate means that at any particular time, the longer an element has been at a particular condition rating (CR), the more likely it will transition to a lower CR in the next instant. For example, suppose that $n(t)$ samples of an element have been at kth CR for t years and that $h(t)$ is the failure rate at any time t. Then, $n(t)\ h(t)$ is approximately the expected number of samples of an element that will transition to $(k - 1)$th CR within the following year. More rigorous description of failure rate and other probabilistic properties used to describe durational phenomena is provided by Mishalini and Madanat (*16*) and DeLisle et al. (*17*).

Distributions of durational phenomena at a particular rating are typically skewed and consequently are rarely normally distributed. For example, Figure 2 shows the frequency distribution of durations at different condition ratings for structural decks. It is observed that distributions of durations for condition ratings are skewed toward the left (lower age). The most frequently used distributions for such data are the Weibull and lognormal. It has been shown by DeLisle et al. (*17*) that the Weibull distribution generally provides the best overall fit for infrastructure deterioration data. This fact has been verified by analysis of reconditioned inspection data of most of the bridge elements in New York state during preliminary analysis of the work presented in this paper.

Weibull distribution parameters βI and ηi, $i = 1, 2, \ldots, 6$, obtained by fitting duration data for different condition ratings can then be used to estimate any other distributional characteristic such as percentiles and probabilities of exceeding any specified duration t. For example, DeLisle et al. (*17*) have obtained $\beta 6$ and $\eta 6$ for CR6 for structural deck elements as 27.3 and 1.2, respectively. Then, the probability that a bridge deck entering into CR6 at some particular time remaining at CR6 for more than 10 years is obtained from Equation 5 as $P(T_6 > 10) = S_6\,(1) = e$ $= 0.74$. Hence, the probability that the deck will transition to CR5 in 10 or less years is $1 - 0.74 =$

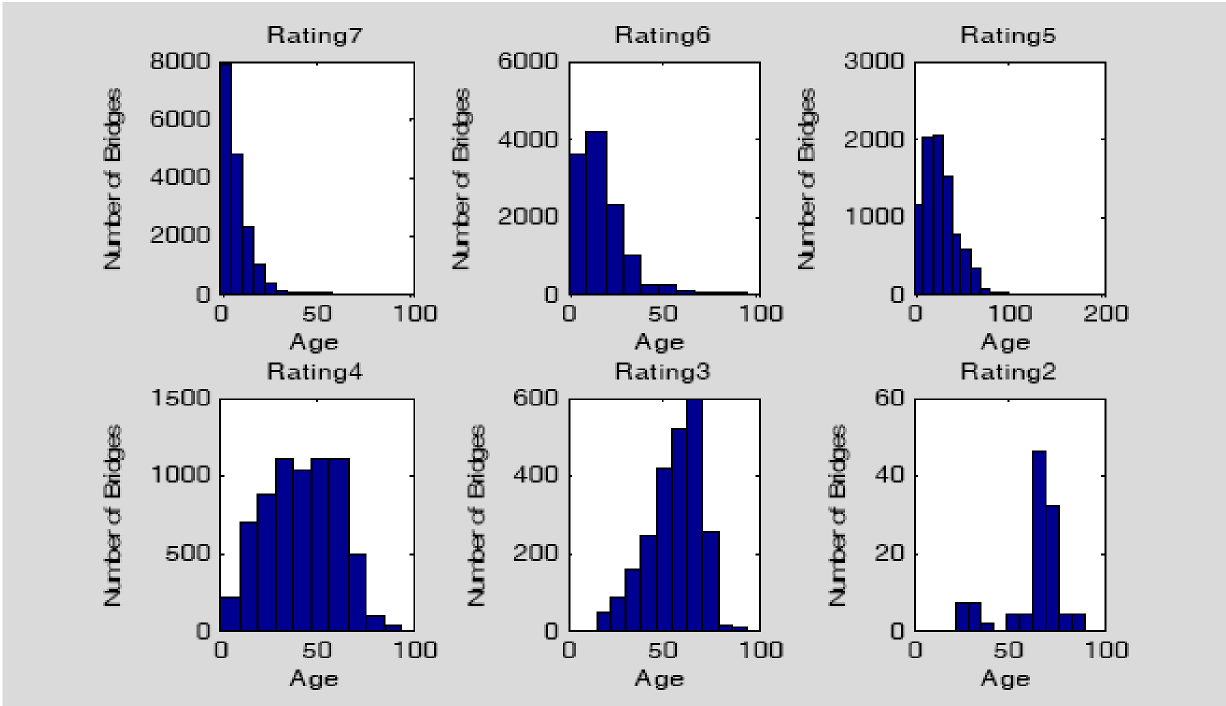


FIGURE 2 Frequency distribution of durations for different CRs.

0.26. Similarly, the probability that a deck element that has been at CR6 will remain at CR6 for an additional 10 years can be estimated as $S_6(20)/S_6(10) = 0.68$. Alternatively, the probability that a structural deck that has been at CR6 for 10 years will move to CR5 within the next 10 years is 1 – 0.68 = 0.32. This type of information is extremely important for effective bridge management and cannot be obtained by other approaches, including the Markov chains approach. For the Weibull distribution, mean T_i for a condition rating i is calculated by

$$E(T_i) = \eta_i \Gamma(1 + \frac{1}{\beta_i}) \quad (6)$$

where Γ is the gamma function. Using Equation 1, mean durations for different CRs are obtained cumulatively, i.e., CR7 to CR6 = E(T7), CR7 to CR5 = E(T7) + E(T6), etc. Although mean T_i represents average duration that a bridge element will stay at ith CR, some of the elements will transition to (i – 1)th CR during this period. This information can be calculated by percentile of CR, given by

$$t_p = \eta[-\ln(1-p)]^{1/\beta} \quad (7)$$

where t_p is the duration after which P % of the samples of an element will move to lower CR. For example, T5 for CR5 to CR4 is 23 years and t25% = 11 years. This means that 25% of the samples of an element will transition to CR4 within 11years, although mean T5 is 23 years.

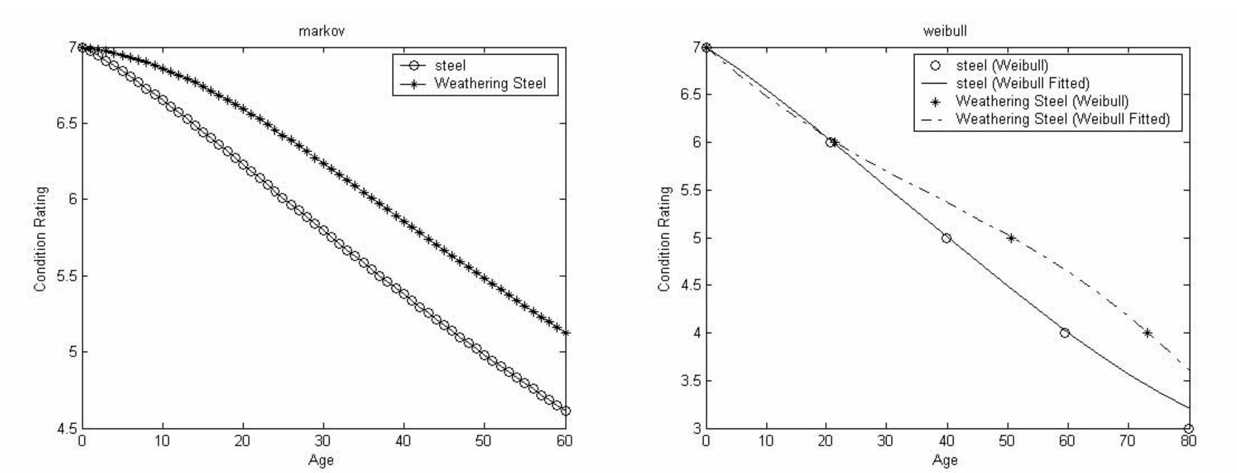
EXAMPLES OF BRIDGE ELEMENT DETERIORATION RATES

Applications of Markov chains and Weibull distribution-based approaches in calculating deterioration rates are illustrated through several case studies as described next. For the Weibull-based approach, figures for case studies also show plots of condition ratings versus age using third-order polynomial curves regressed with very good fit coefficients to deterioration curves obtained by the Weibull-based approach.

Case 1: Primary Member, Plate Girder, Steel Versus Primary Member, Plate Girder, Weathering Steel

Figures 3*a* and 3*b* show deterioration plots for plate girder type primary members made of steel and weathering steel, respectively. It has been observed that Markov chains follow the original data more closely and the Weibull-based approach generally gives higher deterioration rate than that by Markov chains. It is observed from Figure 3*a* using Markov chains that plate girders made of weathering steel deteriorate slower than those made of steel from the beginning. On the other hand, it is observed from Figure 3*b* that plate girders made of steel and weathering steel deteriorate at the same rate during first 20 years. Weathering steel girders deteriorate slower than steel girders beyond 20 years age.

Significant differences between Markov and Weibull distribution-based approaches in Figure 3 can be explained on the basis of their inherent assumptions. One of the main assumptions of Markovian process is of independence from history or duration independence or lack of memory. The Markov chains approach also cannot take into consideration censored condition data. Censoring takes place when the duration at a specific condition level is not completely observed. This happens for the latest condition rating on file or when the condition rating of a bridge element is improved. For the latest condition rating, it is unknown how long



(a) **(b)**

FIGURE 3 Deterioration plots for plate girders made of steel and weathering steel in bridges owned by NYSDOT: (*a*) using Markov chains and (*b*) using Weibull-based approach.

the element will remain at its present condition, but the duration is at least the number of years already recorded. When an improvement occurs, it is unknown how long the element would have remained at its prior condition level if the improvement had not occurred. As before, the duration is at least the number of years recorded at the prior condition rating before the improvement occurred. In both instances, the durations are referred to as right-censored and must be handled appropriately.

In Markov chains, there are only two choices regarding censored observation: disregard them or treat them as actual durations. Either of these choices may lead to seriously biased estimates. Weibull distribution-based approach can handle censored data by explicitly considering duration dependency in the analysis. For example, ($\beta 7$, $\eta 7$) pairs for plate girders of steel and weathering steel in Figure 3 are (3.2473, 23.155) and (2.722, 24.059), respectively. Values of $\beta 7 > 1$ clearly show that the deterioration hazard rate is increasing and that it is durationally dependent. Hence, results obtained by the Weibull-based approach can be considered to be more reliable probabilistically.

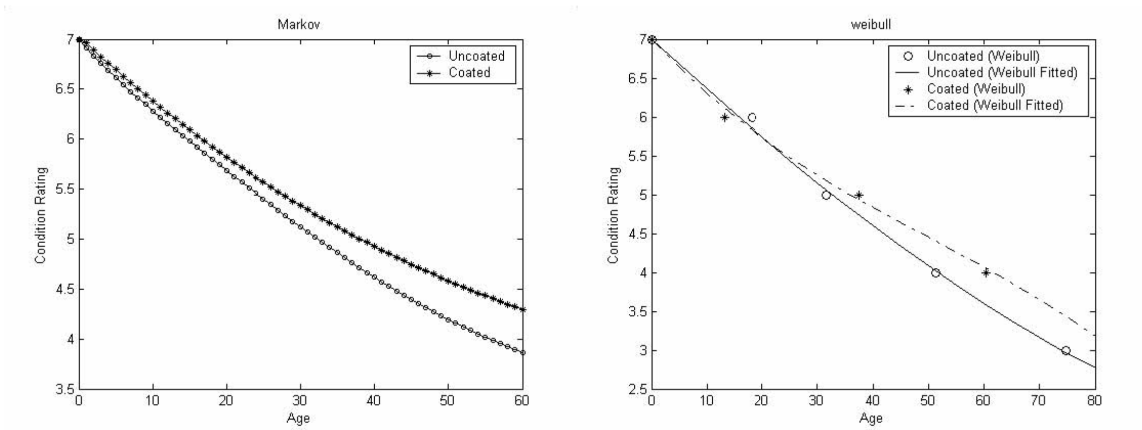
It is observed that both steel and weathering steel plate girders deteriorate identically until 20 years to CR6. Beyond 20 years of age, weathering steel plate girders deteriorate slower than steel plate girders. In fact, the CR of 60-year-old steel plate girders drops to 4 where it is close to 4.64 for weathering steel girders of the same age. This clearly shows that weathering steel plate girders perform significantly better than steel plate girders.

Case 2: Structural Deck, Cast-in-Place Uncoated Rebar, NYSDOT Versus Structural Deck, Cast-in-Place Epoxy-Coated Rebar, NYSDOT

Figures 4*a* and 4*b* show deterioration plots for structural decks with coated and uncoated rebars in bridges owned by NYSDOT using Markov chains and Weibull-based approaches, respectively. It is observed from plots in Figure 4*a* using the Markov chains approach that the CRs of structural decks with uncoated rebars drop from 7 to 5 in 32 years. For structural decks with epoxy-coated rebars, this happens in 38 years. When the Weibull-based approach is used, CRs drop from 7 to 5 in 31.5 and 37.6 years for structural decks with uncoated and epoxy-coated rebars, respectively. Hence, deterioration rates using both Markov chains and Weibull-based approaches are similar for structural decks with coated and uncoated rebars during first 30–40 years. Durations for CRs to drop from 7 to 4 are 49 and 62 years by the Markov chains approach for structural decks with uncoated and coated rebars, respectively. These durations are 43 and 60 years, respectively, by the Weibull-based approach. It is obvious from plots in Figure 5 that structural decks with epoxy-coated rebars perform significantly better as the decks become older. This is because of higher corrosion in decks with uncoated rebars.

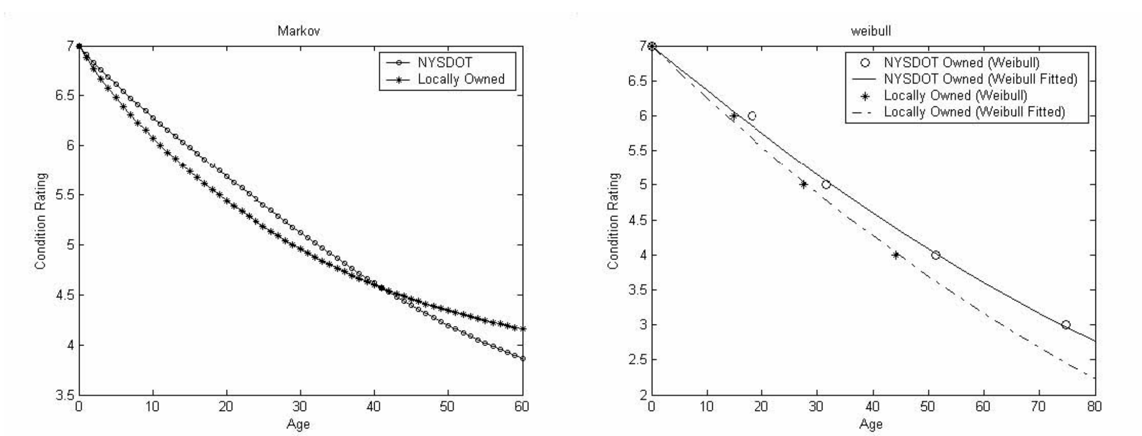
Case 3: Structural Deck, Cast-in-Place Uncoated Rebar, NYSDOT Versus Structural Deck, Cast-in-Place Uncoated Rebar, Locally Owned

Figure 5 shows plots of structural decks with uncoated rebars and owned by NYSDOT and by locally agencies (denoted as locally owned) using Markov chains and Weibull-based approaches, respectively. It is observed that decks of locally owned bridges deterioriate slightly



(a) **(b)**

FIGURE 4 Deterioration plots for structural decks with coated and uncoated rebars in bridges owned by NYSDOT: (*a*) using Markov chains approach and (*b*) using Weibull-based approach.



(a) **(b)**

FIGURE 5 Deterioration plots for structural decks with uncoated rebars: (*a*) using Markov chains approach and (*b*) Weibull-based approach.

faster than those owned by NYSDOT. Higher rates of deterioration in locally owned bridge decks may be because of several factors such as different design standards, different levels of maintenance, and other external factors.

Case 4: Pier Cap, Concrete, NYSDOT

Figure 6 shows deterioration plots for concrete caps in bridges owned by NYSDOT. It is observed from Figure 6 that the concrete piers cap deterioration predicted by both Markov and Weibull is almost similar for the first 30 years. The behavior of the Markov chains plot for pier caps older than 30 years is because of large scatter in inspection data. It is observed from Figure 6 that pier caps

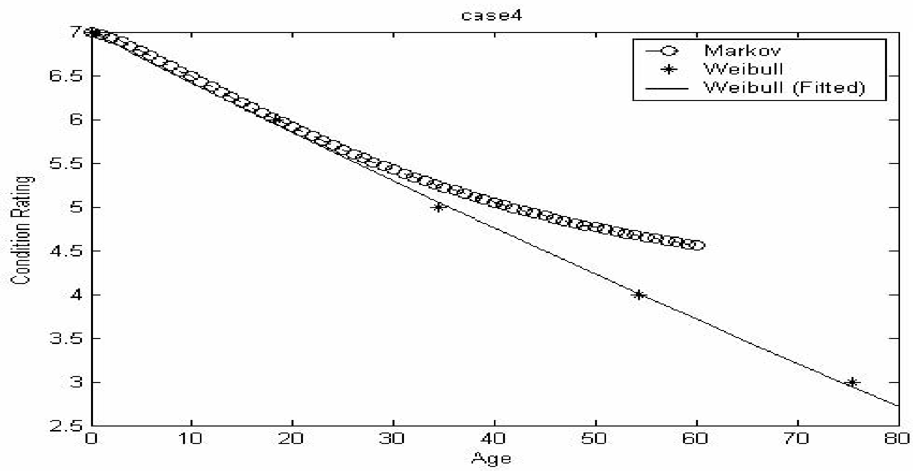


FIGURE 6 Deterioration plots for concrete pier caps in bridges owned by NYSDOT.

deteriorate to CRs 6, 5, and 4 in 18, 34, and 54 years. This information can be used for bridge management decisions, such as estimating and scheduling needed maintenance and rehabilitation work. It can also be useful in estimating the remaining life of a pier cap when deciding between a superstructure replacement and complete bridge replacement.

Case 5: Abutment Bearing (or Pier Bearing), Elastomeric

Figure 7 shows deterioration plots for elastomeric abutment bearings. It is observed from Figure 7 that abutment bearing deterioration predicted by both Markov and Weibull is the same for the first 30 years. Different behavior of Markov chains plot for abutment bearings older than 30 years is because of large scatter in inspection data of abutment bearings older than 30 years. It is observed from Figure 7 that abutment bearings deteriorate from CR 7 to CRs 6, 5, and 4 in 17, 36, and 58 years, respectively.

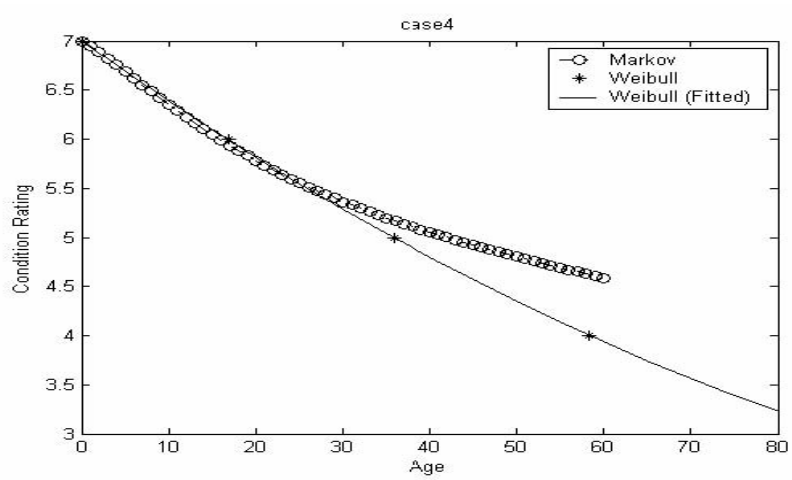


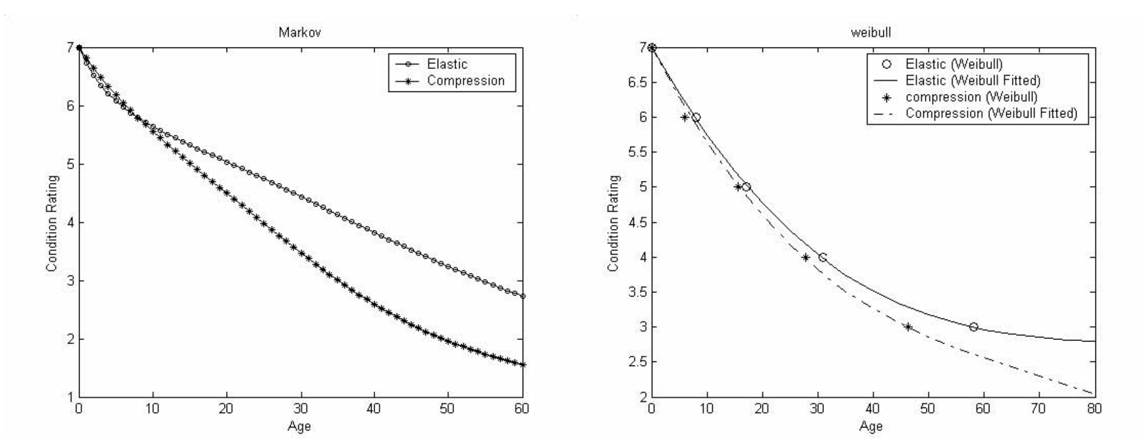
FIGURE 7 Deterioration plots of elastomeric abutment bearings in bridges owned by NYSDOT.

Case 6: **Abutment Joints with Elastic (Filled) and Compression (Including Armored) Materials**

Figure 8 shows deterioration plot for abutment joints with elastic (filled) and compression materials for Markov chains and Weibull-based approaches, respectively. It is observed that deterioration rates for both types of materials are almost the same during the first 9 years using Markov chains and during the first 30 years using the Weibull-based approach. The different behavior of plots by Markov chains than those of Weibull-based approach is because of segmentation of data in 10-year groups to derive the first-level Markov chains and larger scatter in inspection data of abutment joints with elastic material. From plots of Weibull-based approach, it is observed that CRs of abutment joints with elastic materials drop from CR7 to ratings 6, 5, and 4 in 8, 15, and 31 years, respectively. For abutment bearings with compression materials, the ratings drop to 6, 5, and 4 in 6, 16, and 28 years, respectively.

CONCLUSIONS

This paper presents the development of Markov chains and Weibull distribution-based approaches for the calculation of deterioration rates of different elements. An extensive data conditioning has been done by incorporating specific filtering logic based on detailed and available work history and extensive experience of NYSDOT engineers on the life cycle of different bridge elements. Deterioration curves have been developed using Markov chains and Weibull-based approaches by classifying bridges elements according to factors, such as NYSDOT region, material types, and design types. It is observed that deterioration plots for the Markov chains approach are significantly influenced by the duration independence assumption and scatter in inspection data and cannot account for unobserved duration at a rating because of bridge elements being built before the inspection started. On the other hand, deterioration plots by the Weibull-based approach are more reliable because of its ability to handle duration dependent and right censored data. The two approaches are programmed into a C^{++}-based computer



(a) **(b)**

FIGURE 8 Deterioration plots for abutment joints with elastic (filled) and compression materials: (*a*) using Markov chains and (*b*) using Weibull-based approaches.

program developed at the City College of New York. This program can be used to generate reliable deterioration rate plots for bridge elements inspected by the NYSDOT for bridge management decisions.

ACKNOWLEDGMENT

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Bridge Inspection

Local, Frequency, and Thermal Imaging

Supporting Local Bridge Inspections in Kansas with Web-Based Automation

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The Kansas Department of Transportation Bureau of Local Projects (KDOT BLP) is responsible for condition assessment and needs analysis for more than 20,000 bridges that are owned and maintained by municipalities, counties, and other government authorities. These bridges are often on critical travel routes for school buses, farm operations, and local delivery, as well as general rural transportation. The vast majority of the local structures are inspected by contractors. KDOT BLP has implemented a web-based system to collect bridge conditions statewide, with the objective of maximizing compliance with inspection schedules for all types of inspections, minimizing software implementation obstacles, and eliminating data delivery delays. The paper presents a review of the key design characteristics of the system, agency-specific implementation challenges, integration with the Pontis bridge management system database, software usability review, and an assessment of the software's fulfillment of the stated agency automation objectives. The paper also discusses technical details of the declarative software design, coupled with extensive database back-end programming support, as well as a comprehensive, fine-grained security mechanism, which combined provide KDOT BLP with a flexible, extensible bridge condition assessment system.

The Kansas Department of Transportation's Bureau of Local Projects (KDOT BLP) has responsibility for bridge inspections on more than 20,000 bridges in local jurisdictions. In order to comply with the U.S. National Bridge Inspection Standard (NBIS), these bridges must be inspected every 2 years by the local agencies directly or by their designated inspection agent. Local agencies have struggled to comply with this requirement for a variety of reasons, including funding and staff resources, and several attempts to use dedicated software products for bridge inspection, including versions of Pontis 4, have not proven successful. As a consequence, these agencies have been lagging behind in scheduling and completing their bridge inspections.

In order to address the data collection difficulties that the local jurisdictions have encountered, a web-based application has been developed and deployed for their use, focused on collecting the NBIS required information. The inspection system provides security-controlled access from any Internet-connected computer to all Kansas local agency bridge information stored in a Pontis 4.4 database through a web browser, thereby reducing the difficulty of training and deployment considerably. It controls access to bridges so that only each agency, or its inspection consultant, can access and modify data from their own bridges.

This paper provides a summary of the key problems that were driving the design of the web application, the solutions implemented, and an `verview of the software itself.

KEY PROBLEMS

The key problems that the bridge inspection application was expected to address were:

- Client–server software and bridge data distribution problems;
- Issues with staff training and availability, particularly the practice of relying on part-time inspection staff and its consequent problems maintaining continuity;
- Poor data quality, evidenced by errors and warnings generated from the FHWA's NBIS data validation system and the related Pontis data validator;
- Low levels of computer expertise and hardware availability in local agencies;
- Problems maintaining control of data access and modification privileges;
- Insufficient integration with KDOT corporate security infrastructure; and
- Inadequate local agency compliance tracking and notification mechanisms.

Issues that are not requirements for the application but relevant to the inspection program compliance rates include

- Uncertain bridge ownership and maintenance responsibility;
- Problems physically locating bridges;
- Logistical challenges of travel distance and site characteristics when conducting field inspections; and
- KDOT's desire to continue to standardize on the Bridgeware database specification for the long run.

It was evident that a centralized, web-deployed application, rather than a conventional client–server alternative, would provide immediate resolution of a number of the issues identified. Previous attempts to deploy the client–server bridge management system (BMS) Pontis 4 had been largely unsuccessful because of training hurdles, a significant data exchange management burden, lack of National Bridge Inventory (NBI) data check validations in earlier versions, and inadequate support staff expertise and availability levels within the BLP. Although a number of Kansas local agencies did successfully collect the NBIS bridge data with Pontis 4 and submit it to the BLP, the process was never institutionalized statewide successfully, and inspection program compliance levels continued to deteriorate.

The local agencies have only limited data collection responsibilities in actuality, and it became evident that a streamlined application, of whatever technology, would be more appropriate to their inspection program needs. Each local agency must perform a basic NBI inspection every 2 years, or less for structures of special concern. A basic inspection consists, in essence, of confirming all the bridge inventory, geometric, and service characteristics, and performing the condition assessment, which focuses, for most structures other than culverts, on the deck, superstructure, and substructure ratings as well as several other appraisal items. By focusing on the confirmation of existing data and providing a streamlined means to enter the condition and appraisal data, with a streamlined data entry interface, KDOT BLP hoped to expedite the inspection process for local agencies and improve inspection schedule compliance. Rather than burden local agencies with learning the powerful but complex Pontis interface, BLP wanted a simplified interface that would provide a list of bridges and a few form pages to collect the data for each bridge.

THE SOLUTION

A web-based solution was designed for the BLP based on the prior web deployment work and input from agency staff. The forms and business rules were drastically simplified related to those in the Pontis BMS, and new web application technologies were employed, including AJAX techniques, to improve the usability and performance of the application. The BLP application focused on two main requirements: provide controlled access to just the bridges of concern to the local agency, including several different lists that show various information about these bridges, and provide immediate access to the relevant data fields for each bridge. Subordinate requirements for a standard, printable bridge report, similar to the Federal Structure Inventory and Appraisal (SIA) form, as well as immediate field-by-field data validation on demand also were incorporated in the design. Pontis provides mechanisms for validating data according to federal standards, so the design incorporated interfaces to its software capabilities. As a Pontis licensee, KDOT BLP has access to these through the Pontis third-party American Petroleum Institute (API) specification. In order to take advantage of these Pontis tools, the application uses a standard Pontis database and is completely compatible with existing versions of Pontis.

Technical Architecture

The BLP application utilizes Microsoft.NET technology for the web application itself and relies on a third-party toolkit of custom web controls that provides all controls such as data grids, buttons, and other interface objects. The database for the application is Oracle 10g, Release 2, and the web server technology is Internet Information Server 6, both running on dedicated Windows Server 2003 hosts. Communication from the web application to the database uses the Oracle Data Provider (OLEDB) for .NET and ODBC for the reporting subsystem, over a TCP/IP network. All reports were implemented as coded HTML reports or with Crystal Reports XI, Release 2 (CRXIR2). CRXIR2 is .NET-compatible and provides a web report viewer capability that the application utilizes for end user reporting. There is no end user report tool in the system, although the bridges to include in each report can be selected ad hoc to drive the built-in reports.

The application is deployed to two distinct Internet Information Server (IIS 6.0) websites with connections to a central common Pontis database. This provides an external facing website using forms authentication for local agency use and an internal-facing website for KDOT BLP and other KDOT users that uses their network credentials to control access to the application. The two websites use a common code basis with only configuration changes necessary to offer the two alternatives. This provides for streamlined application administration and simplified updates in the future. These two websites do not operate in a “web farm” or “web garden” configuration, which would be the case for more elaborate load-balanced application deployments.

Compatibility with Pontis was an intrinsic aspect of the technical architecture. The web application does not share any user interface or middleware code with Pontis 5.x, but it does utilize the Pontis 4 C++ DLL capabilities to perform data validations. Further, it uses the same database security tables as Pontis 5.0, permitting KDOT BLP to access the local bridge data either with their proprietary web application or with Pontis 5.0 as needed. The administrative capabilities of Pontis 5 are used to define desktop bridge lists and bridge access groups, and perform other program administrative tasks common to both applications. The KDOT BLP application manages its own users and user roles, as well as mailing lists for automated e-mail reports.

User Interface Organization

The user interface for the web inspection application is organized into five main elements: a customizable bridge list, for navigating through the structure inventory bridge-by-bridge; batch tools, which provide data validation routines, maps, and inspection reports based on the current bridge list; detailed data entry pages, used for reviewing and editing detailed data; a scheduling and compliance tracking report system, which provides the ability to generate compliance reports ad hoc for a subset of structures; and administrative pages for the application. In addition, a reporting system based on Crystal Reports components permits review, print, and download of report results.

Bridge List

The bridge list (Figure 1) provides several tools for finding bridges and creating subsets of records. Each list is based on an SQL query, which must include the main bridge identifier. These lists are the same query definitions that Pontis 5.0 uses to display bridge information and are stored in the same table used by Pontis 5.0. The lists only display bridges that the user is authorized to view by their access privileges, which are configurable in the database through the administration screens. These filters are compatible at the database level with Pontis 5.0 as well.

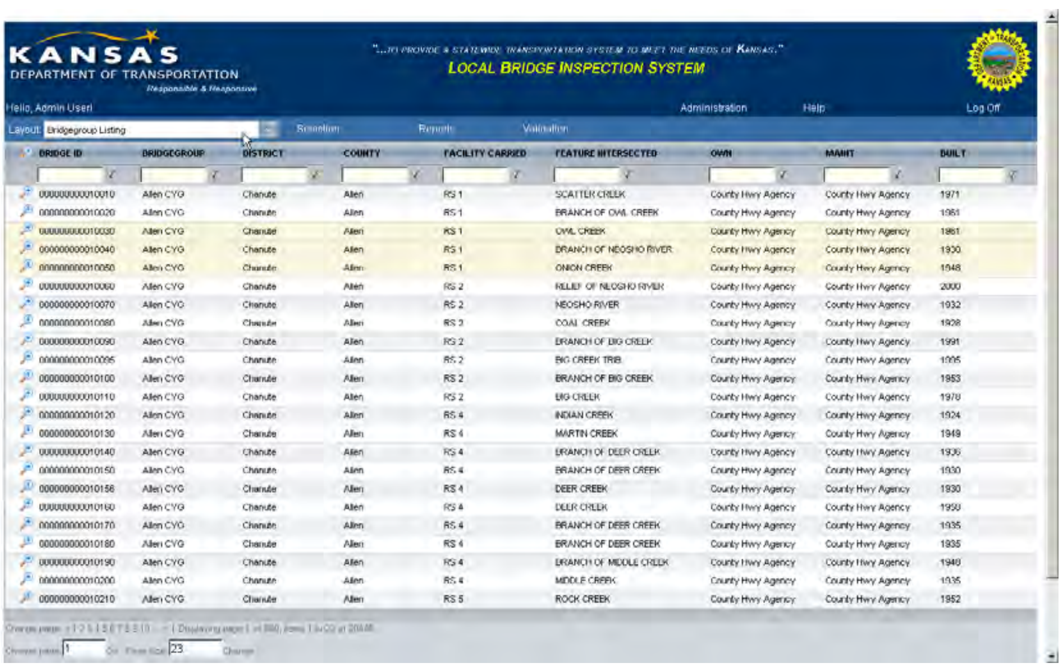


FIGURE 1 Bridge selection list example.

Several predefined lists are provided, such as:

- All bridges,
- Bridges by jurisdiction,
- Bridge condition listing,
- Uninspected bridges,
- Bridges with under roadways, and
- Tunnels.

The lists can be manipulated using the desktop grid control tool to sort and filter by any or several columns concurrently. This permits a generic list to be refined by the user to show the exact set of bridges that are necessary for the task at hand. There are a number of filter criteria options provided that can be set for several columns at once (Figure 2).

Batch Tools

These bridge lists drive reports and batch operations used to process a number of bridges at one time, either for all the bridges on the list or for the specific structures that are of interest. In the KDOT BLP application, the data validation tool relies on the current bridge list and specific bridge selections to identify the bridges that should have data checking performed. Figure 1

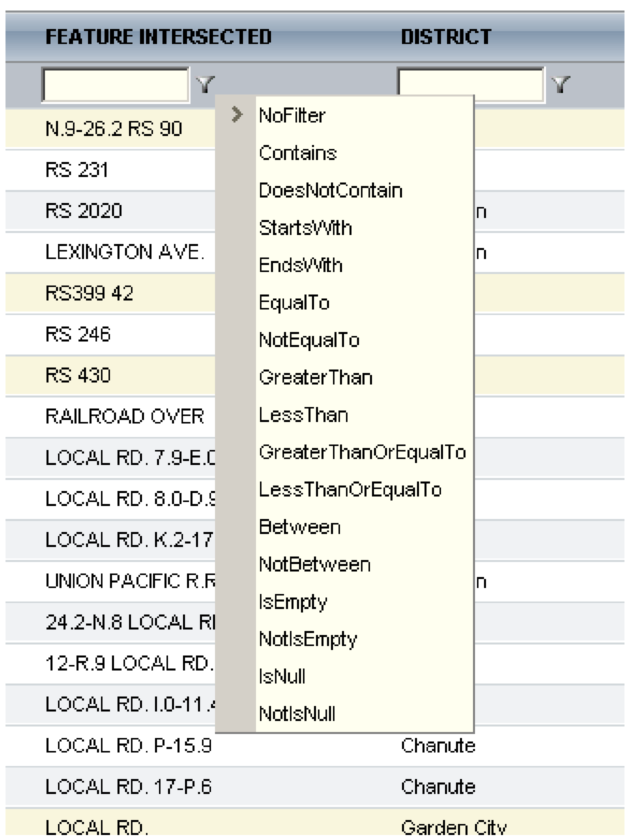


FIGURE 2 Bridge list filtering criteria.

shows a typical list with a few bridges selected. These structure selections can control a batch operation or it may operate over the entire bridge list. The validation tool utilizes the Pontis 4.4 edit check program directly, called from within the web application, in order to ensure that validation results from the web interface are absolutely identical to the results that would be generated by Pontis 4.4 when run in client–server mode, and moreover, because the Pontis edit checks match the NBIS edit checks, the validations performed by the KDOT BLP application will enforce all federal data validation rules for NBI data intrinsically. The batch tool first performs the edit checks and reports the number of warnings and errors, then provides the user with the opportunity to generate a validation report for further review (Figure 3). The same validations that are performed by the batch tool are also performed in the data editing detail pages automatically, before any data are actually updated in the database, to further ensure a high degree of data quality.

Standard bridge inspection reports can also be generated based on the list. These reports are based on the FHWA SIA report. These reports provide a single-page summary of the bridge information and latest inspection condition, which can become part of the formal inspection report from the local agency or its consultants. These reports can be saved as PDF for redistribution to interested parties or to use as a permanent digital record if PDF digital signing is implemented by KDOT BLP in the future.

A mapping "mash up" based on the public domain Google Maps API is also included in the batch tools. This mapping interface provides pinned map locations for all or selected bridges on the desktop list on demand, and shows key information for each bridge when the user's mouse pointer hovers over the structure. This simple mapping capability can also be used to access driving directions if necessary during field inspection operations. The map locations are based strictly on the latitude and longitude of the bridge derived from the database (Figure 4).

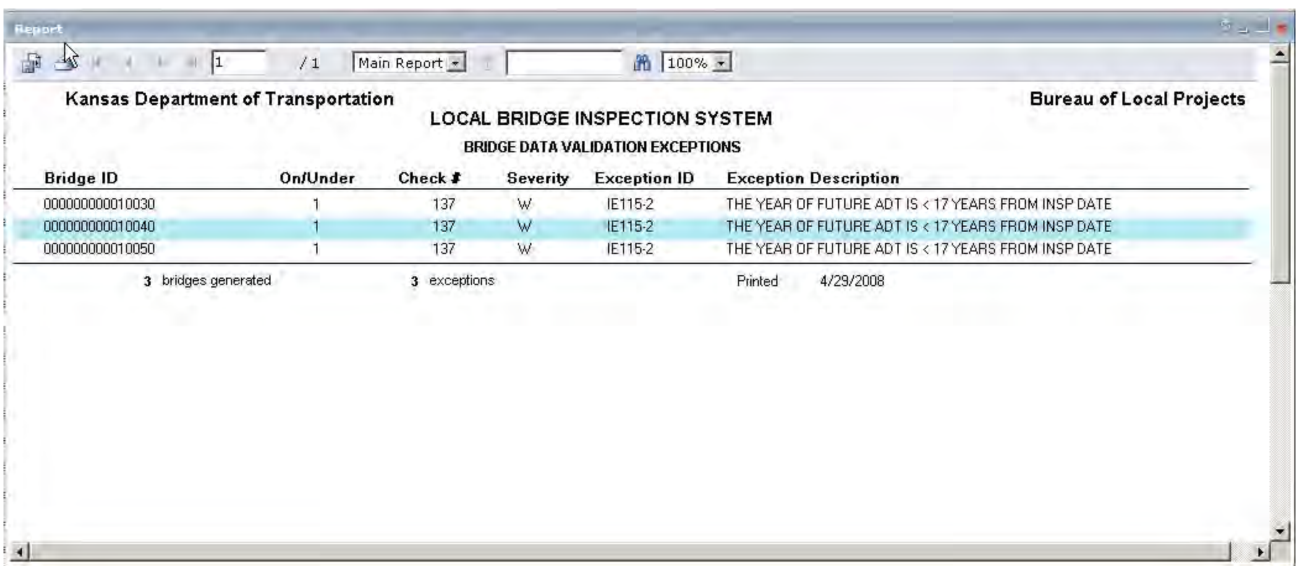


FIGURE 3 Data validation error display.

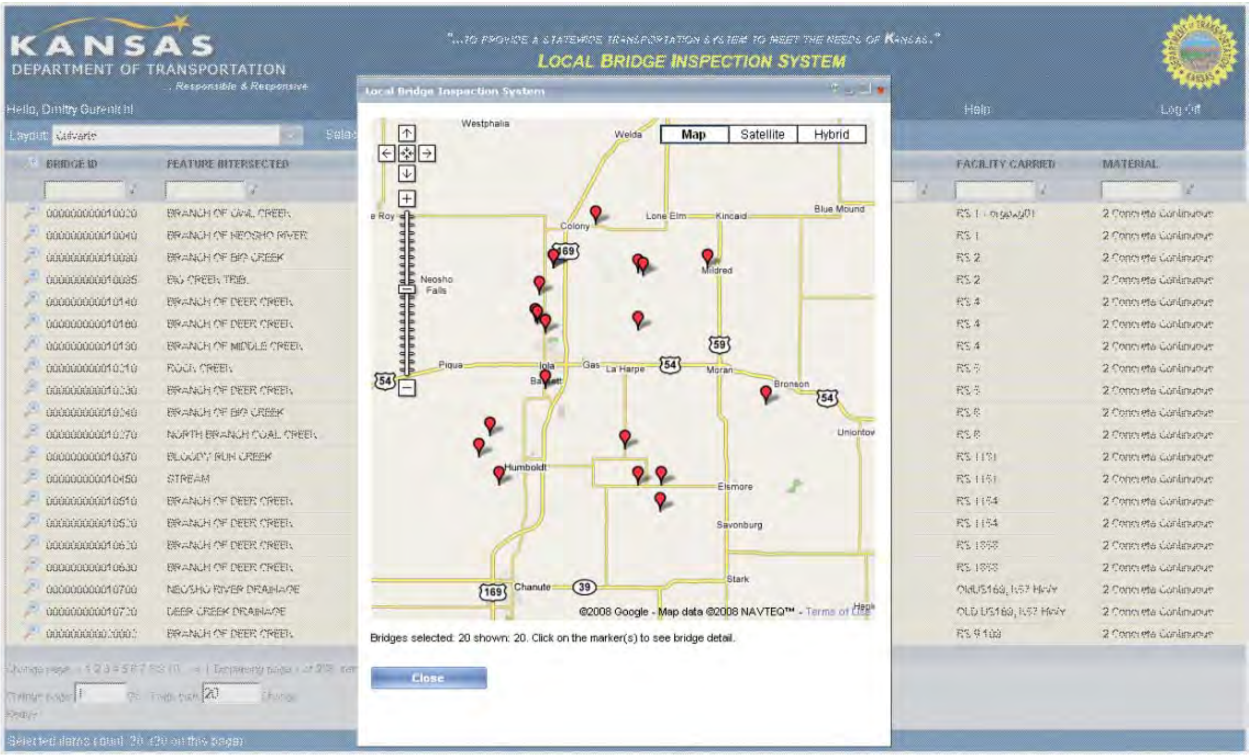


FIGURE 4 Google Maps mashup.

Detailed Data Entry Pages

Detailed data for each bridge are shown on a series of tab pages that are generated automatically when a bridge record is selected using the desktop list magnifying glass icon. The data forms are organized into several tab pages, and also provide capabilities for inspection approval, by qualified users, report generation, and individual bridge validation (Figure 5). For each field, a declarative editing option is provided, which means that the display and editing mode for a field can be changed, for example, to a pick list from free-form editing, or to a checkbox instead of entering Y or N, simply by changing the field settings in a database table. The set of tabs is predefined by design and the tabs cannot be manipulated to change order or labeling except through changes to an underlying database table. This form configuration process is purposely not accessible through the web application directly. With the inspection navigator, the user may move between multiple bridge inspections as needed.

Two types of inspections are defined in the system: interim and inventory. The interim inspections are performed every 2 years, but only selected data items are changeable during those inspections, notably the condition and appraisal data items such as deck and substructure condition. The system streamlines data entry requirements for these interim inspections. Inventory inspections are performed when a bridge is added to the inventory, or significantly rehabilitated, and the interface permits access to a larger set of data fields for these inspections accordingly.

It should be noted that the design intentionally omits the ability to add and remove entire structure records. The KDOT BLP has retained control over this business process to ensure that

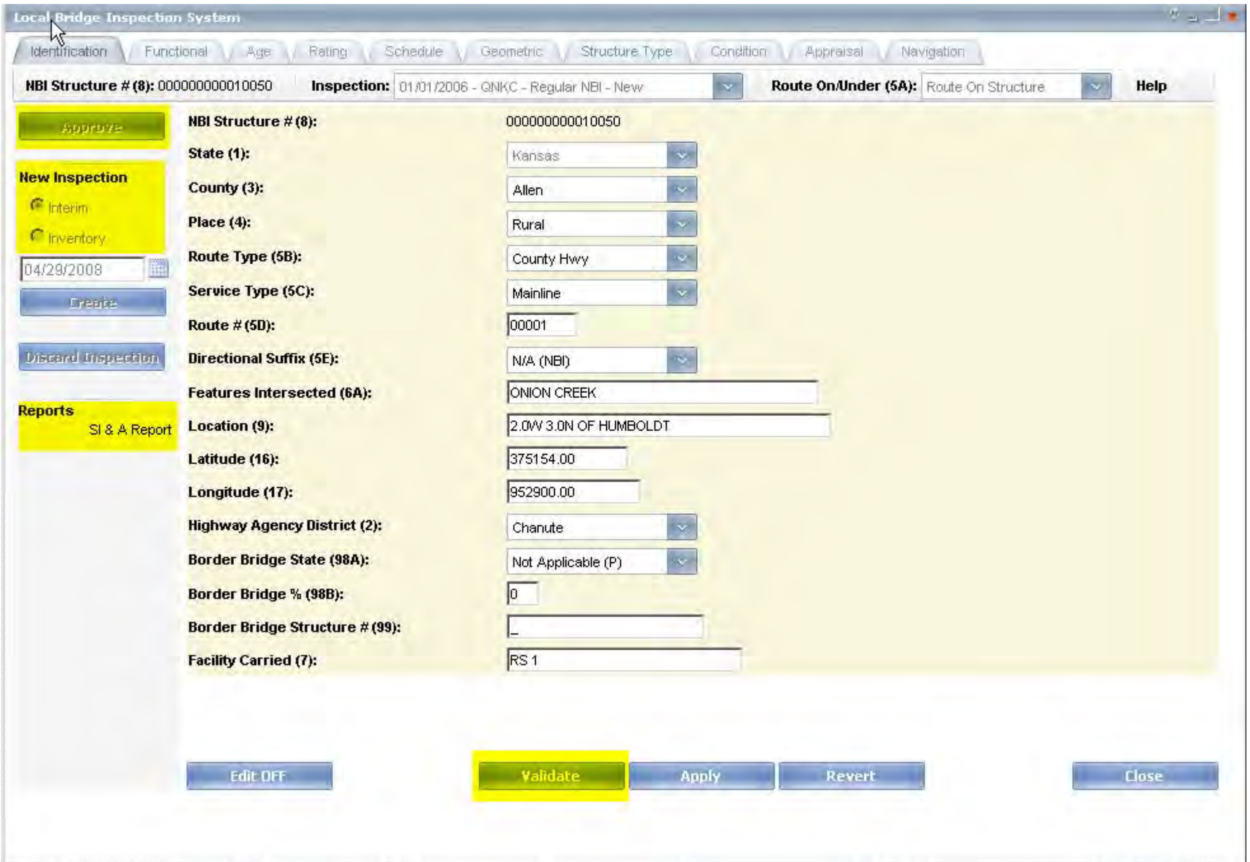


FIGURE 5 Data input forms example.

the bridge records are created with uniform minimum data standards, for example, for the bridge location, identification number, and other critical items. Further, formal removal of a structure requires additional administrative records management beyond merely removing the entry from the database, and the BLP staff preferred to not permit bridge record removal through the web application.

Data validation occurs on each tab page, either on demand, by using the Validate button, or automatically whenever data are saved. If validation fails, a warning message screen is displayed and the user must correct the data problems before the updated record can be saved (Figure 6). This dramatically reduces the potential for entry of erroneous data to the system and overcomes a singular limitation of both Pontis 4.4 and the previous web application prototype.

In addition, an SIA report can be printed for the current inspection at any time from within the detail tab pages.

Scheduling and Compliance Tracking Report Subsystem

The scheduling and compliance tracking report subsystem is used to prepare reports on upcoming and overdue inspections, based on date ranges and inspection type criteria. The report generation interface provides the ability to determine cutoff thresholds in days or months to determine whether a bridge is overdue or coming up due for an inspection (Figure 7). This

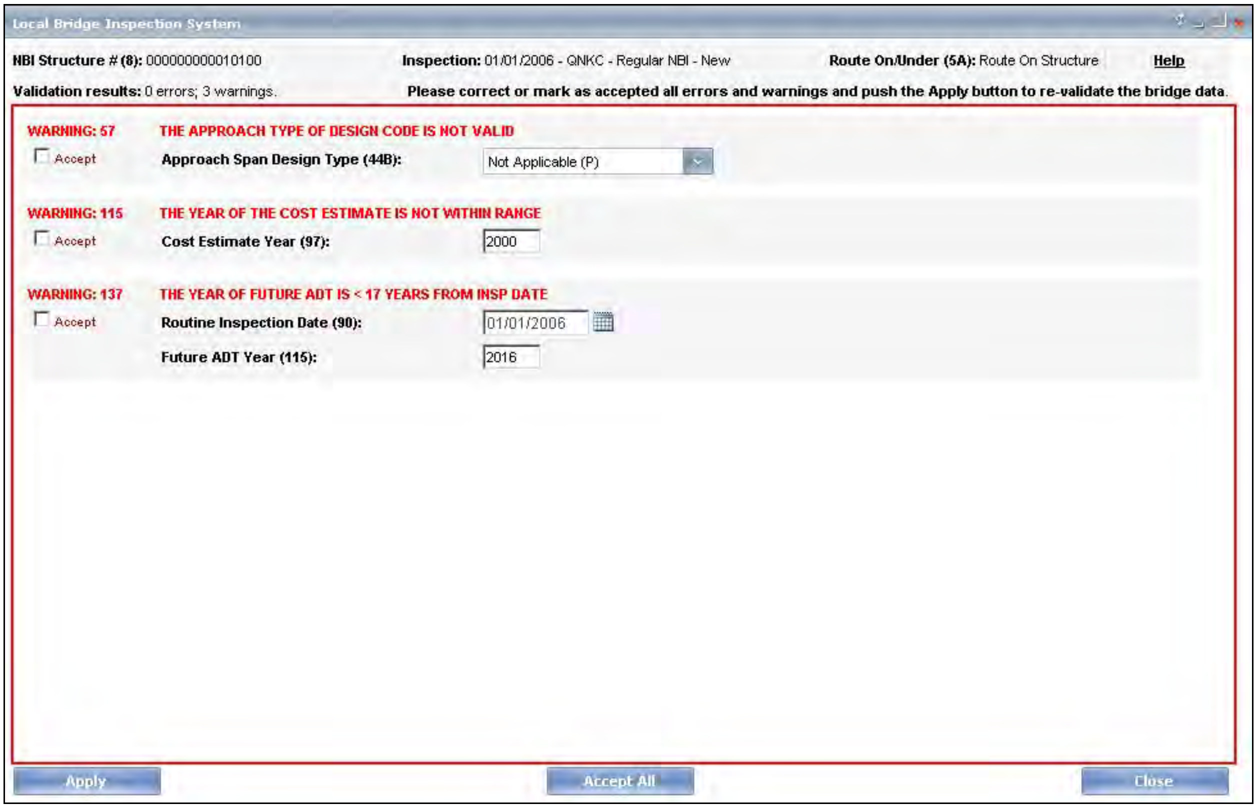


FIGURE 6 Popup display for data corrections.

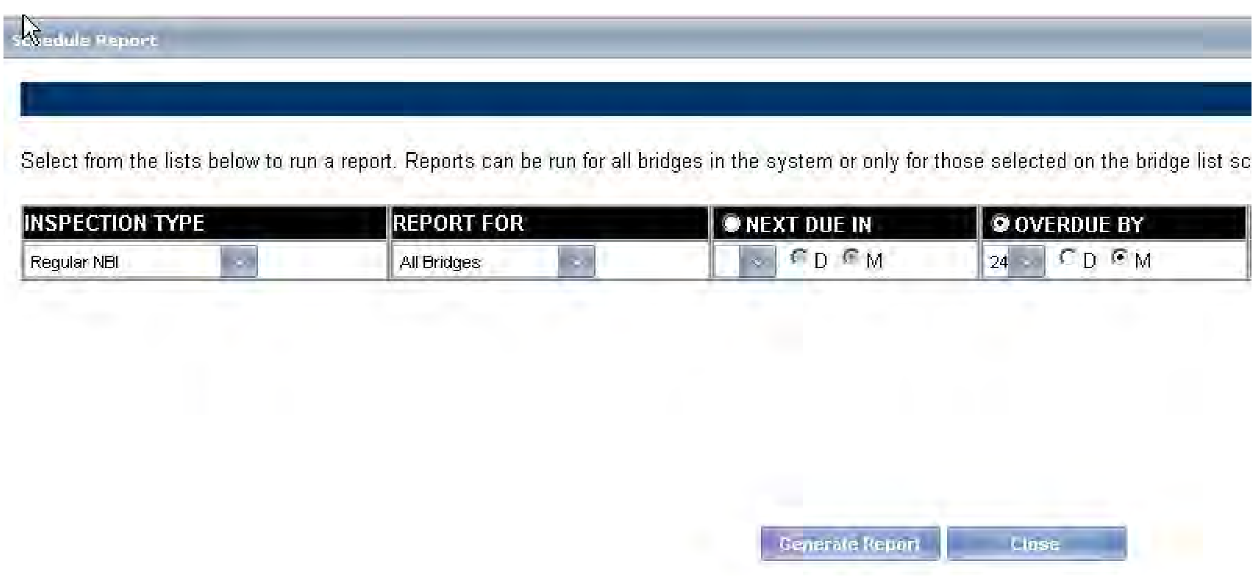


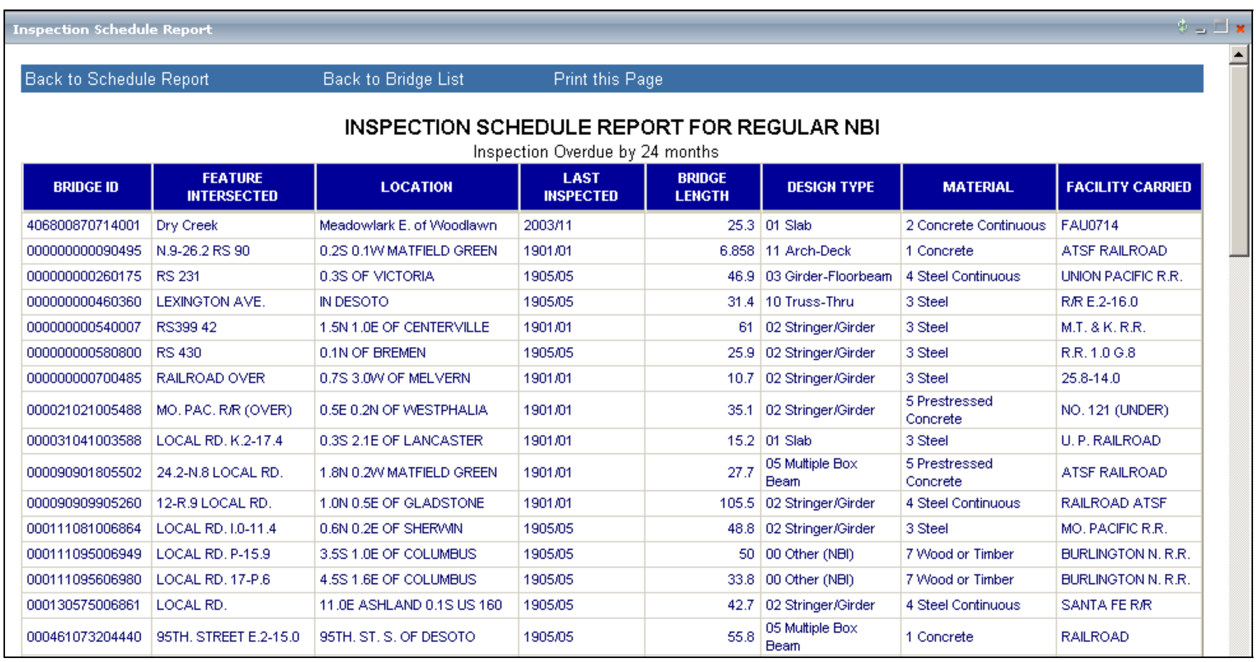
FIGURE 7 Schedule report generation interface.

schedule reporting subsystem provides KDOT BLP staff with the ability to determine which bridges need to be inspected, and contact the agency responsible, as well as estimate the number of bridges that will be coming due in a user-defined future time period. The on-screen inspection schedule reports can be browsed or formatted for printing (Figure 8). These reports can be generated by management and local agencies on demand, and will be a critical tool for improving KDOT BLP compliance with the NBIS inspection and reporting requirements. Agencies that are not in compliance risk the loss of federal funding for bridge work.

Within the database, automated reports can be generated and delivered electronically by e-mail to the bridge owners, on a scheduled basis, to notify them of bridges that are in arrears for inspection or that are coming due (Figure 9). This automated notification capability is managed within the Oracle database itself using Oracle job scheduling tools, but the report content is identical to the reports generated by the web application, and the report bridge selections and e-mail distribution lists are shared from the web application's administrative tables directly.

Administrative Subsystem

The application provides a web interface to common administrative tasks, for privileged users. While all users may use the password change capability, only users with the KDOT BLP web-administrator role have access to the full range of administration tasks (Figure 10). All the data for application configuration and administration, such as user role privileges, are stored in the actual database and are compatible with the Pontis 5.0 application framework. Storing the administrative information in the database permits multiple instances of the application to utilize the same administrative data concurrently and simplifies deployment of the application on multiple user-facing websites.


Inspection Schedule Report

Back to Schedule Report   Back to Bridge List   Print this Page

INSPECTION SCHEDULE REPORT FOR REGULAR NBI

Inspection Overdue by 24 months

BRIDGE ID	FEATURE INTERSECTED	LOCATION	LAST INSPECTED	BRIDGE LENGTH	DESIGN TYPE	MATERIAL	FACILITY CARRIED
406800870714001	Dry Creek	Meadowlark E. of Woodlawn	2003/11	25.3	01 Slab	2 Concrete Continuous	FAU0714
000000000090495	N.9-26.2 RS 90	0.2S 0.1W MATFIELD GREEN	1901/01	6.858	11 Arch-Deck	1 Concrete	ATSF RAILROAD
000000000260175	RS 231	0.3S OF VICTORIA	1905/05	46.9	03 Girder-Floorbeam	4 Steel Continuous	UNION PACIFIC R.R.
000000000460360	LEXINGTON AVE.	IN DESOTO	1905/05	31.4	10 Truss-Thru	3 Steel	R/R E.2-16.0
000000000540007	RS399 42	1.5N 1.0E OF CENTERVILLE	1901/01	61	02 Stringer/Girder	3 Steel	M.T. & K. R.R.
000000000580800	RS 430	0.1N OF BREMEN	1905/05	25.9	02 Stringer/Girder	3 Steel	R.R. 1.0 G.8
000000000700485	RAILROAD OVER	0.7S 3.0W OF MELVERN	1901/01	10.7	02 Stringer/Girder	3 Steel	25.8-14.0
000021021005488	MO. PAC. R/R (OVER)	0.5E 0.2N OF WESTPHALIA	1901/01	35.1	02 Stringer/Girder	5 Prestressed Concrete	NO. 121 (UNDER)
000031041003588	LOCAL RD. K.2-17.4	0.3S 2.1E OF LANCASTER	1901/01	15.2	01 Slab	3 Steel	U. P. RAILROAD
000090901805502	24.2-N.8 LOCAL RD.	1.8N 0.2W MATFIELD GREEN	1901/01	27.7	05 Multiple Box Beam	5 Prestressed Concrete	ATSF RAILROAD
000090909905260	12-R.9 LOCAL RD.	1.0N 0.5E OF GLADSTONE	1901/01	105.5	02 Stringer/Girder	4 Steel Continuous	RAILROAD ATSF
000111081006864	LOCAL RD. I.0-11.4	0.6N 0.2E OF SHERWIN	1905/05	48.8	02 Stringer/Girder	3 Steel	MO. PACIFIC R.R.
000111095006949	LOCAL RD. P-15.9	3.5S 1.0E OF COLUMBUS	1905/05	50	00 Other (NBI)	7 Wood or Timber	BURLINGTON N. R.R.
000111095606980	LOCAL RD. 17-P.6	4.5S 1.6E OF COLUMBUS	1905/05	33.8	00 Other (NBI)	7 Wood or Timber	BURLINGTON N. R.R.
000130575006861	LOCAL RD.	11.0E ASHLAND 0.1S US 160	1905/05	42.7	02 Stringer/Girder	4 Steel Continuous	SANTA FE R/R
000461073204440	95TH. STREET E.2-15.0	95TH. ST. S. OF DESOTO	1905/05	55.8	05 Multiple Box Beam	1 Concrete	RAILROAD

FIGURE 8 Inspection schedule report example.

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TO ALL RECIPIENTS: PLEASE DO NOT REPLY TO THIS MESSAGE.
It was auto-generated by the KDOT Bureau of Local Projects Bridge
Inspection Compliance Tracking System. You may contact Bridge
Inspection Program Management at the KDOT Bureau of Local Projects
for further information about this report.
==========================================================================================
==========================================================================================


Arrears Report of Bridges Overdue for REGULAR NBI Inspection
Criteria ( Days ) : >=30 & <= 59 Days Overdue
Report Run Date    : WED, 28 FEB 2007 10:19:14 CST
Report Sort        : ORDER BY v1.BRIDGEGROUP ASC, v1.COUNTY, v1.PLACECODE, v1.STRUCT_NUM ASC
==========================================================================================

Bridge Group: Lincoln CYG
==========================================================================================
Bridge           County Place   Featint / Facility        Last Insp.   Scheduled     Days    Freq.
==========================================================================================
000000000530190 / 105 / 00000 / BACON CREEK / RS 396     / 01/01/2005 / 01/01/2007 /    -31 / 24
000530701704202 / 105 / 00000 / STREAM / LOCAL RD.       / 01/01/2005 / 01/01/2007 /    -31 / 24
000530729004344 / 105 / 00000 / SALINE RIVER / MINOR COL / 01/01/2005 / 01/01/2007 /    -31 / 24
000530755204460 / 105 / 00000 / TABLE ROCK C / LOCAL RD. / 01/01/2005 / 01/01/2007 /    -31 / 24

==========================================================================================
# of bridges:      4
==========================================================================================


==========================================================================================
# of bridges meeting criteria in all groups:       4
==========================================================================================
```

FIGURE 9 E-mailed inspection schedule report.

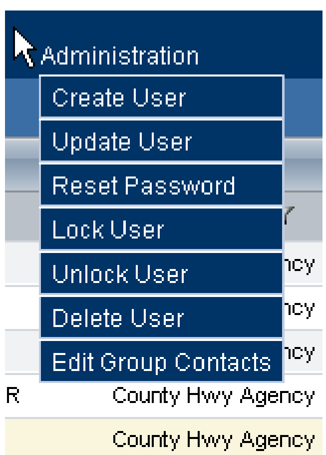


FIGURE 10 Administrative task options.

Security Organization

The application uses a standard .NET authentication provider, which manages the user login process, that interfaces with a proprietary application role management solution, or authentication provider, that is compatible with the Pontis 5.0 authorization implementation at the database level but shares no code with that software application. The authorization Both Forms authentication, with a standard login page, and Windows Integrated logins, where automatic logins depend on the user's corporate network access credentials, are supported. This permits KDOT staff users to connect with their normal intranet logins, while external users (the local agencies and their consultants) connect to the application with a conventional user identifier and password. For forms authentication, the security provider persists (stores) all user information in a dedicated user table structure managed by the application, while Integrated logins are authenticated against KDOT's Windows Domain Controllers.

The application defines a fixed set of five roles of increasing privilege with a browse-only role provided for ad hoc access to the application by KDOT management and other parties. The authentication provider conforms to the .NET provider specification, so the BLP may upgrade the authentication subsystem or substitute an entirely different authentication provider in the future, such as the Lightweight Directory Access Protocol/Active Directory with little impact on the application itself. The relatively simple role privilege mechanism may also be extended in the future if additional fine-grained control over application privileges, or even management of individual data items, is deemed necessary.

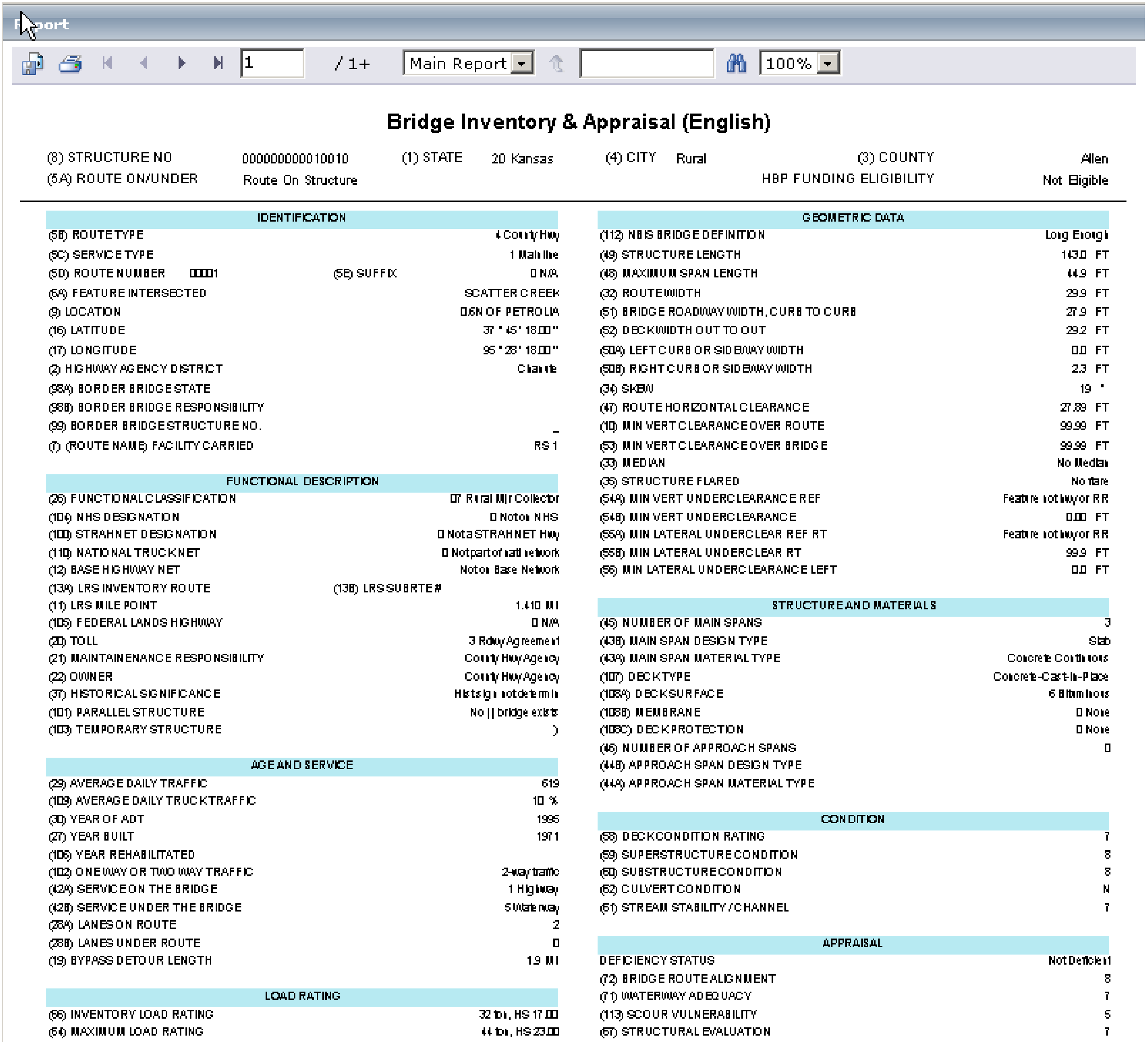
Reporting System

The reporting system employs Crystal Reports technology for report definition and generation. All reports are created in Crystal Reports externally to the system, then hard linked in the web application code to menu items. Within the application, the reports are automatically coordinated with the current bridge list during report generation. For display within the KDOT BLP application, a Crystal Reports web report viewer tool is used to provide standardized report page navigator, data export, and print capabilities to the user (Figure 11).

FUTURE DIRECTIONS

At the time of this writing, the current application is in the initial deployment phase. There are already several enhancements and new functionality requirements that have been identified by the Bureau of Local Projects. A partial list includes:

- Providing a public website for access to a controlled set of key bridge information without editing capabilities;
- Additional reporting capabilities, including more data mining reports, end user ad hoc report design, and the ability to deploy reports to a widely accessible agency report server;
- Supporting display and editing additional KDOT BLP data items;
- Providing a single-page, printable data entry form;
- Providing qualified users with the ability to add and remove structures from the system, with auditing;
- Improving management of the e-mail reporting subsystem;
- Providing the ability to send text messages to subscribers when an event of interest occurs, such as when bridge inventory data change, when a bridge becomes overdue for inspection, or when a bridge fails data validation;



Report

1 / 1+ Main Report 100%

Bridge Inventory & Appraisal (English)

(8) STRUCTURE NO	000000000010010	(1) STATE	20 Kansas	(4) CITY	Rural	(3) COUNTY	Allen
(5A) ROUTE ON/UNDER	Route On Structure					HBP FUNDING ELIGIBILITY	Not Eligible

IDENTIFICATION	
(5B) ROUTE TYPE	4 County Hwy
(5C) SERVICE TYPE	1 Mainline
(5D) ROUTE NUMBER 00001 (5E) SUFFIX	0 N/A
(6A) FEATURE INTERSECTED	SCATTER CREEK
(9) LOCATION	0.6N OF PETROLIA
(16) LATITUDE	37 ° 45' 18.00"
(17) LONGITUDE	95 ° 28' 18.00"
(2) HIGHWAY AGENCY DISTRICT	Chanute
(98A) BORDER BRIDGE STATE	
(98B) BORDER BRIDGE RESPONSIBILITY	
(99) BORDER BRIDGE STRUCTURE NO.	_
(7) (ROUTE NAME) FACILITY CARRIED	RS 1

FUNCTIONAL DESCRIPTION	
(26) FUNCTIONAL CLASSIFICATION	07 Rural Mjr Collector
(104) NHS DESIGNATION	0 Not on NHS
(100) STRAHNET DESIGNATION	0 Not a STRAHNET Hwy
(110) NATIONAL TRUCK NET	0 Not part of natl network
(12) BASE HIGHWAY NET	Not on Base Network
(13A) LRS INVENTORY ROUTE (13B) LRS SUBRTE #	
(11) LRS MILE POINT	1.410 MI
(105) FEDERAL LANDS HIGHWAY	0 N/A
(20) TOLL	3 Rdwy Agreement
(21) MAINTAINENANCE RESPONSIBILITY	County Hwy Agency
(22) OWNER	County Hwy Agency
(37) HISTORICAL SIGNIFICANCE	Hist sign not determin
(101) PARALLEL STRUCTURE	No || bridge exists
(103) TEMPORARY STRUCTURE	)

AGE AND SERVICE	
(29) AVERAGE DAILY TRAFFIC	619
(109) AVERAGE DAILY TRUCK TRAFFIC	10 %
(30) YEAR OF ADT	1995
(27) YEAR BUILT	1971
(106) YEAR REHABILITATED	
(102) ONE WAY OR TWO WAY TRAFFIC	2-way traffic
(42A) SERVICE ON THE BRIDGE	1 Highway
(42B) SERVICE UNDER THE BRIDGE	5 Waterway
(28A) LANES ON ROUTE	2
(28B) LANES UNDER ROUTE	0
(19) BYPASS DETOUR LENGTH	1.9 MI

LOAD RATING	
(66) INVENTORY LOAD RATING	32 ton, HS 17.00
(64) MAXIMUM LOAD RATING	44 ton, HS 23.00

GEOMETRIC DATA	
(112) NBIS BRIDGE DEFINITION	Long Enough
(49) STRUCTURE LENGTH	143.0 FT
(48) MAXIMUM SPAN LENGTH	44.9 FT
(32) ROUTE WIDTH	29.9 FT
(51) BRIDGE ROADWAY WIDTH, CURB TO CURB	27.9 FT
(52) DECK WIDTH OUT TO OUT	29.2 FT
(50A) LEFT CURB OR SIDEWAY WIDTH	0.0 FT
(50B) RIGHT CURB OR SIDEWAY WIDTH	2.3 FT
(34) SKEW	19 °
(47) ROUTE HORIZONTAL CLEARANCE	27.89 FT
(10) MIN VERT CLEARANCE OVER ROUTE	99.99 FT
(53) MIN VERT CLEARANCE OVER BRIDGE	99.99 FT
(33) MEDIAN	No Median
(35) STRUCTURE FLARED	No flare
(54A) MIN VERT UNDERCLEARANCE REF	Feature not hwy or RR
(54B) MIN VERT UNDERCLEARANCE	0.00 FT
(55A) MIN LATERAL UNDERCLEAR REF RT	Feature not hwy or RR
(55B) MIN LATERAL UNDERCLEAR RT	99.9 FT
(56) MIN LATERAL UNDERCLEARANCE LEFT	0.0 FT

STRUCTURE AND MATERIALS	
(45) NUMBER OF MAIN SPANS	3
(43B) MAIN SPAN DESIGN TYPE	Slab
(43A) MAIN SPAN MATERIAL TYPE	Concrete Continuous
(107) DECK TYPE	Concrete-Cast-In-Place
(108A) DECK SURFACE	6 Bituminous
(108B) MEMBRANE	0 None
(108C) DECK PROTECTION	0 None
(46) NUMBER OF APPROACH SPANS	0
(44B) APPROACH SPAN DESIGN TYPE	
(44A) APPROACH SPAN MATERIAL TYPE	

CONDITION	
(58) DECK CONDITION RATING	7
(59) SUPERSTRUCTURE CONDITION	8
(60) SUBSTRUCTURE CONDITION	8
(62) CULVERT CONDITION	N
(61) STREAM STABILITY / CHANNEL	7

APPRAISAL	
DEFICIENCY STATUS	Not Deficient
(72) BRIDGE ROUTE ALIGNMENT	8
(71) WATERWAY ADEQUACY	7
(113) SCOUR VULNERABILITY	5
(67) STRUCTURAL EVALUATION	7

FIGURE 11 Crystal Reports inspection report example.

- Adding an inspector and inspection team credentials management feature, to ensure that known, qualified inspectors are performing local bridge inspections, and integrating these credentials with the user authorization mechanism;
- Providing direct access to program elements that currently are controlled by editing the database records directly, such as data editing displays and validation rules; and
- Linking in other websites supporting the local bridge inspection process, such as federal and state inspection standards, training manual repositories, or Wiki forums dedicated to bridge inspection.

This list is incomplete and will doubtless be adjusted in the future. The BLP anticipates another version of the software in the winter of 2008–2009 to incorporate new functionality identified by both user experience with the current release and new requirements generated from ongoing agency business process improvements.

ACKNOWLEDGMENTS

The authors acknowledge the support of the Kansas Department of Transportation and the Bureau of Local Projects during the preparation of this paper.

Integrated Management and Inspection System for Maryland Counties and Cities

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Counties and cities own slightly more than half of the bridges on the National Bridge Inventory with the remainder being predominately owned by state departments of transportation. Counties and cities often face unique challenges and needs that make approaches taken by larger entities impractical. The State of Maryland also has a nearly even split between state and county or city ownership of its 5,000 bridges over 20 ft in length. A new integrated inspection and management software system is being adopted across the State of Maryland by many of these counties and cities: large and small, rural and urban, to meet their desires of better inspecting and managing their bridge inventories. The system is currently in use by the City of Baltimore and several of the state's largest counties covering well over 2,000 structures that range in size from major 3,000-ft spans to simple spans and large culverts. The software is designed to provide more efficient and less error prone on-site collection and entry of inspection data. The counties rely on private consulting firms to perform their inspections. Through the end of 2007 a total of 12 different firms have utilized the software in counties and cities throughout the state. They have adopted an entirely new process, moving from a largely paper-based approach to an integrated electronic one. This new approach has allowed the counties and cities to improve their analysis and accuracy of inspection data compared to the past approaches. Through the software a wide array of new capabilities are now available to both the inspectors and local government owners.

With more than 5.5 million people and geography that spans coastal areas to mountainous regions, Maryland contains a great amount of diversity in many areas. This is seen in the needs of its bridge community as well. There are ~5,000 bridges in Maryland on the National Bridge Inventory (NBI). Of these bridges slightly more than half are owned by the state and the remainder, 2,200, are owned by its 23 counties, the City of Baltimore, and several smaller entities. Counties' and cities' departments of public works or engineering offices are mandated to ensure that the inspection of these structures is performed to federal standards. To accomplish this private consulting firms with inspection expertise are employed to perform the inspections and generate detailed reports on each bridge for the county or city. Additionally, the local owner must report to the state the basic inspection details required by the FHWA for the annual NBI submission.

Inspection and management has traditionally been a very paper intensive process at the local level. As an example, the City of Baltimore with its 373 bridges would receive inspection reports collectively totaling more than 10,000 pages in length at the end of every 2-year inspection cycle. The task of reviewing and ensuring quality assurance and quality control (QA–QC) on this volume of information and this type of format is overwhelming. In addition to generating paper reports, the inspection consultant is also responsible for manually entering some very limited information for each bridge into a Microsoft Access file for submission to the state. Counties and cities were largely lacking any tools to assist them in managing their bridge population: prioritizing, searching, keeping detailed history, accessing all related files, planning future actions, tracking problems and deterioration on members, sharing structure information with other departments, and all of the other day-to-day tasks a bridge manager faces. This problem was especially acute in entities that owned more than 50 bridges and had greater amounts of information to attempt to organize in a usable manner. The problem was found to be growing at a time-linear rate as each new inspection cycle brought another wave of thousands of pages of documents. For medium and large counties this typically led to the need for ever-increasing space in file rooms for bridge-related documents.

To cope with the challenge of organizing all bridge information from field reports, capital and maintenance planning, drawings and plans, and correspondence, a number of counties, cities, and consultants in Maryland have sought and implemented a state-of-the-art computerized inspection and management system. Since each entity has its own unique requirements the system had to be extremely flexible and robust in dealing with their individual needs and internal organization. InspectTech's BridgeInspect software suite was used as the solution to meet the needs for an integrated end-to-end system. The core system was customized to correspond to the users' exact specifications with interfaces, work–process flow, maintenance items, additional county- and city-specific data and inspection forms, and security settings. The software contains two primary parts: an inspection component and a management system. The inspection software has both a field and stand-alone version that runs on tablet or laptop computers for use by inspectors while at the bridge site and a web-based office version for integrating the field data and finalizing the reports. This software contains all digital versions of the necessary forms needed to generate a complete inspection report (typically 20 to 30 pages in length). The management system is a completely web-based program that can be accessed securely from any office computer or from home with a correct username and password. County and city personnel are able to access all information on bridge structures from current and historical inspection reports, pictures, sketches, memos, and maintenance needs along with having numerous tools such as geographic information system mapping, full searching, and cost estimating components.

This paper is organized in the following sections. Section 2 provides background information on the former systems used for handling inspections and management. Section 3 provides an overview of the overall project goals and a high-level view of the main system components. Section 4 discusses the inspection process and software used to create a final report while Section 5 presents the details of the software used to enable the bridge management process. Section 6 ends this paper with conclusions and remarks on the overall project implementations.

BACKGROUND

Bridge System Information

The system has been utilized in nine of Maryland's counties and cities that collectively represent more than 70% of the state's population (Figure 1). Those counties that own the most bridges and have the greatest need for the system also tend to be those located in more dense urban and

FIGURE 1 Maryland counties and cities where inspection software has been used.

suburban areas. However, the most rural county (least dense population) in the state, Garrett County, has also adopted the software to assist it with its own needs, demonstrating the flexibility of the system to handle a wide range of organization needs.

The software has been utilized to inspect more than 2,000 structures in Maryland. This is composed of nearly 1,000 large structures (greater than 20 ft in length) and a nearly equal number of small structures (less than 20 ft in length). In addition the software has been used on a number of special bridges such as pedestrian bridges and the Washington Metropolitan Area Transit Authority's Metro system, which extends into suburban Maryland with long elevated track structures. The bridges the system must handle span a wide variety of features from rivers to major Interstate highways (some up to 12 lanes) and from railroads to major tidal waters. In the more urban areas the high volume of usage often requires significant maintenance of traffic to perform even minor tasks or inspections. Inspections of structures spanning railroad tracks or the Metro system require significant coordination with outside agencies that makes even the simplest tasks far more complex to achieve.

Most of the counties and cities have a regular maintenance and capital planning program to achieve their minor and major repair needs. Information obtained from the inspection program forms the basis for identifying bridge deficiencies and obtaining the necessary data for prioritization and funding requests.

Federal and State Requirements

The bridge inspection and management program is designed to meet and exceed the requirements specified by the FHWA and Maryland State Highway Agency (SHA). FHWA's National Bridge Inspection Standards (NBIS) require the collection of more than 200 pieces of inventory and appraisal data on all 20-ft or larger structures (*1*). The regulations additionally specify requirements (training and experience) needed for personnel performing the inspections. Maryland's SHA administers these NBIS details as well as adds an additional layer of requirements (*2*). As with most other states, Maryland collects element-level inspection data (*3*). Bridges have been divided into their primary elements and the evaluations of each of these elements are defined by requiring inspectors to quantify the amount of each element based on a varying number of condition states. In addition to element-level data, Maryland SHA has added on several additional inventory related fields that are not present in the NBI data and are required to be collected.

Inspection Process

All of the counties and cities utilize an engineering consulting firm to perform their bridge inspections. In Maryland, SHA provides a set of preselected consultant firms from which the counties can choose or be assigned a firm to perform the inspections (Figure 2). Firms that have utilized the new inspection software include Alvi Associates, David Schmidt Co., Greenhorne & O'Mara, Kennedy Porter Associates, Mercado Associates, M&N Diving, Parsons Brinckerhoff, STV Inc., Tuhin Basu Associates, Wallace Montgomery & Associates, Whitman Requardt & Associates, and Wilson T. Ballard. Some of these consultants have acquired the software independent of their own accord so they could continue to utilize it to perform inspections in areas that do not yet have it. The consultants perform an in-depth hands-on inspection of the structures and when necessary will perform new load rating calculations. The traditional deliverables prior to this inspection cycle were paper inspection reports for each structure along with updating a Maryland SHA Access file containing NBI and element-level data.

(a)

(b)

(c)

FIGURE 2 Diversity of Maryland bridges: (*a*) several bridges over I-170 in Baltimore City; (*b*) Hooper Island Bridge (Dorchester County); and (*c*) Roddy Covered Bridge (Frederick County).

Original Systems

Prior to implementation of the integrated inspection and management system the entities typically relied on a variety of differing formats in order to meet their needs. One of the main features was the utilization of the state's Microsoft Access file to store basic inventory data, condition rating information, and element level coding. In addition Excel spreadsheets were often kept for each task or bridge to keep track of soundings, coating conditions and ratings, guardrail and approach data, as well as maintenance information. The bulk of the narrative data of each inspection report is stored in printed reports kept in file cabinets and shelves for review and reference. In addition many of the files composing these reports are saved as PDF or Word documents. One of the main problems faced in this old piecemeal system was that when questions would arise on a bridge or bridges, information had to be either manually retrieved from a hard copy of a report or compiled from the correct Excel, Access, or Word files. Another issue was that when emergency situations developed, to obtain information the user typically had to be located physically within the primary administration building in order to electronically or physically access any of the information.

SYSTEM OVERVIEW

System Goals

Each of the counties and cities independently chose to implement an integrated inspection and management system. There were a number of common reasons and goals that the entities had as well as ones unique to their particular situation. First, all of the entities desired a system that would enable considerable efficiencies and time savings on both the inspection and management components. Starting with the inspection process there was a desire to establish a high level of quality and consistent format for all consultants to use. For several of the counties a key driving factor was to eliminate all errors from inconsistent data and dramatically increase the reliability of the information obtained via inspections since the data are utilized to make critical maintenance and capital planning decisions. Everyone also desired to unify all bridge information (inspections, as-built drawings, load ratings, work orders, etc.) into one place available from all computers securely via the Internet from any office or individual's home. This integrated bridge information and management system is able to serve a variety of user needs from maintenance to bridge managers to higher-level executives. Each subgroup within an implementation requires different permissions and functions to meet their unique needs and prevent a user from inadvertently corrupting data they should not have access to. Overall, the common goal was to dramatically streamline the inspection and management process for bridges and as a result save time and money, and increase the operating efficiency and safety of the bridge network.

System Structure

In order to meet the goals of each county and city, InspectTech's core BridgeInspect Collector and BridgeInspect Manager software was chosen and customized per the specific county or city specifications. Figure 3 shows the overall system architecture and how the various components

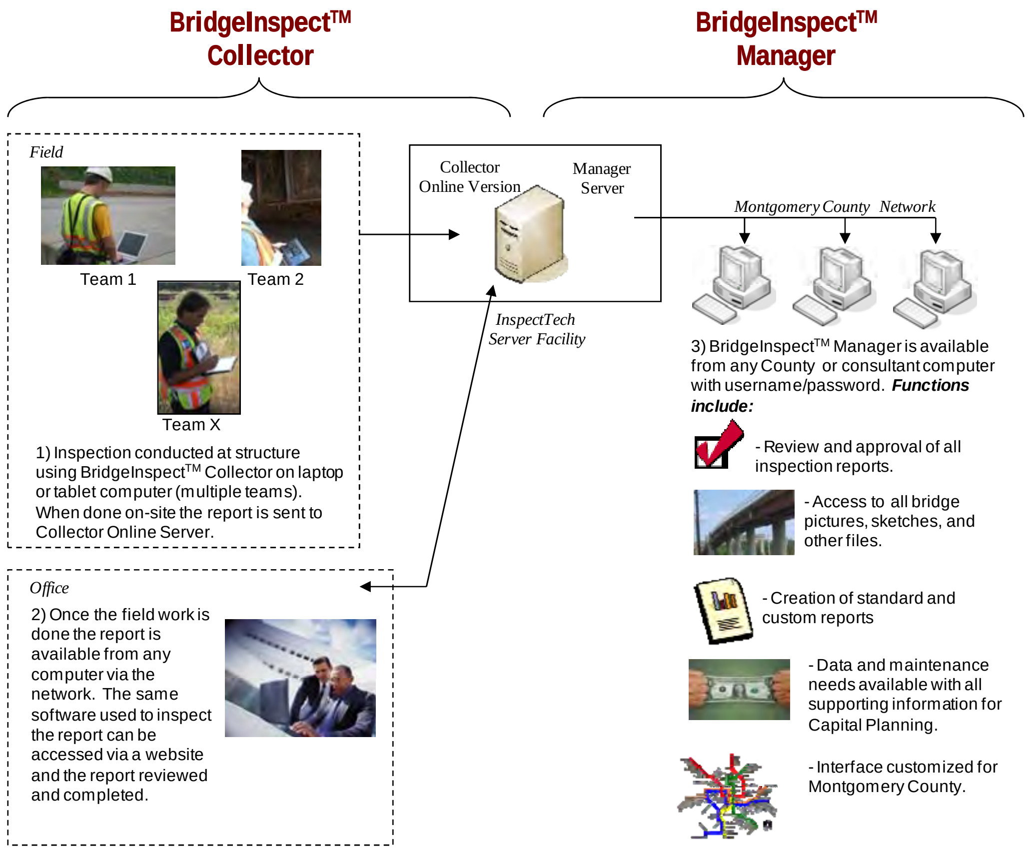

FIGURE 3 Software components by function (Montgomery County, Maryland, implementation).

fit together from field module to office-based management system. The system is composed of three primary components. The first component shown in the upper-left corner of Figure 3 is the field inspection software. This software runs on individual laptop or tablet computers and is taken to the bridge site in order to start the inspection report. Once the field work is done and the laptop is back in the office or at another location with an Internet connection the data collected are uploaded to the online inspection module (Figure 3, bottom left). Once the data are submitted to the online inspection module it is available for continued editing and final review by any office-based personnel. Detailed sketches or load rating analysis sections can be added to the report with multiple users working on different sections of the same bridge report. Once all sections have been added the report can go through internal QA–QC by the consultant project manager. When the consultant project manager is satisfied with the report he submits it to the owner entity for review and final approval. The county or city accesses the report using their bridge management software via the Internet (Figure 3, right). The bridge manager can approve the report and then data are available for usage in all of the various bridge management modules. The bridge management software runs entirely off a server computer and accessed securely through encrypted connections from any Internet computer.

Technical Requirements

The system software is designed to run on the typical computers that consultants and office-based managers already have. To access the online inspection or management software the consultants need only to be using Internet Explorer 6.0 or higher browser along with an Adobe Acrobat Reader program in order to view the completed reports, which are generated as PDF files. The stand-alone inspection software used in the field has more additional requirements as shown in Table 1. It is also recommended that for optimal performance the software be run on a computer designed for outdoor usage. Standard laptop or tablet computers are not designed for usage in outdoor weather or lighting conditions. On sunny days, glare or washout can render the screen unreadable on a regular computer. Additionally, rain or accidental drops could also severely damage a standard laptop. It is recommended that a semi-ruggedized or full ruggedized computer with outdoor optimized screen be used for field inspection work. Many of the input screens uti-lize drop down lists for entry and value selection, which make the added feature of a pen-based touchscreen optimal but not necessary for the field computer.

BRIDGE INSPECTION SOFTWARE

The bridge inspection software was implemented to correspond to the unique needs required by FHWA, SHA, each county's or city's own bridge department, and the consultant team performing the inspections. The field and Internet modules proved extremely useful by allowing the rapid collection of data and then the sharing of the information online between all members of the inspection team even when located in different offices. The intuitive layout of the software application required little training. The software contains all of the forms and entry screens necessary to complete a full bridge inspection report. This report is generated as a PDF file and can be printed out as a standard paper report or submitted electronically to the bridge management system. There are several core features and modules that allow the inspectors to quickly and reliably enter data and then generate the completed reports on bridges ranging from highly complex structures to simple box-beam bridges.

TABLE 1 Field Computer Hardware and Software Requirements

Component	Requirement
Processor speed (CPU)	800 MHz*
Operating system	Windows XP, 2000, Vista (all versions)
RAM	256 MB*
Screen resolution	1024 × 768 pixels (or higher)
Free disk space	1 GB (or more for storing pictures and sketches)
PDF Reader	Adobe Acrobat Viewer 5.0 or higher

Intuitive User Interface

The user interface, shown in Figure 4, was designed to facilitate quick and easy data entry. Overall, inspectors have found the software to literally be something they can just "pick up and go." From the top of the screen the user is able to quickly drill down into their desired entry form by selecting first the main tab and then the specific subtab that they would like. The main tabs correspond to the primary categories or groupings that inspectors normally use. Typical main tabs are SI&A (federal NBI fields), Pontis (element-level information), Condition Forms, county- or city-specific forms (additional fields the county or city has added), and Report Sections (forms related to report completion: pictures, organization, etc.).

The forms in the main window are dynamically generated and offer a number of powerful features to the user. Color coding is used to visually show what information has been preloaded from the last report and has not changed (yellow backgrounds) or data that are new or has changed (white background). The forms look identical to the standard paper ones that the inspectors have used in the past. However, additional features have been added in to make them more interactive. Unlike paper forms users can type as much information as necessary into any single field. Text areas will automatically scroll when necessary to support large amounts of data.

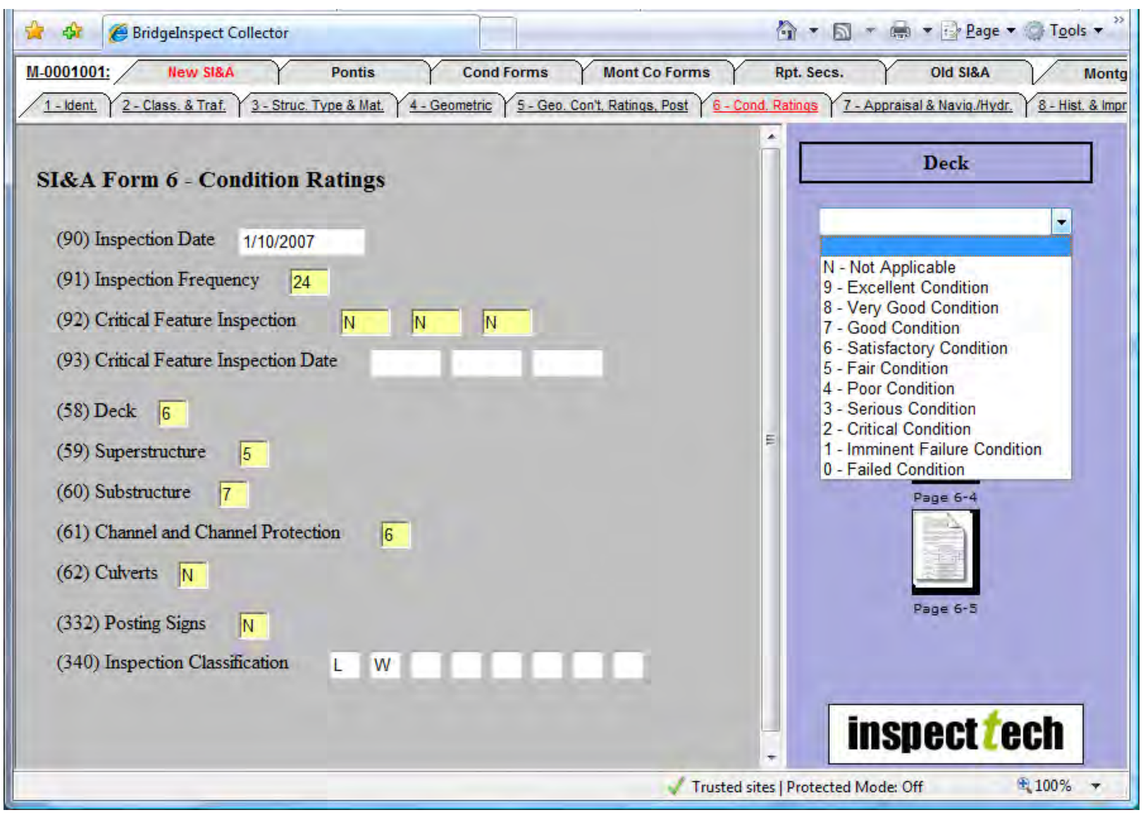

FIGURE 4 Inspection software screen showing tabbed menu across the top, main entry area on the left and bottom, and information sidebar on the right.

The same information often appears in multiple places on different forms. For example, the 58-deck rating is shown on an SI&A sheet, summary sheets, and even the heading of a section within the element-level data. Fields that appear on different forms but reference the same information are automatically linked in the software. Thus, information needs only be entered once on one form and is automatically filled in on all other forms. In addition to text areas on the forms other input options include check boxes and radio buttons. The user interface works on desktops, laptops, and tablet computers. Some of the consultants use touch-screen enabled computers and have the added advantage of being able to point and tap as the method of entering data, which the software fully supports. For long forms that cannot be displayed in their entirety within the main window vertical and horizontal scrollbars will automatically appear along either axis that needs to allow for the ability to scroll.

Integration of Manuals and Selection Lists

When one clicks on or selects a field in the main window the information sidebar located on the right of the screen (Figure 4) is automatically updated. On the top of the information bar is the expanded field title and description. Immediately below it is a message, if applicable, describing any restrictions on the field (i.e., Maximum Character Length 3). These restrictions often are imposed by federal or state database requirements. Most fields have a list of predefined options that the inspector can code for that item. A drop-down selection list is available showing all available options and brief descriptions for each item. Below the selection lists are thumbnail images of any coding manual pages from FHWA, state, county, or city sources in which the selected field is referenced. By clicking on the link, the manual page is instantly opened up as a PDF file on the inspector’s screen. This feature integrates hundreds of pages of detailed information in an easy-to-access format. When element-level inspections are conducted, an additional feature is that if the inspector clicks in a specific condition state field for a given element the side bar also displays the full text description of that condition state for that element along with feasible actions.

Support for Adding Pictures, Sketches, and Other Files

The BridgeInspect Collector software supports the integration of pictures, sketches, diagrams, and other digital file types. Integrating, labeling, and organizing pictures, sketches, and other files into reports is traditionally a painful process for inspectors. The interface shown in Figure 5 allows inspectors to select any file on their computer or any attached device to load into the report. Inspectors utilizing a digital camera can either download their pictures to the laptop or tablet or directly insert their camera’s memory card. Thumbnails of the pictures are automatically created of the file for display on the screen. Descriptions that describe the attached file can be added. In addition to pictures, sketches, diagrams, and output from any program (Word, Excel, Microstation, etc.) can be inserted in the correct location by importing them as a PDF file.

Maintenance Documentation and Cost Estimates

One of the critical tasks inspectors have is documenting maintenance needs that exist on the bridge and their corresponding priorities. The software supports an integrated list of county- or city-specific bridge maintenance items and cost estimates. Inspectors can select an item code, assign a priority, and reference pictures. Total cost estimates are automatically generated.

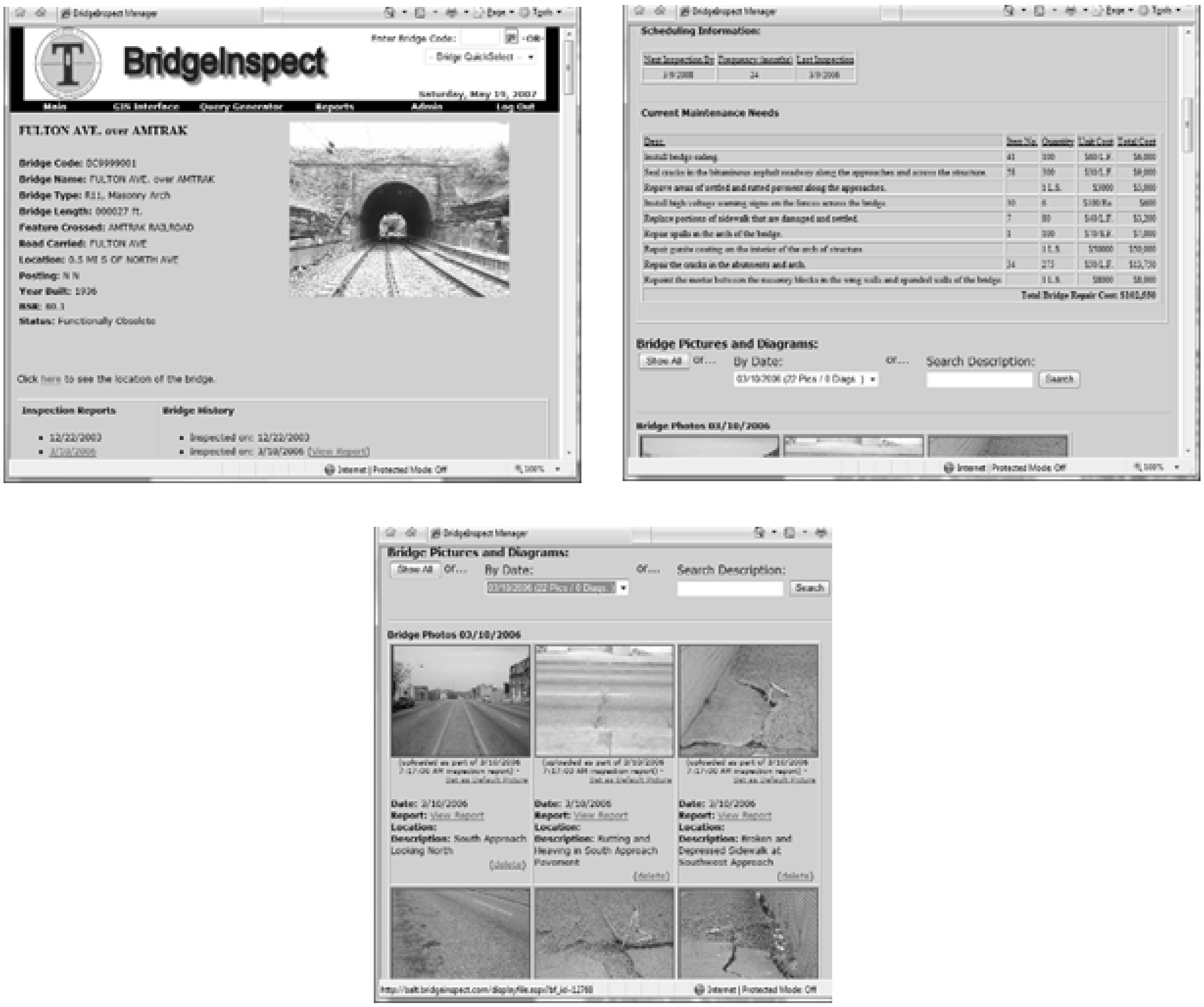

FIGURE 5 Bridge detail screen (selections from top to bottom).

Inspectors still retain the ability to override automatic cost estimates based on unique conditions and create entries for problems not on the standard item list.

Instant Formatting and Compiling of Reports

In the past inspectors have complained of spending far too much time trying to type and format reports. The customized software automatically stitches together a completely formatted report by organizing all information entered (narrative, pictures, sketches, PDF attachments, etc.). Formatting of the pages is done automatically and several sections are created with no effort by the inspector such as the cover, table of contents, summary sheets, and others, saving the inspectors considerable time in the office and allowing more time to be spent in the field. In addition to generating a PDF file for printing a complete paper report, the report data can be submitted electronically to the bridge management system as well as exporting relevant fields to the state's required database format.

BRIDGE MANAGEMENT SOFTWARE

User Interface Overview

The bridge management software is accessible via a secure website. Government personnel can reach the website from any Internet-enabled computer with their secure username and passwords. Based on their login information they are given varying permissions to access different modules within the management system. The manager software uses a drop-down menu based system that provides much the same interaction and experience of a stand-alone program. At any time users can select individual bridges by a drop-down list in the upper right corner or enter a specific bridge number. Like the inspection software the management program is extremely intuitive and easy to use.

Bridge Detail Page

When an individual bridge is selected or clicked on within a search result or from the interactive map, users are taken to the bridge detail page. This page serves as the integration of all information on the bridge structure. If users within the organization have any question on the structure they can come here to find out current conditions, historical conditions, trending, past and future work that has been scheduled, drawings, pictures (current and historical), and any digital file that has been attached to the bridge [memos, computer-assisted drafting (CAD), etc.]. The page is composed of many sections each of which provides a snapshot of information. Also on the page are links to full documents available. A user is able to retrieve complete inspection reports (latest or any historical) as a PDF file by clicking on a link. Summaries of all of the current maintenance needs for the structure are also present. Since pictures can be linked directly to the maintenance needs by the inspectors, managers are also able to drill down into the maintenance needs for specific pictures and detailed descriptions. A popular item present on the detail page is the load rating summary and link to the full detailed calculations. The detail page serves as a powerful index that allows the county management personnel to quickly reference information on a specific bridge that could fill a file cabinet.

Full Searching

The management software provides the capability to search across any field or combination of fields from within the inspection reports. This includes all inventory data, condition ratings, and maintenance information. If a manager needs to quickly identify all prestressed adjacent concrete box beam bridges with superstructure ratings of 5 or less and that were built before 1970, it can be done with a couple of clicks. Queries from very simple to extremely complex nested logic strings can be created and saved for future usage. Results are displayed in tabular form and indexed by bridge. Users can click on the bridge name or code to go directly to the bridge detail page for more information or click on a mapping link to plot all the bridges on the mapping interface to determine visually where the structures are located.

Document Storage

In order to serve as the single source for all bridge information, the management system allows for the uploading and direct storage on the server of any digital file. Files can be associated with dates and given descriptions during the submission process. As some examples, the management software can store Word documents describing agreements with utilities running on the structure, e-mails regarding actions to be done, CAD drawings, or a variety of other information. For other users to access this information stored on the server they have a direct link to download the file onto their computer. In order for the file to be opened, the user accessing the data will need that specific software application to read a file of that type (i.e., a Word file requires Microsoft Word to be installed).

Mapping Interface

The management software integrates directly with Google Maps (Figure 6). Using latitude and longitude coordinates already present in the inventory data, all bridges can be plotted directly onto an interactive map of the city or county. No additional software needs to be installed on the user's computer. Users can select between the roadway layer, a satellite layer, or a combination of both. The map allows for the county users to zoom in and out and see a very high level of detail in aerial photos of the structure. In addition to displaying the entire set of the bridges, the software can also display only an individual bridge or other subsets of bridges that are outputted from reports or that meet a specific search criterion.

Automatic Change and Trend Spotting

Managers need ways to see and track problems over time. All structures are deteriorating. The key question is how fast is the deterioration occurring and does it need to be addressed. The management software has built-in tools to automatically flag bridges on which the main federal condition rating values or element-level information has changed. Users can click on links to automatically see only ratings that have changed between the last inspection cycle and current cycle. The listing of changes can also be sorted based on the magnitude of the trend. Figure 7 shows an example of a condition rating that dropped two values (8 to 6) on a bridge that was only 3 years old. This ability allows managers to quickly find areas of concern, often before they become major problems.

Administrative Features

The software allows the administrative features necessary to handle the addition, editing, and deletion of bridges as well as new users. A user who is an administrator in the software can create accounts for others with specified permissions and correct access by structure or module. Additionally, accounts can be set up for the consultant inspectors to allow them permissions to retrieve data for performing inspections automatically from their laptop computers. This is especially useful when a new inspection cycle is starting. A variety of tasks can be done to allow for the full system maintenance and importing and outputting of data to external programs.

CONCLUSIONS

Bridge owners and inspectors of all sizes can significantly benefit from adoption of a customized

FIGURE 6 Interactive mapping interface showing bridges matching a search and specific bridge highlighted.

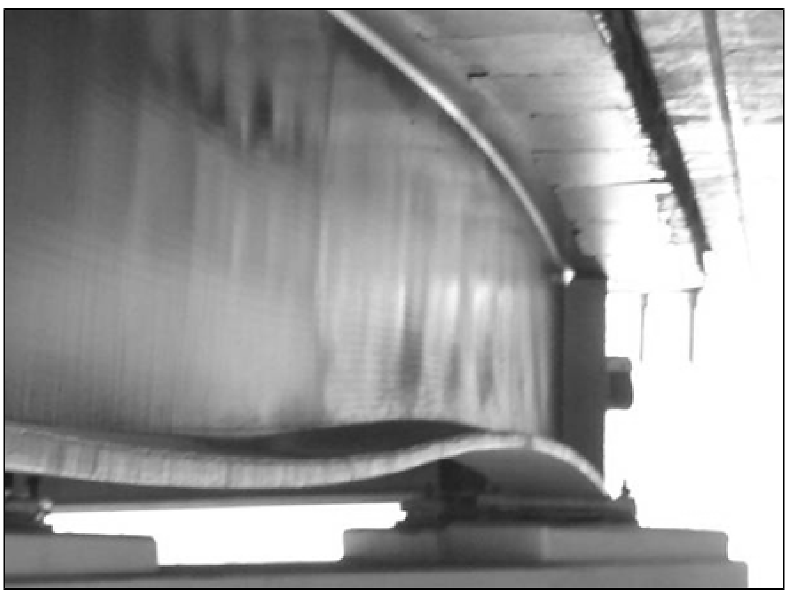

FIGURE 7 Example of problem identified in analysis of trends and changes.

and integrated inspection and management system as demonstrated in Maryland. The traditional processes of inspection with disjoint or incomplete databases and extensive usage of static paper reports lead to considerable inefficiencies, are prone to errors, and lack the flexibility or functions necessary for bridge managers or inspectors. The implementations across the State of Maryland demonstrate how both consultant inspectors and bridge owners can utilize the software to facilitate better communication, quicker results, and much more in-depth and usable information. Bridge inspection is far more than just collecting data for storage in a file cabinet with little practical usage. Effective bridge inspection software helps to highlight and provide quick and easy access to turn data into useful information. Problems can be quickly identified and documented, and action plans developed. The management software creates a unified location for the full documentation and plans on all bridges to be accessible throughout an organization, helping to prevent internal communication errors. The user-friendly nature of the software requires little training and fits well with the standard inspection flow. It has been demonstrated that the software does not create an extra burden for the inspectors but is instead a powerful tool that allows them to do their job without having to focus on tedious clerical work in organizing a report and entering duplicate information. Overall, the system has been extremely well received and has allowed the counties and cities to achieve their primary goal of integrating all bridge information in a single location for quick and easy access with tools to prioritize and highlight problem areas.

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Condition-Based Bridge Inspection Frequency

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Twenty-four years of historical database of the National Bridge Inventory (NBI) provide valuable information for bridge management investigations that can be used to minimize the uncertainty in the prediction of bridge conditions and as a result enhance decision making. Some bridge owners and stakeholders have suggested over the years that the mandated 2-year NBI inspection interval is not optimal in many situations and therefore potentially results in unnecessary inspection expenditures that may be better allocated for higher-priority maintenance activities. However, some bridge design–condition combinations may require more frequent inspections. The objective of this paper is to use the available NBI data for investigating the condition ratings and time-in-condition of various bridge design types and characteristics. An empirical probabilistic approach to investigate bridge inspection intervals potentially benefits all stakeholders, owners, and users. An optimal policy for inspection intervals can be expected to reduce the total inspection program cost and avoid unnecessary traffic delay or lane closure in order to inspect a bridge. An inspection interval can be based on the probability that the current bridge condition is about to change, thereby enabling appropriate decisions and actions to be taken at the optimal time. The proposed approach in this research effort investigates whether a future inspection can be based on bridge characteristics and external factors; current bridge condition; time spent in current condition; and the bridge condition before and after any observed maintenance or repair took place. The findings presented add to the knowledge base of bridge management and seek to advance the understanding of factors that may govern an empirical approach to bridge inspection intervals.

Thermal Imaging for Bridge Inspection and Maintenance

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This paper will discuss the research related to the development of thermographic methods for the inspection and maintenance of highway bridges. Subsurface deterioration in concrete structures presents a significant challenge for inspection and maintenance engineers. Cracking, delaminations, and spalling that can occur as a result of corrosion of embedded reinforcing steel can lead to potholes and even punch-through in concrete decks. The condition assessment of these structures typically requires lane closures to provide hands-on access for inspection, making inspections costly and logistically difficult. A new generation of infrared (IR) cameras provide the opportunity to perform inspections in a noncontact manner that can reduce the number of lane closures required, improve safety by providing a tool for monitoring conditions between periodic inspections, and improve the inspector's ability to rapidly determine the extent of deterioration to support maintenance and repair activities. The primary challenge to applying IR technology in the field is determining if the appropriate environmental conditions exist to provide measurable temperature contrasts between deteriorated and sound concrete, which is required for reliable imaging of deteriorated areas. Research results reported in this paper will describe the initial results of a study to develop effective guidelines for the application of IR thermography in the field.

There are approximately 600,000 bridges in the National Bridge Inventory (NBI), of which more than 60% are constructed with concrete superstructures. Most of the 600,000 bridges have substructures and decks constructed from reinforced concrete elements. With the average age of a bridge in the inventory being 44 years (*1*), there is an increasing focus on the condition assessment, repair, and rehabilitation of concrete components in bridges. Defects such as spalls and delaminations in the concrete that occur as a result of corrosion of embedded reinforcing steel reduce the durability of the structure and may require immediate repair. As a result, there is a need to improve the ability of maintenance and inspection personnel to detect and quantify subsurface deterioration so that repairs can be made. Traditional inspection methods such as hammer sounding can detect subsurface deterioration in many cases, but require arm reach access to the surface being inspected. This frequently requires lane closures or other traffic disruptions, and can be time-consuming for the inspectors. The application of infrared (IR) thermography for detecting deterioration in concrete bridge components may provide a means for imaging large areas of a structure from a distance, reducing inspection times, and averting traffic disruptions. However, the technique is dependent on certain environmental conditions to provide thermal gradients in the concrete so that subsurface deterioration can be detected. This paper discusses experimental testing designed to identify the optimum conditions for the detection of subsurface features in concrete bridge components. The experimental testing involves monitoring the thermal response, using both IR cameras and traditional instrumentation, of a large concrete block with targets embedded at various depths. Environmental parameters including wind speed, and solar loading and ambient temperatures are assessed to determine their influence on the detection of subsurface features in the block.

BACKGROUND

New methods to detect subsurface defects in concrete structures have been emerging in recent years. These new methods include ground-penetrating radar (GPR), impact echo (IE), and ultrasonic pulse velocity (UPV), among many others. These new methods supplement traditional methods, such as hammer sounding and chain drag techniques, which may not adequately address current demands in bridge inspection (*2*). The primary disadvantage of these traditional methods includes the time required to implement the methods over the entire bridge, and surface access required. To achieve the necessary access, the use of specialized inspection vehicles and traffic control is frequently required, which leads to traffic disruptions. This is equally true for newer methods such as GPR, IE, and UPV, each of which requires access to the surface of the component under inspection. IR thermography, the technology this study focuses on, has gained increasing use in many industries in recent years due to the advancement of imaging technology (*3*). Portable hand-held thermal cameras (IR cameras) are capable of imaging large areas from a distance, minimizing traffic interference, and allowing for the assessment of large areas in a small time period. Additionally, data are presented in real time during the inspection and typically do not require postprocessing. However, IR thermography has some limitations, such as the camera's inability to determine the depth of the features imaged in most cases. Perhaps more significantly, the method relies entirely upon the environment to provide thermal contrasts necessary for defect detection.

The basic principle behind thermographic inspections of civil structures is that environmental variations result in a thermal gradient in a concrete structure. The transfer of heat through the structure is disrupted by the presence of subsurface defects, such as delaminations. The disruption in thermal transfer manifests in an observable change in temperature at the surface of the structure, which in turn is detected by an IR camera and displayed on an image. IR cameras infer the temperature of a material by measuring the electromagnetic radiation emitted from the surface.

All materials emit radiation in the IR range when their temperature is above absolute zero. The rate at which this energy is emitted is a function of the temperature of the material and its emissivity. The emissivity of a material is the characteristic rate of thermal radiation, relative to a black body. For concrete, emissivity values are typically between 0.92 and 0.97, but vary according to the specific material composition and surface texture.

However, there are many other factors that influence the IR image produced. First, the composition of the structure under inspection can be of different materials; anomalies such as dirt, moisture, staining, and coatings can affect emissivity of the surface and therefore alter its appearance when observed with the IR camera**.** As a result thermal contrast may appear in an image that is not related to any defect or flaw. Second, the environmental conditions surrounding the structure, such as ambient temperature, solar loading, humidity, and wind speed, all have an effect upon the thermal conditions produced and hence the IR image.

Much of the time the thermal gradient created by environmental conditions is relatively small. Direct sunlight (solar loading) can produce a significant thermal gradient by heating the surface of the structure through radiant heating. However, wind can mitigate this effect by drawing away heat through convective cooling. The ambient temperature (air temperature) will also have an effect through convective thermal transfer and may be adequate to produce the necessary thermal conditions to make subsurface defects observable. The humidity of the air itself will also affect the convective properties of the air. As a result of these factors, determining optimum conditions under which an inspection using an IR camera would be effective can be difficult. The research reported here examines the effect of environmental conditions on the thermal contrast resulting from Styrofoam targets embedded at different depths in a large concrete block. Ambient temperature information (°C), solar loading (W/m^2), wind speed (m/s), and relative humidity (%) are measured and analyzed in relationship to the thermal contrast created by the embedded targets as a means of determining the optimum conditions for IR inspections of highway bridges.

RESEARCH APPROACH

To evaluate the effect of environmental influences upon a concrete structure, a large test block was constructed as shown in Figure 1. Styrofoam targets of known thickness, area, location, and depth were embedded in the concrete. These were cast at depths of 1, 2, 3, and 5 in. on the north and south faces, and depths of 1 and 3 in. on the east and west faces. The north and south faces of the test block were 8 ft x 8 ft, and the block is 3 ft thick. Thermocouple arrays were manufactured and cast into the block to provide internal block temperature measurement through its thickness.

FIGURE 1 Photograph of test block during construction.

The environmental data were measured using an on-site weather station, collecting the data for variables such as ambient temperature, solar loading, wind speed, humidity, and rainfall. Data were collected and stored by a stand-alone computer system at 1-min intervals, along with the thermocouple temperature readings at the same intervals. A LabVIEW data controlling environment was created to display and store this data on the computer.

The IR research camera (FLIR S65) was used to constantly monitor the surface of the block. As with the data stored by the thermocouples and the weather station, the IR camera is set up to store its data directly onto the stand-alone computer at 10-min intervals. The research, currently focusing upon the data collected from the north and south faces of the test block during various weather trends and conditions, highlights a variety of conditions in which the defective area is visible with the IR equipment (Figure 2).

The image shown in Figure 2*b* corresponds to the concrete test block being monitored by the IR camera from the south side, which is exposed to the sun in the southern sky. Temperature differentials (thermal gradients) were measured by the thermocouples, while thermal contrasts were taken between the surface area directly above the targets and the surface area directly above sound concrete.

The IR image displays a two-dimensional temperature representation of the entire face of the block and a temperature at any one point can be taken by choosing that corresponding pixel. To represent measured temperatures at the locations of the embedded targets, individual pixels are chosen from the IR camera image to represent each of the four targets (the pixel locations are fixed for all subsequent analysis). A pixel is also chosen to represent sound concrete, away from the embedded targets. The difference in temperature between the pixels in the sound area of the concrete block and the individual targets represents the thermal contrast apparent in the image, and provides a means of quantifying the thermal variations apparent in the IR images.

(a) **(b)**

FIGURE 2 (*a*) Completed test block being monitored and (*b*) IR image of same block showing subsurface targets at 1, 2, and 3 in. (5-in. deep target is not apparent).

PRELIMINARY RESULTS

The research reported here describes results observed on the south side of the block, which is exposed to direct sunlight. At this stage in the research, fully developed conclusions are yet to be drawn; however, useful results have already emerged. Figure 3 shows the results from the first week in January 2008, which is presented as an example of the data being collected and studied in this research. Each graph compares an environmental influence with the response in the form of a thermal contrast at the four defect locations. Figure 3*a* shows ambient temperature changes over the week, Figure 3*b* shows solar loading for the same week, while Figure 3*c* shows the week's changes in wind speed as measured by the on-site weather station.

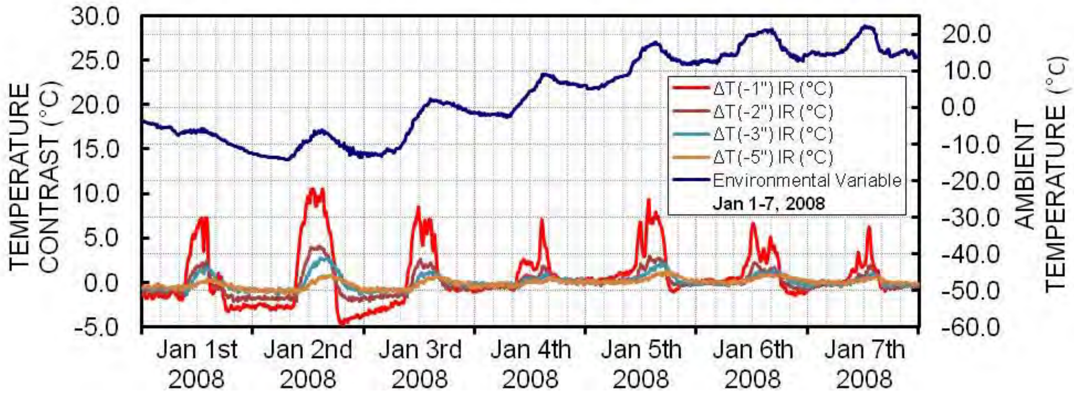
In Figure 3, the left axis always shows the thermal contrasts as seen by the IR camera and the right axis is the environmental variable to which it is being compared (always the blue line), with the duration of 1 week running across the x-axis with minor increments of 4 h.

What should be pointed out is that the main focus of this research is to determine a "good day" versus a "bad day" for detecting subsurface defects with an IR camera. For example, on a good day, there would be a large clear range of significant thermal contrasts for all four depths, which are identifiable for a reasonably long period. This occurs in the case of January 2, 2008, with an almost sinusoidal waveform in the thermal contrast data being driven by environmental conditions. While on the other hand, a relatively bad day can be seen, as represented by an insignificant thermal contrast which in this case occurs on January 4, 2008. At this point, the second looks to be an example of a model good day.

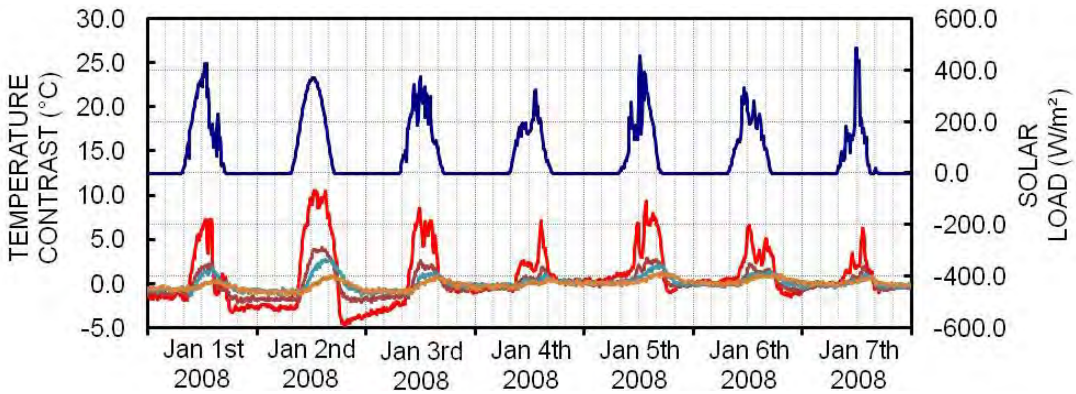
Examining the data for January 2, there is positive change in ambient temperature that day of approximately 10°C, there exist thermal gradients of about 10°C for the 1-in. target, 4°C for the 2-in. target, 3°C at the 3-in. target, and approximately 1°C at the 5-in. deep target. These contrasts can be expected to be clearly visible to the IR camera for a relatively long duration, when compared to the short contrast period of January 4, for example. Figure 3*b* shows the solar loading, which is consistent throughout the entire day of January 2, again especially prominent when compared to that of January 4. If one looks at Figure 3*b* for the entire week, a pattern can be easily identified. Despite the magnitude of the solar loading, if the solar loading curve is not relatively consistent throughout the entire day, i.e., no cloudy regions (which are signified by a sharp drops in the solar loading curve), then there will be diminished thermal contrasts evident in the IR image.

Another contributing factor to the loss in thermal contrast clarity can very often be the wind speed, which can be seen in Figure 3*c*. Here it can be noted that during January 2 there is a relatively low wind speed (a maximum of approximately 5 to 10 m/s) compared to the frequently higher wind speeds for the other days in the week (approximately 20 to 25 m/s). It should be noted that the ability to observe thermal contrast relies upon the ability of the concrete to build up heat on one side of a defective region of poor thermal conductivity, for example, where a subsurface defect exists. The effect of the higher wind speed is to carry the stored heat away from the surface of the structure, equalizing the temperatures across the surface, and hence diminishing the contrasts perceived in the IR image.

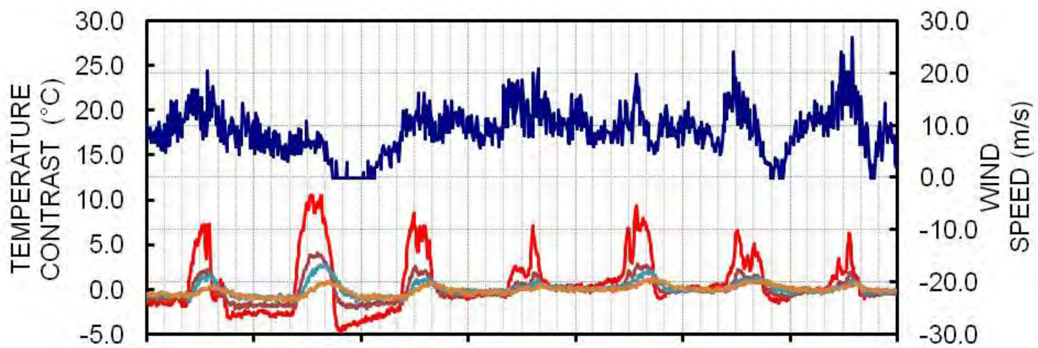
It should also be noted that the diurnal variations in ambient temperatures and the day-to-day trends that are apparent in Figure 3*a* do not appear to have as significant an effect on the thermal contrast as the solar loading. For example, on January 1, there is a negative ambient temperature trend and little daily temperature variation, yet thermal contrast still exists in the



(a)



(b)



(c)

FIGURE 3 (*a*) Ambient temperature versus 1-, 2-, 3-, and 5-in. thermal contrasts; (*b*) solar loading versus 1-, 2-, 3-, and 5-in. thermal contrast; and (*c*) wind speed versus 1-, 2-, 3-, and 5-in. thermal contrast. (Note the environmental variables shown as the upper blue line on each graph.)

images due to the solar loading that occurs on that day. On the other hand, the warming trend that is apparent beginning on January 2 does not appear to result in improved contrasts.

It can also be observed that there is staggered development of peak thermal contrasts within an individual day of data. Using again the data from January 2, the 1-in. target reaches its 10°C contrast around 1 p.m., with the 2-in. target about 4°C shortly after that at roughly 2 p.m., the 3 in. about 3°C a little after that at 3 p.m., and the 5-in. target showing approximately 1°C of contrast about 1 to 2 h later at around 5 p.m. This information corresponds to the thermal wave propagation through the structure. In this specific case and with these specific environmental contributing factors, the propagation rate seems to be advancing at approximately 1 in. per hour. Further analysis should bring more detailed guidelines for this phenomenon.

Figure 4 shows the actual thermal contrast images for the days from January 1 through 7, 2008, to give a visual representation of the images that could be expected when the block is subjected to similar environmental trends. The images shown in the figure are selected for their clarity from the 1,440 images collected each day on the block. These images generally indicate that even on a day when environmental conditions are not optimum, a thermal contrast is achievable, albeit short-lived with a smaller image temperature span.

It can be further noted that, with respect to Figure 3*b*, even though the thermal contrast is apparent during the days of interrupted solar loading, it is not very useable regarding inspection due to its short-lived and often diminished contrast when compared to that on a day where solar loading is uninterrupted. In addition to this, from the same graph, it seems that a high solar loading is useless if it is not accompanied by a good spells of clear skies for that day. January 2 has a far lower maximum solar loading than, say, January 7, but the thermal contrast for January 7 is still much smaller for each of the targets. With respect to Figure 3*c*, the wind speed trend throughout the week shows that only during low daytime wind conditions of less than about 10 m/s is the ideal thermal contrast model able to reach its potential for all target depths.

CONCLUSIONS

This paper has presented results from a study to determine optimum environmental condition for the inspection of concrete structures using IR cameras. A test block with embedded Styrofoam targets was used to determine the relationship between the thermal contrast imaged by an IR camera and the environmental variables of ambient temperatures, solar loading, wind speed, and humidity. Example results for a single week were presented and qualitative descriptions of observations for the data were provided. Initial results of the study suggest that clear skies, resulting in uninterrupted solar loading, provides greater contrast in thermal images than days when peak solar loading is higher but loading is intermittent due to passing clouds. This result is not unexpected, as the total thermal energy introduced into the block on a clear day can be

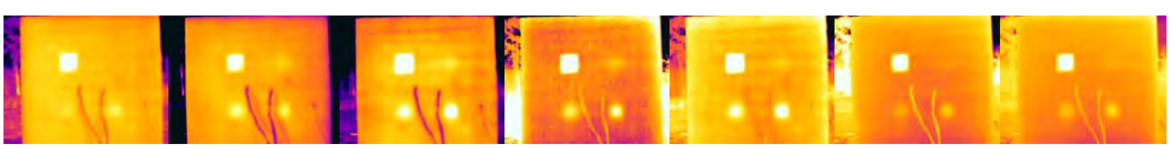


FIGURE 4 The most detailed IR image of each day with respect to the 7 days of data, January 1–7, 2008.

greater, establishing a greater thermal gradient in the block and therefore greater contrast in the images. The effects of wind speed and ambient temperature variations were also presented. It was observed that the daily temperature variations and day-to-day trends did not have as significant an impact on thermal contrast as solar loading.

Three months of data have been collected on the south side of the test block. These data are being analyzed to determine quantitatively the effects of environmental variables on the thermal contrast in the test block. Correlation models are being developed to identify optimum conditions for the inspection of concrete structures exposed to direct sunlight. The observation point for the thermal cameras has been moved to the north side of the block, which is never exposed to direct sunlight. Analysis of the behavior of the north face of the test block is ongoing.

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Application of Prioritization and Optimization Routines

Multiattribute Prioritization Framework for Bridges, Roadside Elements, and Traffic Control Devices Maintenance

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A highway management system consists of four important facilities: pavements, bridges, roadside elements, and traffic control devices. Most road agencies focus on the pavement for maintenance operations as it is directly related to the user costs. Due to limited budgets, maintenance of other facilities is often ignored or deferred. Budget constraint often forces agencies to work on the crisis maintenance basis. This research realizes an important role of these facilities and aims to develop a prioritization framework for their maintenance operations. As the highway management system includes various facilities, objectives, and functions having different measurement scales, the benefits under different dimensions are measured into different units. It makes the decision-making process complicated. In order to eliminate this difficulty, all the units are needed to convert into a nondimensional uniform unit so that decisions can be made under the same platform. This research places an emphasis on developing a multiattribute prioritization framework for bridges, roadside elements, and traffic control device maintenance, which allows intangible objectives that are difficult to quantify on an absolute numerical scale to be considered without the need to convert the units into monetary scale. Expert opinions acquired by questionnaire surveys were obtained for the selection of influential factors contributing to the multiattribute prioritization process. Finally, utility functions are derived in this paper based on the weighting factors and related performance indictors. This proposed framework may help planners select a reduced number of alternatives from all available alternatives while ensuring that the selected alternatives are the best possible options.

An effective highway management system consists of four important facilities: pavements, bridges, roadside elements, and traffic control devices (*1*). Among them, pavements and bridges can be considered as major components. On the other hand, roadside elements (i.e., guardrails, barriers, utility poles, drainage, rest areas, right-of-way) and traffic control devices (i.e., signs, pavement markings, traffic lights) are peripheral elements used in a highway management system. These elements are designed to provide directional information and increase higher safety and comfort to highway users. Most road agencies focus on the pavement for maintenance operations as it is directly related to the user costs. Due to limited maintenance and operational budgets, the maintenance management of other facilities is often ignored or deferred. Limited budgets often force agencies to work on the crisis maintenance basis. This research realizes an important role of these facilities (bridges, roadside elements, and traffic control devices) and aims to develop a prioritization framework for their maintenance operations.

An effective prioritization framework provides the final value for ranking different alternatives so that the most optimum alternative can be identified. As a highway management system includes various facilities, objectives, and functions having different measurement scales, the benefits under different dimensions are measured into different units. It makes the decision-making process complicated. In order to eliminate this difficulty, all the units of benefits are needed to convert into a nondimensional uniform unit so that decision-making action can be implemented under the same platform (*2*).

This research places an emphasis on developing the multiattribute prioritization framework, which allows many intangible objectives that are difficult to quantify on an absolute numerical scale to be considered without the need to convert the units into monetary scale. The concept of utility functions is applied in order to transfer them into nondimensional units. This proposed framework may help decision makers select a reduced number of alternatives from a larger number of all available alternatives while ensuring that the selected alternatives are the best possible options (*3*).

Therefore, the objective of this paper is to develop a multiattribute prioritization framework for bridges, roadside elements, and traffic control device maintenance. Expert opinions were acquired by questionnaire survey for the selection of influencing factors like different objectives, their respective performance indicators, and the corresponding weighting factors. Finally, utility functions are derived in this paper based on the weighting factors and related performance indictors to help highway agencies better prioritize their facilities for maintenance in the most efficient manner.

BACKGROUND

This section discusses literature relevant to this research. One of the key issues in highway management is to conduct the trade-off analyses among different facilities and maintenance alternatives under budget constraints. A number of road agencies in the United States and many other countries implement mathematical techniques for prioritizing their pavement maintenance works. Among the different facilities of highway, pavement has been emphasized for decades in most of the countries and the Highway Development and Management Tool (HDM-4), RTIM, and the Highway Economic Requirements System (HERS), etc., have been introduced as powerful tools to serve this purpose. Recently, bridge maintenance also has been a focus in some developed countries. However, most of the agencies ignore or defer the maintenance operations of roadside elements and traffic control devices. There is no rigid framework for prioritization methodology for bridges, roadside elements, and traffic control devices. In general, priority assessment methods vary from simple subjective ranking to sophisticated mathematical programming (*4*). These methods usually incorporate four major central steps: information, identification of needs, priority analysis, and output reports (*4*). Most agencies implement their own prioritization framework based on their own experience and actual practice.

A number of studies have recently been carried out for optimizing bridge maintenance. Lee and Kim (*5*) proposed optimal maintenance methods for bridge decks using a generic algorithm (GA). Hong and Hastak (*6*) identified the practices and experiences of engineers and inspectors for bridge maintenance. In addition, maintenance impact and deterioration rate were estimated by use of a questionnaire survey and optimal strategies were determined. Yang et al. (*7*) predicted the evolution in time of the reliability of deteriorating bridge structures based on lifetime function and studied proactive, reactive, and necessary maintenance effects. Yehia et al. (*8*) developed a decision-support system for bridge deck maintenance considering a certain number of problems. Factors considered in this paper are defects, maintenance effect on service life, traffic flow and funding availability. Considering roadside elements maintenance, Jha and Abdullah (*9*) worked on optimizing the highway life cycle with GA. Different factors such as maintenance cost, discount rate, and condition were taken into account in their research. A comprehensive literature survey indicates that some important factors for bridges and roadside elements such as user cost, travel time savings, visual quality, accident cost, and safety measures are difficult to incorporate in optimization models.

Generally, optimization presents a more complicated mathematical programming, which could be implemented for single and multiyear planning. Alternatives are selected to satisfy a specific objective function such as the maximization of maintenance effectiveness and the minimization of maintenance costs (*4*). In a single-objective decision-making process, best alternative(s) can easily be identified by this objective function. There exists a single identifiable optimal solution that gives the best objective function value. In multi-objective prioritization, none of the alternatives can be superior in all objectives value. There might be a set of optimal alternatives (*10*). Therefore, the multiattribute prioritization process provides planners some technical tools to help them resolve a decision-making process where conflicts among alternatives exist. Another important capability of the multi-objective prioritization analysis is that it allows many intangible objectives that are difficult to express on an absolute numerical scale (*3*). An effective decision-making process among a set of choices containing various alternatives needs an internal process that evaluates the objectives and other factors and develops a unique choice as an output. Among the various renowned processes, the utility function method is one of the most prominent direct assessment methods that measures and represents the preference of decision makers in the form a real-value function (*11*).

At present, several studies are available that discuss multi-objective characteristics of highway management systems. Li and Sinha (*2*) proposed a multicriteria decision-making methodology for highway asset management programs. Moreover, the need for a total highway management system was addressed by Sinha and Fwa (*1*). Important objectives and their performance indicators for different highway facilities including bridges, roadside elements, and traffic control devices were outlined in (*1*). Based on the findings of Sinha and Fwa (*1*), our present research defines related objectives and performance indicators that might be useful in actual practice of highway facilities maintenance through available literature and expert opinions. Their relative weights and utility functions are also developed for a comprehensive decision-making process in maintenance operations.

METHODOLOGY

This research defines important objectives and their corresponding performance indicators. Expert opinions acquired by questionnaire survey were the foundation of this selection. Nonprobability judgment sampling that occurs when researchers prefer to select samples to conform to some criterion was used in this study (*12*). Experts were considered to have at least 10 years of experience in highway maintenance. Three key decision makers were selected from each of the three public highway agencies in Thailand: Department of Highways (DOH), Office of Transport and Traffic Policy and Planning, and Ministry of Transport and Communication of Thailand. There was a total of nine experts. A preliminary questionnaire was prepared after a comprehensive literature survey. Important factors for bridges, roadside elements, and traffic control device maintenance found from previous research (*1*) were listed in this questionnaire. The preliminary questionnaire survey was carried out to define important objectives and their performance indicators in the maintenance of these facilities. Nine experts of three public agencies in Thailand were requested to select important objectives and their performance indicators from lists. They were also allowed to add any new factor if it should also be a concern in these maintenance operations.

After the preliminary questionnaire survey, this research made it possible to define relevant objectives and performance indicators for bridges, roadside elements, and traffic control device maintenance. As different benefits are measured by various means and with noncommensurable units under different goals, developing utility functions was the next step of this research. Moreover, it was also necessary to determine relative weights of these factors. The final questionnaire survey was carried out to serve these purposes. Experts were asked for their opinions regarding defined weighting factors of different objectives and performance indicators. In order to develop utility functions, experts marked different values of performance indicators with different utility scores. Regression analysis was used to develop the utility equation.

Figure 1 shows the framework of this research methodology. Important issues of this research are the selection of related objectives and performance indicators, the determination of their relative weighting factors, the development of utility function, the result investigation, the validation, and the development of a multiattribute prioritization process.

RESULTS AND DISCUSSIONS

Preliminary Questionnaire Survey

Table 1 lists all the important objectives and their performance indicators. Observation shows that the objectives are almost the same for these three facilities. Service, condition, safety, cost, socioeconomic factor, and energy should be the important objectives for bridges and traffic

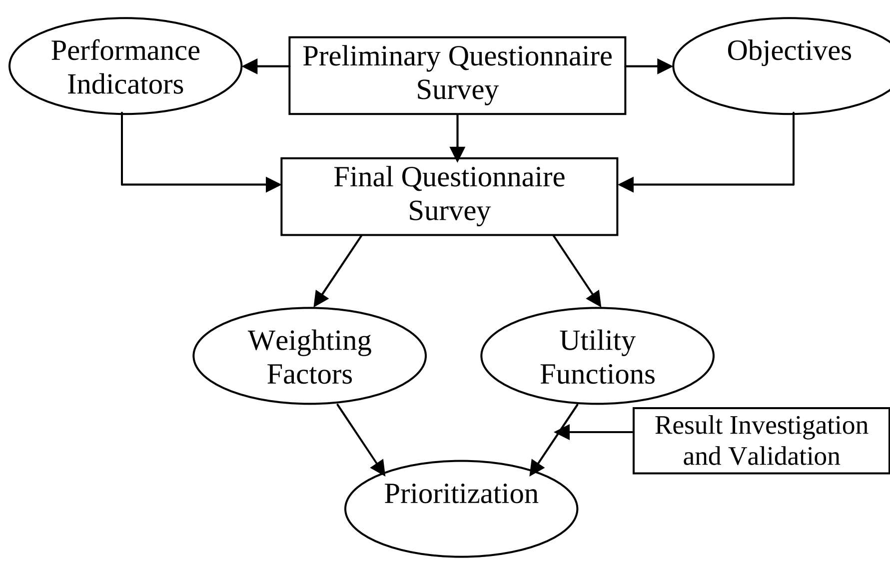

FIGURE 1 Research methodology.

TABLE 1 Objectives and Performance Indicators

Objective	Performance Indicators		
	Bridges	**Roadside Elements**	**Traffic Control Devices**
Service	• Travel speed • Traffic volume	• Travel speed • Clear roadway width	• Traffic volume • Delay time
Condition	• Load capacity • Remaining service life • Deck, superstructure, and substructure deterioration index	• Deflection/displacement • Ditch erosion	• Visibility • Physical deterioration
Safety	• Load capacity • Clear deck width • Occurrences of accidents	• Occupant risk • Road side slope • Roadside hazardous • Sight distance	• Sight distance • Luminance
Cost	• Agency costs • User costs	• Agency costs • User costs	• Agency costs • User costs
Socioeconomic Factor	• Travel time savings • Visual quality • Saving in accident costs	• Saving in accident costs • Visual quality	• Travel time delay • Fuel waste • Pollution • Driver satisfaction
Energy	• Fuel consumption	—	• Fuel consumption

control device maintenance. For the roadside elements, energy can be neglected. However, five other objectives must be fulfilled in this case. Table 1 also summarizes the performance indicators of each objective. For instance, travel speed and traffic volume are the performance indicators of service for bridge maintenance. In total, there are 14 performance indicators for bridges, 12 for roadside elements, and 13 for traffic control device maintenance.

Final Questionnaire Survey

Bridges

The outcome of experts' opinions reveals that bridge condition should be the most influential factor in maintenance planning. As shown in Table 2, the bridge condition contributes 28.333% among other objectives. Service, safety, and cost are viewed as mid-level prioritization factors since they contribute 20.556%, 20%, and 18.333%, respectively. The other objectives (i.e., socioeconomic factors and energy) receive lower scores, 12.777%. In addition, traffic volume, agency cost, and deck, superstructure and substructure deterioration index are the most influential factors among all the performance indicators. All the weighting factors of each objective and their performance indicators are listed in Table 2.

TABLE 2 Weighting Factors of Objectives and Performance Indicators

Facility	Objective	Weight (%)	Performance Indicator	Weight in Objective (%)	Overall Weight (%)
Bridges	Service	20.556	Traffic volume	38.889	12.562
			Traffic speed	38.889	7.994
	Condition	28.333	Load capacity	34.444	9.759
			Remaining service life	27.778	7.870
			Deck, superstructure, and substructure deterioration	37.778	10.704
	Safety	20.000	Load capacity	35.556	7.111
			Clear deck width	26.667	5.333
			Occurrences of accidents	37.778	7.556
	Cost	18.333	Agency costs	60.000	11.000
			User costs	40.000	7.333
	Socioeconomic Factor	8.333	Travel time savings	38.571	3.214
			Visual quality	25.714	2.143
			Saving in accident costs	35.714	2.976
	Energy	4.444	Fuel consumption	100	4.444
Roadside Elements	Service	21.111	Travel speed	36.667	7.741
			Clear roadway width	63.333	13.370
	Condition	20.556	Deflection/displacement	56.667	11.648
			Ditch erosion	43.333	8.907
	Safety	28.889	Occupant risk	20.556	5.938
			Road side slope	19.444	5.617
			Roadside hazardous	28.333	8.185
			Sight distance	31.667	9.148
	Cost	13.889	Agency costs	48.889	6.790
			User costs	51.111	7.099
	Socioeconomic factor	15.556	Saving in accident costs	64.444	10.025
			Visual quality	35.556	5.531
Traffic Control Devices	Service	23.131	Traffic volume	44.444	10.281
			Delay time	55.556	12.851
	Condition	24.141	Visibility	64.444	15.558
			Physical deterioration	35.556	8.584
	Safety	24.798	Sight distance	57.778	14.328
			Luminance	42.222	10.470
	Cost	11.909	Agency costs	61.111	7.278
			User costs	38.889	4.631
	Socioeconomic factor	9.677	Travel time delay	43.333	4.193
			Fuel waste	21.111	2.043
			Pollution	15.000	1.452
			Driver satisfaction	20.556	1.989
	Energy	6.343	Fuel consumption	100	6.343

Roadside Elements

Roadside elements such as guardrails, barriers, utility poles, drainage, rest areas, and right-of-way are normally used for safe and systematic movement of vehicles along the highway. Logically, safety should be the major concern for the maintenance of this facility. It is also supported by the expert opinion in this study. Expert opinion regarding the weighting factors as shown in Table 2 indicates that safety should have the highest priority with 28.889%; condition and service follow with around 20% weight. Among the 12 performance indicators, clear roadway width and deflection or displacement are the most important factors.

Traffic Control Devices

Traffic control devices include various signals, such as signs, pavement markings, and traffic lights that control vehicles on highways. After accumulating all the opinions, it is found that three objectives; service, condition, and safety, should be the major factors for the maintenance of this facility. Despite having the least importance, cost, socioeconomic factors, and energy are also important for an effective decision in traffic control device maintenance. These six objectives include 13 performance indicators, among which visibility, delay time, and sight distance have the major importance. Table 2 lists all the performance indicators of traffic control device maintenance and their weighting factors.

Decision priority for maintenance of each of the facilities depends on the aforementioned objectives and performance indicators shown in Table 2. As their measurement units vary, it was essential to ensure that the prioritization is conducted in the same common platform. For simplicity, this study uses utility values to bring them into the same unit. Utility values from zero to 10 are used in this study where zero is the least preferable option and 10 is the highest preferable option. Regression analysis of the scores given by the experts produces the expected utility equation for each performance indicator. For bridge maintenance, load capacity is the performance indicator for both the objectives condition and safety. Therefore, only one utility function is developed for this indicator that can be used for both condition and safety. For the same reason, one function is developed for sight distance and visual quality in roadside element maintenance. Similarly, for traffic control devices delay time and travel time delay can be represented by one function. Fuel waste and fuel consumption can also be considered as one. Tables 3, 4, and 5 list all the utility equations developed in this research for bridges, roadside elements, and traffic control devices, respectively. With these equations, utility values can be determined if the measurement unit of a performance indicator is known. In addition, Figures 2 to 5 depict several sample graphs produced by the regression analyses.

By use of the relative weights and the utility functions of different performance indicators, it is also possible to construct utility equations of different objectives. These equations reduce the computational efforts and make it easier to prioritize the alternatives while considering various contributing factors in maintenance planning. Table 6 lists all the utility functions of the objectives for bridge maintenance. Similarly, Tables 7 and 8 summarize the utility function of each objective involved for roadside elements and traffic control device maintenance. Utility values are unitless for all the equations.

TABLE 3 Utility Functions for Bridge Maintenance

Performance Indicator	Utility Function	Unit of x
Traffic volume (AADT)	$y = 5.5365\text{Ln}(x) - 41.517$	x = AADT
Traffic speed (km/h)	$y = -0.0017x^2 + 0.4374x - 18.375$	x = km/h
Load capacity (sufficiency rating)	$y = -0.0004x^2 - 0.0598x + 9.9286$	x = sufficiency rating
Remaining service life (years)	$y = -0.00101x^2 + 0.1975x$	x = year
Deck, superstructure, and substructure deterioration index (SI&A)	$y = -1.1625x + 11.1375$	x = SI&A
Clear deck width (m)	$y = 8.5425\text{Ln}(x) - 19.50$	x = width (m)
Occurrences of accidents	$y = 1.1938x + 0.925$	x = No./year/bridge
Agency costs (Baht/bridge)	$y = -2.4\text{E-}06x + 11.911$	x = Cost (Baht/bridge)
User costs (Baht/km/PC)	$y = 4.875x - 29.025$	x = Cost (Baht/km/PC)
Travel time savings (min)	$y = 0.4622x + 1.2222$	x = minute
Visual quality (m)	$y = -0.048x + 12.25$	x = meter
Saving in accident vosts (Baht/bridge)	$y = 1.76\text{E-}05x + 1.0222$	x = Baht/bridge
Fuel consumption (Baht/km/bridge)	$y = 4.575x - 8.225$	x = Baht/km/bridge

TABLE 4 Utility Functions for Roadside Element Maintenance

Performance Indicator	Utility Function	Unit of x
Travel speed (km/h)	$y = -0.0029x^2 + 0.7962x - 43.689$	x = km/h
Clear roadway width (m)	$y = -1.125x + 11.725$	x = width (m)
Deflection/displacement	$y = 2.2625x + 0.85$	x = obstacle No./km
Ditch erosion	$y = 0.0935x + 0.975$	x = % increased
Occupant risk	$y = 0.0945x + 0.925$	x = % minimization
Road side slope (gradient)	$y = 5.3113\text{Ln}(x) + 13.642$	x = gradient
Roadside hazardous	$y = 2.25x + 1.175$	x = obstacle no./km
Sight distance/visual quality (m)	$y = -0.0475x + 12.375$	x = m
Agency costs (Baht/km)	$y = -9.3\text{E} - 05x + 10.111$	x = Baht/km
User costs (Baht/km/PC)	$y = -1.0357x^2 + 19.175x - 77.407$	x = Baht/km/PC
Saving in accident costs (Baht/km)	$y = -2.7\text{E} - 11x^2 + 3.11\text{E} - 05x + 0.8476$	x = Baht/km

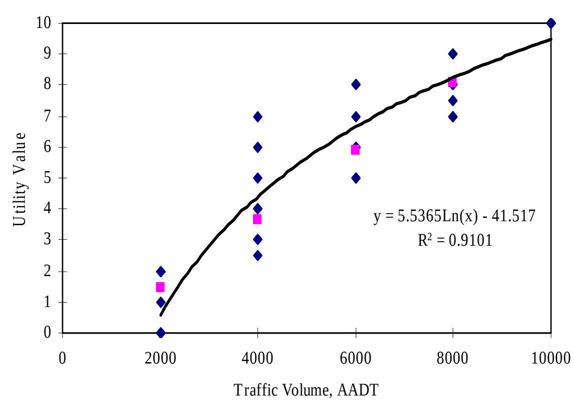

FIGURE 2 Utility equation of traffic volume for bridge maintenance.

TABLE 5 Utility Functions for Traffic Control Device Maintenance

Performance Indicator	Utility Function	Unit of x
Traffic volume (AADT)	$y = 5.608\text{Ln}(x) - 41.973$	x = AADT
Delay time (min)	$y = -0.00556x^2 + 0.45x + 0.6222$	x = min
Visibility	$y = -0.00092x^2 + 9.3603$	x = % reflection
Physical deterioration (condition)	Working = 0, Not Working = 10	
Sight distance (m)	$y = -0.045x + 12.05$	x = m
Luminance	$y = -0.08675x + 10.35$	x = % reflection
Agency costs (Baht/intersection)	$y = -0.0001x + 11.106$	x = Baht/intersection
User costs (Baht/intersection/PC)	$y = 0.3781x + 1.05$	x = Baht/intersection/PC
Pollution	$y = 0.0956x + 1.0444$	x = % reduced
Driver satisfaction	Satisfied = 0, Not Satisfied = 10	
Fuel consumption (Baht/intersection/PC)	$y = 0.47375x + 1.05$	x = Baht/intersection/PC

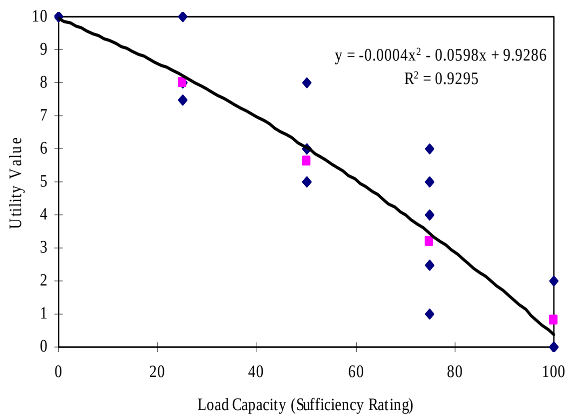

FIGURE 3 Utility equation of load capacity for bridge maintenance.

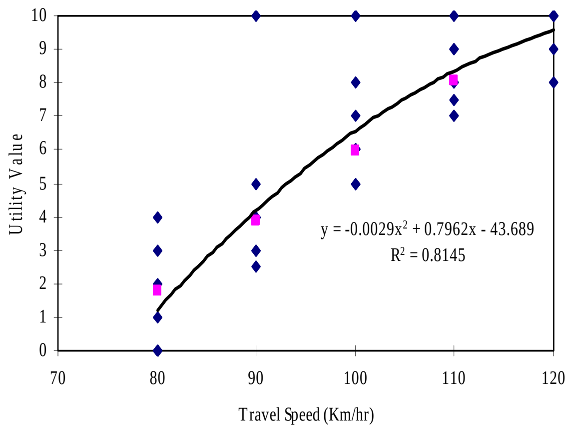

FIGURE 4 Utility equation of travel speed for roadside element maintenance.

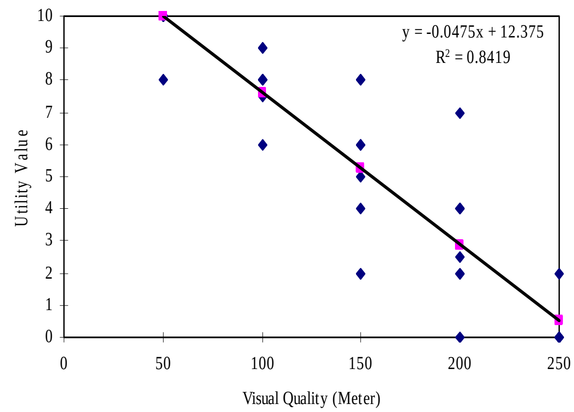

FIGURE 5 Utility equation of visual quality for roadside element maintenance.

TABLE 6 Utility Functions of Different Objectives for Bridges

Objective	**Utility Function**
Service	$U_{se} = 3.383(\ln X_1) - 0.00066X_2^2 + 0.1701X_2 - 32.517$ U_{se} is the utility value of service X_1 is traffic volume in AADT X_2 is traffic speed in km/h
Condition	$U_c = -0.00014X_1^2 - 0.0206X_1 - 0.00028X_2^2 + 0.0549X_2 - 0.4392X_3 + 7.627$ U_c is the utility value of condition X_1 is load capacity in sufficiency rating (SR) $SR = S_1 + S_2 + S_3 + S_4$ S_1= structural adequacy and safety S2= special reduction S3= essentially S4= serviceability and functional obsolescence (excellent condition = 100 and worst = 0) X_2 is remaining service life in year X_3 is bridge structure inventory and condition appraisal, SI&A
Safety	$U_{sa} = -0.00014X_1^2 - 0.0213X_1 + 2.278(\ln X_2) - 0.451X_3 - 1.32$ U_{sa} is the utility value of safety X_1 is load capacity in SR X_2 is clear deck width in meters X_3 is number of accidents per year
Cost	$U_{co} = -1.44{*}10^{-6}X_1 + 1.95X_2 - 4.4634$ U_{co} is the utility value of cost X_1 is agency cost in Baht/km X_2 is vehicle operating cost in Baht/km/PC
Socioeconomic Factor	$U_{so} = 0.1783X_1 - 0.0123X_2 + 6.286{*}10^{-6}X_3 + 3.986$ U_{so} is the utility value of socioeconomic factor X_1 is travel time savings after maintenance in minutes X_2 is visibility distance in meters X_3 is savings in accident costs after maintenance in Baht
Energy	$U_e = 4.575X_1 - 8.225$ U_e is the utility value of fuel consumption X_1 is fuel consumption in Baht/km/PC

TABLE 7 Utility Functions of Different Objectives for Roadside Elements

Objective	Utility Function
Service	$U_{se} = -0.00106X_1^2 + 0.2919X_1 - 0.7125X_2 - 8.594$ U_{se} is the utility value of service X_1 is travel speed in km/h X_2 is clear roadway width in meters
Condition	$U_c = 1.2821X_1 + 0.0405X_2 + 0.9042$ U_c is the utility value of condition X_1 is number of deflected or displaced roadside elements per kilometer X_2 is percentage improved of ditch erosion after maintenance
Safety	$U_{sa} = 0.0194X_1 + 1.033(\ln X_2)+0.6375X_3 - 0.015X_4 + 7.094$ U_{sa} is the utility value of safety X_1 is percentage minimization of occupant risk after maintenance X_2 is roadside slope X_3 is number of hazards per kilometer X_4 is visibility distance of roadside elements
Cost	$U_{co} = -4.55*10^{-5}X_1 - 0.5294X_2^2 + 9.72X_2 - 34.62$ U_{co} is the utility value of cost X_1 is agency cost in Baht/km X_2 is vehicle operating cost in Baht/km/PC
Socioeconomic Factor	$U_{so} = 1.74*10^{-11}X_1^2 + 2*10^{-5}X_1 - 0.0169X_2 + 4.9463$ U_{so} is the utility value of socioeconomic factor X_1 is savings in accident costs after maintenance in Baht X_2 is visibility distance in meters

In order to calculate the utility values, two groups of data are required depending on the performance indicators. The first data group is the existing pavement condition data; for example, traffic volume, deterioration index, and remaining service life, etc. The second data group type is the improvement data after maintenance is applied. Some examples are travel time saved after maintenance, the amount of pollution reduction, and percentage of ditch erosion improved, etc. Historical data or observation in previous maintenance operations might be useful to obtain these values.

Prioritization Process

The objective functions of this prioritization model are to maximize the following expressions.

$$Ub_{total} = 0.206U_{se} + 0.283U_c + 0.20U_{sa} + 0.183U_{co} + 0.083U_{so} + 0.044U_e \quad (1)$$

$$Ur_{total} = 0.211U_{se} + 0.206U_c + 0.289U_{sa} + 0.139U_{co} + 0.156U_{so} \quad (2)$$

$$Ut_{total} = 0.231U_{se} + 0.241U_c + 0.248U_{sa} + 0.119U_{co} + 0.0968U_{so} + 0.063U_e \quad (3)$$

where

Ub_{total} = the total utility value for bridges

Ur_{total} = the total utility value for roadside elements
Ut_{total} = the total utility value for traffic control devices

U_{se} = the utility value of service
U_c = the utility value of condition
U_{sa} = the utility value of safety
U_{co} = the utility value of cost
U_{so} = the utility value of socioeconomic factor
U_e = the utility value of fuel consumption

For any alternative, utility values of all the objectives and their relative weights provide the total utility value of that option. Finally, preference in the prioritization should be adopted according to the higher utility value.

TABLE 8 Utility Functions of Different Objectives for Traffic Control Devices

Objective	Utility Function
Service	$U_{se} = 2.4924(\ln X_1) – 0.0031X_2^2 + 0.25X_2 – 18.309$ U_{se} is the utility value of service X_1 is traffic volume in AADT X_2 is travel time savings after maintenance in minutes
Condition	$U_c = –0.00059X_1 + 0.356X_2 + 6.032$ U_c is the utility value of condition X_1 is visibility in percentage of reflection X_2 is 0 if device works and X_2 is 10 if device does not work
Safety	$U_{sa} = –0.026X_1 – 0.0366X_2 + 11.3322$ U_{sa} is the utility value of safety X_1 is sight distance in meters X_2 is percentage of reflection
Cost	$U_{co} = –6.11*10^{-5} + 0.147X_2 + 7.195$ U_{co} is the utility value of cost X_1 is agency cost in Baht/intersection X_2 is user cost in Baht/intersection/PC
Socioeconomic Factor	$U_{so} = –0.0024X_1^2 + 0.195X_1 + 0.10X_2 + 0.0143X_3 + X_4 + 0.6479$ U_{so} is the utility value of socioeconomic factor X_1 is travel time savings after maintenance in minutes X_2 is fuel consumption in Baht/intersection/PC X_3 is pollution reduction in percentage X_4 is 0 if driver is satisfied and X_4 is 10 if driver is not satisfied
Energy	$U_e = 0.4738X_1 + 1.05$ U_e is the utility value of fuel consumption X_1 is fuel consumption in Baht/intersection/PC

Validation

The concepts of linear and nonlinear regression analysis are used to accumulate all the obtained experts' opinions and to examine best-fitted utility functions. Coefficient of determination, R^2, the proportion of the total variation in the dependent variable that is explained or accounted for by the variation in the independent variable, is checked for the fitness of these functions (*13*). Moreover, *t*-statistic value and significant level of each of the variables are observed to judge their appropriateness. Variables of developed utility functions are well fitted for the 95% confidence level.

CONCLUSION

A prioritization framework for bridges, roadside elements, and traffic control device maintenance is developed in this study by acquiring the opinions of experts who are usually engaged as decision makers in Thailand. Questionnaire surveys are carried out in two different steps, each with in-depth interviews. During the preliminary stage, this study identifies different objectives in maintenance and their respective performance indicators that are necessary to be incorporated in the prioritization process. After obtaining these identified factors and having several interviews of experts, the final questionnaire survey is conducted for the determination of relative weights of different objectives and their corresponding performance indicators. Finally, utility functions of different performance indicators are developed. Final decision for the prioritization in maintenance operation should be made after calculation of the total utility value of each of the facilities for a specific maintenance action. Higher utility should have higher priority in maintenance operation.

This study ignores all the uncertainties and risks that might be associated in bridges, roadside elements, and traffic control device maintenance. Accumulating the certainty, uncertainty, and risk would make this framework more realistic and effective. Therefore, further study can be carried out to associate the uncertainty and risk management issue with the outcome of this study. Moreover, this study only presents a framework for bridges, roadside elements, and traffic control device maintenance. Developing a prioritization framework for other facilities in the highway management system would enhance highway maintenance and help the planners conduct trade-off analyses among different facilities.

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Project Prioritization Using Multiobjective Utility Functions

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Bridge management systems (BMS) have traditionally evaluated project alternatives by comparing benefit–cost ratios of various potential strategies and selecting the alternative that maximizes these ratios. Calculated benefits have included contributions from improved conditions and increased strength, vertical clearance, or reduced accidents. Combining these benefits into a single project benefit has been problematic because of the lack of a common scale necessary to cumulate the overall project benefit in a meaningful way. Utility functions provide a means to combine various project attributes into a single unitless benefit that offers a potential for improvement over current BMS practices. Applying utility functions to bridge management practice can extend the practice to incorporate project attributes such as scour potential, seismic risk, collision risk, and other factors that are currently excluded from project benefit calculations. This paper will present the results and findings of a pilot application of project prioritization using utility functions within the California Department of Transportation.

The California Department of Transportation (Caltrans) is responsible for the preservation of more than 13,000 state highway bridges. Preservation activities on these bridges consist of condition-based needs caused by deterioration, risk-based needs such as scour and seismic vulnerabilities, safety needs driven by changing bridge rail crash test standards, and operational needs such as reduced load capacity and deficient clearances.

Known bridge preservation needs plus those identified through the bridge inspection program and forecasted future needs exceed the annual preservation funding levels by a factor of about two. In an effort to maximize the benefit of available preservation dollars, Caltrans strives to prioritize the "best" projects based on a number of factors. Evaluating potential projects to determine the "best" ones is not a simple exercise due to the varied nature of the project benefits. To help bridge managers find the "best" projects, bridge management system (BMS) tools have been developed that provide an objective comparison of project needs and potential benefits. The "best" projects are those that have the maximum benefit–cost (B/C) ratio. The measure of benefit varies among commercially available management system software, but typically ignores the benefit of reduced risks resulting from scour, seismic, and safety upgrades. Ignoring the benefit of risk reduction can lead to network-level optimization and project-level prioritizations that do not reflect an agency's true objectives.

In California, the State Highway Operation Protection Plan (SHOPP) identifies preservation needs in five areas: rehabilitation, scour, seismic, bridge rail upgrade, and mobility upgrades (raising and strengthening). The SHOPP consists of funding commitments to projects for a 4-year programming horizon (*1*). State law also mandates the development of a 10-year SHOPP plan that defines the unconstrained needs for all transportation assets (*1*). BMS software tools are effective at evaluating needs and prioritizing projects in only two of the five defined program areas. The three risk-based programs (scour, seismic, and rail) account for approximately 40% of the total SHOPP bridge dollars. The needs in the three risk-based programs must be evaluated using criteria that is outside the capability of the BMS. Projects that have components of condition and risk benefit have no objective way of being combined for prioritization. To address the full spectrum of bridge needs, Caltrans has begun to utilize multiobjective utility theory to prioritize potential bridge projects for the SHOPP. This paper presents the formulations and findings from the initial work that was done for the 2008 SHOPP.

The application of multiobjective utility theory provides a means to capture all factors that are typically considered by bridge managers when making project-level decisions. A zero-to-one value function captures the decision preferences and risk tolerances for any number of attributes. The individual value components can then be weighted and combined to determine an overall project utility based on all the attributes. The generic form of the total project utility is given by:

$$U_t = \sum \alpha_i \beta_i X_i \qquad (1)$$

where

U_t = total project utility
α_i = 0 or 1 indicator noting rather attribute is addressed by project or not
β_i = attribute weighting factor
X_i = attribute value coefficient

Expanding the generic utility form to address the attributes used to develop the SHOPP in California results in the expanded form of the total project utility:

$$U_t = \alpha_1\beta_1 X_1 + \alpha_2\beta_2 X_2 + \alpha_3\beta_3 X_3 + \alpha_4\beta_4 X_4 + \alpha_6\beta_5 X_5 \qquad (2)$$

where

U_t = total project utility
α_i = indicator that attribute is addressed or not
β_1 = rehabilitation or replacement weighting factor
X_1 = rehabilitation or replacement value coefficient
β_2 = scour weighting factor
X_2 = scour value coefficient
β_3 = rail upgrade weighting factor
X_3 = rail upgrade value coefficient
β_4 = seismic weighting factor
X_4 = seismic value coefficient
β_5 = raising and strengthening weighting factor
X_5 = raising and strengthening value coefficient

The specific attributes included in Equation 1 are not limited in number; however, the influence of individual attributes will diminish with each additional item included in the equation.

The individual value functions can contain numerous parameters. For example, the scour function developed includes an assessment of the scour risk based on the National Bridge Inventory (NBI) scour item (113), the average daily traffic (ADT), and detour length (DL) around the bridge. Combining these three bridge-level attributes in a value function captures the potential for scour and a measure of the consequence if the scour affected the operation of the bridge. The scour value function can be expressed in mathematical terms or in graphical form. The graphical form is presented below.

Figure 1 presents a graphic form of the value function for scour with a fixed ADT and DL. The figure indicates that for a bridge with scour risk of 8 as defined in the *FHWA Recording and Coding Guide* (*2*), the value coefficient of a scour mitigation project would be zero, indicating that a project to address scour would provide no benefit. As the scour risk increases to a critical point [scour code 3 (SC 3)] the value coefficient increases to 0.75 out of a maximum of 1.0, indicating a potential project that has significant benefit.

Value functions can be developed for the other attributes included in the total project utility. The value function developed for rehabilitation and replacement projects uses the Bridge Health Index (BHI) (*3*), ADT volumes, DL, and repair urgency defined by the inspecting engineer. The graphic form of the rehabilitation and replacement value function is shown in Figure 2. A bridge in good condition as indicated by a high BHI results in a very small value coefficient. As the condition of the bridge deteriorates the value of the rehabilitation and replacement component of the total project utility increases.

The graphic form of the value functions helps with visualization of the concept, but the mathematical equations of the curves are more useful and can be readily solved in simple spreadsheet applications. The mathematical formulation of the value functions is logit curves (*4*) of the following form:

$$X_i = 1/(1 + e^{-C_i}) \qquad (3)$$

where

X_i = the value coefficient for each component of the utility, and
C_i = a function of the significant decision parameters for each value component.

In developing the C_i variable for each of the utility components every attempt was made to capture the key decision parameters and to draw drew upon readily available bridge information in the BMS. The variable C is shown in Table 1 for all value components.

The priority formulas shown in Table 1 had been previously created to prioritize each funding component relative to other similar types of projects, but these priority formulas had no common basis to be combined together to determine an overall project priority. By use of the utility function concept, it was possible to combine the various value functions into a total project utility by weighting each of the attributes.

The weight associated with each component of the total project utility can be determined by a number of methods (*5*); however sensitivity analysis of the resulting component utilities and the total project utilities by experienced bridge engineers proved most effective. The weights developed for each attribute are shown in Table 2.

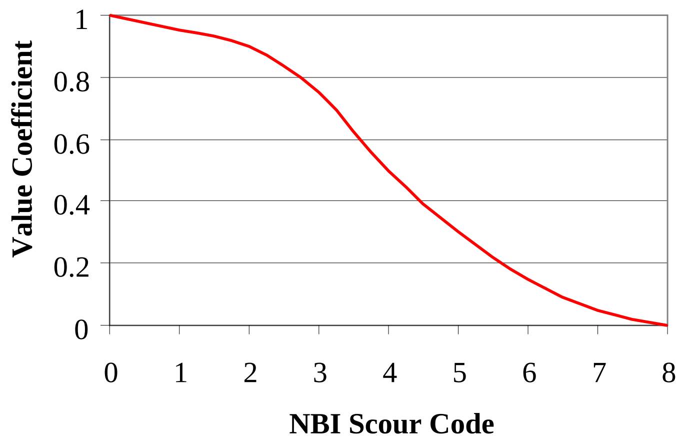

FIGURE 1 Scour value function.

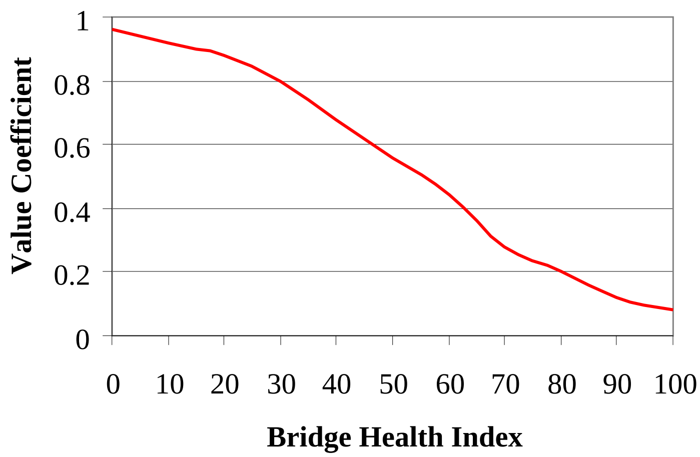

FIGURE 2 Rehabilitation and replacement value function.

TABLE 1 Variable *C* for Value Components

Utility Component	Key Parameters	C_i
Rehabilitation and replacement needs	BHI, ADT, repair urgency (U), and DL	– 2.5 + .000001[(100 – BHI – ΔBHI)TEV]/100 + .00000001(ADT)(DL) + 0.5(10 – U)
Scour needs	NBI SC, ADT, and DL	– 4 + (8 – SC) + 0.000001(ADT)(DL)
Bridge rail upgrade needs	Caltrans rail upgrade score (RS)	– 2 + RS
Seismic retrofit needs	Caltrans seismic priority (S_v), ADT, and DL	– 1.5 + S_v + 0.000001(ADT)(DL)
Mobility needs (raising/strengthening)	Pontis improvement benefit (P) (*6*)	– 4.5 + 0.00015(P)

TABLE 2 Utility Function Weights

Attribute	Weight
Rehabilitation and replacement needs	25
Scour needs	20
Bridge rail upgrade needs	10
Seismic retrofit needs	25
Mobility needs (raising/strengthening)	20
Total	100

Applying the value functions and weights to all the components included in the project results in a unitless total utility prior to the proposed project. For the initial Caltrans effort, we assumed that the post-project utility would be 1.0 for each component that was included in the project. This assumption simplified the analysis greatly. The post-project assumption was based in part on an internal policy within Caltrans to address all needs when initiating a SHOPP project. A logical enhancement of the initial work would be to estimate the effectiveness of various repair alternatives in terms of the value functions. The post-project utility can be determined by defining the expected post-project state of each value attribute. For example, a scour critical bridge with an NBI scour rating of 3 would result in a preproject scour value coefficient of 0.75. Upon completion of the project, the scour risk would have been mitigated with an expected scour rating of 8 and a corresponding post-project utility of zero. In this example the net scour benefit of the project would then be 0.75 or the difference between pre- and post-project utility.

The real power of the utility approach is realized when one compares projects with a combination of risk reduction and condition improvement. The following simple example demonstrates the concept. Consider two similar bridges with a combination of condition and scour needs as shown in Table 3.

The example shown in Table 3 demonstrates how the risk and condition value functions are weighted and combined together to create a two-component project utility that indicates bridge A would provide four times the utility benefit of bridge B. If the project costs are known, or can be estimated, it is possible to define a B/C ratio using the net change in utility from the project divided by the project cost.

In Caltrans the value functions and weights were applied to every state-owned bridge in the BMS. Bridges with higher utility cost ratios were evaluated for potential programming. Bridge replacements had higher utilities because they often received utility value contributions from multiple components in the total project utility. The total project utilities were reviewed by

TABLE 3 Utility Application Example

Structure	Scour Risk (NBI Item 113)	Scour Value Coefficient	Condition Value Coefficient	Total Utility (weighted sum of coefficients)
Bridge A	Scour Critical – 3	0.75	0.20	0.25(.20) + 0.20(.75) = 0.20
Bridge B	No Scour – 8	0.00	0.20	0.25(.20) + 0.20(0) = 0.05

experienced bridge engineers and found to have a high correlation to the priorities that would have been assigned in practice.

Applying the estimated project costs generated utility B/C ratios for each proposed project. Working with the utility B/C ratios indicated that we had overlooked a significant factor. The cost associated with the net improvement of utility is based on many factors, but is certainly related to the magnitude of the project. It became apparent that the size or value of the bridge needed to be incorporated into the net project utility to counteract the difference in project costs due to the size of the structure. There are several ways to account for the size of the structure in the utility B/C ratios. Caltrans opted to multiply the net utility benefit by the value of each bridge using the total element value (TEV) as defined in the BHI formulas (*3*). Using the latter approach, we achieved utility B/C ratios that could be compared across structures of various sizes, materials, and compositions. The final project utility B/C ratios are given by the following equation:

$$\text{Project utility B/C ratio} = U_t(\text{TEV})/\text{Project Cost} \quad (4)$$

The application of weighted value functions to develop a single unitless total project utility that incorporates condition and risk attributes was shown to have significant promise for project prioritization. The objective incorporation of risk into the decision-making process is a significant step in the advancement of BMSs. The value functions are flexible and can adapt to specific agency programs and practices as demonstrated. Value functions, project utilities, and utility cost ratios are computationally simple and can easily be solved with commercial spreadsheet programs. The concepts presented in this paper demonstrate a simple way to combine disparate attributes into a single utility function that can be optimized for any value component or the total project utility. This paper has demonstrated the application of the utility concept to varying types of bridge needs; however, a similar approach could be used to evaluate projects across asset classes. The potential application of this approach in the area of asset management shows real promise.

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APPLICATION OF PRIORITIZATION AND OPTIMIZATION ROUTINES

Multiobjective Optimization for Bridge Management Systems

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Optimizing investment funding levels and combinations of treatment types and timings, as an aid to management decision making, are vital functions of any bridge management system (BMS). Bridge managers and engineers are finding that their constituents require bridge projects and programs to perform not only as provided by least long-term cost solutions, but also to satisfy other objectives such as safety, minimum traffic flow disruption, and risk. NCHRP Project 12-67, published as NCHRP Report 590, has developed a multiple objective optimization methodology and software to facilitate implementation of balanced decision-support practices at the network and bridge-levels. Relative to existing BMS analytical frameworks, the new analysis includes several innovations: the use of utility theory and appropriate techniques of weighting, scaling, and amalgamation to combine multiple decision objectives; separation of bridge-level, network-level, and program-level components of the optimization so the engineer can work with each one separately yet maintain consistency in either a top-down or bottom-up management style; a framework for incorporating structure vulnerability and indirect costs in the analysis; the ability at the program level to apply both budget and performance constraints; and a digital dashboard presentation style to help engineers visualize the economic and performance trade-offs. The products of the study addressed many unmet needs in bridge management and opened the door to significant further research and development. Many aspects of the report are now being designed into AASHTO's Pontis 5.2 BMS.

Optimizing investment funding levels and combinations of treatment types and timings, as an aid to management decision making, are vital functions of any bridge management system (BMS). Bridge managers and engineers are finding that their constituents require bridge projects and programs to perform not only as provided by least long-term cost solutions, but also to satisfy other objectives such as safety, minimum traffic flow disruption, and risk.

NCHRP Project 12-67, published as NCHRP Report 590 (*1*), has developed a multiple objective optimization methodology and software to facilitate implementation of balanced decision-support practices at the network and bridge levels.

The addition of multiple objective capability to a BMS adds important realism and policy relevance. But it brings with it the potential for added complexity and data requirements, attributes that are not widely demanded in these systems. A challenge for the research team was to redefine the analytical framework of the system to break it into manageable pieces that better fit agency business processes and encourage a higher intensity of use of decision support features. Combined with a new generation of graphical user interface, the study products give an entirely new look and feel to the analysis, with the goal of complementing relevance with usability.

Inherent in multiple-criteria decision making is the value trade-off, i.e., the decision maker is faced with a problem of trading off the achievement of one criterion against another criterion due to conflicting objectives. In such a case, an important aspect of the decision-making process is to be able to capture these value trade-offs effectively. Multicriteria methodologies were developed for the bridge decision-making problem based on the concepts of value and utility theory. Mathematical strategies were advantageously used to handle the multidimensionality of the problem in order to capture the decision makers' preference structures effectively in a practical manner.

The network-level model aims to obtain the optimal selection of candidate projects from a networkwide candidate list to yield maximum network benefits subject to multiple constraints. The problem was formulated as a Multi-Choice Multi-Dimensional Knapsack problem. The most promising heuristics were explored and a strategy was developed that balanced theoretical precision with an appropriate level of practicality while selecting the solution he

.uristic for implementation in the final software product. This strategy was based on evaluating the heuristics in terms of optimality, computational speed, accuracy, robustness, and simplicity. The computational experiments provided very useful insights into the optimization heuristics reflecting the appropriateness and applicability to the bridge program optimization problem. The incremental utility/cost (IUC) heuristic performed very well in terms of both computational speed and optimality and was implemented in the software product.

The bridge-level methodology includes a life-cycle cost framework, preservation and functionality models, candidate definitions, and their evaluation. In many ways this is similar to AASHTO's Pontis (*2*) approach, with similar models for network policy optimization, bridge element deterioration, and functional improvements. The methodology includes a bridge-level optimization that maximizes utility by selecting from an array of scoping and timing alternatives. A recursive approach was selected that is consistent with input data available from existing BMSs, but that gives a deterministic result as is needed at the bridge level.

MULTIPLE OBJECTIVES

The foundation of any decision analysis is a clear statement of goals and performance measures. To describe the consequences of alternative bridge actions and enable trade-offs between competing different goals, it is necessary to identify a set of goals and a set of performance measures for each goal. For purposes of this study, the consequences of bridge actions are evaluated on the basis of the following general goals:

- Preservation of bridge condition,
- Traffic safety enhancement,
- Protection from extreme events,
- Agency cost minimization, and
- User cost minimization.

Selecting Performance Measures

A set of performance measures for each goal clarifies the meaning of each goal and is required to measure the consequences of alternative bridge actions. Performance measures are also sometimes referred to as attributes or criteria. Some desirable properties for the set of performance measures for each goal are (*3*):

- Completeness: A set of performance measures is complete if it is adequate in indicating the degree to which the goal is met.
- Operational: Since the idea of decision analysis is to help the decision maker choose the best course of action, the performance measures must be useful and meaningful to understand the implications of the alternatives, and to make the problem more tractable.
- Nonredundancy: The performance measures should be defined to avoid double counting of consequences.
- Minimal: It is desirable to keep the set as small as possible to reduce dimensionality.

A large number of performance criteria can be derived from a bridge management database, including basic National Bridge Inventory (NBI) items (*4*), Pontis analytical products (*2*), and new measures not currently accounted for in most BMSs, such as vulnerability to extreme events. The software product of the study provides a menu of 23 different performance measures, but many are completely or partially redundant with each other so the number of measures used in practice would be far less. The measures that have generated the greatest interest among early users of the products are:

- Life-cycle cost, as a single quantity or divided into agency and user costs;
- A condition measure, either health index (*5*) or some combination of NBI condition ratings; and
- A measure of vulnerability and/or risk.

It is the authors' conjecture, based on their own experience, that a selection of just three measures in these categories would be considered adequate in a wide variety of applications.

Combining Performance Measures

Implicit in any decision-making process is the need to construct, either directly or indirectly, the preference order, so that alternative candidates can be ranked and the best candidate can be selected. When multiple criteria are considered together, it is often true that no dominant alternative will exist that is better than all other alternatives in terms of all of these criteria. For example, one cannot maximize service levels and at the same time minimize costs.

As a result, the decision maker is faced with a problem of trading off the achievement of one criterion against another criterion. In such a case, an important aspect of the decision-making process is to be able to capture these value trade offs effectively. Hence, it is desirable to explore the decision maker's preference structure in some direct fashion and to attempt to construct some sort of preference order directly.

An important class of decision-making techniques that attempt to construct the preference order by directly eliciting the decision maker's preference is predicated on what is known as utility theory. This, in turn, is based on the premise that the decision maker's preference structure can be represented by a real-valued function called a utility function. Once such a function is constructed, the selection of the alternative candidates can be done by use of an optimization method. Broadly speaking this technique involves three steps:

- Weighting assigns relative weights to the multiple criteria, reflecting their relative importance.
- Scaling translates the decision makers' preferences for each performance criterion onto a 0–100 scale. This involves developing utility functions.
- Amalgamation combines the single-criterion utility functions into one measure based on mathematical assumptions about the decision makers' preference structure. This involves deriving the functional forms of multicriteria utility functions.

There are two general approaches for establishing utility function weights, distinguished by their context. One approach is to conduct a structured expert elicitation exercise that is intentionally removed from the context of projects and programs under consideration, to try to obtain more general insights into the relative importance of policy goals. In the research study several methods were evaluated and tested, some using the NCHRP Project Panel as the group of experts. The method preferred by the researchers and panel was a pair-wise comparison method known as the Analytic Hierarchy Process (AHP) (*6*). This technique, which involves the construction of a hierarchy of "levels" and pair-wise comparisons at each level to estimate the relative weights, has been found to be capable of handling quantitative, qualitative, tangible, and intangible criteria.

Another completely different approach is to experiment with the construction of real projects and programs in an optimization framework, adjusting the utility function weights iteratively to try to develop the most satisfactory program in terms of both project selection and forecast achievement of policy goals and objectives. This method was built into the study's software product. Both approaches are valid and can be used together: for example, the AHP can be used to generate an initial set of weights for the optimization model, weights that are later refined when actual programs or projects are evaluated.

ANALYTICAL ARCHITECTURE

Much of the analytical structure of the study products was motivated by experience of the project panel and researchers in the use of AASHTO's Pontis BMS (*2*). Used by nearly all of the states, Pontis solidifies the de facto data requirements, concepts, and jargon of decision support for bridge management. Yet, it has been widely believed that the decision support features of Pontis are underutilized, and that the perceived "black box" nature of the analysis might be a barrier to more widespread implementation.

Important progress on this issue was made by the Florida Department of Transportation when it developed a Project Level Analysis Tool (PLAT) (*7*) as an add-on to Pontis. The PLAT uses the Pontis database and predictive models, but presents the results from a bridge-level perspective as a set of scoping and timing alternatives for future work on the bridge. The probabilistic orientation of the network level was translated to a deterministic presentation of possible near-term futures for a bridge, using a graphical presentation to help engineers visualize current structural and economic health and future possibilities.

Existence of the PLAT didn't compromise the network-level perspective of bridge management at all. In fact, Florida later developed a companion tool for visualizing network-level trade-offs of performance versus budget. But immediate interest in PLAT made it clear that an essential business process of bridge management was being missed by Pontis.

To better support project planning in a BMS, it would be necessary to decouple bridge-level from program-level analysis, and engineer the program-level analysis so it can make full use of bridge-level decisions. This architectural change would follow a pattern set by the Ontario Bridge Management System (*8*), where bridge-level analytical products and decisions are stored in a database for later use by a program planning tool.

As a result, the analytical architecture of the NCHRP 12-67 study divides up the analysis as in Figure 1. The analytical database is a new feature separate from the BMS, with the purpose of communicating bridge-level results asynchronously to a separate network-level programming process.

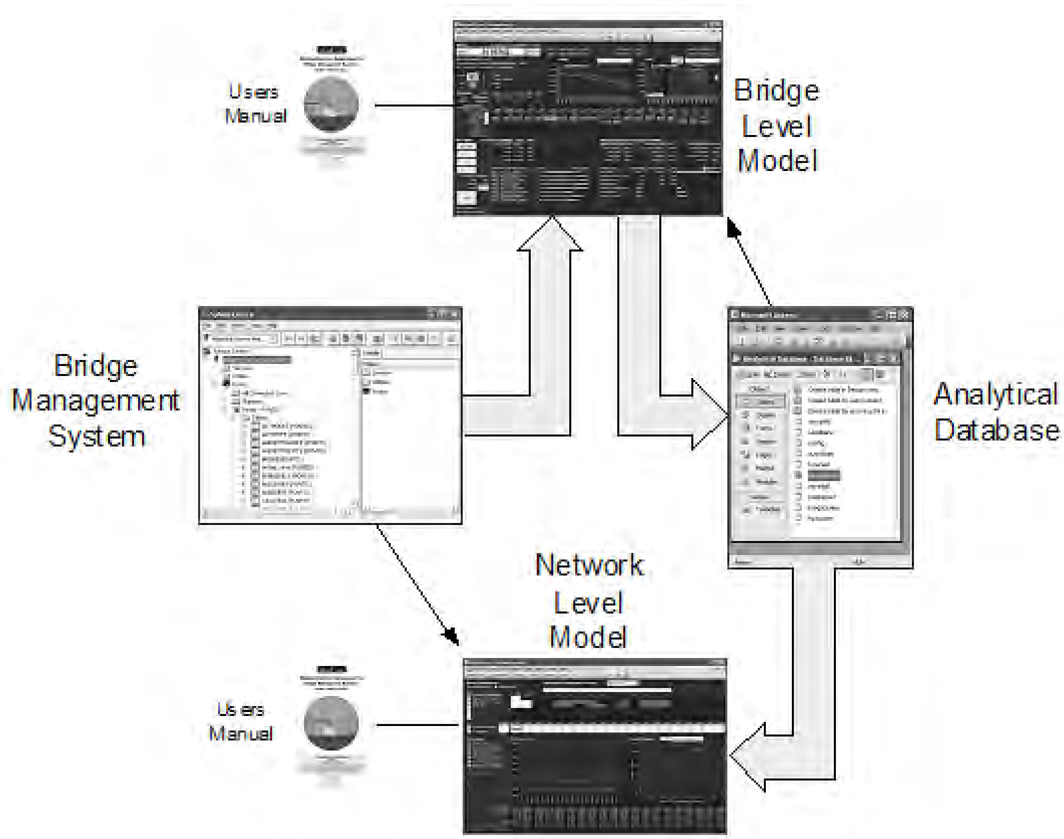

FIGURE 1 High-level architecture of the NCHRP Report 590 analysis framework.

BRIDGE-LEVEL ANALYSIS

At the bridge level, the analytical framework generates and evaluates a set of “candidates,” which are alternative life-cycle activity profiles each distinguished by a set of scope items and an implementation year. The evaluation model estimates the initial cost and future costs and performance of each candidate. A utility function is then calculated from this information.

Life-Cycle Costs

Each life-cycle activity profile is modeled as an infinite time series of annual cash flows representing various types of agency and user costs. Agency costs are concentrated in discrete interventions, each of which represents all the work done on the bridge in a given year.

Figure 2 is a schematic depiction of a typical pattern of life-cycle costs. The diagram shows an analysis conducted in 2007 with a view to developing a program to start in 2008. For the example candidate described in the diagram, the first intervention occurs in 2011, followed by a period of inaction until 2023. The significant time intervals and milestones are described in the following sections.

First Waiting Period

From the start of 2008 to the start of 2011 is the initial waiting period, the time interval over which the bridge deteriorates gradually and its repair needs accumulate, prior to a first intervention. During this period, elements on the bridge are modeled to deteriorate according to Markovian transition probabilities. Those that reach their worst condition state, and are allowed to stay there, incur a risk of unprogrammed or emergency work. If any functional deficiencies exist on the bridge, user costs of accident risk or delays may occur.

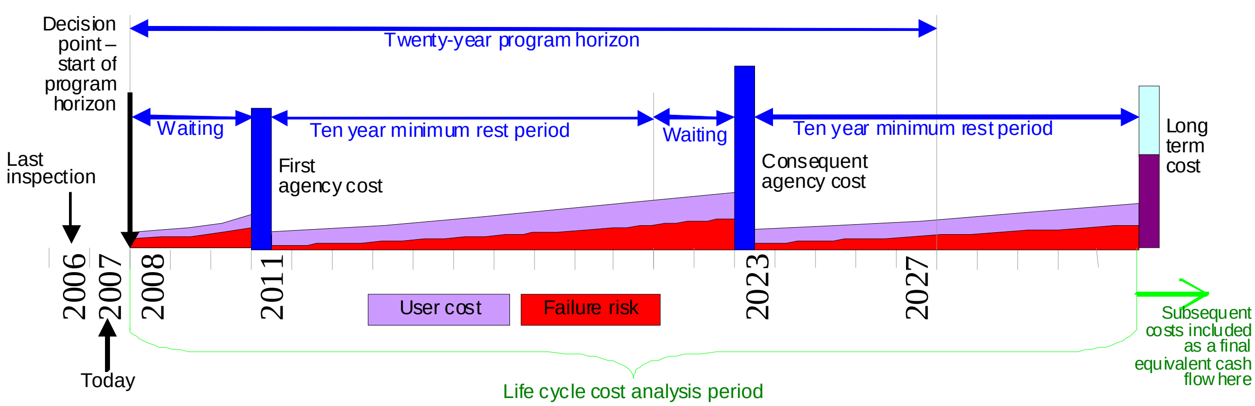

FIGURE 2 Example life-cycle activity profile.

First Intervention

At the start of 2011, the agency implements an intervention that may address some or all of the built-up needs. With the intervention, the condition of elements is improved, and functional deficiencies and vulnerabilities may be corrected.

First Rest Period

Following the first intervention is a period mandated by agency policy, when no action may be taken on the bridge. Deterioration of bridge elements continues, and failure risk costs may be incurred. Also, any functional deficiencies that were not remedied by the first intervention would cause user costs to be incurred.

Consequent Waiting Periods, Interventions, and Rest Periods

If the program horizon is long enough, additional cycles may be modeled. For every consequent intervention that is modeled, a corresponding waiting period before, and rest period after, are also modeled. The final rest period of the analysis may extend beyond the end of the program horizon; if so, it extends the life-cycle cost analysis period accordingly. No interventions may be programmed beyond the program horizon. As such, with the conventions adopted here, a waiting period may not extend beyond that point.

Long Term

The end of a life-cycle activity profile, as modeled here, occurs at a point where the following year may bring either an intervention or the start or extension of a waiting period. We do not model the possibility of interventions beyond this point, however, but instead use less-detailed models to collapse all subsequent life-cycle costs into a final cash flow at the end of the analysis period. This is similar to a salvage value analysis, except the structure is modeled as a going concern in perpetuity.

Discounting

Since all of the costs occur at various times in the future, they are processed in a standard engineering procedure called net present value analysis. Each cost item is discounted (reduced in value) by an amount that depends on how far in the future it occurs. Naturally if a cost needs to be incurred, the analyst prefers to put it off as long as possible, because then it matters less to the analyst. The discount factor represents how much less it matters for each year that the cost can be delayed.

Discounting makes the analysis relatively insensitive to costs that occur far in the future. The effect is enhanced by the final rest period, where agency policy mandates that a fixed period, usually 10 years, must elapse before any further intervention costs can be incurred. This further reduces the sensitivity of the model to costs that occur far in the future.

Decision Variables

The most essential decision to be optimized in the bridge-level analysis is the scope and timing of the first intervention. When multiple candidates are defined for a bridge, they differ in terms of the first intervention. Timing of the first intervention determines the length of the first waiting period whose duration may vary from zero to the full length of the program horizon. Consequent interventions are forecast for programming and for life-cycle cost analysis, but are not the subject of decision making by the bridge maintenance planner. The following types of first interventions are analyzed:

- Do-nothing: models the costs and performance of the structure if no programmed action is taken on the bridge any time during the analysis period.
- Auto maintenance, repair, rehabilitation, and improvement (MRR&I): an intervention that includes all cost-effective maintenance, repair, rehabilitation, and functional improvement actions, as indicated at the time of intervention.
- Auto replace: an intervention that replaces the structure with a new, larger structure.
- Custom: up to three interventions whose scope is determined by the engineer.

To make the scope of work of each intervention easy to understand and analyze, the model framework defines a concept of scope items. Each scope item is a general class of work, such as painting or concrete repair, applied to all elements on the structure to which the work is applicable. For example, a scope item of “total paint system replacement” involves recoating all painted steel elements on the structure.

Optimization

The decision to be made by the bridge maintenance planner has two dimensions: scope and timing. The planner must decide how long the first waiting period should be, and then must decide what kind of intervention to undertake. Naturally, if the waiting period is extended, deterioration will continue and it is likely that the scope and cost of the first intervention will increase. Intervention scope and timing are therefore interrelated. If the program horizon is long enough, the decision becomes a multistage process. After the first intervention and subsequent rest period, the need may arise for another intervention. The scope and timing of the second intervention are conditional on the decisions made in the first intervention. So we end up with a tree structure of decisions with potentially a very large number of branches.

The familiar tool of recursion provides a means to divide the problem into smaller parts that are easier to analyze and require less computation. The algorithm exploits the fact that each stage of the decision tree is similar to earlier stages. Setting aside rest periods where the only prescribed intervention is do-nothing, the decision points during a waiting period share the following common features:

- They all start with the same structure of forecast initial conditions, either at the start of the program horizon or at the end of the previous rest period.
- They all have the same set of alternative scopes (types) of interventions to be considered: do-nothing, rehabilitation, or replacement (for the first intervention, custom

candidates may also be considered). Do-nothing in this case means merely putting off consideration of any intervention to the next programming period (which is typically 1 year).

- The choice of intervention scope in every programming period is based on the same utility function.
- The structure of future consequent interventions is the same, differing only in how soon the detailed simulation of interventions is replaced by a less detailed long-term cost model.

With these stipulations, we can apply a recursive approach as described in Figure 3. The optimization works in either or both of two modes:

- Optimizing, where all program years are investigated and the one with highest utility is chosen; or
- Worst tolerable performance (WTP), where an intervention is considered only in the first year that one or more performance measures fail to meet performance thresholds.

If both optimization methods are chosen, the algorithm stops as soon as a performance threshold fails, but may select a candidate in an earlier year if the latter has higher utility. The WTP mode requires an intervention to be taken even if no candidate has a positive benefit (that is, a utility greater than that of the do-nothing candidate).

PROGRAM-LEVEL ANALYSIS

The bridge-level optimization produces an evaluated set of candidates that are stored in a database to be made available for programming. Each candidate has a cost and a vector of performance measures. At the program level, the challenge is to select a candidate from each bridge, in such a way that networkwide utility is maximized, and all cost and performance constraints are satisfied if possible.

This challenge can be recognized as a special case of integer programming problems known as the knapsack problem. It is one of the most well-known integer programming problems and has received wide attention from the operations research community during the last four decades. Although recent advances have made possible the solution of medium-size instances, solving this computationally hard problem remains intractable for full-size databases (*9*).

One of the oldest and most common applications of the knapsack problem is the capital budgeting problem (*10*). The large domain of its applications also includes cutting stock, investment policy for the tourism sector, allocation of databases and processors in a distributed data-processing network, delivery of groceries in multicompartment vehicles, multicommodity network optimization, and daily management of a remote sensing satellite (*9*). More specifically the program optimization problem is a Multi-Choice Multi-Dimensional Knapsack (MCMDKP) problem.

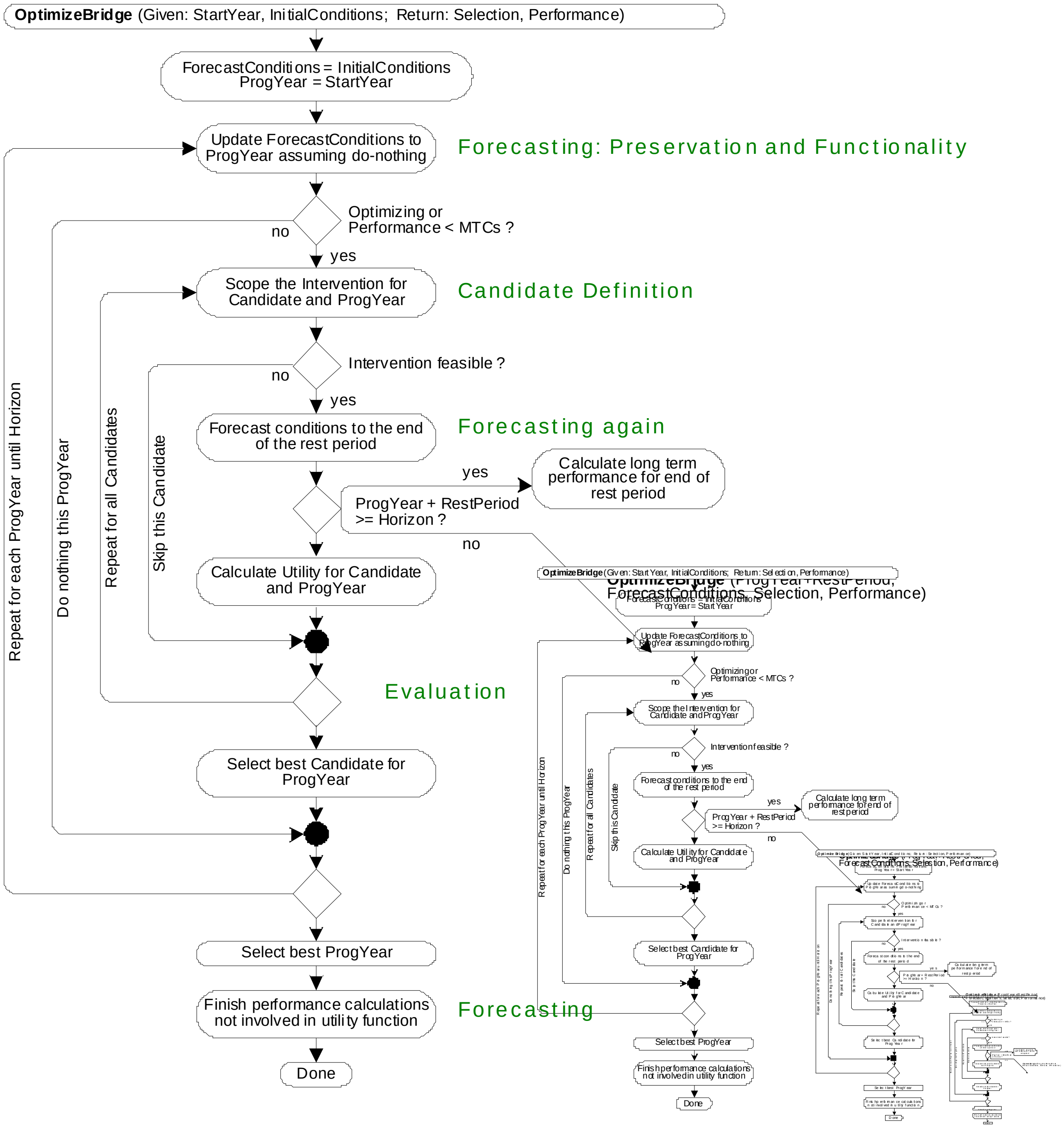

FIGURE 3 Recursive bridge-level optimization.

Accuracy Versus Responsiveness

The MCMDKP problem is considered NP-hard in the sense that no known deterministic polynomial algorithm exists. This means that the time requirement for the optimal solution grows exponentially with the size of the problem.

As an example of exponential growth, an optimization algorithm that takes 1 min on a test data set of 1,000 bridges might take roughly 3 days on an average state's inventory of 12,000 bridges and roughly 45 days on Texas' inventory of 50,000 bridges. In contrast, a polynomial algorithm's times for the same problem might be 1 min for 1,000 bridges, 2.5 h for 12,000 bridges, and 1.7 days for 50,000 bridges. For a suitably defined problem, solution times can be even faster than this. If the solution method could be reduced to simple sorting, it would have "*n*(log *n*)" execution time that would turn 1 min for 1,000 bridges into 13 min for 12,000 bridges and 31 min for 50,000 bridges. Obviously this faster behavior is preferred since it allows the user of the system to investigate many program alternatives.

There are two classes of methods that exist to solve this problem: exact methods and heuristics. Exact methods are guaranteed to arrive at the optimal solution but are typically associated with lower computational speeds. On the other hand, heuristic methods strive to achieve "good" approximate (near optimal) solutions quickly. So there is a trade-off between the accuracy and computational speed of the solution methods. The largest network we are dealing with is one with 50,000 bridges. Each bridge could have as many as five possible interventions including the do-nothing alternative. This means that are about a quarter million items or 0/1 variables for the knapsack problem.

This is a huge integer programming problem, so approximate heuristic methods are very appropriate. The performance of a solution method can be evaluated based on:

- Accuracy, i.e., how close is the solution to the true optimal solution?
- Computational speed, i.e., how long does it take to solve?
- Robustness of the method, i.e., how sensitive is its performance to variation in inputs?
- Simplicity, i.e., will the prospective users understand it well enough to use it effectively?

The accuracy depends on the type of solution method being implemented and the problem structure. The computational speed depends on a number of factors including type of method (theoretical computational complexity), coding language, coding efficiency, specific problem instances (realistic data sets), network parameters, and computer configuration.

Solution Method

Several heuristic methods were evaluated and tested to determine which one would give the optimal balance of accuracy and responsiveness for a program-level bridge management optimization, with acceptable robustness and ease of use. The algorithms selected for detailed evaluation were the IUC heuristic, the Lagrangian relaxation method (*11*), and the pivot-and-complement method (*12*).

These were tested on a set of example bridge management problems, ranging from 100 bridges to 50,000 bridges, generated by the Florida PLAT (*7*). For evaluation of accuracy, the problems were also solved to exact solutions using an industrial-strength integer programming software system, ILOG CPLEX (*13*). In the largest cases with CPLEX and the Lagrangian relaxation technique, memory requirements or execution time prevented complete testing.

The IUC method proved to be fastest, and very close to the most accurate. It is based on the classic linear relaxation of the MCMDKP problem, which is much more easily solved than the integer program by using a "greedy algorithm." Investment alternatives are arranged in decreasing order of reward to cost ratios. The algorithm simply scans down the list to fill the knapsack. Since the algorithm is based on sorting, it has *n* log *n* performance.

The IUC heuristic can be generalized to incorporate multiple constraints if we recognize an order of priority among the constraints. The strategy is to divide it into two separate single-constraint knapsack problems that are solved in tandem (Figure 4):

List A – Simple budget-constrained problem: $\mathrm{IUC(A)} = \dfrac{\Delta\,\mathrm{Utility}}{\Delta\,\mathrm{Cost}}$

List B – Feasible region problem: $\mathrm{IUC(B)} = \dfrac{\Delta\,\mathrm{Performance}}{\Delta\,\mathrm{Cost}}$

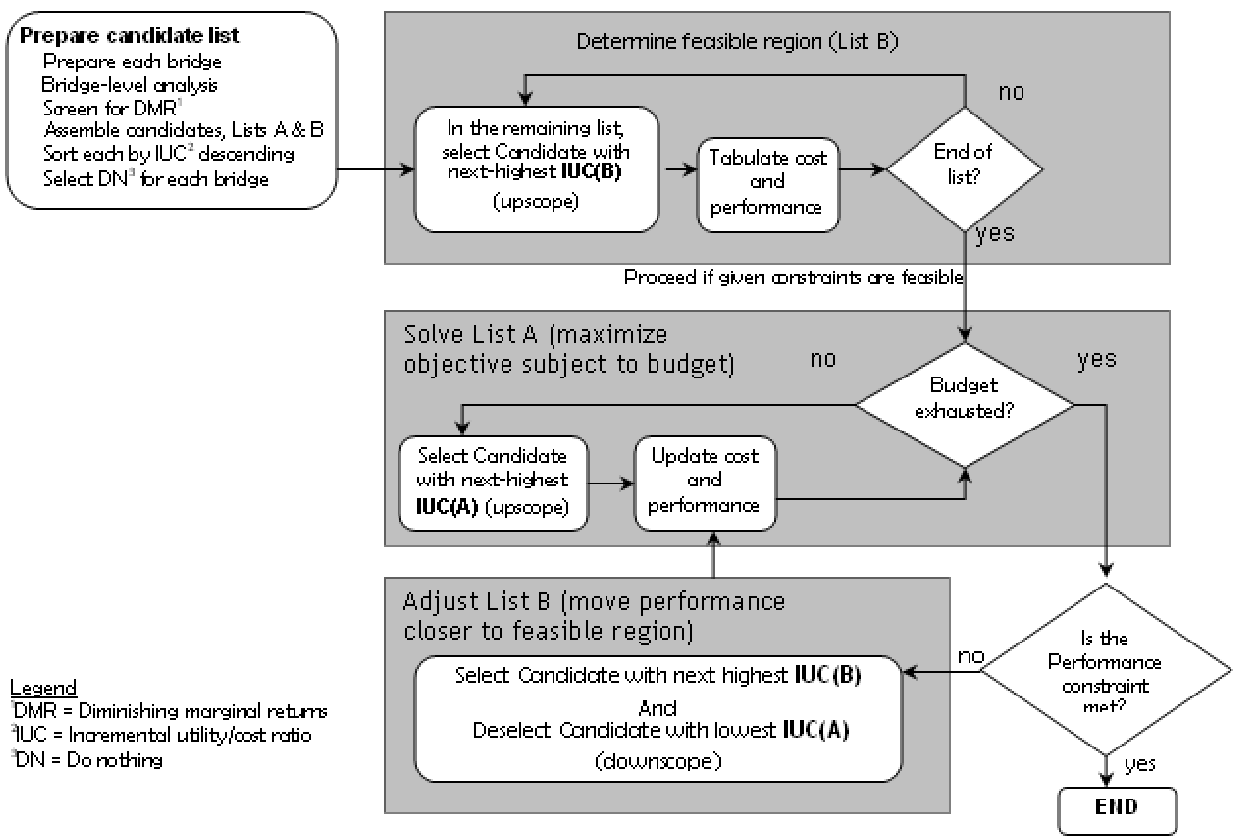

FIGURE 4 Flowchart for the IUC heuristic to solve the multiconstraint knapsack problem.

In the figure, List B describes the trade-off between the two performance criteria that present the constraints: performance and cost. The list represents an ordinary MCKP problem that maximizes performance subject to a budget constraint and can be solved using the ordinary IUC algorithm. The numerator of IUC(B) is the utility function that comprises multiple performance criteria. List A is the underlying optimization problem that has only the budget constraint. In the event that there is no feasible solution for the given combination of budgetary and performance constraints, such lack of a solution is detected in List B before proceeding into the main iteration of the procedure.

Typically, the first pass through the List A optimization does not satisfy the performance constraint. In that case List B is used to identify the candidate most responsible for the constraint violation. Then a second pass is made through List A. The process is repeated until both constraints are satisfied. If the performance constraint is not binding, the optimal solution will be one of the candidate combinations found in List A but would not necessarily be visited by the IUC algorithm on List B.

DIGITAL DASHBOARDS

The software products of the study include two visualization tools called digital dashboards: one for bridge-level analysis and one for network level.

The bridge-level dashboard (Figure 5) presents the current and forecast condition of a bridge's elements, and any functional or structural deficiencies that might exist. It presents the possible futures of a bridge as a table and graph of scope and timing alternatives. The graphs make it easy to see how changes in scope or timing might affect the future performance of the structure. Tools are included to help the engineer develop custom scope items and evaluate their effect on future performance.

For network-level analysis, the network dashboard (Figure 6) presents the key trade-offs between performance and funding. The program manager can specify annual budgets and the weight given to different performance measures. The graphics show how these settings affect future costs and performance. The optimization model can find the best performance achievable for any given funding level or the funding level required to achieve any given level of performance.

CONCLUSIONS AND NEXT STEPS

The study presented a novel new combination of methods that together make up a new state-of-the-art bridge management analytical framework. The various analytical tools were shown to be compatible with each other, to make effective use of readily available data, and to be usable within a user-friendly decision support system. They accomplished the project objectives of incorporating multiple objectives effectively while increasing the usefulness of bridge management systems.

Conclusions in the final report of the study identified a few additional data requirements, most notably a new system for assessing the vulnerability of structures to natural and man-made hazards. The research found certain deficiencies in commonly used methods for forecasting

FIGURE 5 Bridge-level dashboard.

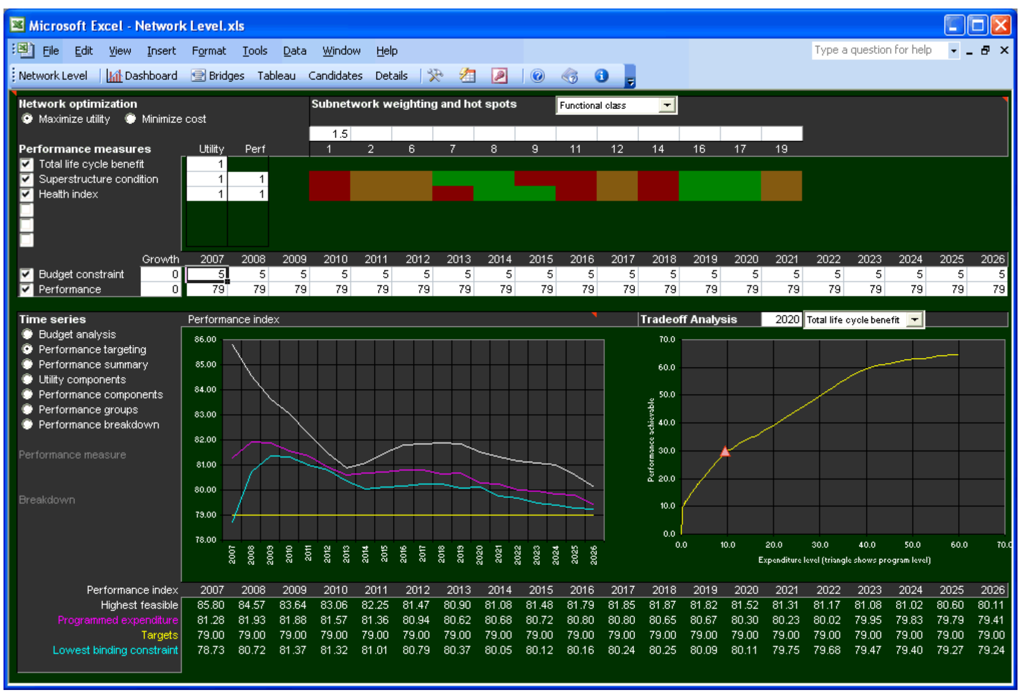

FIGURE 6 Network-level dashboard.

bridge deterioration and for translating those methods to NBI condition ratings. Future research will be necessary to correct these deficiencies.

Major new analytical and presentation methods described in this paper have encountered widespread interest since completion of the project. The AASHTO BRIDGEWare Task Force, which oversees development of Pontis, has decided to implement most of them in release 5.2 of that product, scheduled for completion in 2009.

ACKNOWLEDGMENTS

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The opinions and conclusions expressed or implied are those of the authors, and not necessarily those of the Transportation Research Board or the National Academies.

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Bridge Decks and Stay Cable

BRIDGE DECKS AND STAY CABLE

Rapid Concrete Bridge Overlays

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This presentation describes the implementation of rapid concrete bridge deck overlays and deck repairs in Virginia. Very early strength latex-modified concrete (LMC-VE) overlays and patches are being constructed at night or on weekends and opened to traffic with only 3 hours of curing rather than the 3 to 4 days of curing required for conventional overlays. The reduced lane-closure time for construction results in construction cost savings and reduced traffic delays, fuel consumption, and accidents. Evaluations indicate the overlay should last as long or longer than conventional overlays. Virginia Department of Transportation (VDOT) spends approximately $4 million per year on deck overlays. Because of the large savings in the cost of traffic control, LMC-VE deck overlays cost at least 25% less than conventional overlays. It is estimated that VDOT can save up to $1 million annually using LMC-VE deck overlays. The greater benefit is the reduction in traffic delays, fuel consumption, and accidents. The eastbound lanes on Interstate 64 over the Rivanna River near Charlottesville were overlaid with two Friday-night-through-Tuesday-morning lane closures in the spring of 2006. A conventional overlay would have required approximately 2 weeks of continuous lane closures. The user delay savings were estimated to be $519,000. The construction cost was approximately $750,000.

Prioritization Strategy for Replacing Florida's Deck Panel Bridges

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ATIQ ALVI
PB America

Precast deck panel bridges have a long history of poor performance in Florida. A spate of localized failures on major highways led to a decision by the Florida Department of Transportation (FDOT) to replace selected bridges on Interstate 75 in Districts 1 and 7 by full-depth, cast-in-place concrete slab over 10 years. This presentation describes the strategy developed by a University of South Florida research team to prioritize the replacement. A progressive degradation model was developed from a careful analysis of localized failures, on-site forensic investigation of deck panel bridges during their replacement, review of historical inspection data, and finite element analysis. This model was subsequently integrated in PANEL—custom software written for this project. A special database containing inspection records for more than 130 precast deck panel bridges in Districts 1 and 7 extending over 20 years in electronic form was created for PANEL. This allowed PANEL to automate the prioritization process. PANEL permits users to specify weighting factors for parameters such as safety, importance, and cost. This information was then used to create lists that ranked the order in which the replacement was to be carried out. Weighting factors used for ranking were calibrated using the latest inspection data. As the database can be updated easily to include new inspection information and photographs, PANEL provides a dynamic resource that can be used by FDOT to review and revise its prioritization strategy in the future to take into consideration the latest available information. This paper provides an overview of the 3-year research study.

Condition Assessment, Rehabilitation Planning, and Stay Cable Replacement Design for the Hale Boggs Bridge in Luling, Louisiana

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The Hale Boggs Bridge opened to traffic on October 5, 1983. At the time, it was the first U.S. cable-stayed crossing over the Mississippi River. The polyethylene protective sheathing was damaged in many of the cables before and during installation and after opening the bridge to traffic. Repairs were attempted to correct the defects in cable sheathing. Many of the repairs performed poorly and failed to protect the main tension element. The condition of 39 out of 72 cables indicated a critical need for repair, and timely action was recommended. To address these damages and to assure the structural integrity of the bridge structure, several strategies involving a range of repair and replacement options were evaluated using life-cycle cost analysis. It was concluded that the strategy to replace all cables presents the best value among evaluated alternatives. The design of the complete 72-cable array replacement is the first occasion on which this process is attempted in North America. The final design of the replacement cables is heavily influenced by the geometric restrictions of the existing anchorage locations. The replacement cables are being designed for a 75-year design life and incorporate the latest advancements in corrosion protection and vibration control. Maintenance of traffic design is an essential part of the project. The bridge is a critical regional link and hurricane evacuation route. Traffic maintenance is designed to be as unobtrusive to the public and commerce as practical. This paper describes efforts associated with cable condition assessment, rehabilitation strategy, and design considerations at the 30% design completion stage of Louisiana Department of Transportation and Development Contract entitled Phase III Final Plans, S.P. No. 700-45-0107.

The Hale Boggs Memorial Bridge, also known as the Luling Bridge, opened to traffic October 5, 1983. Designed by Modjeski and Masters and Frankland and Lienhard, it was the first cable-stayed bridge over the Mississippi River and, at the time, had the largest navigation channel span of its kind in the western hemisphere. The bridge was constructed by Williams Bros. and IHI, Inc., of Japan (Ishikawajima–Harima Heavy Industries Company). Figure 1 shows the bridge alignment and approach road network, and Figure 2 depicts a bridge elevation.

The Hale Boggs Bridge is a twin-pylon, cable-stayed bridge with a main span of 1,222 ft and two stayed-side spans of 495 and 508 ft. The bridge crosses the Mississippi River between Luling and Destrehan, Louisiana, carrying four lanes of Interstate 310 traffic. The pylons are a modified A-shape, and the deck cross section is composed of twin 14-ft deep steel trapezoidal

FIGURE 1 Hale Boggs Memorial Bridge alignment [Louisiana Department of Transportation Development (LaDOTD) photo].

FIGURE 2 Bridge elevation.

box girders with a total width of 82.33 ft, all made of weathering steel. Figure 3 shows the bridge during construction, highlighting superstructure configuration.

The stay cables are arranged in two planes and are anchored into steel cross girders at the deck level and into the pylon legs at the main piers. At each pylon, there are six cable group locations along each edge of the bridge, three each on either side of the pylon. There are 24 such cable groupings and a total of 72 cables. Cables are grouped by pairs or fours. In elevation, the cables are arranged in a semifanned pattern.

Stay cables were prefabricated and are composed of parallel, 1/4-in. diameter, 240 ksi wire bundles consisting of 103, 211, 271, or 307 wires. Near the anchorage sockets, the bundle flares out slightly from its compact shape in the free length, and each wire is anchored against a steel plate by means of cold-formed button heads. The wire bundle is wrapped with a 1/4-in. diameter, spiral-wound, seven-wire strand for holding it concentrically with the sheathing pipe. The wire bundle is encased within a black high-density polyethylene (PE) pipe. The space between the PE pipe and the steel cable and voids among the wires are grouted with cement.

HISTORY OF CABLE PERFORMANCE

After 25 years in service—years that encompassed frequent repairs to the cables, protective sheathing, and anchorage components—the Hale Boggs Bridge will undergo the complete replacement of its stay cable system.

The potential for stay cable durability performance problems arose during the construction of this bridge. The most significant were those associated with damage to the protective high-density PE sheathing of the main tension elements of the prefabricated stay cables. On the Luling Bridge, the protective sheathing around many of the cables was damaged before and during installation and after the bridge opened to traffic.

FIGURE 3 Hale Boggs Bridge under construction (LaDOTD photo).

Twenty-one cables required repairs before and during cable erection. Many of repairs performed before installation failed during pressure grouting and repairs were repeated in place. In April 1985, cracks were detected in two back stay cables' protective sheathing. The cable manufacturer electrowelded the cracks in November of that year. Later, cracks developed in these PE weld repairs. In the winter of 1985, new cracks were detected in three other cables. In 1990, after being exposed for 7 years, all cables were wrapped with ultraviolet (UV) protection (Tedlar) tape after existing splits and cracks were filled with epoxy. The first evidence of damage to the cable wrapping tape was detected in 1995. Subsequent inspections showed the existence of exposed and rusted stay cable wires, unplugged grout ports, extensive water leakage, cementitious grout efflorescence, and rust at the deck-level anchorage sockets.

Despite these ongoing inspection and repair efforts through the 1980s and 1990s, the stay cables' corrosion protection system continued to exhibit signs of deterioration, including rusting and water leakage in the anchorages and splitting of the protective sheathing. Concern for the fracture-critical components continued.

INSPECTION AND CONDITION ASSESSMENT

In 2002, Louisiana's Department of Transportation Development (LaDOTD) authorized initiation of a detailed condition assessment and rehabilitation study of Luling Bridge stay cable array. The department's vision was to assess cable array condition and project future maintenance and rehabilitation needs and costs. The state of the art in cable condition assessment technology and improvements in stay cable design, materials, and fabrication had changed dramatically since the bridge was designed in the 1970s, during the infancy of cable-stayed bridge construction in North America. The department was motivated to extract the fullest potential service life possible for its valuable infrastructure asset, taking advantage of stay cable engineering innovations.

Phase I

Phase I of the evaluation program was intended to assess and categorize the cable conditions, select techniques to assess the extent of problems in the stay cables and their main tension element, and to ascertain the overall integrity of the stay cable array. This initiating phase of the investigation included:

- A review of bridge details and prior bridge evaluations and investigations;
- Laser-based cable force measurements, geometric survey, and cable intrinsic damping measurement addressing cable susceptibility to wind-induced vibration;
- Representative cable and anchorage inspection and sheathing dissection;
- Development of an analytical model of the superstructure;
- Assessment of cable array safety, reliability, and potential failure scenarios; and
- Development of conceptual maintenance recommendations.

The cable force measurement and analysis (*1*) showed no evidence that stay cables had suffered significant structural damage to date; nevertheless, cable durability was a significant concern. Inspection of deck-level anchorages showed signs of rust and dripping water from anchorages, and ingress and collection of water inside the sockets. Inspection of selected cable surfaces revealed the ineffective repair of cover pipes that originally split during the grouting process, creating the potential exposure of steel wires to the environment and corrosion. Further cable damping measurements revealed that most of the cables possessed low damping capability and were susceptible to wind-induced vibrations, potentially introducing repetitive bending stresses near anchorages and impairing the cables' fatigue endurance.

From the Phase I investigation, it became apparent that categorizing and studying stay cable arrays' long-term durability and maintenance needs and costs would require a more-detailed inspection protocol and methods to assess extent of cable damage. Certain defects and damage in the cables were discovered to be potentially hidden from inspectors' view by the protective tape or the PE sheathing. Reliable techniques were needed for identifying hidden damage such as

- Unfilled splits in PE sheathing,
- Epoxy-filled splits in sheathing,
- Damage to UV protection tape, and
- Grout voids or damage.

Phase II

In 2004, Phase II commenced, therefore, with planning and development of a comprehensive cable condition analysis approach methodology. Technology was needed for identifying hidden cable damage and for accessing the stay cables for their full length.

Nondestructive Testing Approach Validated

A range of nondestructive testing (NDT) techniques capable of detecting flaws in materials and assemblies were sought to augment visual inspection. Full-scale mock ups of stay cables with intentionally inflicted PE flaws in the form of splits, with and without epoxy repair and wrapped with UV protective tape, were fabricated. Candidate NDT methods offered advantages and disadvantages in terms of effectiveness, efficiency, adaptability to field conditions, and cost (*2*).

Inspection Access Considerations

The second challenge confronting the Phase II condition assessment was efficient, safe access for inspectors over the entire length of the cables. It was quickly determined that the best approach would be to use a manned inspection vehicle that crawls along the stay cables. Although the design and fabrication of the inspection vehicle would require considerable time and cost, this approach offered some significant benefits.

- Thoroughness: inspectors were able to cover every inch of the cable sheathing and combine visual inspection, hammer sounding, and thermography.
- Improved comfort and safety: compared with the alternative of rope rigging and bosun's chairs, the inspection hoist allowed investigators to concentrate more fully on the work at hand and worry less about rigging and personal safety.

- Reusability: LaDOTD expressed interest in utilizing an inspection buggy for future inspection, maintenance, and repairs to the Luling Bridge.
- Minimal user and environmental impact: the inspection could be carried out quickly and with minimal disruption of traffic on the bridge. Traffic control limitations restricted inspection teams to shoulder closures, thus limiting additional congestion and auto exhaust emissions.

Hoist Design and Fabrication

Working to the consultant's requirements and specifications, an Ohio-based specialty fabricator designed the inspection vehicle. The buggy is capable of carrying two inspectors and equipment, while traveling at a rate of about 50 ft per minute. It provides convenient hands-on access to multiple cables in a cable group; is simple to install, operate, and move; and meets all Occupational Safety and Health Administration safety requirements. By early 2006, the vehicle basket had been fabricated from aluminum for weight reduction advantages. The basket is attached to the lower cable of each group by a pair of rollers. Lift is provided by a hoist wire rope installed parallel to the cable, and a standard wire rope hoist attached to and operated from the basket. Safety brakes were provided both on the hoist and upper roller. The inspection buggy is shown in use during inspection in Figure 4.

Phase II Inspection Procedures

The inspection began in April 2006 and, with 5½ mi of cable to ride, was expected to take 12 weeks to complete. Two inspectors rode in the hoist, observing, hammer-sounding, and digital

FIGURE 4 Inspection buggy in use.

tape testing the cable cover pipe and UV protection tape to locate damage. Infrared thermography was used selectively to detect open splits in the PE cover pipes beneath the UV protection tape, as well as potential voided zones in cementitious grout.

Observations of main tension element (steel wire) were made where these were exposed by defects and deterioration in the protective elements. As the inspection progressed, the inspection teams carefully recorded their observations. Cable diameter was verified periodically and wherever an extraordinary change in cross section was visible. A laser distance meter was used to measure the locations of inspection points from either the upper or lower cable exit points. Thanks largely to good weather, the inspection was completed in 9 weeks.

Additionally, cable anchorages were inspected. Anchorage inspection included visual examination of the bearing plates, shim plates, and the exterior of the anchorage sockets. End caps were opened to observe the condition of the wire buttons and inside of the socket. Each cable, immediately above the anchorage, passes through an anchorage box within the cross girder. The cables were not accessible for direct visual inspection in this region. In order to gain access to the cables, the neoprene washers at the exit points had to be removed and the inspection was performed using a videoscope.

Inspection Observations

Damage observed in the anchorage zones mainly comprised corrosion of sockets and button heads, missing or broken seals at the joint between the transition pipe and PE pipe, and open grout ports. With time, precipitation leaked into the anchorage boxes from gaps in and around neoprene washers at the cable exit points and collected behind the bearing plates. Broken seals and ports also had allowed water to accumulate within the cable in the transition zone and inside the sockets. Because of poor access, detailed assessment of corrosion and cross-section loss of wires inside the socket and in the transition zones could not be performed. Hence, there remained uncertainty regarding the condition of the main tension elements in these areas.

Inspection of the free length discovered a variety of damage conditions in the cable-free length. These included

- Longitudinal and transverse splits in the PE pipe; some covered by UV protection tape, and some not (Figure 5);
- Bulges and holes in the PE;
- Burst and damaged tape (Figure 6), exposure and degradation or corrosion of grout filler and steel wires (Figure 7);
- PE joint separations and failure of previous repairs and grout voids and delamination; and
- Damage to UV protection tape (Figure 8).

Cable Condition Rating

Three levels of damage severity were established for cable condition rating; Level 1 or satisfactory, Level 2 or poor, and Level 3 or critical. These condition ratings were intended to semiquantitatively reflect the cables' future durability for rehabilitation planning.

The severity levels were assigned to observed condition of the corrosion protection barrier elements, namely the UV protection tape, PE pipe, and cement grout; their ability to

FIGURE 5 PE pipe split; UV protection tape missing.

FIGURE 6 Burst and damaged protective tape.

FIGURE 7 PE sheathing split, wire corroded, and wire not providing protection.

FIGURE 8 Separation of PE transverse joints.

protect the main tension element; the condition of the steel wires in the anchorage zones—based on conditions observed within the anchorage sockets; and the need for action or repair.

Condition Level 1 encompassed cables with sound and well-functioning repairs as implemented earlier to the damaged PE pipes, in the form of welding or epoxy fillers. Normal aging and weathering of UV protection tape, minor delamination of PE from grout filler, and light rust inside the anchorage sockets and on wire buttonheads fell into this category. Level 1 anomalies would not require any corrective action until future inspections revealed increasing deterioration rates.

Condition Level 2 included UV protection tape damages and nicks, grout voids, repair sleeves, PE bulges, major tape wrinkles, and moderate rust inside the anchorage sockets. At a minimum, locations with this level of damage severity need to be monitored routinely and a plan for repair or correction should be developed for implementation in the near future.

Condition Level 3 encompassed ineffectively repaired and unrepaired PE splits, holes in PE pipes, exposed grout and wires, transverse PE joint cracks and separations, and heavy rust inside the anchorage sockets and on wire button heads. This level of damage severity necessitates near-immediate repair and close monitoring until repairs are implemented.

Using the cable condition rating system, condition rating factors for each cable were established, based on damage accumulated along the free length of cables and within anchorages. Any cable containing at least one indicator of Level 3 critical damage was automatically rated with Severity Level 3. All cables, without exception, contained at least one form of damage or anomalies with Severity Level 2. Thirty-nine of 72 cables were rated as being in critical condition (Level 3), and the remaining 33 cables were rated as being in poor condition (Level 2).

REHABILITATION STRATEGIES AND COST STUDY

The selection of the most appropriate course of action for bridge service life extension was selected based on economic and technical–operational factors. To compare and provide guidance for maintenance and rehabilitation decisions, five repair scenarios, representing a wide range of repair estimates derived for correcting defects and damage to the stay cable array noted during the condition assessment, were studied in a life-cycle cost analysis (LCCA):

1. Base case: do minimal repair only, to protect exposed wires along the free length of the cables. Include monitoring and inspection regimen prescribed for Level 3 and Level 2 damages; this strategy has the highest potential for cable accelerated degradation and failure, and highest vulnerability to storm-related damage.
2. Repair all: repair free length of all cables, clean corroded sockets, and provide drainage for anchorage boxes. Included is the monitoring and inspection regimen prescribed for Level 2 damages. New repairs were assumed to be necessary every 20 years. This strategy represents moderate potential for cable failure and moderate vulnerability to storm-related damage.
3. Repair–replace option 1: replace 20 cables—13 cables rated with anchorage Severity Level 3 and seven cables rated with most extensive Severity Level 3 damages; repair remaining 19 cables rated with Severity Level 3 and all Level 2 cables. Included is the monitoring and inspection regimen prescribed for Level 2 damages for the repaired cables. New repairs are

assumed to be needed every 20 years. Low to moderate potential for cable failure and low to moderate vulnerability to storm-related damage are anticipated.

4. Repair–replace option 2: replace all 39 cables rated with Severity Level 3, and repair all cables rated with Severity Level 2. Includes monitoring and inspection regimen prescribed for Level 2 damages for the repaired cables. New repairs are assumed to be needed every 20 years. Low potential for cable failure and low vulnerability to storm-related damage are expected.

5. Replace all: replace all cables. No potential for cable failure and no added vulnerability to storm-related damage for the bridge structure are expected. Visual inspection will be prescribed only once every 20 years, and force measurement every 5 years.

Life-Cycle Cost Analysis

An LCCA was performed for the selected strategies over a 75-year planning horizon. For present value calculations, a real discount rate of 3.8% was assumed, reflecting a nominal rate of 5% and inflation rate for the overall economy of 1.2%.

The costs associated with the various options were divided into three groups: initial, distributed–periodic, and vulnerability costs. The initial costs are related to the installation of a monitoring system, repair, or replacement of cables that will occur 1 year after the project begins. Distributed–periodic costs are related to inspection, cable force measurement, maintenance of the monitoring system, and future periodic repairs. Vulnerability costs are related to the potential repair–replacement of cables and structural repair to the bridge superstructure due to loss of load carrying capacity of the cables from ongoing corrosion–fatigue and extraordinary storm events (hurricanes).

Costs in each category were assigned two components: agency cost and users' cost. Agency cost refers to the actual cost of implementing an event such as contract cost for repair or inspection. Users' cost refers to cost borne by the users of the bridge, i.e., drivers, for delays or detours related to activities on the bridge.

Results of Cost Analyses

The first-pass LCCA indicated that the base case presents the lowest cost ($0.7 million), while the replace-all scenario reflected the highest potential cost ($16 million) when only initial agency costs are considered. The costs of other strategies increment almost linearly between these two extremes in the order described above.

When distributed, periodic, and vulnerability agency costs were added, the cost for the base case ($17.5 million) became costliest. In this case, the repair-all strategy results in the lowest total cost ($12.7 million) when only agency costs are accounted for. With the users' costs added, all four strategies involving repair and replacement compete closely (around $20 million) while the cost for the base case ($35 million) is significantly higher than others.

In accordance with economic efficiency theory, a strategy with the lowest present value should be selected as the preferred strategy. If this rule were to be followed, the repair-all strategy should be adopted when only agency costs are considered, and the replace-all strategy should be selected when agency and users' cost are both considered.

However, taking into account uncertainties and approximations affecting cost estimates and the sensitivity of present values to variations of costs and discount rate, such a determination cannot be made with comfortable certainty. The only strategy that can be ruled out with certainty is the base case. The total cost of this strategy was considerably higher when compared to the lowest-cost strategy.

Furthermore, it was recognized that cost efficiency may not be the only parameter to be considered in a decision-making process. The location of a bridge structure in a highway transportation network is one parameter that might significantly influence the decision making. LCCA of a bridge isolated from the regional highway system, though effective and useful, cannot include or quantify all contributing cost aspects. The LCCA results were evaluated in conjunction with network- or system-strategic considerations. Another intangible but crucial parameter not considered here is the public safety concern, especially in case of emergencies related to evacuations caused by natural disasters known to the region. Bridge closure and traffic limitations necessitated by an unpredictable need for repairs to deteriorating cables, for example, may have consequences above and beyond economically measurable factors.

It was concluded, therefore, that when the LCCA results are considered along with the effect of costs not included in the analysis, anticipation of lower discount rates, and concerns related to highway network system and public safety, the replace-all strategy presented the best value among selected strategies. The long-term benefits of replacing all cables outweigh the higher initial investment when compared to other strategies.

DESIGN OF CABLE REPLACEMENT

Cable Replacement Design Strategy

Based on the bridge's history and the results of the in-depth 2006 inspection, LaDOTD will extend the Hale Boggs Bridge's service life by replacing all of the stay cables. The project is the first to include the replacement of a complete stay cable array in North America. Its goal at a minimum is to restore the load-carrying capacity of the bridge to its as-designed state without detriment to the bridge structure.

Having already acquired an in-depth understanding of the bridge, its structural condition, and need for improvement, the consultant was asked to broaden the scope of the work to include developing and providing the design drawings, specifications, special provisions, and cost estimates required to solicit bids for the replacement. The project is charged with developing a complete, cost-effective cable replacement scheme that bidders can implement directly with minimal engineering. The project must be designed to minimize the impact on traffic and on the "maintenance of traffic" costs by maximizing the number of traffic lanes kept in service on the bridge at all times.

The cable replacement project has passed the 30% design completion phase. Construction is expected to begin by the first quarter of 2009. For its investment in this historic span, LaDOTD will have gained a cable-stayed bridge with an unimpaired stay cable array, one that incorporates the advances in stay cable technology developed over the past quarter-century—and promises to serve the public for many years to come. Design progress to date is discussed by task below.

Design Task A: Assess Current Conditions and Design Requirements

At a minimum, cable replacement must restore load-carrying capacity of the bridge. Forces of all 72 stay cables on Luling Bridge were measured using the laser-based vibration technique. These new results were compared to estimated forces from measurements performed in 2002, and the final dead loads taken from the table of cable loads from the record drawings.

The target geometry to be created by the new stay cable array may not match the existing condition and is being designed taking into account structural capacity, serviceability, roadway vertical geometry constraints, and hydraulic drainage considerations. Hence, a complete survey of the bridge superstructure was performed to establish the current state.

Bridge cross sections at 50-ft intervals and coordinates of crossgirders, towers, piers, and bearings, needed for constructing a model representing the existing geometry of the bridge superstructure, were determined.

Design Task B: Design Replacement Cable

Parameters considered in identifying candidate cable systems included adaptability to existing openings, passages, and anchorage zones; accommodation of erection process related to available spaces near anchorage areas; durability issues and past performance; corrosion protection requirements; ease of future inspection and repair; availability of material and sources in the United States; and cost. There exist two stay systems deemed suitable for replacing the stays at the Luling crossing.

Parallel Wire Stay

The specification of a parallel wire system would result in a direct replacement of the current system with an updated, prefabricated parallel wire stay, including improvements in the corrosion protection systems. It is possible that no structural modifications to the bridge would be required at all if installing a similar parallel wire cable array.

Worldwide, this system is preferred for longer-span cable-stayed bridges. It provides a more compact stay cable with greater stiffness than the parallel strand equivalent. The wires are individually galvanized and wrapped to provide corrosion protection. However, it is difficult to inspect the individual wires, and individual wire replacement is not possible. At present, parallel wire stay cable systems are only fabricated overseas; North American stay cable suppliers do not produce galvanized wire to the required specifications, nor do they manufacture parallel wire stay cable systems.

Parallel Strand Stay

Since 1990, the parallel strand cable has been far more prevalent for medium-length cable-stayed bridges. Presently, strands are individually greased and sheathed, providing individual corrosion protection, and a wedge retention system that allows for individual strand installation and replacement has been widely adopted.

Structurally, the stiffness of strand is less than that of wires. The system’s principal disadvantage for replacing the stay cables of the Luling Bridge, however, is that the anchorages are larger in size than those of an equivalent wire system. Stay sheathing pipes are larger as well, increasing cable aerodynamic profile and wind loads on the bridge superstructure.

A compact parallel strand stay cable sheathing pipe and strand installation method has recently been made available by one stay cable supplier, to overcome some of the aforementioned disadvantages of the parallel strand stay. It offers an alternative to be evaluated for use on a bridge designed originally for a parallel wire cable. However, the feasibility of future individual strand removal and replacement after completion of cable erection for inspection and repair purposes has to be resolved.

Ongoing stages of the final design process are further evaluating suitability. The following sections of the paper discuss issues related to incorporation of a parallel strand-based system.

Superstructure Modifications Needed for Candidate Cables

To have the least impact on the bridge, it is believed most straightforward to match stiffness of the new cable to that of the existing. It might be possible to reduce the cable size to an equivalent strength cable, but extensive modeling and analysis will be required to verify that other parts of the bridge do not become overloaded as a result of the lower stiffness of the stay cables.

The degree of anchorage modification required will depend on the selected stay cable supplier's anchorage and stay pipe dimensions. The design team is evaluating feasibility of adapting the range of options offered by prevailing stay cable systems, extracted from manufacturers' published data.

Review of the anchorage zone dimensions has indicated that in order to allow a new parallel strand system to be installed, existing anchorage openings will need to be enlarged to accommodate the larger strand anchorages. These alterations increase construction costs and require heightened attention during construction to prevent damage to adjacent stay cables.

Means are being devised to avoid enlarging the existing holes in these zones, adapting chair-like spacer assemblies. A spacer assembly can be designed to create distance between the anchor plates and the existing holes, therefore clearing the steel anchor pipe in the cable system.

Corrosion Protection Requirements

The stay cable is required to safely withstand constant tensile forces resulting from superstructure dead load and fluctuating live load stresses during service. The high-strength main tension element (MTE) materials are inherently more susceptible to corrosion and deterioration, than are mild steel counterparts used for concrete reinforcement and structural steel fabrications.

In North America, the stay cable design has evolved in 30 years from the bonded post-tensioning tendon technology, which relies on metal or plastic ducts and cementitious grout–filler. It is noted that the oldest seven-wire prestressing strand-based stay cable system in the United States is only 24 years old. Because there exist no scientific models and durability performance databases for reliably forecasting all the corrosion processes to which the MTE is susceptible, and because stay cable sheathing obscures the MTE from view during periodic bridge inspection, the present corrosion protection philosophy for the tensile element of cable-stayed bridges in the United States and in Europe is to provide reliable, compatible, and permanent multilayer corrosion barriers which prevent MTE corrosion over the full length of the cable.

Presently available corrosion barriers encompass, grease- or wax-filled individual strand sheathing, individually fusion-bonded epoxy coating, cementitious grout, and other fillers or blocking compounds, and high-density PE (HDPE) or steel sheathing. Sacrificial zinc coatings for strand, frequently used outside the United States for stay cables, are also available. The Post Tensioning Institute's (PTI) *Recommendations for Stay Cable Design, Testing and Installation*

[4th edition (2001) and 5th edition (2007)] (*3*) requires a minimum of two nested, qualified barriers for corrosion protection of the MTE. It is noted that the PTI Cable-Stayed Bridge Committee does not recognize the sacrificial zinc layer to be one of those barriers.

The transition zone between cable free length and the cable anchorage, as well as the anchorage itself, also require corrosion protection. In the transition zones of stay cables, where continuous corrosion barriers applied directly to the MTE are interrupted along the path to the structural anchor, equivalent and compatible materials are provided to maintain end-to-end protection for the stay cable MTE in these zones.

Cable Stressing Method

Cable stressing can be performed strand-by-strand or by preloading the entire cable (multistrand method). The strand-by-strand method has many practical advantages when compared to multistrand method. These include the use of single strand hydraulic rams that are significantly lighter, smaller, and easier to operate than rams required for stressing the whole assembly. Because of the unique condition of the anchorage spaces of the Luling Bridge, it is preferable that the individual method be utilized for stressing of the new replacement cables. Detensioning of existing cables, however, will require the use of a single large ram for the cable, similar to that used for original stressing of these cables.

Wind Load Considerations

The wind load imparted on the bridge by the cables is a direct function of the stay cable sheathing diameter. The existing parallel wire system is compact, and care must be taken to ensure that the replacement system’s wind profile does not overload the structure. It will be beneficial to use a stay cable system that minimizes wind loads.

Finite Element Analysis and Local–Global Structural Integrity Check

A 3D model of the bridge has been developed to determine the response of the structure during each step of the cable replacement process. It is expected that the replacement scheme will impart minimal changes to the distribution of loads through the structure. The model assesses variation in the loads induced in the structure during the replacement and to verify that the replacement stay cables perform in the same way as the existing stay cables. The components of the bridge will be checked to ensure they possess sufficient capacity to carry any increased loads during the work or after completion, if differences are found.

Design Task C: Design Temporary Cable

To protect the bridge structure from excessive force variation during removal of existing cables, and to eliminate the potential for rupture of adjacent cables, temporary cables will be used to carry the dead loads and balance the forces when existing cables are removed.

In addition, inspection of existing cables has shown that damage exists in both free length and anchorage regions of cables. These damages generate uncertainties in the current overload-carrying capacities of existing cables. Temporary cables are designed for force levels determined by structural analysis and the geometry of the bridge, including wind and hurricane events.

Connection of these cables to the superstructure at the deck level and to the tower was designed. The contractor will be required to provide the suggested design or propose a comparable design for LaDOTD approval.

The temporary cable erection concept has been developed. It consists of a cable saddle on top of the tower and a spreader (waler) beam at the deck anchorages. This system will require that a main span and a back span cable are replaced simultaneously, so that the forces in the temporary cables are approximately balanced. This system has a significant advantage since it does not require that access be maintained to the top of the pylon and that all stressing operations can be completed at deck level. The system is designed to accommodate the range of stay cable angles and therefore allows for reuse of the saddle and temporary stay cable.

The 3D finite element model developed for the bridge structure is being used to assess variation in the loads induced in the structure during the placement and stressing of the temporary cables and their effect on the global structural integrity, and to verify that the cable connections to the bridge will not result in local overstress or damages.

The required size of the temporary cable will be finalized after detailed computer analysis. The temporary cables are being analyzed for wind stability.

Design Task D: Design Peripheral Bridge Elements

Peripheral design tasks include design of the drainage system for the deck-level anchorage boxes, evaluation of deck-level surface drainage, taking into account geometry changes created by cable replacement, enhanced security and antivandalism measures for stay cables near deck level, and damping requirements for the replacement cables.

Security and Antivandalism

Cable security has become an increased concern for bridge owners, particularly since the events of September 11, 2001. In response to increasing concern over the vulnerability of prominent transportation structures to vandalism or a terrorist attack, FHWA commissioned the Blue Ribbon Panel on Bridge and Tunnel Security. The culmination of their work, Recommendations for Bridge and Tunnel Security, was published in September 2003. This document provides a framework for assessing the risk of a bridge or bridge component in terms of importance, occurrence, and vulnerability to an attack. The three risk factors are assessed qualitatively to identify a facility risk score. This score is then compared against the cost, effectiveness, and availability of potential mitigation options to determine whether counter measures are prudent. Based on the guidelines of the FHWA document, preliminary analyses have indicated that only the free length of cable above the lower end exit point up to approximately 10 ft above deck level would generate a risk score that necessitates consideration of counter measures. An initial list of countermeasures has been developed for further consideration:

- Monitoring:
  - Remotely monitored video;
  - Increased patrol by local authorities, state police, and LaDOTD personnel;
  - Signage notifying the public that the area is under video surveillance; and
  - Installation of “dummy cameras” as a deterrent.
- Strengthening:

    - Fiber wrapped lower sections of cable free length; and
    - Welded blast plates in the anchorage zone.
- Access restrictions and increase standoff distances:
    - Taller concrete barrier near the lower anchorage; and
    - Installation of additional fences near the cable anchorages.

Cable Damping Requirement

The Phase I scope lead to the conclusion that the existing stay cables of the Luling Bridge are susceptible to wind-induced vibration, requiring vibration suppression. Damage to seals and scarring of PE cover pipes at cable exit points were partly attributed to the past uncontrolled vibration experiences. After design of the replacement cables is in its final stage, a vibration study will be performed on the replacement cable system to determine susceptibility to wind-induced vibrations. Vibration suppression measures will be designed.

The measures may include one or more cable surface modifications, internal dampers at cable exit points, and external viscous damper away from cable exit points.

Anchorage Drainage

An anchorage box drainage system has designed to address the issue of water ponding at the lower cable transition zone.

Design Task E: Design Maintenance of Traffic

Maintenance of traffic (MOT) is a critical part of the cable replacement construction process. The Luling Bridge is a critical regional link, and traffic interruptions need to be as unobtrusive to the public and commerce as is practical. Additionally, the bridge is located in a hurricane region and is part of a hurricane evacuation route. Limitations on traffic control that impact evacuation capacity, including seasonal timing, have to be considered. The MOT design is being coordinated closely with LaDOTD district and headquarters offices.

Several traffic control scenarios were initially identified and studied. The overall objective of the MOT is two-fold: provide enough work area to facilitate construction and minimize lane closures and impact on traffic. The optimal scenario will most likely involve a combination of alternatives depending on work zone width requirements throughout construction. To this extent, a movable barrier system that can accommodate multiple lane configurations based on construction needs at various times of the day would be the optimal solution.

The ideal scenario would be to maintain traffic on each side of the bridge during construction to eliminate the need for crossovers. This scenario will allow the ramps to remain in service, resulting in minimal disruption to local traffic. If the predominance of construction work can be accomplished within a 12 ft, 3 in. work area, then two lanes in each direction can be provided for traffic. This will provide a relatively narrow but workable area. If a particular construction operation requires more space, then the barrier can be moved during off-peak travel times to provide a wider work area. As long as the maximum construction work area width requirements do not exceed 23 ft 4 in. for any operation during off-peak hours, then the desired scenario is achievable.

Design Task F: Determine Construction Sequence

A construction procedure for the replacement activities has been devised to address the cable replacement cycle process. The construction will be limited to one side of the bridge, and each quarter of the bridge at a time. The sequence starts with placement of a saddle support on top of each leg of pylon. This will be followed by installation of a proposed highline or cableway system, to aid in stay cable removal and installation.

Highline System

The highline system can simplify installation of both the temporary and permanent stay cables by supporting the stay cable pipes and reducing cable sag. It will be essential for the removal of the existing stay cable. The flexibility of the stay cable is significantly less than, and the weight significantly more than, when it was installed, due to the cement grout installed after erection. Using the highline, the stay cable can be supported at intermediate locations and lowered to the deck under controlled conditions.

Just as importantly, using the highline will reduce the need to position a mobile crane on the bridge deck, and allow the contractor to work within a smaller footprint on the bridge for the majority of the operations. The highline design will be the responsibility of the contractor, who will have specific capacity requirements, depending on the stay system components and preferred work methods.

Construction Space Requirements on the Deck

It is believed that the proposed sequence and the use of a highline system will allow the majority of the construction operation to be carried out within a space of about 12 ft on the right side of each bound. There could be occasions when for some operations, such as installation of large equipment and supports, a wider space may be needed. Nevertheless, it is believed that this proposed sequence will allow two-lane traffic on the construction side of the bridge for most of the time, and one-lane for occasional off-peak hours. Further analysis is ongoing to assure that the combination of construction loads and live load will be within a range that is safe for the structure.

Finite Element Analysis of Construction Sequence

The selected new stay cable array installation and construction sequence will induce force variations and deformations at the cross beam ends and the tower. It is expected that the replacement scheme selected for this project will result in minimal changes to the distribution of loads throughout the structure. This will be confirmed as the design progresses.

Nevertheless, design of new and temporary cables and development of construction sequence are affected to varying degrees by the process. Structural analysis is supporting the design effort by determining the effect of these variations both locally and globally.

At the local level, connections of the temporary cables to the cross beam and tower and their local effects on the structure are investigated. The effects of modifications to the existing structure for accommodating replacement cable installation are analyzed locally. Also, eccentric forces generated by cutting cables among a group of cables need to be studied.

At the global level, the structure as a whole is analyzed for force variations from installing temporary cables, removing existing cables, and stressing of replacement cables. This requires, at a minimum, analytical modeling of the bridge superstructure.

The structural analyses may determine limitations for force variations, and for cable and connection design details. With this information, the construction sequence can be revisited and modified if necessary. The analysis could also limit the live load at certain stages of the construction.

Structural analysis will also be used to specify and optimize force application at various stages to produce the desired final force array and geometry with minimal iterations.

CONCLUDING REMARKS

The final design of stay cable replacement for the Hale Boggs Memorial Bridge is the first of its kind in North America and will progress through the summer of 2008. This paper, submitted for the Transportation Research Board's 10th International Bridge Structure and Management Conference in Buffalo, New York, describes design progress approaching the 60% design completion stage in April 2008 and highlights some of the key design issues confronting the design team. The paper summarizes the decision process for selecting cable replacement as a means for extending the bridge's service life. The stay cable array replacement project will be advertised for bid in late 2008, and it is anticipated that cable replacement will commence in early 2009.

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Structural Performance, Monitoring, and Remaining Life

Federal Highway Administration's Long-Term Bridge Performance Program

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With the recent passage of the Safe, Accountable, Flexible, and Efficient Transportation Equity Act: A Legacy for Users, the FHWA is initiating the Long-Term Bridge Performance Program (LTBPP). The LTBPP is an ambitious 20-year research effort that is strategic in nature and has both specific short-term and long-range goals. It will be similar to the Long-Term Pavement Performance program that has been underway for more than 15 years. The objectives of the LTBPP are to collect, document, and make available high-quality quantitative performance data on a representative sample of bridges nationwide. The collected data will be used to develop greater knowledge regarding bridge performance and degradation, develop better design methods and performance predictive models, and support advanced management decision-making tools.

Long-Term Structural Monitoring for Asset Management

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Scour and other natural hazards have the potential to undermine the structural stability of highway bridges and the piers that support them. This has led to bridge collapse in the past, and significant efforts have been undertaken to address the potential danger of scour and other hazards to pier stability. To effectively manage these assets, reliable systems for measuring and reporting structural conditions are required. However, there remains a lack of reliable, cost-effective, long-term monitoring devices capable of determining the structural stability of bridge piers. This paper will present the results of efforts to utilize an array of low-cost tilt sensors, deployed on both the pier and the superstructure of the bridge, to monitor long-term structural behavior. The sensors are deployed in an array so that multiple sensor outputs can be integrated to increase signal to noise ratios, eliminate erroneous readings, and provide systematic sensor redundancy. Additionally, using increased numbers of sensors allow for the application of more-advanced signal processing schemes including correlation algorithms developed to measure and better understand structural behavior. The application of such monitoring devices and how they integrate with overall bridge management strategies will be discussed.

Scour and other natural hazards have the potential to undermine the stability of highway bridges and the piers that support them. Scour occurs when flowing water removes material from around bridge piers, thus creating scour holes beneath footings that can jeopardize the stability of the bridge (Richardson, 2001). Other hazards, such as erosion and unexpected settlement, can also result in a loss of subsurface support. Unexpected superstructure behavior, such as locked bearings, can manifest in unexpected structural movements that can lead to bearing failures and even collapse. These effects have caused bridge collapse in the past. Significant efforts have been undertaken to address the potential danger of scour, and a number of scour monitoring devices have been developed to address this need (Schall, 2004). However, there remains a lack of reliable, cost-effective, long-term monitoring devices capable of monitoring the condition of bridge piers when other events, such as unexpected settlement or bearing conditions, occur. These events typically affect a bridge over a long time period, perhaps years, and separating these conditions from normal structural movements is challenging.

Tilt sensors have been proposed and are currently utilized in systems intended to monitor short-term events, such as a scour event that occurs during a time of flooding (Richardson, 2003). These systems frequently include a limited number of tilt sensors, due to cost and other considerations. As a result, the systems lack redundancy, and the measurement configurations assume rigid body rotations along a specific axis will occur. Thus, the systems may be incapable of identifying unusual structural motions, such as vertical displacement. For example, systems that utilize sensors located on the pier may not detect a movement in which the pier itself does not tilt, such as sinking of the entire structure due to settlement. In addition, a long-term gradual tilt may remain undetected or assumed to be sensor drift characteristics rather than actual structural movements.

In order to address these issues, a long-term multiple-sensor system is being developed. The concept of this system is to utilize an array of low-cost tilt sensors, deployed on both the pier and the superstructure of the bridge, to monitor the stability of a bridge pier. The system is being developed to measure both changes in rotation (tilt) as well as vertical displacement of a pier, allowing for a more complete understanding of the behavior of the pier than is available using currently available technologies. The sensors are deployed in a high-density array, such that multiple-sensor outputs can be integrated to eliminate erroneous readings, provide systematic sensor redundancy, and increase signal-to-noise ratios. Signal processing correlation algorithms are being developed that use sensor density and location to better measure and understand long-term bridge rotations and displacements. The system is intended to provide a tool for long-term asset management, allowing owners to monitor remotely the behavior of a bridge over many years, and provide notification if long-term structural motions are occurring that may lead to structural collapse.

This paper will present initial results and developments for the system being developed. The equipment being used to develop the system is described, as well as algorithms intended to provide bridge owners with quantitative data on structural motions of bridge piers.

PROJECT EQUIPMENT

The system being developed employs electrolytic tilt sensors for monitoring the tilt of a bridge pier and structural displacements. Electrolytic sensors were selected over a microelectro-mechanical systems-based technology due to their greater stability and precision. Shown in Figure 1, the sensors chosen are dual-axis tilt sensors that operate based on a five-pin configuration housed in an electrolytic fluid-filled capsule. Inside the capsule, an air bubble is enclosed which orients itself perpendicular to gravity. As the sensor tilts, the air bubble moves causing a change in the sensor impedance and the resulting voltage output is directly related to the tilt angle of the sensor.

Lab testing has been conducted using a custom-designed 32-channel data acquisition system developed by Fuchs Consulting, Inc. (Figure 2). The system supplies the power to the tilt sensors, logs the output from the sensors at a specified rate, provides temperature measurements, and offers a computer interface for analyzing data.

In order to test the sensors and data acquisition system in the lab, a test bridge has been designed and constructed on which the sensors can be mounted. Photographs of the bridge can be seen in Figure 3. Both the pier and three girder lines are composed of extruded aluminum parts from 80/20, Inc. The pier has a triangular base with three spring-loaded feet on threaded struts that can be adjusted to tilt the pier in all directions. Testing has been completed in order to calibrate the movement on the threaded struts with respect to the actual tilt of the pier in two dimensions.

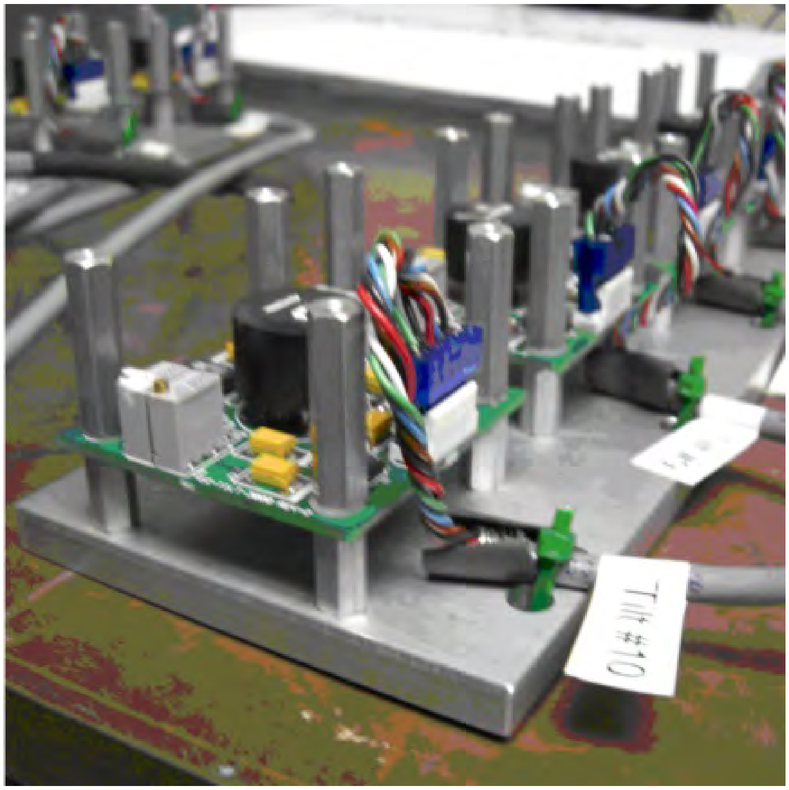


FIGURE 1 Photograph of electrolytic tilt sensors.

FIGURE 2 Photograph of data acquisition system.

(a) (b)

FIGURE 3 Photograph of test bridge (*a*) and sensors on pier and (*b*) superstructure.

SENSOR CHARACTERIZATION

The beginning stages of the project involved characterizing the electrolytic tilt sensors. Testing was completed in order to calibrate the sensors as well as determine the sensor noise, warm-up periods, temperature effects, and resolution. Using the rotary stage shown in Figure 4, calibration of the sensors was completed by rotating each sensor in 23 arcminute (0.38°) increments through its linear range.

The voltage output was recorded using the data acquisition system, and calibration plots were created for each axis of the 10 sensors. R^2 values for these calibration tests were typically greater than 0.99, thus showing the highly linear behavior of the electrolytic tilt sensors.

Along with calibration, tests have been run in order to determine temperature effects, specifically to verify the nominal temperature compensation factor. The temperature compensation factor accounts for the change in calibration factor as the temperature varies. The calibration factors for electrolytic tilt sensors vary with temperature due to the changing viscosity of the fluid within the sensor. In order to verify this factor, a test was completed in which the rotary stage was placed in a temperature chamber. While being kept at a constant temperature, the sensors were rotated through their linear range. This test was repeated at three different temperatures, 20°C, 30°C, and 40°C, in order to determine the calibration factor at each temperature. These values were then compared with the expected calibration factors computed by applying the nominal temperature compensation factor to the original calibration factors for each of the three temperatures. The error between the experimentally collected and the expected values was then computed, with the results ranging from 1% to 9%. Because the error was found to be less than 10% in all cases, it was decided that the nominal temperature compensation factor is acceptable for use in postprocessing.

Other tests have also been run in order to characterize the sensors. Among these were tests to evaluate drift, identify noise levels, and determine experimental resolution of the overall system. The resolution of the system is dependent on several components, including the analog-to-digital converter, the test setup, and the sensors themselves. Based on the laboratory setup, the minimum resolution of the system was found to be no more than 28 arcseconds (0.0077°). Although a smaller resolution may have been possible under different conditions, the resolution achieved is sufficient for this project's application.

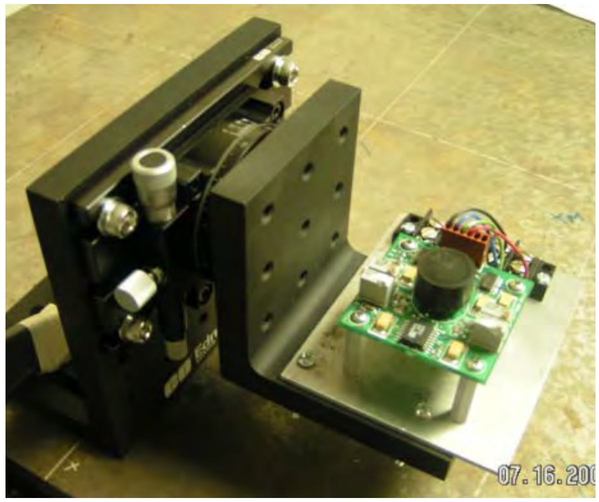


FIGURE 4 Photograph of rotary stage with mounted sensor used for sensor calibration.

ALGORITHM DEVELOPMENT

One aspect of this system that sets it apart from currently available monitoring systems is the use of multiple sensors. The use of multiple sensors allows one to achieve measurement redundancy, increase sign-to-noise ratios, identify inconsistent sensors, and utilize signal correlation. These are main points of interest when it comes to algorithm development. Currently under development are algorithms to achieve the following: initial filtration, calibration, and temperature compensation, movement computation, determination of sensor consistency, sensor correlation, and critical movement identification.

Initial Filtration

To address potential noise sources or error in field measurements, an initial filter has been developed that screens raw data for errant noise spikes that may occur across all data channels. If left untreated, these noise spikes may cause inaccurate movements to be calculated in future postprocessing. The filter is immediately applied to the raw data during postprocessing. The data file from the data acquisition system consists of 32 channels, including two axes of tilt and one temperature output for each of 10 sensors as well as an internal and external temperature reading. The initial filter has been developed to address errant readings that are consistent among all of the channels. As a result, the filter is designed to consider only one of the 32 channels for filtering. Errant data points that occur in individual channels are addressed in algorithms to test sensor consistency within the sensor array, as described in a later section.

The filter was developed by utilizing the “3 Sigma Edit Rule” in which an outlier is that which deviates more than three times the standard deviation from the mean. In order to increase the filter’s ability to detect multiple outliers, thus making it more robust, the Hampel identifier was added (Näsi, 2005). This method replaces the mean with the median, and the standard deviation with the median absolute deviation from the median.

Another important aspect of the filter is the amount of data considered. In order to account for potential varying data, the filter only considers then nearest 48 points when computing the median and median absolute deviation from the median. Thus, the filter is able to account for large variations in the data, such as may occur with diurnal temperature variations, and still accurately detect outliers.

Detected outliers are then replaced in the data set using a median value from nearby points. This method was chosen over replacement with an average or neighboring point in order to account for situations when the data vary greatly or when two consecutive points are deemed outliers.

In order to show the effectiveness of the filter, the following figure contains direct output from the MATLAB initial filter. In this case, approximately 10% of the data are noise points generated from intermittent data collection errors.

Figure 5 shows the results of the filter on a set of data in which the temperature varied daily. As shown with the blue circles, the filter detects the errant points and replaces them with valid points, shown as black x's. The filter has been shown to detect upwards of 99% of the noise spikes. It is designed to error on the side of false positive readings rather than leaving noise spikes undetected. In this way, the noise spikes are eliminated, thus resulting in no inaccurate movement calculations in future postprocessing.

Calibration and Temperature Compensation

Along with the initial filter, algorithms have been developed to convert the raw voltage output of the sensors to angular measurements in degrees. Included in this conversion are the calibration scale factors and the temperature compensation factors.

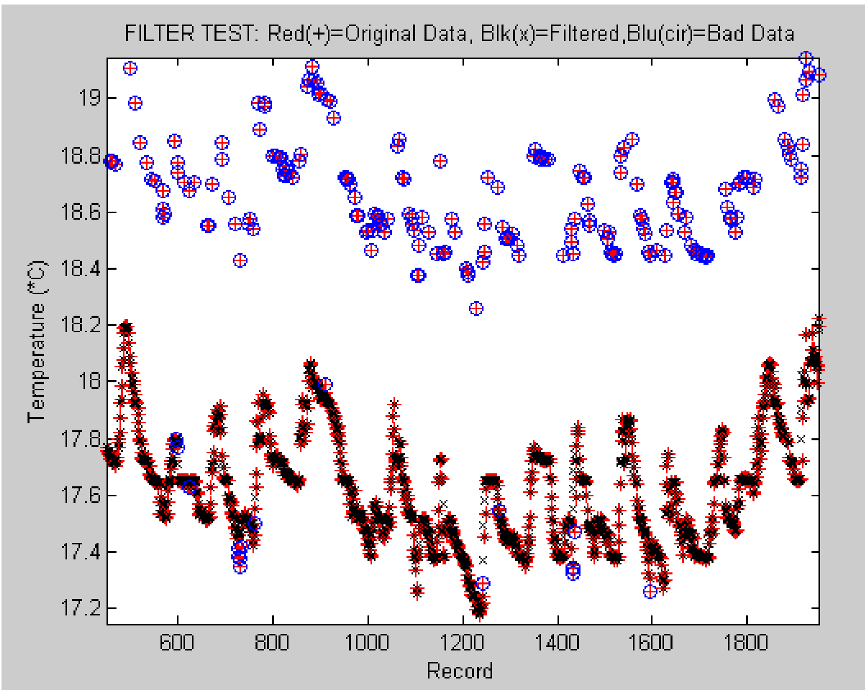


FIGURE 5 Results from initial filter applied to temperature output.

Movement Computation

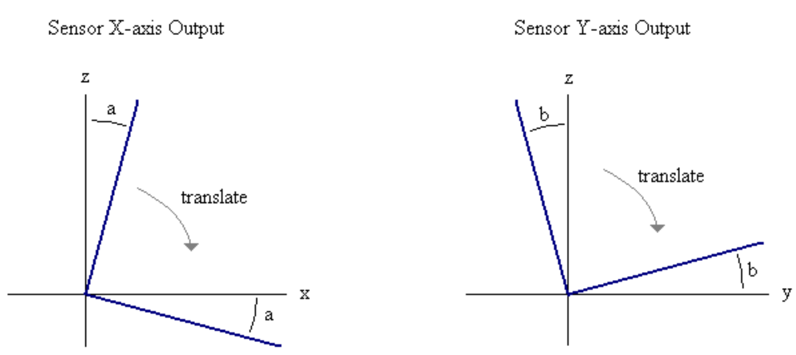
One of the goals of this project is to develop a system that can detect both short-term and long-term movements. In order to do this, a method was developed in which the change in tilt is computed over several different time periods. In order to detect instantaneous movements or movements that occur quickly such as scour events, short time periods are used. On the other hand, in order to monitor long-term gradual movements or tilt that occurs gradually over a period of months due to subsurface erosion or locked bearings, longer time periods are utilized. In this way, the system will be sensitive to both instant and gradual tilts. Algorithms have been completed to assess tilt data over 1 h, 12 h, 24 h, 7 days, 28 days, and 90 days.

In order to determine the magnitude and direction of pier movements, the programmed algorithms compute the change in tilt over a set time integral using vector comparison. The output from both the *x* and *y* axis of each sensor is converted into a unit vector translated to parallel to the sensor's base in the direction of the *x* and *y* axis, respectively. Once in unit vector form, the normal vector to the module base is determined by computing the cross product of the two unit vectors. Figure 6 and Equation 1 show how this is completed.

$$\text{normal vector} = n1 = \begin{bmatrix} i & j & k \\ x_a & y_a & z_a \\ x_b & y_b & z_b \end{bmatrix} \quad (1)$$

This process is repeated for each data point in time. To determine the movement over a set time period, a normal vector from the beginning of the time period, *n*1, is compared to that from the end of the time period, *n*2. The angle between the two normal vectors is computed based on Equation 2 below.

$$\cos\theta = \frac{n1 \bullet n2}{|n1||n2|} \quad (2)$$



a ≡ tilt output for x-axis (degrees)
$x_a = \cos(a)$
$y_a = 0$
$z_a = \sin(a)$

b ≡ tilt output for y-axis (degrees)
$x_b = 0$
$y_b = \cos(b)$
$z_b = \sin(b)$

FIGURE 6 Method for converting sensor output into unit vectors.

Thus, the resulting theta is the magnitude of the change in tilt over that time period in degrees. In addition, the direction of that movement is also computed. The MATLAB file is programmed to compute the change in tilt over each of the time integrals under consideration.

Sensor Consistency

The use of multiple sensors allows for invalid sensor measurements to be identified and excluded from postprocessing. In this way, a sensor can go bad without jeopardizing the entire system, and users can be notified of the sensor failure without having to monitor system performance on a daily basis. This concept differs from currently available technologies, which count on individual sensors to report accurate data at all times. Thus, the algorithms presented in this section were developed to identify and disregard inconsistent sensors.

The system is currently setup such that Sensors 2, 3, and 4 form an array on one side of the pier, while Sensors 5, 6, and 7 form an array on the opposite side as shown in Figures 7 and 8.

FIGURE 7 Photograph of sensor array configuration.

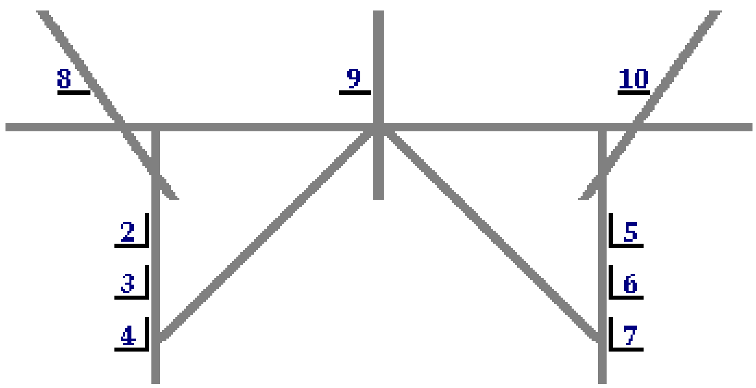


FIGURE 8 Diagram of sensor array configuration.

The sensors are configured in this way so that each side of the pier has three sensors in order to achieve sensor redundancy. Because of this, each set of three sensors can be considered in order to pinpoint an errant sensor. By comparing the outputs from the sensors within a particular array, a dissimilar output can be detected and disregarded. This is the method used to ensure sensor consistency.

The algorithms developed first check for errant readings in each sensor individually. Once each sensor is checked individually, sets of sensors are considered. Within each set, an average movement is determined. Each sensor's tilt movement is then compared with that average. If any movement varies more than a set threshold from that average, a sensor inconsistency exists. The output furthest from the average belongs to the inconsistent sensor. If a sensor is shown to be inconsistent for a certain percentage of the data, then that sensor is disregarded for all future postprocessing.

In the case that a sensor is deemed inconsistent and is disregarded, the program checks the remaining two sensors in the set in order to ensure consistency between them. In order to do this, the difference between the two sensors' movements is compared to a threshold value. If above the threshold, the two sensors are not consistent with one another. If the sensors are found to be inconsistent for a certain percentage of the data, a message is sent to the user stating that the sensors in the set are inconsistent and further review is necessary.

Figure 9 contains direct output from MATLAB in which one sensor in a set is inconsistent.

In the graph in Figure 9, the top three lines are the sensors' zeroed outputs, offset by 3 degrees for viewing purposes. The bottom lines are movements computed over a 1-day period from the sensors' outputs. During this test, the test bridge was tilted three times resulting in the three jumps in the bottom lines. Two ½-degree movements occurred as well as one larger movement. As shown, Sensor 4 unexpectedly began to output irregular data after Record 4000.

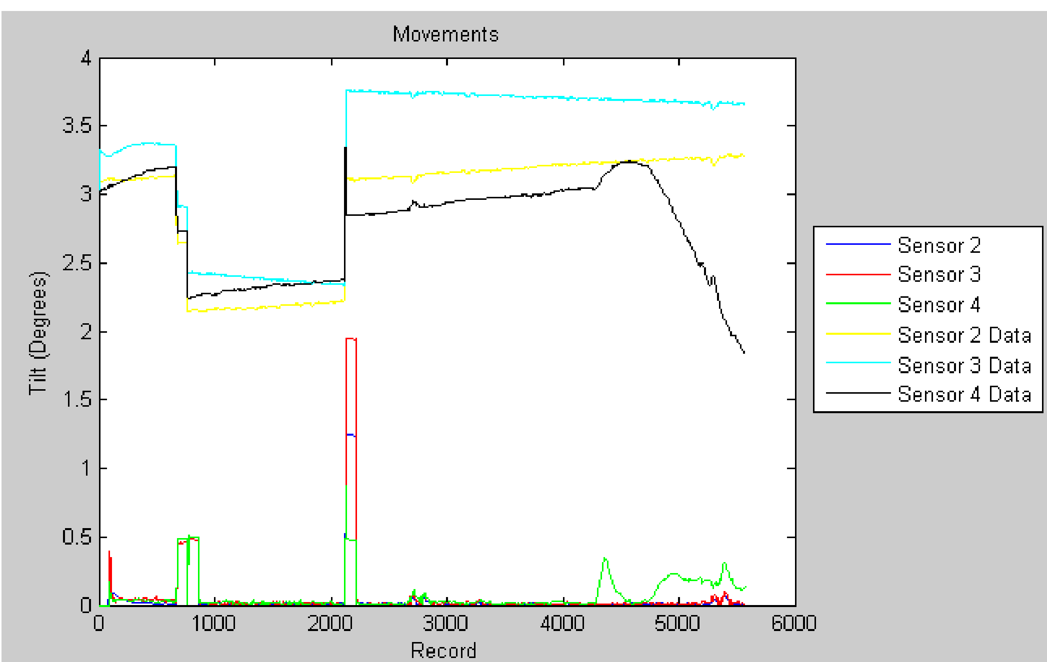


FIGURE 9 Example of data in which Sensor 4 becomes inconsistent.

The algorithms detect that irregularity and a message is given to the user reporting that Sensor 4 is inconsistent for a 1-day period and will be disregarded in further processing. Although the sensor failure is not desired, it is important to know that due to sensor redundancy in the system, the failure of Sensor 4 will not cause a loss in the effectiveness of the system.

Once all inconsistencies are identified, the remaining sensors in the array—those not disregarded, whether that be all three or just two— are averaged in order to determine one value for the movement of the array. This process is completed for each array. Thus, once this portion of the program is completed, one value of tilt movement for each array remains for each time step. These values can then be used for sensor correlation.

Sensor Correlation

While sensor consistency algorithms were developed to identify inconsistent sensors, sensor correlation algorithms were developed to more accurately interpret the movement of the pier. Here, the concept of signal correlation is used both as a check of sensor outputs as well as a tool to more thoroughly model the behavior of the pier. The system is designed with two sensor arrays, one on each side of the pier. Once the movement for each array of sensors is determined through sensor consistency, sensor correlation is used to compare these values and combine them into an overall movement of the bridge pier with respect to a universal coordinate system.

In order to ensure consistency between the sets, a method is used in which the difference between the two sides' outputs is computed and compared to a predetermined threshold. If above the threshold, the sets are deemed inconsistent. If they are deemed inconsistent a certain number of times, the user is alerted that further review is necessary in order to determine what is occurring.

Once the sets are shown to be consistent with one another, they are combined to determine an overall movement of the bridge pier. To do this, the magnitude of the movement from each set is averaged. In addition, the directions from each set are translated to a universal coordinate system and averaged. Thus, an overall direction of the tilt of the pier is recorded for each time step as well as the magnitude of that movement.

Critical Movement Identification

The final aspect of algorithm development which is currently under consideration is determining which movements are critical. The program will record movements computed over six different time integrals, but which movements are not normal movements due to diurnal and seasonal temperature variations? In addition, how can the program distinguish between sensor drift and actual tilt? Work is ongoing towards answering these questions and determining the best method to be applied in algorithms. Fuzzy thresholds will likely be used in order to detect these critical movements. Model data are being constructed in order to test the program's ability to distinguish between long-term drift and gradual tilt movements.

SYSTEM VERIFICATION AND IMPLEMENTATION

Upon completion of algorithm development, the system will be tested on the test bridge in order to verify the program's ability to successfully detect and alert the user of critical movements. In addition, model data will be created in order to further test the programs effectiveness with long-term gradual movements. Once verified, a field system will be developed, including the enclosures, software and installation scheme for the sensors. This field system will be implemented on a bridge in New York State for final testing.

CONCLUSION

Scour and other natural hazards have the potential to undermine the stability of bridge piers. The goal of this project is to develop and implement a system of tilt sensors capable of monitoring the long-term motion of bridge piers. Initial tests were completed in order to determine sensor characteristics such as calibration factors, temperature compensation factors, resolution, and drift effects. Algorithm development included utilizing multisensor reasoning in order to program initial filtration, calibration and temperature compensation, movement computation, determination of sensor consistency, sensor correlation, and critical movement identification. Within algorithm development, an initial filter was developed in order to eliminate noise spikes that, if left untreated, would result in inaccurate movement computations. Calibration and temperature compensation were completed in order to convert the voltage output into the more user-friendly units of degrees. Movements were computed over several different time integrals in the movement computation algorithms so the system can detect instantaneous events such as scour, as well as long-term gradual movements due to subsurface erosion or locked bearings. Because of sensor redundancy in the system, sensor consistency algorithms were developed in order to identify and disregard failed sensors without inhibiting the effectiveness of the system. In addition, the placement of sensors in arrays allows for a more accurate model of the pier movement to be determined through sensor correlation. Finally, critical movement identification algorithms are under development in order to pinpoint the movements that require notification. Once completed, the result will be a system capable of long-term monitoring of bridge piers. It will be capable of identification and notification of pier rotations and displacements that may lead to structural instability. The system will act as a tool for long-term asset management, allowing owners to monitor remotely the behavior of a bridge over many years.

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Technical Audits of Rail Infrastructure

Description of Existing Infrastructure and Evaluation of Past Performance

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As rail infrastructure networks deteriorate and the demands upon them change, it is important for decision makers to have a sound objective overview of their rail network and the organization that takes care of it, to ensure that it is being, and will be, optimally managed. This objective view can be obtained by conducting a technical audit. The components to be included in a technical audit, with respect to the description of existing infrastructure and the evaluation of past performance, and a proposal of how they are to be structured, are presented.

As an infrastructure network deteriorates and the demands upon it change, it is important for the people responsible for it to have a sound objective view of how it, and the organization that takes care of it, have functioned in the past, as well as the reasons why, and to have plausible scenarios of how both will function in the future. This objective view can be obtained by conducting an audit, of which some examples are the infrastructure audit in New Zealand for the Ministry of Economic Development conducted by Price Waterhousecooper (*1*); the audit of the French National Railway Network conducted by the Swiss Federal Institute of Technology in Lausanne (*2*); and the annual report to Congress on the status of the Nation's Highways, Bridges, and Transit conducted by the U.S. Department of Transportation (USDOT) (*3*).

There are many different components that can be included in an audit. The ones that should be included depend on the specific objectives of the audit. This article confines itself to technical audits, defined to have the following general objectives:

- Describe existing infrastructure,
- Evaluate past performance,
- Evaluate the organization responsible for the infrastructure, and
- Predict future performance of rail networks.

The components that should be used with respect to the first two of these four objectives, assuming that all four objectives are to be met, are discussed. The components of the audit used to describe existing infrastructure consist of extent, location, value, and condition. The components used to evaluate past performance consist of identification of a base case (i.e., the expected use of the infrastructure) and additional costs (i.e., the differences between expected and actual use). For each component, the reason for its inclusion is stated, along with the requirements of the component for its inclusion in an audit and a statement of some of the potential problems.

DESCRIPTION OF EXISTING INFRASTRUCTURE

Extent

Reason

The identification of all infrastructure objects to be included in the audit and the determination of their extent, is required as it is the basis for the audit (Table 1). Without the identification of the infrastructure objects and their extent the audit cannot be conducted. Extent can be measured in number of objects and in terms of other units more representative of the significance of the infrastructure objects. An example for bridges is shown in Table 2 using the number and surface area of the bridges. In this example it can be seen that there are significantly more masonry rail bridges than any other type of bridge in terms of number and surface area.

TABLE 1 Example Tabular Display of the Extent of Bridge Infrastructure

No.	Function	Principal Construction Material	No.	Deck Surface Area (m^2)
1	Rail bridge	Concrete	33	1,215
2		Masonry	287	42,121
3		Metal	37	5,188
4		Composite	85	3,423
5	Road bridge	Concrete	25	6,386
6		Masonry	85	4,548
7		Metal	8	281
8		Composite	16	796
Total			576	63,958

TABLE 2 Example Tabular Display of the Replacement Value of Bridge Iinfrastructure

No.	Function	Principal Construction Material	No.	Deck Surface Area (m^2)	Value (mu)
1	Rail bridge	Concrete	33	1,215	6,078
2		Masonry	287	42,121	210,608
3		Metal	37	5,188	25,940
4		Composite	85	3,423	17,115
5	Road bridge	Concrete	25	6,386	31,919
6		Masonry	85	4,548	22,738
7		Metal	8	281	1,405
8		Composite	16	796	3,981
Total			576	63,958	319,784

Requirements

As it is rarely reasonable to take into consideration all of the infrastructure objects individually, an infrastructure model must be constructed that allows the classification of the infrastructure as normally required to simplify analysis and display. The model used depends on the specific objectives of the audit, the accuracy and precision of the results required from the audit with respect to these objectives, and the time period of investigation. It is useful to use the same model throughout the audit. In many cases, organizations that own a large amount of infrastructure have their own models. Some good examples for bridges can be found in highway organizations that have developed asset management systems such as the Swiss Federal Roads Office (*4*) and many of the states in the United States (*5*). Existing infrastructure models must, however, be checked for their suitability for use in an audit.

An infrastructure model is not suitable if it cannot be used to provide sufficiently accurate future predictions. For example, if under the previously defined general objective to predict future performance, one of the specific objectives is to estimate the risk[1] associated with each network link, an infrastructure model that does not take into consideration the relationships between objects cannot be used if it cannot be assumed that that the probability of simultaneous inadequate performing infrastructure objects is negligible (*6*).

When this assumption cannot be made, the relationships between the multiple infrastructure objects must be taken into consideration. One example is the collapse of a retaining wall. If the proximity of the retaining wall to the rails is not taken into consideration, then it is not possible to estimate the consequences of the collapse of the retaining wall and therefore not possible to estimate risk. Another example is the failure of a bridge. Assuming that a bridge failure results in link closure, it is not possible to estimate the consequences of the bridge failure without taking into consideration the possibility of deviating trains on other links, requiring consideration of the spare capacity of the other links in terms of trains per day, the ability of the bridges on the other links to carry the trains, the size of the tunnels to allow the passage of these trains, the type of power supply, and the ability of the signalization equipment to deal with the additional trains, and therefore not possible to estimate risk.

An infrastructure model is not suitable if it is will not yield sufficiently precise future predictions. For example, if under the previously defined general objective to predict future performance, one of the specific objectives is to estimate future intervention costs, all objects classified as one type should be considered to have the same intervention costs per intervention type, deterioration speeds, and intervention effectiveness, with respect to the restoration of the infrastructure; if there are substantially different intervention costs for concrete bridges that are shorter than 4 m than those that are equal to or greater than 4 m, the length of the concrete bridges should be included in the model of the structures infrastructure.

An infrastructure model is not be suitable if it does not allow the achievement of the audit objectives and goals within the time period of investigation; if under the previously defined general objective to predict future performance, one of the specific objectives of the audit is to estimate future intervention costs, it may be possible to achieve very accurate cost estimates for concrete bridges that are equal to or over 20 m in length if the number of columns, abutments, and deck slabs as well as their sizes are known, rather than simply lumping them all together as concrete bridges over 20 m in length. If this data, however, is not readily available, then such data may take years of work to collect and therefore a time period of investigation of a few months would render such a model unsuitable.

A sample of an example model of infrastructure for a railway network where all predictions can be made neglecting the relationships between infrastructure objects is shown in Figure 1. In this example a railway network is divided into four broad groups based on function: (*a*) rails, (*b*) structures, (*c*) signalization equipment, and (*d*) power supply equipment. The structures infrastructure is divided into four general object types based on object function: (*a*) bridges, (*b*) tunnels, (*c*) retaining walls, and (*d*) soil structures. Bridges are divided into groups based on their principal material of construction, and concrete bridges are divided into groups depending on their length. The most detail that can achieved with the model shown in Figure 1 is to be able to make statements with respect to level 4, e.g., there are 10,000 m^2 of concrete bridge structures equal to or greater than 4 m in length.

Potential Problems

Some of the potential problems with the classification of infrastructure are

- Incomplete classification, i.e., not all infrastructure can be included,
- Non-orthogonal classification,
- Inadequate detail with respect to the specific objective to be achieved, and
- Inadequate consideration of interdependencies and correlations of infrastructure objects with respect to the specific objective to be achieved.

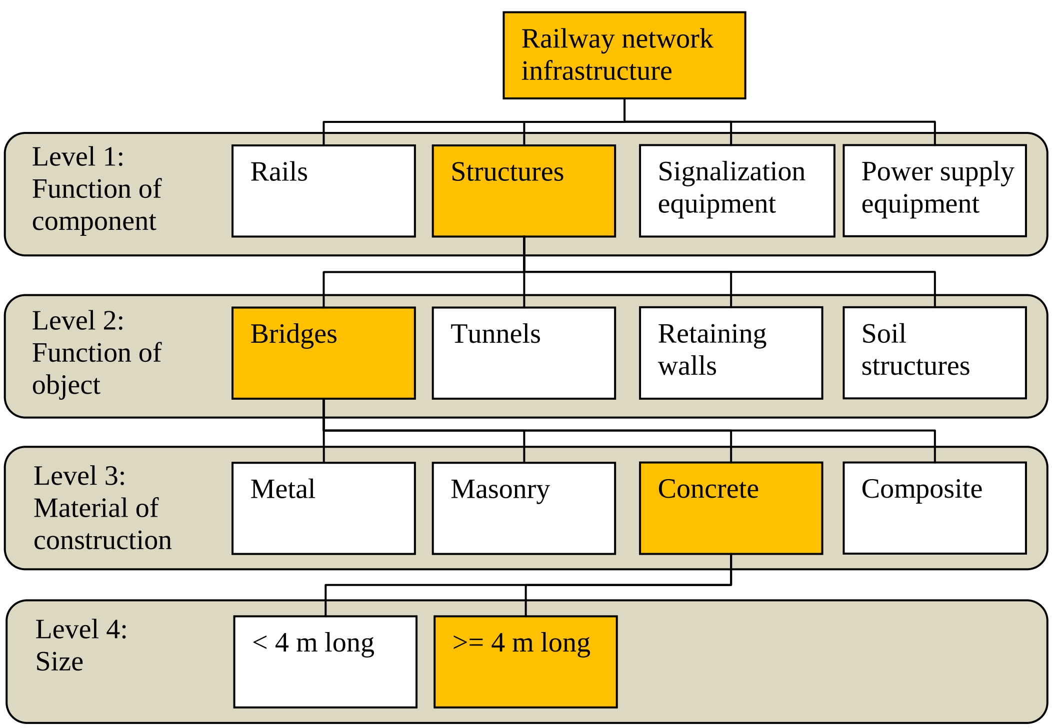

FIGURE 1 Example classification.

Location

Reason

The determination of infrastructure location is often necessary in order (*a*) to have an overview of the role of the infrastructure in the network, and (*b*) to group infrastructure according to properties that can be expected to vary based on geographic location. An overview of the role of the infrastructure is useful in developing future development scenarios, e.g., with respect to which lines to discontinue, or which lines need to have capacity increased. The grouping of infrastructure according to properties that can be expected to vary based on geographic locations, often increases the accuracy of future predictions. The determination of exactly which characteristics to consider depends on the specific objectives of the audit. Examples of properties that can be expected to vary based on geographic locations follow.

- The speed of gradual deterioration of infrastructure. For example, since reinforced concrete bridges deteriorate faster in areas where they are exposed to more wetting and drying cycles with chloride-laden water than those that are not, concrete bridges within a few kilometers of the ocean normally experience faster deterioration than those far inland.
- The likelihood of occurrence of sudden deterioration due to events, such as natural hazards. For example, bridges near a fault line will have a much higher probability of inadequate performance due to an earthquake than those that are far away from fault lines.
- The ability to access the infrastructure to perform interventions. For example, bridges located in steep valleys with no access road will have significantly higher costs of intervention than those that are easily accessible.
- The ability to group interventions when developing work programs. For example, interventions including the replacement of a waterproofing layer on bridges can be performed less expensively when they are performed at the same time as track replacement, i.e., when the traffic is stopped and the rail removed.

An example display of infrastructure location is shown in Figure 2, using a classification that results in four bridge types. The infrastructure is shown in terms of surface area per line segment. The segment widths are varied as a function of the total surface area of all infrastructure types that they contain, e.g., the thickest line segments mean that there is over 40,000 m^2 of bridges on the line segment. The surface area of the infrastructure per type is given as a pie chart per segment, e.g., on the upper left most line segment there are 12,000 m^2 of concrete bridges, 30,000 m^2 of masonry bridges, 3,000 m^2 of composite bridges, and 15,000 m^2 of metal bridges.

Requirements

The determination of the location of infrastructure requires the identification of where the infrastructure to be included in the audit is located. As it is rarely reasonable or helpful to list the exact x, y, and z coordinates of all infrastructure objects, infrastructure is often grouped by zone or line segment, or using linear referencing along a segment.

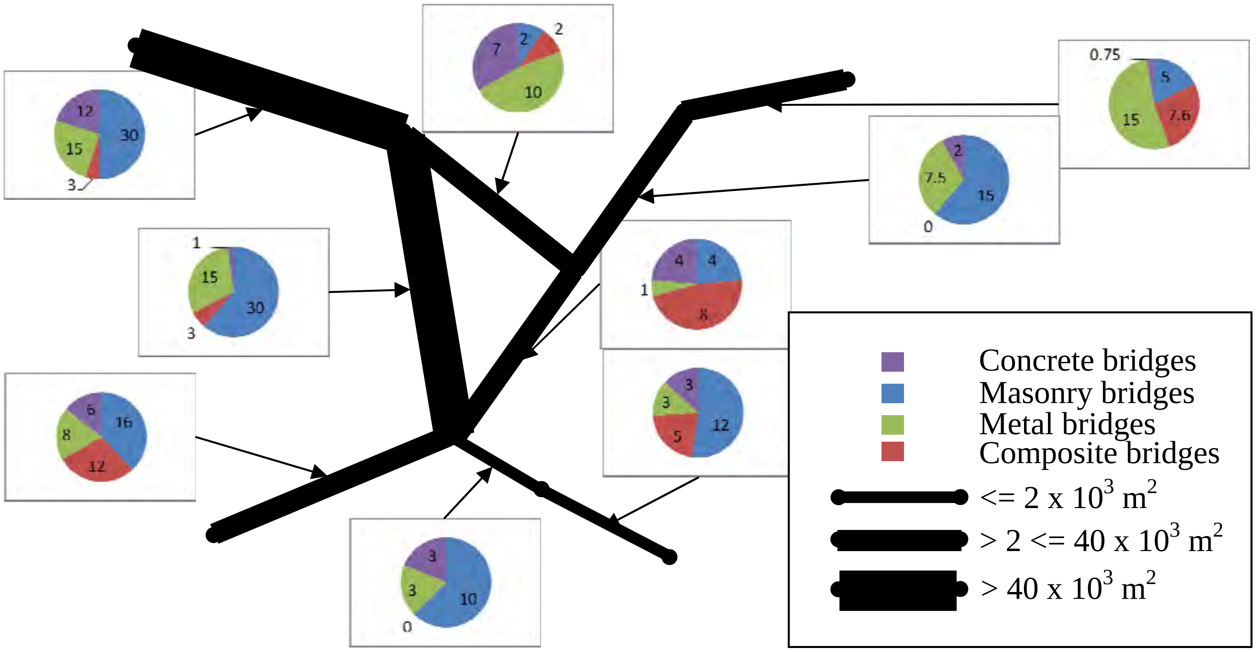

FIGURE 2 Example graphical display of the extent and location of bridge infrastructure.

Potential Problems

The most significant problem with the determination of infrastructure location is the availability of data. If exact location data already exist in database systems then it can be determined for an audit quickly, precisely and accurately. If it is not then the location of the infrastructure must be identified through more approximate methods, such as linear referencing.

Value

Reasons

The determination of infrastructure value is necessary so that the significance of infrastructure objects can be compared. For example, without the use of value, it is difficult to ascertain the significance of having *x* kilometers of track and *y* structures. If value is estimated, it is possible to say that there is *z* monetary units worth of track and 1.5*z* monetary units worth of structures. This makes a high-level comparison direct and understandable. It is important, however, to realize that this does not translate directly into maintenance costs, since intervention strategies and, therefore, costs can be significantly different. For example, it may cost 1% of the replacement value per year on average to maintain a group of bridges but it may cost 5% of the replacement value per year to maintain the rails.

Requirements

There are different ways to measure value (*7*) and the relevancy of each depends on the objectives of the audit. Two of the most common ways are (*a*) the current replacement value, and (*b*) the use value.

The current replacement value is the cost of replacing an infrastructure object today, either with or without the demolition costs of the old infrastructure, although with demolition costs is normally preferable. The estimation of current replacement value requires the consideration of all costs associated with the replacement of the infrastructure object, including installation costs, traffic deviation costs, labor costs, and material costs.

The use value is the actual value for the user. The estimation of use value requires the estimation of the benefits of the users because of the existence of the infrastructure object and what the user would do if the infrastructure object could no longer be used. Use value can be used if the inadequate performance of infrastructure objects can be considered to be mutually exclusive, i.e., when the probability of simultaneous inadequate performance is negligible. This assumption requires two supporting assumptions, namely that there is a negligible probability of simultaneous inadequate performance of infrastructure objects due to loads exceeding resistance and it can be assumed that once inadequate performance of an infrastructure object occurs that it can and will be repaired before the inadequate performance of another infrastructure object happens. It is only possible to make these assumptions for infrastructure networks in a limited number of cases. If these assumptions are not possible the use of use value will give erroneous results. For example, if the use value of traveling on a railway link during a specified time frame is x, and there is only one bridge in the railway link then the use value of the bridge is x. If, however, there are two bridges in the link then the use value of each of the bridges would be x and therefore the use value of the bridges on the link would have a value of $2x$. The use value would be double counted.

An example of bridge value, in tabular form, for bridge infrastructure is shown in Table 2. It can be seen that masonry rail bridges comprise approximately two thirds of the bridge infrastructure in terms of replacement value. The use of value becomes particularly useful when fundamentally different infrastructure objects are to be compared, e.g., track and bridges.

Potential Problems

A potential problem that occurs in the estimation of the value of infrastructure is the use of market values when they do not exist, i.e., treating infrastructure objects as if they can be sold, when they cannot, as in the case where it is owned by a government that has the responsibility to provide a certain level of service to the public for the least amount of money. For example, if it is assumed that a bridge has a certain value at construction and that this value decreases with time due to depreciation, as is done with many accounting methods, there can be the perverse situation where a bridge has no value but functions completely well and is used daily by hundreds of trains. When such a case occurs there is no way that this value is representative of anything meaningful to infrastructure managers.

Condition

Reason

The assessment of the physical condition of the infrastructure is normally required to provide a basis from which to predict future costs, and to evaluate past agency costs. In this section the assessment of the physical condition of the infrastructure to provide a basis from which to predict future costs is discussed. The assessment of physical condition to evaluate past agency costs is discussed in the section entitled past performance.

Requirements

In analyzing the physical condition of infrastructure it is necessary to take into consideration both the physical condition of the infrastructure and the deterioration processes that caused the physical condition. It is often beneficial to group the deterioration processes into two categories, deterioration processes that result in

- Infrastructure that deteriorates gradually, i.e., infrastructure where scheduled inspections can determine the advancement of the deterioration of the physical condition of the infrastructure through predefined condition states and there is sufficient time to perform an intervention before the infrastructure provides an inadequate level of service once it is in a specified condition state.
- Infrastructure that deteriorates suddenly, i.e., infrastructure where scheduled inspections cannot determine the advancement of the deterioration of the physical condition of the infrastructure through predefined condition states or there is not sufficient time to perform an intervention before the infrastructure provides an inadequate level of service once it is in a specified condition state.

For infrastructure that appears to be deteriorating gradually, condition states can be used to rate the condition of the object or its components, for example scales such as 1 to 5 or “failed” to “excellent.” Condition states can be defined qualitatively or quantitatively or using a combination of both. Depending on the classification of the infrastructure, it may be possible to use combined or weighted condition states which attempt to assess the condition of an entire infrastructure object based on the condition assessment of its elements, e.g., if a bridge is composed of a deck in condition state 3, and two abutments in condition state 2, and all elements have an equal weight, than the bridge may have a condition state of 2.33 [(3 + 2 + 2)/3].

For infrastructure that appears to be deteriorating suddenly, age and operating conditions can be used to assess physical condition, for example scales such as 1 to 5 or “beginning of service life” to “end of service life.” In order to compare the physical condition of infrastructure that appears to be deteriorating gradually and infrastructure that appears to be deteriorating suddenly it is often advantageous to build condition states from the attributes used to assess the physical condition of the latter. For example, if a piece of signalization equipment is 17 years old and its life expectancy with only routine maintenance is expected to be 38 years old, the equipment may be considered to be in condition state 3, assuming the definition of the condition states are condition state 1 is from 1 to 7 years old, condition state 2 is 8 to 15 years old, condition state 3 is 16 to 23 years old, condition state 4 is 24 to 31 years old, and condition state 5 is 31 to 38 years old.

An example display of the condition of four infrastructure types can be seen in Figure 3. In this example, condition is ranked in five categories of increasing physical damage using the average condition of the infrastructures weighted with respect to their value. The average condition of each infrastructure per type is given as a pie chart per segment. It can be seen in this example that the bridges of the top-left segment with more than 40,000 m^2 of bridges (Figure 2), are in poor condition, and the concrete bridges are in the worst condition (4.8). The masonry bridges are also in a poor condition (4.2) albeit not as poor as the concrete bridges. The metal bridges are in an acceptable condition (3.5), and the composite bridges are in a good condition.

Potential Problems

One of the potential problems associated with the use of condition as a performance indicator is related to how condition states are defined. Condition states are best defined in reference to physical condition alone. It is then easy to establish guidelines as to what condition the infrastructure object is in. When other criteria are used, such as time to intervention, judgment is required with respect to the action that needs to be taken in the future and when they should be made, which essentially requires the assessment of how fast the infrastructure will deteriorate and how much the interventions will cost and that it is clear what the optimal intervention strategy is taking into consideration the maintenance policies of the organization. It is considerably clearer, and therefore less prone to error, to define condition state purely on physical condition. The other factors can be introduced later in the audit.

Another potential problem is the inconsistency of the definition of condition states across infrastructure object types. For example, if catenaries are divided into three condition states and

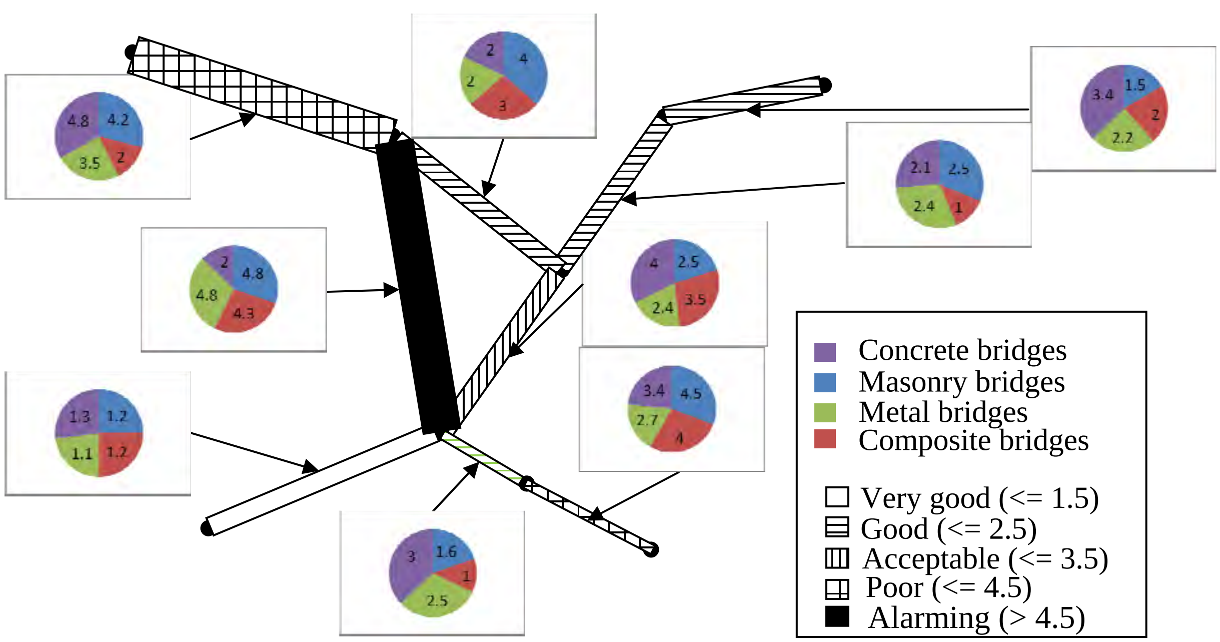

FIGURE 3 Example graphical display of condition of bridge infrastructure.

bridges are divided into five condition states it is difficult to compare between them. Generic definitions that are the same for the condition states of all infrastructure objects should be used.

Detailed definitions of the condition states should, however, be generated for each object type so that it is clear what is meant and so that the results are reproducible.

PAST PERFORMANCE

Base Case: Expected Infrastructure Use

Reason

The determination of infrastructure use is necessary for the assessment of past infrastructure performance. For example, a comparison between the speed at which trains were expected to travel over specified line segments, and the speeds at which they actually traveled, can be an indicator of inadequate infrastructure performance, and can allow estimation of the severity of the inadequate performance.

Requirements

In order to determine, summarize, and display infrastructure use, it is often necessary to rely on use indicators. Some example use indicators are

- The number of trains that were supposed to travel on each line segment,
- The number of passengers that were supposed to be able to be carried on each line segment, and
- The number of tons of goods that were supposed to be able to be carried over each link segment.

The deviations from these expected values can be used as indicators of the inadequate performance of infrastructure. The simple statistics are, however, often not enough to definitively state if there was inadequate infrastructure performance. The statistics often need to be coupled with an explanation of the problem source, e.g., train speed reduced from 120 km/h to 40 km/h due to uncertainty of bridge load carrying capacity.

An example display of four infrastructure use indicators can be seen in Figure 4. In this example, the four use indicators are:

- Train speed, i.e., trains are expected to travel at *w* km/h along the line segment (the line segments are coded to illustrate the expected speeds on each line segment);
- Number of trains, i.e., the line segment was expected to carry *x* trains per day;
- Number of passengers, i.e., the line segment was expected to have the capacity to carry *y* passengers per day; and
- Number of tons of goods, i.e., the line segment is expected to have the capacity to carry *z* tons per day.

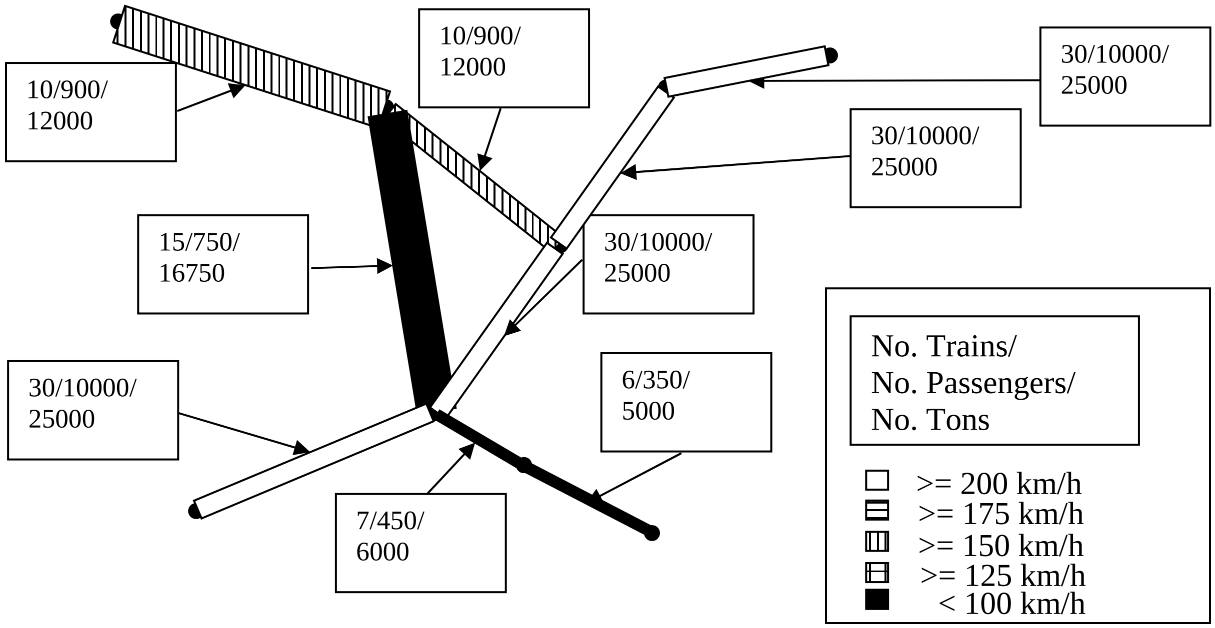

FIGURE 4 Example graphical display of infrastructure use, including expected speed, number of trains, number of passengers, and tons of goods per day.

Numerous use indicators can be combined if it will be of use in the identification of infrastructure problems, for example, tons of goods, and train speed, e.g., the line segment is expected to support a train carrying *x* tons of goods traveling at *y* km/h.

Potential Problems

A potential problem with determining infrastructure use indicators is to capture everything that the infrastructure is currently being used for, to summarize it in a meaningful manner, and to structure the indicators in the best way to help identify past infrastructure performance problems.

Another potential problem is the consideration of changing use in the past, e.g., for the most recent 5-year period the expected speed of trains on a specific link might have been 120 km/h where as prior to this period the expected speed may only have been 100 km/h. These differences should be clearly displayed as they may cause confusion in the analysis of past performance indicators.

Additional Costs: Differences Between Expected and Actual Use

Reason

The determination of the costs, i.e., the differences between expected and actual use, is necessary for the assessment of past performance.

Requirements

The evaluation of past infrastructure performance depends on the past costs of infrastructure use with respect to a base case, i.e., the expected infrastructure use, explained in the previous section. In order to effectively complete this part of an audit it is often necessary to rely on indicators of past additional costs, i.e., costs that were incurred in addition to those that were expect to have been incurred. All additional costs can be grouped as shown in Table 3, which includes the group of people considered to bear the costs and the type of cost that may have been incurred due to the inadequate performance of the infrastructure, as well as indicators that can be used to represent each of the costs and the base values to which they are normally compared, in case there is insufficient time or data to conduct a detailed past cost analysis.

Agency costs can be grouped into those resulting from a change in the behavior of people, i.e., labor costs, and changes resulting from a change in the environment, i.e., material costs. As it is difficult to separate agency labor and material costs, they are normally grouped together and treated as one.

In order to determine if the infrastructure has been costing the agency more than it should have, infrastructure condition coupled with past expenditures can be used as an indicator. There is a minimal cost and condition evolution of infrastructure, i.e., there is a minimum amount of money that must be spent to ensure the adequate performance of infrastructure and there is a

TABLE 3 Additional Cost Types

Cost Bearer	**Type of Cost**	**Indicator of Additional Costs**[b]	**Base Value of Indicators**[b]
Agency	Labor and material costs[a]	Expenditures	Expected expenditures (to achieve past condition evolution)
User	Labor and material costs[a]	Expenditures	Expected expenditures
	Negative physical/mental impact	Comfort levels	Expected comfort levels
		Number and type of injuries[c]	Expected number and type of injuries Normally assumed to be 0
		Number of deaths[c]	Expected number of deaths Normally assumed to be 0
	Negative economic impact	Number and amount of delays	Expected number and amount of delays
Public	Negative economic impact	Number and type of injuries[c]	Expected number and type of injuries Normally assumed to be 0
		Number of deaths[c]	Expected number of deaths Normally assumed to be 0
	Negative environmental impact	Amount of material consumption[a]	Expected amount of material consumption
		Noise levels	Expected noise levels
		Pollution levels	Expected pollution levels

[a] Use of materials has a cost both to the agency or the user and the general public.
[b] All costs and indicators of costs are past values.
[c] Injuries and deaths are considered to have a cost both to the user of the infrastructure and the general public.

certain condition of infrastructure that accompanies this expenditure. Deviation from this minimal expenditure is an indication of inadequate infrastructure performance or inadequate business processes related to the maintenance of the infrastructure.

Infrastructure condition or past expenditure alone, except in extreme cases, provides little information with respect to past infrastructure performance, since neither infrastructure condition nor expenditures are expected to be constant from construction until the end of use.

Users have costs due to the inadequate performance of infrastructure that can be grouped as

- The labor and material costs used to maintain vehicles and to repair vehicles following an accident;
- The negative physical or mental impact of inadequate infrastructure performance, including discomfort, injuries, and deaths; and
- The negative economic impact on the user, including lost time, lost business, and destroyed goods.

An indicator of labor and material costs are past expenditures. Indicators of negative physical and mental discomfort are past comfort levels, numbers of injuries, and numbers of deaths. An indicator of negative economic impact is delays and number of derailments.

The general public has costs due to the negative impact on the economy, e.g., due to the lost economic output due to injured persons, and due to the negative impact on the environment, e.g., the reduction of negative noise, air, water, and soil impacts. Indicators of negative economic impact are the number of injuries and deaths. Indicators of the negative environmental impact are past material consumption, noise levels, and pollution levels.

The evaluation of past performance often requires the combination of the performance indicators. The combination of the performance indicators can be done using weighting factors based on the preference of the infrastructure managers, e.g., *x* derailments equal *y* speed limitations which equals *z* noise levels.

One way to ensure good weighting factors is to translate the performance measure into monetary units. Another way is to use utility theory (*8*). It is also possible to use more complex weighting factors, which can be developed through the techniques such as the Analytical Hierarchy Process (*9*).

Potential Problems

The use of technical serviceability indicators as performance indicators is a problem because without a direct correlation to the additional costs incurred due to inadequate performance it is difficult if not impossible to compare results between infrastructure object types. For example, it is not possible to directly compare a deflection of *x* mm on five of 100 bridges with the deviations in rail geometry of *y* mm per kilometer of track, unless the limits are translated into a common denominator, e.g., numbers of derailments or speed limitations. Indicators should be chosen to reflect costs.

The combination of performance criteria is a problem as it can lead to the misinterpretation of the results by decision makers.

SUMMARY

There are many different components to be included in an audit and the exact components to be included depend on the specific objectives of the audit. This article confined itself to technical audits, defined to have the following general objectives:

- To describe existing infrastructure,
- To evaluate past performance,
- To evaluate the organization responsible for the infrastructure, and
- To predict future performance of rail networks.

The components that should be used with respect to the first two of these four objectives, assuming that all four objectives are to be met, were discussed. The components of the audit used to describe existing infrastructure consist of extent, location, value, and condition. The components used to evaluate past performance consist of identification of infrastructure use and evaluation of performance. For each component, the reason for its inclusion was stated, along with the requirements of the component for its inclusion in an audit and a statement of some of the potential problems.

NOTE

1. Risk is defined as the probability of occurrence of inadequate infrastructure performance multiplied by the consequences of inadequate infrastructure performance.

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Benefits and Overview of Instrumentation-Based Structural Health Monitoring for Bridge Management

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Structural instrumentation-based monitoring is a useful tool for health monitoring, inspection, maintenance, and safety assurance. Several illustrative case studies are presented. Real-time, in-service performance measurements provide invaluable information to make inspections, schedule maintenance, ensure structural integrity, evaluate new materials and structural systems, and validate analytical evaluations. A properly designed monitoring system can provide real-time performance data that can be used to build a historical record as part of an inspection program. For a complex structure, performance information may be a more useful and cost-effective inspection tool than visual observations. As an example, CTLGroup designed and installed a monitoring system on the Jindo Grand Bridge in Korea to measure long-term performance, including static force distribution among stay cables. Dial-out capabilities can be incorporated into a system to provide immediate alerts if measured data exceed predetermined thresholds. Such systems can used to insure structural safety and integrity under normal loads and extreme events. The Federal Reserve Bank in Boston, Massachusetts, was instrumented and monitored during construction of the central artery tunnel to provide immediate alerts of detrimental effects and assure structural safety and continuity of operations. A video-based system is being developed to alert bridge owners of barge impacts and help identify offending barges. Similar systems have been implemented for bridge impacts due to traffic. Measured performance data can also be used to reduce maintenance costs. CTLGroup developed and installed a system to monitor the performance of four large-capacity dampers in the movable roof structure of Safeco Field in Seattle, Washington. Damper maintenance intervals can be determined based on performance data, rather than the supplier's recommendations, saving significant effort and money. Structural monitoring is often used to evaluate the in-service performance of new materials, including fiber-reinforced polymers. A monitoring system can be used to measure changes in behavior over time under normal service as well as periodic load tests. Load tests can be monitored remotely, and performed with minimal disruption to traffic. Structural monitoring can also be a useful tool for the validation of analytical studies. Various sensor configurations provide an understanding of a structure's response to various loading, including thermal and environmental loadings.

Estimating the Remaining Life of the George Washington Bridge Wire Rope Suspenders Using Destructive and Nondestructive Test Data

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The George Washington Bridge (GWB) is a two-level suspension bridge that crosses the Hudson River between upper Manhattan, New York, and Fort Lee, New Jersey, and forms part of Interstate 95. In 2006, approximately 162 million vehicles crossed the bridge. The GWB has a total of 592 wire rope suspenders that were installed in 1931 and have been in service for more than 75 years. Each suspender consists of a pair of wire ropes looped around the main cable. The configuration of the wire ropes consists of a total of 283 wires in a formation of six outer strands around an independent wire rope center. The wire ropes transfer the bridge loads from the main floor beams to four main suspension cables. Each wire rope loop is connected to the main floor beam with two-button sockets. Each floor beam is supported by two pairs of suspenders, or eight wire ropes at each end. In the late 1990s, a specially developed nondestructive testing (NDT) technique or magnetostrictive sensor, a long-range guided waves technique, was performed on several (21 suspenders) in-service wire ropes. Some of those wire ropes were then removed for destructive testing (ultimate breaking strength and fatigue testing) and detailed visual inspection. In 2005, the validated NDT technique was performed on a total of 283 rope legs that included in-service suspenders tested in the late 1990s. Based on the results from all the studies, the remaining life factor of safety and deterioration rates of the suspender ropes were developed. Using this information, a capital plan was developed to replace all suspenders. Disruption to traffic during construction was a major factor that influenced the capital plan and the strategy developed for the suspender replacement. This presentation describes the procedures followed to determine the remaining life and deterioration rates of the suspenders. Engineers and managers of long-span suspension bridges will find the discussions and results presented very useful.

Accelerated Construction, Fiber-Reinforced Polymers, and Corrosion Evaluation

National Perspective on Accelerated Bridge Construction

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The United State's 4-million mile highway system is considered the most extensive and heavily traveled highway network in the world. Perhaps no other public asset is as central to the national economy and the daily lives of Americans. The unprecedented increases in traffic volumes and increased funding levels coupled with aging infrastructure have caused highway construction activities to intensify in recent years. Although highway construction is unavoidable, its excessive construction time should be avoided. It is costly and it can expose workers to traffic hazard and the motorists to substandard conditions longer than necessary. It affects the safety and welfare of the construction workforce and traveling public as well. Transportation agencies are committed to offering motorists high-quality, longer-lasting highways and bridges while reducing construction time. Front-burner priorities are to stem the annual loss of more than 40,000 lives to accidents and reduce the $63 billion generally attributed to congestion. They operate, however, against a backdrop of challenges that include intensified construction activities needed to restore a system built largely in the 1950s and 1960s; capacity that has increased little in the past two decades; and growing communities and increasing traffic volumes. Accelerated bridge construction can help minimize traffic delays and community disruptions by reducing onsite construction time and by improving quality, traffic control, and safety. The following topics will be covered in this presentation: (*a*) what is accelerated bridge construction; (*b*) why now; (*c*) its advantages; (*d*) innovative use of prefabricated systems; and (*e*) U.S. prefabricated bridge success stories. The attendees will learn about the state-of-the practice information relative to means, methods, details, and criteria that have been used and proven successful to reduce the negative effects of bridge construction on the mobility of the traveling public, environmental impact, and improve product quality and increase public safety.

ACCELERATED CONSTRUCTION, FIBER-REINFORCED POLYMERS, AND CORROSION EVALUATION

Carbon Fiber-Reinforced Polymer Reinforcing to Extend the Fatigue Lives of Existing Steel Bridges

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The problem of fatigue cracking in existing steel bridges is becoming an increasingly important one because of the natural aging of these structures as well as the increasing traffic loads and volumes to which they are being subjected. Once fatigue cracks have been detected in a steel bridge, the possibilities for rehabilitation, in particular, may be limited in the case of bridges built using older steel grades to which reinforcing plates cannot be easily welded. Even for bridges constructed of weldable steel grades, it can be a significant challenge for engineers to come up with rehabilitation schemes that will allow the services lives of these structures to be extended with confidence. One possible method of rehabilitating structures such as these that has received recent attention is the application of adhesively bonded carbon fiber-reinforced polymer (CFRP) overlays. This approach has a number of potential advantages. Among these are the relative ease with which CFRP overlays can be handled and applied, and the fact that reinforcing the structure in this way does not require welding or bolting. In the proposed contribution, the results of experimental and analytical studies will be presented on the effectiveness of high-modulus CFRP overlays in increasing the fatigue lives of notched steel plates. The experiment study will consist of four parts: fatigue tests on unreinforced notched plates, fatigue tests on notched plates reinforced prior to testing, fatigue tests on notched plates reinforced after cyclic loading until small cracks have been detected, and fatigue tests on notched plates reinforced while under a static tensile load intended to simulate the self weight of the bridge. For each part of the experimental study, the fatigue loading will consist of constant amplitude tensile load cycles (with stress ratio *R* = 0.1). Crack mouth opening displacements will be measured and the crack lengths will be monitored using various techniques. The analytical study will consist of finite element analysis of each of the four test types. For each test type, several finite element models will be constructed, so that the effective stress intensity factor ranges can be determined for a number of crack depths. This information will then be used to perform fracture mechanics calculations. These calculations will be compared with the test data and conclusions will be drawn regarding the effectiveness of this rehabilitation technique and how it could be further improved. It is believed that the findings of this work will provide valuable insight regarding the usage of CFRP to extend the fatigue lives of existing steel bridges.

ACCELERATED CONSTRUCTION, FIBER-REINFORCED POLYMERS, AND CORROSION EVALUATION

In-Situ Repair of Steel Bridges Using Advanced Composite Materials

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EARL FRANKS
New Mexico Deptartment of Transportation

Advances in structural adhesives have permitted engineers to contemplate the use of bonded joints in areas that have long been dominated by mechanical fasteners and welds. In recent years, an advanced, bonded composite repair technology has made great strides in commercial aviation use. Extensive testing, analysis, and successful flight performance history—obtained through joint programs between the Sandia Labs–FAA Airworthiness Assurance Center and the aviation industry—has established the viability and durability of bonded composite patches as a permanent repair on metallic, commercial aircraft structures. With this foundation in place, efforts are underway to adapt bonded composite repair technology to civil structures. Specifically, a long-term study has demonstrated the feasibility of applying composite repairs to steel structures such as mining trucks, oil recovery equipment, buildings, and bridges. This paper presents the application of high-modulus, fiber-reinforced polymer (FRP) composite patches to repair a steel bridge on Interstate 10. It discusses the array of engineering activities that were completed in order to establish this repair technology for use on civil structures. Results from the I-10 bridge repair will be presented. The factors influencing the durability of composite patches in severe field environments will be discussed along with related laminate design, analysis, installation, and nondestructive inspection issues. The goal of this pilot program is to eliminate any obstacles to the use of composite doubler repairs and allow authorities to exploit the engineering and economic advantages for the refurbishment of steel structures.

The unavoidable by-product of a metallic structure's use is the appearance of crack and corrosion flaws. Economic barriers to the replacement of these structures have created an aging infrastructure and placed even greater demands on efficient and safe repair methods. The use of bonded composite doublers has the potential to overcome the difficulties associated with current repair techniques and the ability to be applied where there are currently no rehabilitation options. It is a cost-effective repair method with minimal disruption to the users of the structure. Instead of fastening multiple steel plates to implement a repair, it is possible to bond a single fiber-reinforced polymer (FRP) composite doubler to the damaged structure. Current techniques for strengthening steel structures have several drawbacks, including requiring heavy equipment for installation, poor fatigue performance, and the need for ongoing maintenance due to continued corrosion attack or crack growth. The use of composite doublers, which do not have brittle fracture problems such as those inherent in welds, helps extend a structure's fatigue life and reduce equipment downtime.

Steel bridges are susceptible to fatigue cracking caused by numerous cycles of heavy live loading. Steel superstructure bridges built during the Interstate construction boom of the 1950s and 1960s are reaching or surpassing their initial design lifetime. Also, the future magnitude and frequency of loading on Interstate highways were underestimated and this produced additional unanticipated fatigue scenarios. Depending on the level of maintenance, some bridges are showing visible signs of deterioration. Presently, state departments of transportation are being forced to detour permit loads around such structures to limit the magnitude of loading while attempting to replace a large number of these structures that have all reached retirement age at the same time. This situation has placed even greater demands on efficient, effective, and safe permanent repair and reinforcement methods.

In the wake of the August 2008 Interstate bridge collapse in Minneapolis, Minnesota, a wide range of concerns has arisen over bridges and other civil structures that are being used beyond their initial design lifetime. These concerns are heightened by the fact that many of these structures undergo minimal, and possibly inadequate, inspections. The effect of structural aging and the dangerous combination of fatigue, corrosion, and other environmentally induced deterioration is now being reassessed. The end result of these assessments has been greater emphasis on the application of sophisticated rehabilitation and repair technology along with improved health monitoring approaches.

Mechanically fastened repairs involve the addition of new holes with associated stress risers and new crack initiation sites. The time and labor involved to attach such repairs can be prohibitive. The uniform load transfer provided by the bonded composite joint is very efficient and free from the fatigue cracks that can arise in mechanically fastened joints. In addition, both bolted and welded repairs have inferior crack mitigation capabilities compared to bonded composite repairs (*1*, *2*).

Sandia Labs has completed a comprehensive program to establish bonded composite doublers as a reliable and cost-effective structural repair method for civil and military structures and to develop adequate real-time monitoring and self-healing systems to ensure the long-term integrity of such structures with minimal need for human intervention. This investigation proved the effectiveness of composite materials to strengthen damaged or deficient steel structures by successfully establishing that

1. Composite doubler repair technology is viable for steel structures within both civil and military arenas;
2. Composite doublers are able to withstand extensive damage and nonoptimum installations while improving fatigue life and ultimate strength; and
3. Field installations are feasible and can demonstrate the performance of composite materials in the repair and refurbishment of steel structures.

This research program developed and proved an optimum field installation process using specific mechanical and chemical surface preparation techniques coupled with unique, in-situ heating methods. By encompassing all "cradle-to-grave" tasks—including design, analysis, installation, structural integrity, and inspection—this program is designed to firmly establish the capabilities of composite doubler repairs.

BONDED COMPOSITE DOUBLER REPAIR METHOD

Figure 1 shows schematics of general composite doubler designs and basic design parameters such as ply lay-up, ply orientation, patch shape and taper, and the bond layer. The number of plies and fiber orientation are determined by the nature of the reinforcement required (i.e., stress field and configuration of original structure). The taper at the edge of the doubler is used to produce a gradually increasing stress gradient in the area of primary load transfer. A top ply of fiberglass is installed to supply mechanical and environmental protection for the installation. This type of repair can often provide more cost-effective and reliable repairs than those currently employed. Figure 2 shows two families of aircraft composite doubler repair installations on aircraft.

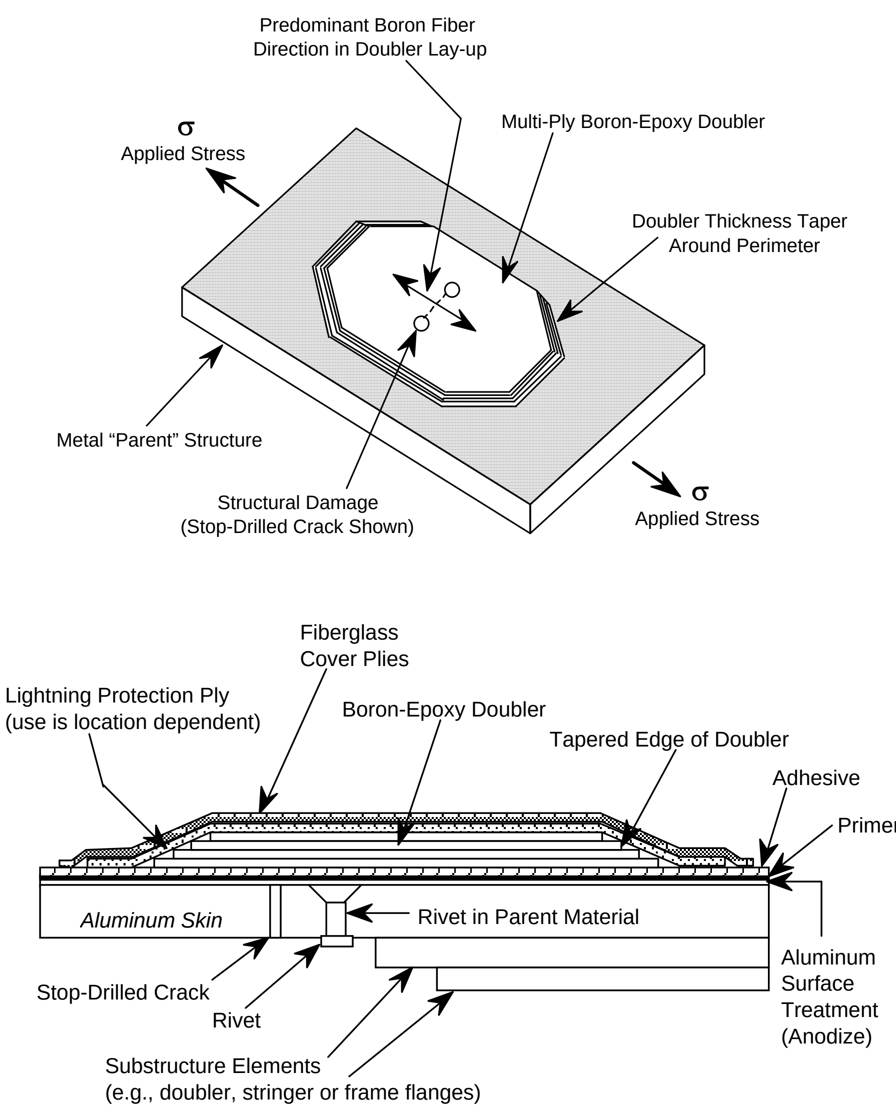


FIGURE 1 Schematics of the bonded composite doubler repair method for any metallic surface.

FIGURE 2 Sample composite doubler aircraft repairs.

Composite doublers offer enhanced safety through improved fatigue life and corrosion resistance (*3*). The engineering advantages of composite doubler repairs include: (*a*) elimination of fastener holes or weld stress risers that produce new crack initiation sites, (*b*) superior flaw growth mitigation performance, (*c*) high strength-to-weight ratio, (*d*) high durability, (*e*) adhesive bonding producing more uniform load transfer and distribution, (*f*) the ability to tailor strength to meet anisotropy needs thus eliminating the undesirable stiffening of a structure in directions other than those required, (*g*) corrosion resistance, and (*h*) formability to complex contours. The economic advantages include rapid repair installations that reduce downtime and do not require future maintenance.

APPLICATIONS FOR COMPOSITE DOUBLER REPAIRS

As a result of U.S. Department of Homeland Security and aging civil infrastructure concerns, increased attention has been placed on the rapid repair and preemptive reinforcement of buildings and bridges (*4*). Bonded composite doublers can be used in lieu of mechanically fastened metallic patches to reinforce or repair damaged structures. Critical infrastructure repair needs represent a broad array of unique facilities, sites, and structures in which its disruption could have significant consequences. Composite doublers can be used to reinforce critical portions of terrorism targets or applied as preventative measures to mitigate damage and avoid catastrophic failures. Figures 3 through 7 show applications for composite doubler repairs and include some damage scenarios experienced in these structures.

Composite doubler repair applications can include such diverse structures as auto and rail bridges, trains and subway vehicles, ships and naval vessels, buildings, electric nuclear power plants, energy generation equipment, aerospace vehicles, transmission towers, manufacturing factories, pressure vessels, tanker trucks, military vehicles and structures, mining equipment, offshore oil platforms, and pipelines.

In the matter of bridge refurbishment alone, the National Bridge Inventory Database (FHWA 2003) indicates that 30% of the 600,000 bridges in the United States are “structurally deficient.” In addition, a majority of the rail bridges in the United States are operating beyond their initial design life. A bridge that is structurally deficient is still strong enough and stable enough for use; however, there are some elements that could be repaired in order to restore the structure

FIGURE 3 Applications for composite doubler repairs—trains and subway vehicles, ships and naval vessels, trucks, and mining equipment.

FIGURE 4 Applications for composite doubler repairs—buildings, power and manufacturing plants, pressure vessels, military vehicles, pipelines, offshore platforms, and oil recovery equipment.

FIGURE 5 Applications for composite doubler repairs—rail and auto bridges.

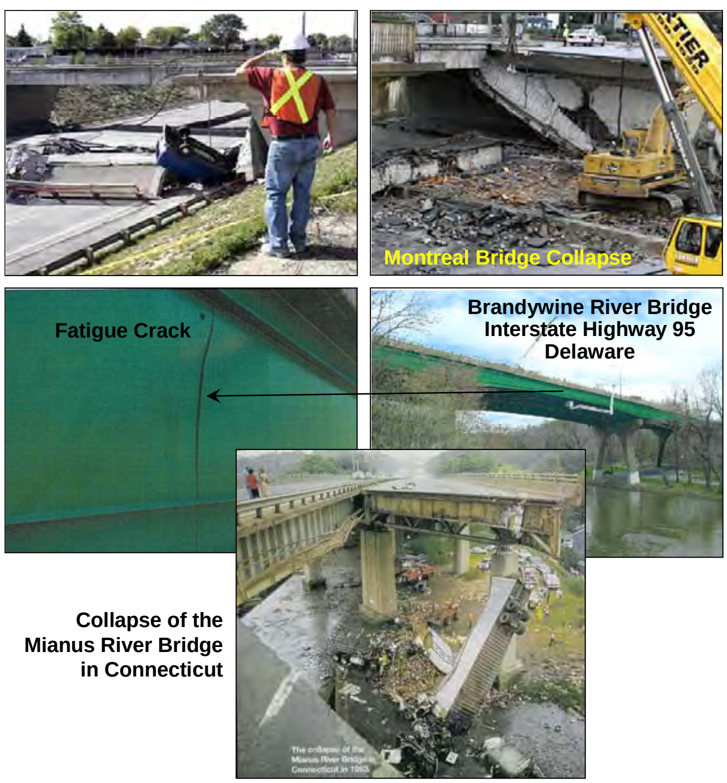


FIGURE 6 Sample failures in bridge structures.

FIGURE 7 Collapse of the I-35 bridge in Minneapolis, Minnesota.

to its original strength and stability. There are other reports that indicate up to 40% of our U.S. bridges are in need of repair. That doesn't mean that 40% are about to fail; rather, it means that preventative maintenance is needed to keep the bridges from further deterioration.

Tom Warne, President, Tom Warne and Associates, LLC, has stated that "Even modest gains in the efficiency of construction and repair could yield huge overall savings." In 2006, the American Society of Civil Engineers (ASCE) issued a report on the status of the U.S. infrastructure. It assessed everything from roads to hazardous waste systems and gave the country's infrastructure an overall grade of "D." The ASCE warned that our "rotting" infrastructure poses risks to safety and economic growth and urged wholesale changes including increased research and development. For metal structures, corrosion flaws reduce the cross section of members, and the effect of repeated loading can generate fatigue cracks.

Steel superstructure bridges built during the Interstate construction boom of the 1950s and 1960s are reaching or surpassing their initial design lifetime. Depending on the level of maintenance, some bridges are showing visible signs of deterioration. Budget restrictions can limit inspections or repairs such that only the more serious problems are addressed. On September 30, 2006, part of an overpass collapsed in Laval, a suburb of Montreal. Figure 6 shows two bridge failures—in Montreal and Connecticut—and one bridge with a large fatigue crack that was discovered and repaired prior to any catastrophic failure.

On August 1, 2007, an I-35 bridge crossing the Mississippi River in Minneapolis, Minnesota, failed (Figure 7). The collapse of the I-35 bridge in Minneapolis prompted many questions regarding the health of similar structures around the world and their associated maintenance programs. On June 15, 2007, a bridge over the Xijiang River in south China's Guangdong Province failed, and in January 1999 a pedestrian bridge spanning the Qi River in southwestern China's Sichuan province collapsed. The government's plans to fix 6,000 "structural deficient" bridges by 2010 were reported recently in *China Daily*. The Minneapolis event is being described as a structural failure and not an act of terrorism.

In Ontario, Canada, 187 bridges missed safety inspections in 2006, according to a report in the *Hamilton Spectator*. The newspaper said Ontario only finished $36 million of $210 million in bridge repairs needed in the province's Golden Horseshoe area. Last year, a provincial report in Manitoba found that of 1,200 bridges in the province, 123 had exceeded their design life of 50 years. Another 222 were at least 40 years old.

DESIGN, INSTALLATION, AND PERFORMANCE ASSESSMENT OF BONDED COMPOSITE DOUBLER REPAIR METHOD

This study developed and proved an optimum field installation process using specific mechanical and chemical surface preparation techniques. In addition, a comprehensive performance assessment of composite doubler repairs was completed to establish the feasibility of this technology for large, steel structures. Damage tolerance, durability, crack mitigation, and ultimate strength tests were performed to quantify the capabilities of composite doubler repairs and to provide insights into the critical laminate design parameters. Fatigue and ultimate strength tests evaluated the overall effect of composite doublers on stress intensity, crack growth, and strength characteristics of steel structures. Bonded composite doubler repairs can extend fatigue life by a factor of 100 while allowing little or no crack growth in a parent steel structure. Furthermore, doublers may reduce stress levels below crack onset and growth conditions such that the flaw is completely arrested. Post-fatigue load-to-failure tests produced residual strength values for the composite-steel specimens. Even in the presence of extensive damage, it was demonstrated that the doubler-reinforced structures were able to achieve residual tensile strengths (i.e., post-damage tensile strength) that exceeded the 65 ksi baseline value for this steel material. Thus, the boron–epoxy doubler was able to return the parent structure to its original strength and load carrying capability.

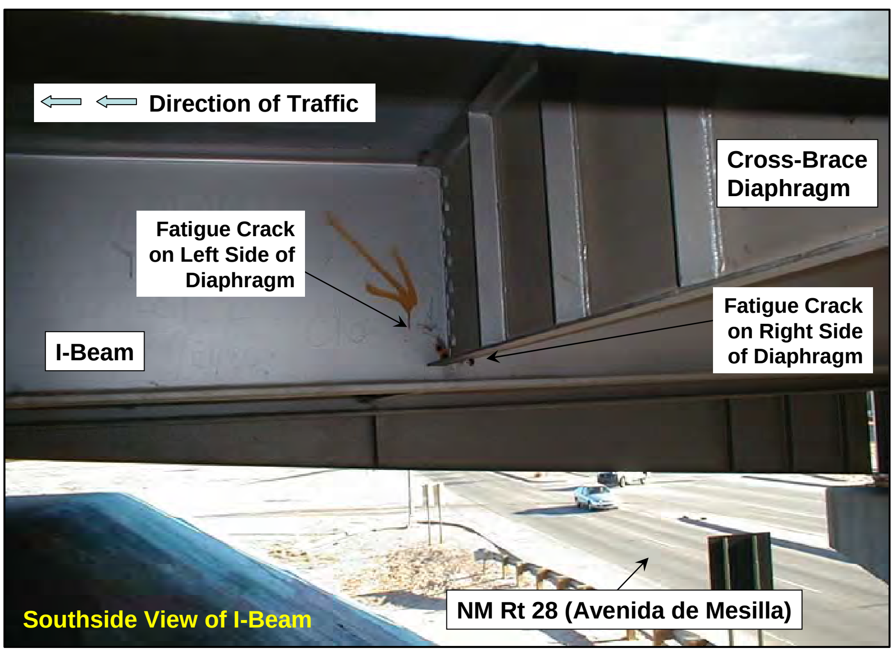
REPAIR OF I-10 BRIDGE

With the technology foundation produced by Sandia Labs in place (*5–10*), the focus turned to producing a field demonstration of bonded composite repair technology on a civil structure. Such a proof-of-concept application was completed in the repair of a steel bridge on New Mexico I-10 highway. This bridge experienced fatigue crack growth in one of its steel I-beams at a joint between the I-beam and a cross-bracing diaphragm. All aspects of the bridge repair were coordinated with the New Mexico Department of Transportation (NMDOT).

Figure 8 shows the I-10 bridge and highlights the bridge construction around the damaged region. An initial site visit was made to

- Measure all structure geometry,
- Measure the profile of the fatigue cracks,
- Conduct a fit check on the preliminary repair doubler designs,
- Perform strain field monitoring to support the final repair design effort, and
- Assess critical surface preparation and installation steps:

(a)



(b)

FIGURE 8 (*a*) Close-up view of I-10 bridge showing example of live loading, and (*b*) view of damaged I-beam and details of the bridge construction in the vicinity of the fatigue cracks.

  - Ability to grit blast the steel surface,
  - Proper sizing of field compressor, and
  - Ability to draw vacuum over all heat blanket regions;
- Complete a heat source–sink analysis to determine:
  - Ability to raise temperatures to 225°F in the repair region,
  - Check proposed heat blanket and vacuum bag arrangement,
  - Check function of high amperage (30A) power cables, and
  - Proper power supply from field generators (2); and
- Coordinate logistics with NMDOT—overall preparation for final repair installation.

The I-beam to be repaired was 0.5 in. thick. The set of fatigue cracks covered an area that was approximately 5 in. long x 4.5 in. high (Figure 8). The large holes visible in the web of the I-beam are stop-drills that were added in unsuccessful attempts to arrest the growth of the fatigue cracks. Eddy current inspections were used to determine the path of the fatigue cracks and establish the tip of the cracks that were not visible to the human eye.

Stress Field Monitoring

After recording the geometry of all structural members and the profile of the fatigue cracks, field measurements were taken to determine the strain field in the vicinity of the damage. A set of 16 strain gages were used to monitor the strain risers around the crack tips and holes, as well as the uniform farfield strains. In order to produce extreme loading conditions—so that the composite doubler could be properly and conservatively designed—a combined load of a stationary dump truck (total weight of 53,120 lb) and a moving semi-truck was placed over the damaged region. Strain data collected from the gages were used to calculate membrane and principle stresses in parent steel plate. The maximum stress levels ranged from 11,000 to 12,000 psi. Principal stress levels away from the damaged region reached a maximum value of 8,000 psi and were determined to be in the longitudinal direction.

Heat Source–Sink Assessment

A critical element in producing field repairs of this nature is the ability to produce sufficient temperatures in the structure to complete the hot-bonding process. As the steel structure becomes thicker, higher energy levels are needed to heat the structure to the desired maximum cure temperature of 225°F. Flanges, structural attachments, and other heat sinks that can remove heat from the repair area exacerbate this problem. Preliminary tests were conducted to determine if the number, size, power, and arrangement of heat blankets were sufficient to provide the necessary composite doubler cure profile. In order to minimize the detrimental effects of the major heat sinks, heat lamps were used to warm the bottom flange of the I-beam and the web area immediately adjacent to the repair region.

Composite Doubler Design for Bridge Repair

It was determined that the optimal composite doubler repair would be part of an overall repair method that also includes a weld process. Cracks in structures should first undergo a material removal (gouge) and replacement (weld) process. Currently this is the final step in the repair procedure. However, in most instances, fatigue cracks reappear in the weld fill areas. Composite doublers placed over the weld will produce a true hybrid repair where the heat-affected zones or unintentional weld porosity pockets are reinforced to avoid subsequent recracking along the weld site. This is a superior repair with a significantly extended fatigue life.

The doublers were designed using laminate design principles to determine the proper E_x, E_y, ν_{xy}, and G_{xy} for the composite lay ups. The required number of plies and the transfer length, which defines the overall repair length, were calculated. An iterative process was then followed whereby specific design parameters were adjusted in an effort to meet general design guidelines as summarized below:

- Design input data:
  - Parent plate thickness = 0.5 in. thick (ASTM A36),
  - Parent plate modulus = 29 x 10^6 psi,
  - Doubler (patch) thickness = 32 plies x 0.00544 in. = 0.175 in. thick,
  - Doubler length, L_r = 12 in. (full thickness length = 7 in.; taper length at each side = 2.5 in.),
  - Doubler width = 8 in.,
  - Stacking sequence: wedding cake stacking sequence (largest ply at the steel interface; sequentially smaller footprint plies are stacked on top),
  - Taper ratio = 15 (when combined with patch thickness, the taper ratio determines the transfer length, β = 0.4),
  - Vacuum bag debulk every four plies,
  - Corner angle = 45 degrees,
  - Patch shape = octagon,
  - Load constant, N = 8, is related to the full thickness length of the patch (the overlap length, or one half the doubler full thickness length, is equal to N times the transfer length β);
- Doubler design notes:
  - 45-degree plies provide better performance in shear loads,
  - Use of corner angles reduces stress riser at steel-doubler interface,
  - Adding width improves load transfer into doubler and decreases bypass strain,
  - Increasing taper ratio reduces peel stresses;
- Doubler design output:
  - Doubler lay up (32 plies) = $[0,0,45,–45,90,0,0,+45,–45,90,0,0,45,–45,90,0]_s$,
  - Stiffness ratio = 0.40, and
  - Effective doubler modulus E_x, = 21.3 x 106 psi; E_y = 12.9 x 106 psi.

The input parameters listed above where used to calculate the doubler lay up (32 plies) and the symmetric, two-sided composite doubler repair design shown in Figure 9. The repair was optimized by varying the number and orientation of the plies, the taper ratio, the load constant, and the patch geometry. Note the addition of a fiberglass cover ply over the boron–epoxy laminate. It is used to provide mechanical (impact) protection, moisture resistance, and ultraviolet (UV) protection. UV rays from the sun will break down an epoxy resin system over time and result in boron fiber loss.

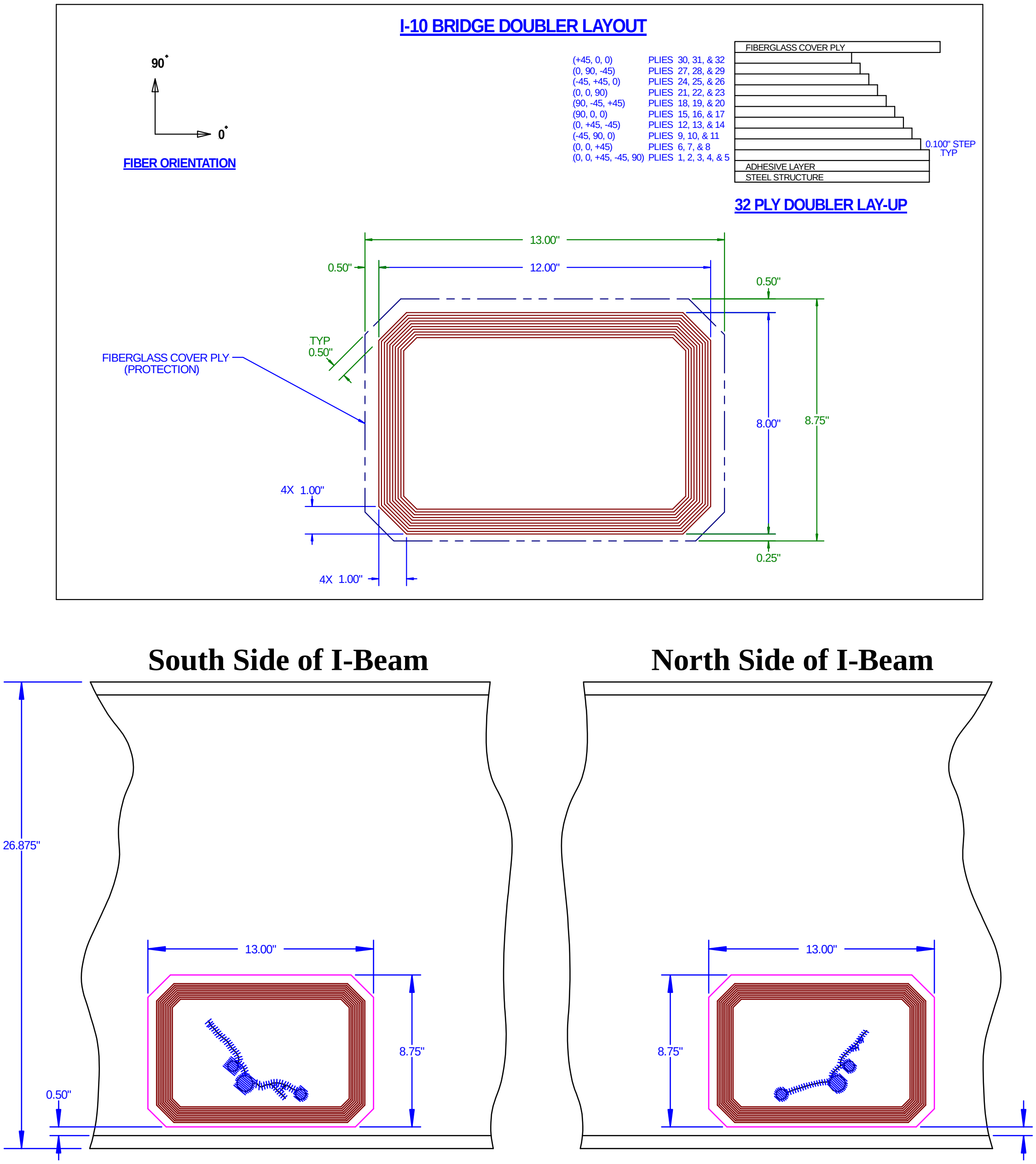


FIGURE 9 Final composite doubler repair design for bridge I-beam repair.

Composite Repair Installation and Strain Field Monitoring

The basic composite doubler installation steps are as follows: (*a*) paint removal and coarse sanding to remove oxide layers, (*b*) grit blasting to produce a proper and uniform surface roughness, (*c*) application of silane and primer to enhance the chemical bond, (*d*) placement of the composite doubler and adhesive film over the region to be repaired, and (*e*) application of controlled heat to cure the boron–epoxy doubler and the adhesive. Figures 10 and 11 provide a

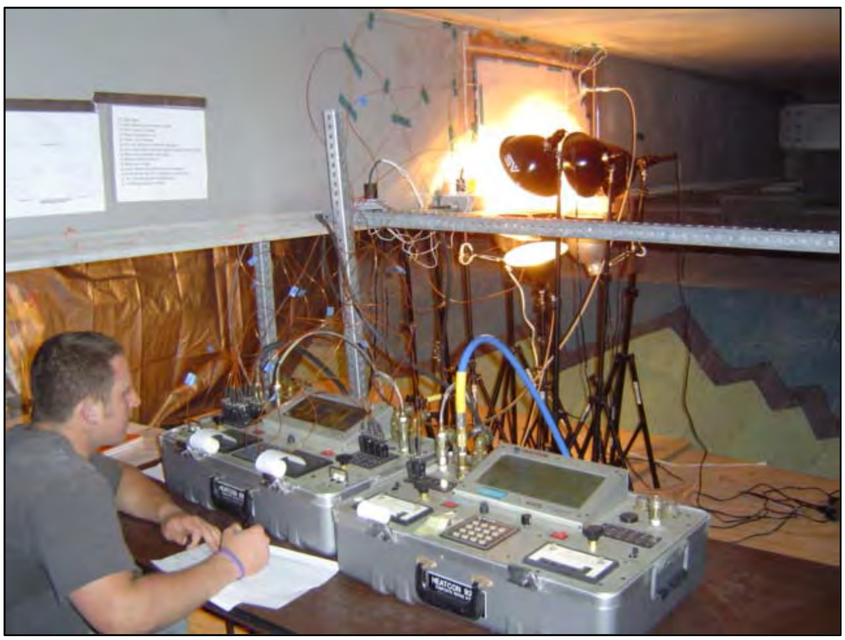
(a)

(b)

(c)

FIGURE 10 Surface preparation and placement of composite repair patch on cracked I-beam: (*a*) sanding to remove excess scale–oxide and grit blasting to produce optimum surface roughness; (*b*) application of silane mixture to grit-blasted surface and primer to chemically enhance the bond; (*c*) boron–epoxy composite doubler prior to placement on I-beam and placing patch in repair area.

(*a*)



(b)

FIGURE 11 Bond and cure of composite patch using heat blankets controlled by hot bonders: (*a*) composite doublers placed on both sides of the cracked I-beam, vacuum bag, and heat blanket set-up; and (*b*) monitoring pressure and temperature during the 3-h cure process (hot bonder control units shown in foreground).

summary of the major bridge repair steps. Five heat blankets were used to produce the cure temperatures and 16 thermocouples were mounted around the repair region to provide the closed-loop feedback to the hot bonder machines. The array of heat blankets over the composite doublers and in the adjacent regions was able to adequately control the temperatures and closely follow the desired cure profile (ramp up of 5°F per minute and 180-min hold at 225°F). The completed composite doubler repairs are shown in Figure 12 prior to the application of a protective paint.

Beside the doublers in Figure 12 are the witness coupons that are used for quality control purposes. A plastic wedge is used to pry these witness coupons away from the steel plate at the bondline as shown in Figure 12. A successful wedge test result is achieved when adhesive material appears on both the metal substrate and the witness coupon. Note the presence of the pink adhesive on both the witness coupon and the parent steel plate in Figure 12. This signifies a good installation and assures that the adhesive layer will fracture at high strains (cohesive failure) rather than disbonding (adhesive failure). Such cohesive failure indicates that the full strength of the adhesive is achieved at the bond line rather than lesser strengths associated with adhesive failure modes.

South Side of I-Beam **North Side of I-Beam**

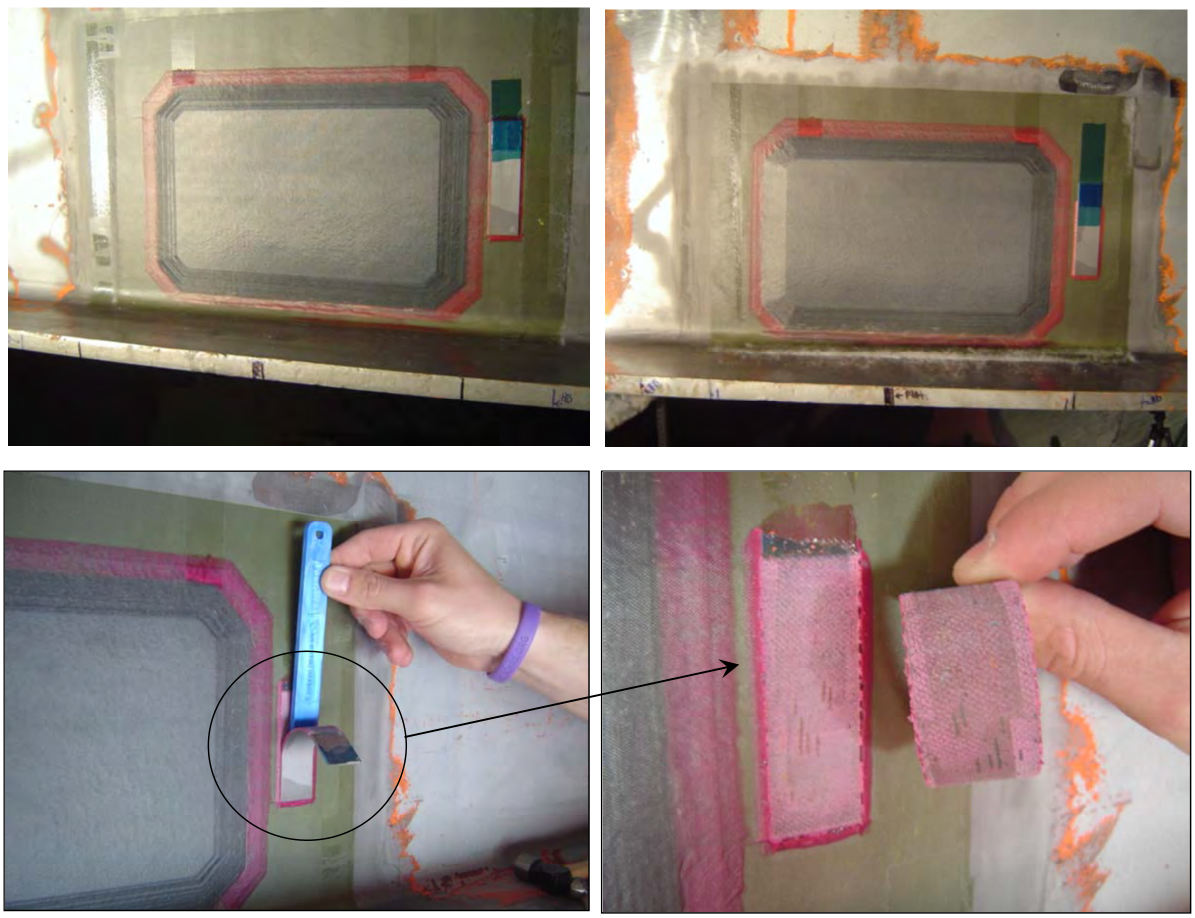


FIGURE 12 Composite doublers after hot bonding process; witness coupons used for quality assurance are shown next to repair patches.

Stress Field Analysis

A post-repair strain mapping was conducted after the composite doubler installation to compare strain fields before and after repairing the bridge cracks. The 16 gages were placed in approximately the same location as during the pre-repair strain field monitoring. The same load scenario applied before the repair was used to induce a combined load of a stationary dump truck (26.5 tons) and moving semi-truck over the repaired region. Dynamic, peak strain measurements were taken when semi-trucks crossed the bridge traveling at approximately 50 mph. Strain readings showed that after the composite repair process, the stress risers associated with the cracks were eliminated. The lateral strains were reduced to essentially zero. The larger longitudinal strains were reduced to less than half the values recorded during the pre-repair visit.

Maximum longitudinal stress levels ranged from 4,000 to 4,500 psi. Principal stress levels away from the damaged region reached a maximum value of 4,600 psi and were determined to be in the longitudinal direction. The lateral stress levels were reduced significantly. Load transfer into the doubler was approximately 50% which agrees with normal repair design principles. Decreases in the longitudinal stresses ranged from 20% to 80% with the largest reductions occurring in the highest (stress riser) areas. The end result is that the stress field in the bridge I-beam is now much more uniform. Also, the stresses in the steel immediately adjacent to the repair patches were not increased as would happen if the patch stiffness was too large. The large lateral stresses, produced primarily by the presence of the fatigue cracks, were reduced more than 90% by the composite doubler patches.

CONCLUSIONS

This Sandia Lab research program evolved the application of composite patches to refurbish thick steel structures typically used in civil and military structures. Instead of riveting multiple metal plates or using welding processes to facilitate a repair, it is possible to bond a single composite doubler to the damaged structure. The use of FRP materials, which do not have brittle fracture problems such as those inherent in welds, can help extend the structure’s fatigue life and reduce equipment downtime. Applications include such diverse structures as buildings, bridges, railroad cars, trucks and other heavy machinery, steel power and communication towers, pipelines, factories, mining equipment, ships, and tanks and other military vehicles. Extreme fatigue, temperature, erosive, and corrosive environments induce an array of structural damage. Current techniques for strengthening steel structures have several drawbacks such as requiring heavy equipment for installation, poor fatigue performance, heat-affected zones around welds, and the need for ongoing maintenance due to continued corrosion attack and crack growth. The use of bonded composite doublers overcomes the difficulties associated with current repair techniques and the ability to be applied where there are currently no rehabilitation options. This study showed that bonded composite repairs have crack mitigation capabilities that are superior to bolted and welded repairs. The economic advantages include rapid repair installations that reduce downtime and do not require future maintenance. This new repair process was successfully demonstrated on a steel highway bridge. Stress monitoring before and after the repair showed that the composite repair will arrest crack growth in the bridge I-beam.

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Corrosion Evaluation of Post-Tensioned Tendons in a Box Girder Bridge

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Corrosion and corrosion-related tendon failures have been disclosed on several segmental box girder bridges in Florida. One of these was the Mid-Bay Bridge near Destin where as many as 11 tendons were replaced. Corrosion of post-tensioned tendon is suspected in several other post-tensioned bridges. State departments of transportation have been interested in identifying the extent of corrosion problems and the time-to-criticality so they can plan and schedule rehabilitation. This paper discusses our work on a post-tensioned bridge in the Midwest. The visual inspection of this bridge revealed (*a*) cracks on the riding deck, (*b*) active cracks on webs and diaphragms, (*c*) corrosion-related distress on the underside of the riding surface, and (*c*) voids along the tendons. Based upon field and laboratory corrosion investigations, tendon corrosion was present at select locations. The primary causes of tendon corrosion are compromised alkalinity of the grout and the presence of voids. The rate of corrosion measurements indicated that these tendons experience moderate to high corrosion rate. The rates of corrosion of tendons help define the time to a defined section loss. This information was used in determining the remaining structural capacity and identifying areas that require strengthening.

Steel structures exposed to soil and water corrode from the first day they are placed in service. The life of the structure primarily depends on the rate of corrosion of the structure in a given environment. The rate of corrosion depends on the availability of water and oxygen in alkaline environments. In acidic environments, corrosion can proceed even in the absence of oxygen and water. There are several examples of water mains and pipelines that experience corrosion on a daily basis. In fact, U.S. Department of Transportation Office of Pipeline Safety (*1*) regulations expressly prohibit operating a coated pipeline without any cathodic protection. The Code of Federal Regulations further stipulates that the level of protection achieved by cathodic protection must be documented every 2 months to ensure that corrosion will not lead to leaks and explosions.

When steel is placed in a high pH environment such as concrete or grout, it does not corrode. This is primarily due to the formation of a film on the surface of steel. Typically, the pH of the fresh concrete is 13 and above. However, this passive film may not form if the pH of the concrete or grout at the steel interface is less than 11. The passive film can also be destroyed when the chloride at the steel–concrete interface is at or above the threshold required for corrosion. In addition, the passive film can also be compromised by carbonation. Carbonation is a process where carbon dioxide from the atmosphere reacts with calcium hydroxide in concrete or grout to form calcium carbonate. This chemical reaction reduces the pH and causes the dissolution of the passive film.

CORROSION OF TENDONS IN POST-TENSIONED STRUCTURES

In post-tensioned structures, the tendons are protected from corrosion by filling the annulus between the tendon and the duct using grout. The anchors are typically coated with mastic to prevent chloride from entering into the post-tensioned ducts. Water and fines in the grout have a tendency to separate and move ahead of the grout mass. This phenomenon depends on the quality of the grout, equipment, and procedures used during pumping operation, and any admixtures added to the grout. This results in voids and low pH grout around the tendons. This typically occurs at certain locations along the duct. These conditions lead to corrosion of strands even in the absence of chlorides. However, the time to failure of strands depends on the rate of corrosion of strands at these locations. Thus, it is essential to measure the rate of corrosion of strands to estimate the remaining life or time to criticality.

The majority of post-tensioned structures are still functioning under full design loads without any problems. However, tendons in post-tensioned structures are typically loaded to 60% or more of their ultimate tensile strength. Any section loss due to corrosion can significantly overstress the remaining wires–strands and can lead to structural failure without any warning sign (unlike the regular reinforced concrete structures). Some of the well-known post-tensioned bridge failures are

- A concrete pedestrian bridge (over U.S. Highway 29) collapse in North Carolina,
- The Berlin Congress Hall roof collapse,
- A post-tensioned concrete bridge collapse in Wales,
- A post-tensioned segmental bridge collapse (Ynys-y-Gwas) in the United Kingdom,

and

- The Koror–Babeldaob Bridge, Republic of Palau, Micronesia, collapse.

Most of these structures did not exhibit any sign of distress before they collapsed.

In other structures (e.g., Varina–Enon Bridge in Virginia), past routine investigations (i.e., bore scope, impact echo) identified voids. These voids were then filled with good quality grout. It was assumed that if voids were completely filled, there would be no further corrosion. However, 3 years later one of the tendons was completely severed due to severe corrosion. This clearly indicated that actively corroding tendons would continue to corrode even if the voids are completely filled with grout.

It is, therefore, important that owners of these structures be proactive in evaluating the condition of the encased strands, since there may be no advanced signs associated with their failures. It is our recommendation that owners perform corrosion evaluations to ensure that tendons are not corroding and the structure has adequate strength.

It is essential to perform corrosion evaluations of the tendons in post-tensioned and cable-stayed structures at the first sign of distress, i.e., wire breaks, cracks, spalls, efflorescence, misalignment, or other similar signs. These are the only signs the owner may see before failure. There is a general tendency to limit corrosion evaluation to evidence of corrosion, voids in the grout, or broken wires and to believe that corrosion can occur only at void areas. If there are no voids, or if the bore scope cannot identify voids, there can be no corrosion. There are several limitations to this line of thinking. For example, a bore scope can be inserted only if the voids are greater than a certain size. Sometimes there may be a very thin layer of carbonated grout present on top of the strands that may prevent one from observing the condition of the strands using a bore scope. Again, impact echo may be able to identify big voids but may miss small voids. Corrosion investigation will also assist in identifying the time to rehabilitation and all suitable rehabilitation.

The author has investigated the corrosion condition of several post-tensioned structures. Based on his experience, voids are more common than expected. Identifying and filling voids with high-quality grout may not arrest corrosion but in fact may accelerate corrosion and strand failures (*2*). Voids are a problem, but not always. Severe corrosion can occur even if the tendons are completely covered by grout (*3*). The author has employed nondestructive techniques to quantify the rate of corrosion of tendons and the time-to-criticality. The following case study is based on the work performed by the author.

CASE STUDY

This structure is a 25-span, 5,800-ft-long cast-in-place box girder bridge (Figure 1). The structure is approximately 25 years old. It is a six-lane divided highway with shoulders on both sides. The structure has three boxes adjacent to each other throughout its length. The total length of tendons in this bridge is approximately 35,000 ft. The tendons that hold the boxes together are inside the web walls of the box girder. Each tendon is made up of several spirally wound, 1/2- to 5/8-in. diameter, seven-wire strands inside a grouted 4-in. diameter, galvanized metal duct. Since the tendons are placed inside the web walls, any wire or strand breaks due to corrosion will not be readily visible to the naked eye. Though all ingredients were present for corrosion, it was not clear whether the visual signs were the result of tendon corrosion or were contributing to tendon corrosion. Thus, it is important to determine the rate of corrosion of strands in situ and to determine how the quality of the grout may influence the rate of corrosion of strands.

FIGURE 1 A view of the cast-in-place box girder bridge.

Problem

There were signs of problems throughout the structure, i.e., efflorescence, cracks (both small and large), and several spalled areas on the underside of the top slab (Figure 2). A previous bore scope inspection indicated numerous small and some large voids in the grout surrounding the tendons in different parts of the structure. The owner prudently embarked on a thorough investigation to quantify the level of corrosion and not just identify corrosion.

Corrosion Investigations

A number of locations were tested in the field. Testing involved exposing the tendons, documenting the visual condition of the wires–strands, including the degree of rusting and/or pitting, documenting the quality of the grout surrounding the tendons, measuring the corrosion rate of strands embedded in the original grout and the alkalinity of the grout. Grout samples were extracted for chloride and petrographic analysis.

Voids in the Grout

Voids with white chalky grout were discovered at several locations (see Figure 3). Based on the measured variations in temperature and humidity, thousands of cycles of condensation will occur that will lead to corrosion. Corrosion activity will be restricted to times of wetting due to condensation. However, presence of a thin layer of grout on the surface of the strands can actually hold this moisture and increase the duration that the strands experience corrosion. Thus, it is important to measure the rate of corrosion of strands while it is still covered by grout.

FIGURE 2 Presence of efflorescence, delamination, and spall observed on post-tensioned box girders.

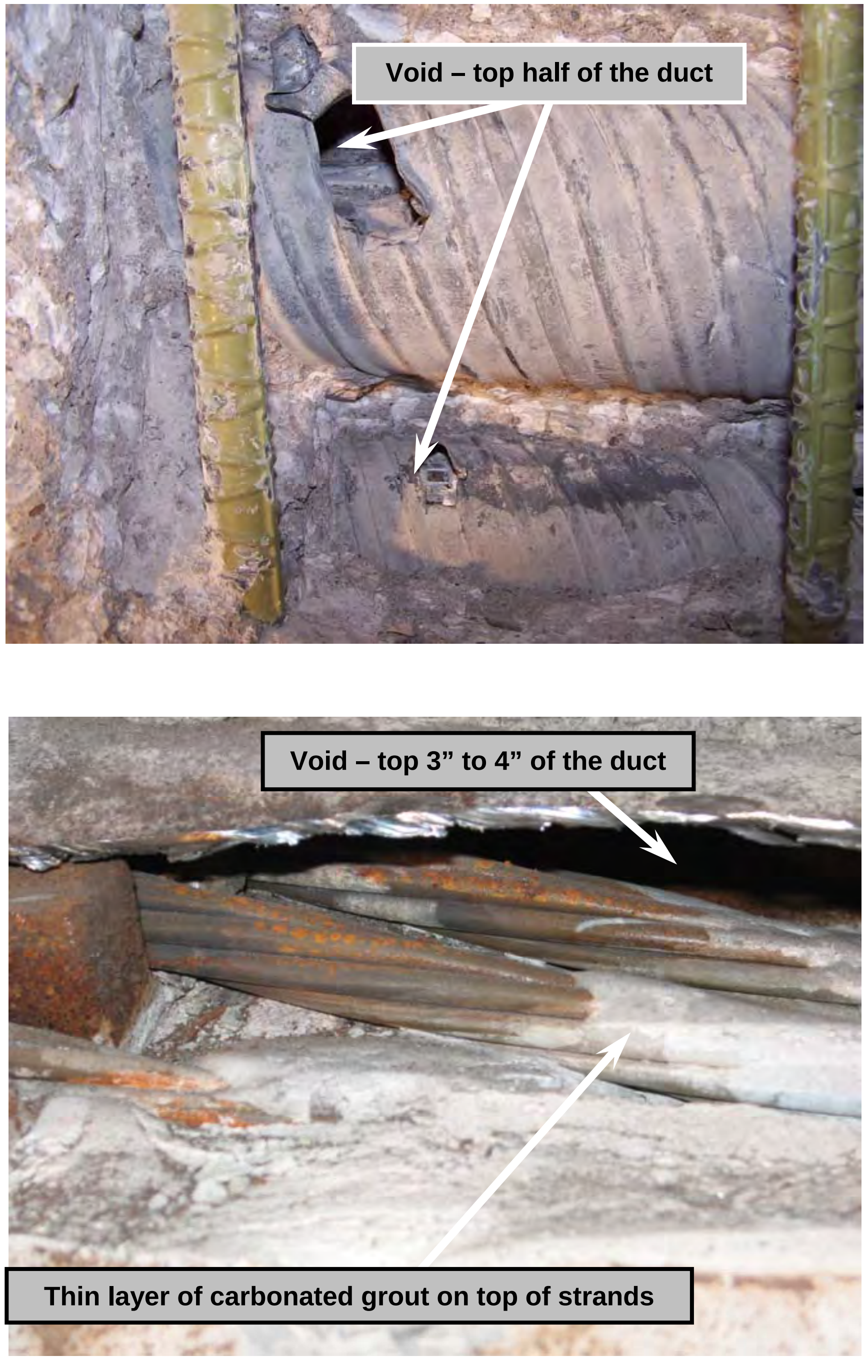

FIGURE 3 Voids (with a thin layer of chalky material) at different sections of the tendon.

Corrosion Potentials

The corrosion potential varied from a low of –263 mV CSE (may or may not have active corrosion) to a high of –104 mV CSE (90% probability of no active corrosion). Though many engineers rely on corrosion potential to decide if a structure is actively corroding, it must be remembered that corrosion potential only refers to the probability of corrosion (*4*). In certain situations measured corrosion potential may not be reflective of the true corrosion activity of the steel embedded in concrete–grout. The moisture content of the grout varied from a low of 10% to a high of 36% indicating that ongoing corrosion is possible. It is, therefore, important to measure the rate of corrosion of strands in addition to corrosion potential.

Chloride Content of the Grout

Grout samples taken from various test locations were immediately sealed in a zip lock bag at the site and transported to the laboratory for further testing. The chloride content of the grout sample was determined based on AASHTO T260 (*5*). The chloride content of the grout samples ranged from a low of 0.0051% to a high of 0.0127% by weight of cement. The highest chloride content was lower than the limit of 0.08% established for new post-tensioned structures.

Alkalinity of the Grout

The alkalinity of the grout varied from 9 to 13 along the strand. At select locations (i.e., couplers), several strands had a thin layer of carbonated or partially carbonated grout. In some cases, high pH grout was found on top of the low pH grout (see Figure 4).

Corrosion Rate

Our past experience with Mid-Bay Bridge in Florida indicated that corrosion potential is not a reliable indicator of existing or ongoing corrosion. SCS measured the rate of corrosion of strands in-situ to capture the ongoing corrosion (see Figure 5). The overall corrosion rate varied from as low as 0.01 mpy (very low) to as high as 1.9 mpy (high). Seven (out of 30) tendons (23%) indicated a high (> 1 mpy) corrosion rate.

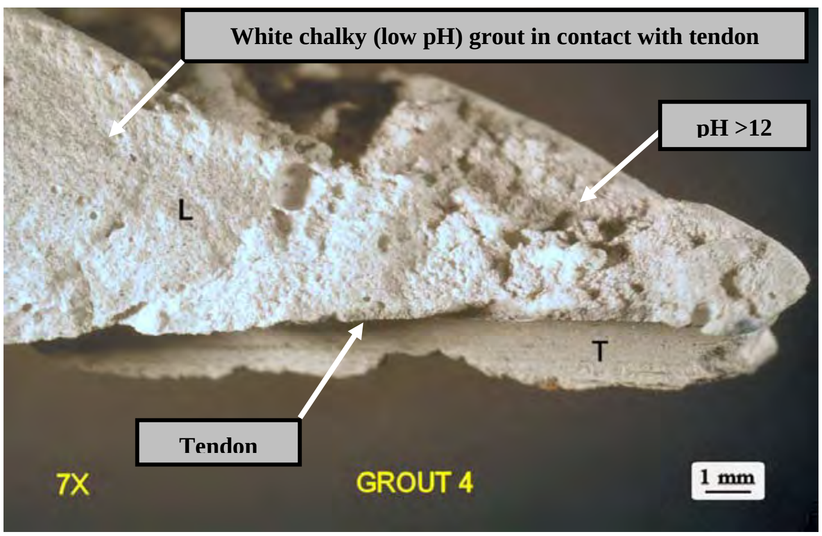

FIGURE 4 Good quality grout on top of carbonated grout.

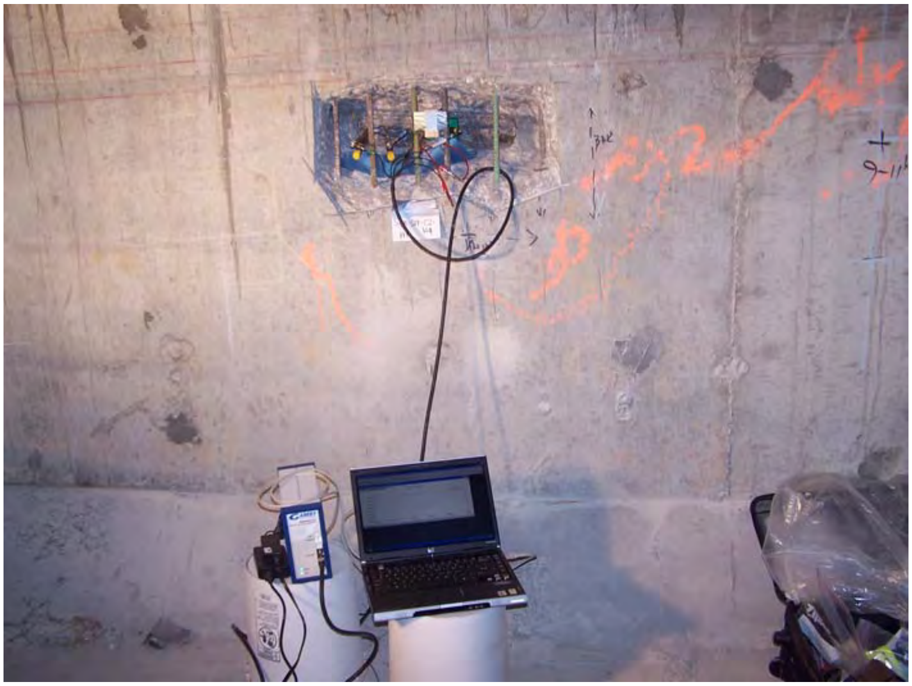

FIGURE 5 In situ nondestructive testing to determine the rate of corrosion of strands.

Petrographic Analysis of the Grout

Several grout samples were evaluated under the microscope to determine the quality of the grout. The water–cement ratio of the grout varied from 0.36 to 0.90. At least some of the voids in the ducts can be attributed to the occurrence of bleeding in the grout. Bleeding within the grout strongly indicates the possibility of segregation of the grout and accumulation of fines along certain sections of the duct. Presence of voids and white chalky grout (see Figure 6) also points to bleeding.

CONCLUSIONS

The moisture content of the grout and the temperature and humidity variation within the structure indicate a possibility of corrosion of strands, particularly where there are voids. However, the strands may not corrode if they are surrounded by good-quality high alkaline grout. Though there were no significant levels of chlorides in the grout, corrosion due to chloride intrusion is possible in both precast and cast-in-place box girders. Rate of corrosion of strands varies significantly from location to location, depending upon the quality of the grout. Since the rate of corrosion depends on the alkalinity of the grout, the chloride level, and the moisture availability, rate of corrosion measurement can delineate areas of concern within the structure. The rate of corrosion combined with past corrosion damage can help determine the time to criticality and provide a measure of section loss to be used in structural analysis to determine the adequacy. Our evaluation here underlines the importance of measuring the rate of corrosion of strands in situ.

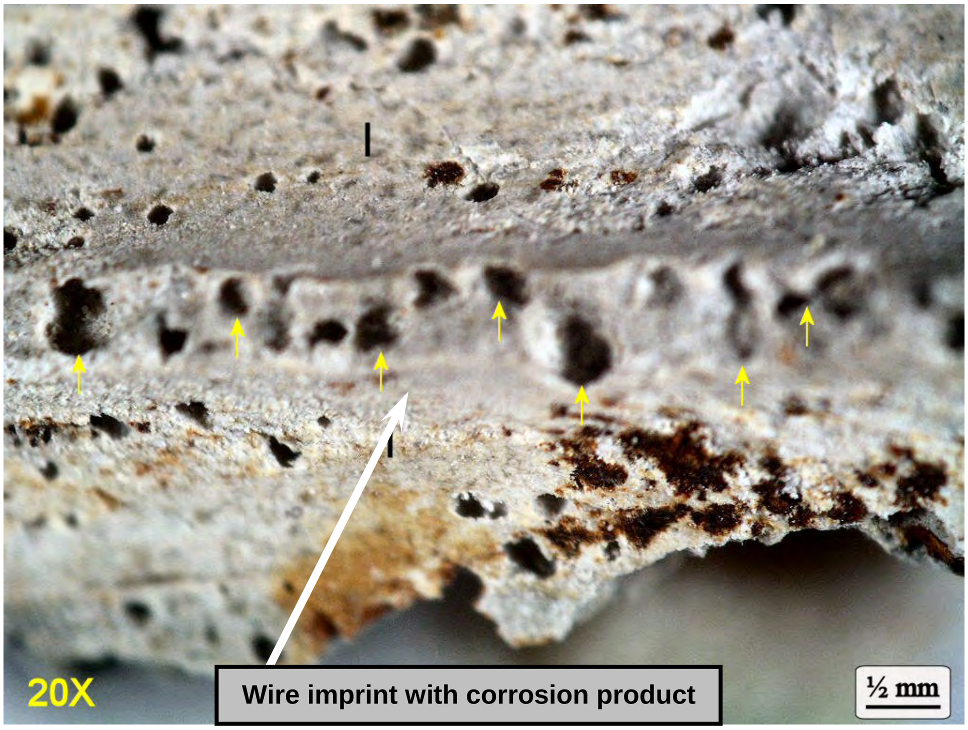

FIGURE 6 Presence of numerous voids adjacent to the wire trace (I). Corrosion products on the wire trace indicate that wires are actively corroding.

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Bridge Modeling and NBI Translator

BRIDGE MODELING AND NBI TRANSLATOR

Modeling Approach of the National Bridge Investment Analysis System

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The FHWA and FTA report to the U.S. Congress on the state of transportation infrastructure in their biannual Conditions and Performance (C&P) Report. The FHWA uses the National Bridge Investment Analysis System (NBIAS) to develop its estimates of bridge investment needs for the C&P Report. This paper details the modeling approach used by NBIAS in determining bridge investment needs, focusing on recent developments in the system not previously described in the literature. NBIAS uses the National Bridge Inventory (NBI) for its input. The system predicts the structural elements, including the quantity and condition of each element, for each bridge in the NBI. Alternatively, one may import element data into the system directly. Once a bridge inventory has been established, NBIAS uses a modeling approach originally adapted from the Pontis Bridge Management System (BMS) to predict maintenance, repair, and rehabilitation (MR&R, also referred to as preservation) and functional improvement investment needs, and then simulates allocation of a given budget to the bridge inventory over time with the objective of maximizing user benefits and minimizing agency costs. When performing an analysis, the system executes a series of simulations with different annual budgets. The system presents its results through a series of reports and interactive views, which allow for interpolating results between the range of budgets analyzed. Distinctive aspects of the NBIAS modeling approach and detailed in the paper include the following:

- **Elements are classified by Highway Performance Monitoring System climate zone. Default deterioration models have been developed for each of the nine climate zones through a separate effort.**
- **Functional improvements may be triggered either through a functional improvement policy matrix that specifies standards for lane width, shoulder width, vertical clearance, and load rating (as in Pontis) or by any functional issue that results in a deduction in the sufficiency rating or in classification of a bridge as structurally deficient (SD) or functionally obsolete (FO) based on FHWA definitions.**
- **When calculating the user benefit associated with making a functional improvement, the system may use either the Pontis improvement models, or a separate set of improvement models developed more recently for the Florida Department of Transportation. Functional improvements are considered needs if the benefit of making an improvement over a specified analysis period exceeds the improvement cost.**
- **Replacement of a bridge may be triggered because replacement is the most cost-effective means to address a preservation or functional improvement need, because functional improvement is needed but otherwise infeasible, or because an engineering rule specifying the minimum tolerable condition for a bridge is triggered. Engineering rules may be specified based on bridge age, health index, sufficiency rating, or whether or not a bridge is SD or FO.**
- **Agency and user cost adjustment coefficients may be specified by state.**
- **Although the system is intended for use as a network-level analysis tool, bridge-level results may be stored for diagnostic or reporting purposes.**

Determining the appropriate level of investment to maintain the U.S. highway bridge network in a state of good repair is a national concern. FHWA reports on the condition of the bridge network, and on investment needs for bridges, in the biannual Conditions and Performance (C&P) Report (*1*). The FHWA uses the computer model National Bridge Investment Analysis System (NBIAS) to perform its analysis of bridge investment needs. FHWA began developing NBIAS in the 1990s as a successor to the Bridge Needs Investment Process model released in 1991. The first version of NBIAS was released in 1999; the current version of the system, Version 3.3, was released in 2007.

The basic modeling approach in NBIAS is derived from that of the Pontis bridge management system (BMS). The Pontis BMS was initially developed for the FHWA beginning in the 1980s (*2*) and is now licensed by AASHTO to more than 40 U.S. state transportation departments and other agencies. The two systems share a common fundamental modeling approach, based on representation of a bridge as a set of structural elements. However, whereas Pontis requires element-level data for its modeling, NBIAS requires only the National Bridge Inventory (NBI) data reported annually to FHWA by each state based on NBI coding guidelines (*3*). Also, NBIAS has a number of features to support network-level analyses as required for development of the C&P Report. Further, over time FHWA has made a number of revisions to the NBIAS modeling approach, such as to support integration of NBIAS results with those from the Highway Economics Requirements System (HERS) used by FHWA for highway investment needs analysis.

This paper details the modeling approach in NBIAS, describes the implementation of the models in the system, and offers a set of conclusions. Because the fundamentals of the modeling approach have been well documented previously in the literature, the focus here is on aspects of the modeling approach that are unique to the system, or recently introduced in the Version 3.3 release.

MODELING APPROACH

NBIAS models preservation and functional improvement needs for each bridge in a network. The system uses data on current conditions to predict conditions over time and determine the optimal allocation of resources for bridge investment given a fixed budget. This section describes the NBIAS modeling approach, with subsections on database generation, the preservation models, the functional improvement models, the scenario simulation process, and measures of effectiveness predicted by the system. Additional detail on the modeling approach is provided in NBIAS 3.3 Technical Manual (*4*).

Database Generation

The first step in working with NBIAS is to create a database of bridges for analysis. To establish a database one must import a metric NBI file coded as detailed in FHWA's *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges* (*3*). Upon initial import, all culverts and other nonbridge structures are omitted. Also, the user may optionally apply a procedure to screen certain bridges from further analysis.

The motivation for the screening procedure is that once a bridge is imported into the system it is assumed that the bridge should be maintained in a state of good repair. If one is interested in evaluating whether or not it is cost effective to maintain a given bridge to begin with, this calculation must be performed through the screening procedure prior to other analyses. To configure the screening procedure, the user specifies what bridges, if any, are considered for screening. If a bridge is considered for screening, the system compares the disbenefit of removing a bridge from service to the cost of replacing the bridge. If the disbenefit is less than the replacement cost by a specified factor, the bridge is screened from further analysis. This disbenefit is calculated as follows:

$$B_a = 365.25 \bullet T\left[\frac{\tau}{100}\left(LC_l^t + \frac{L}{\mathrm{v}_d}C_h^t\right) + \left(\frac{100-\tau}{100}\right)\left(LC_l^c + \frac{L}{\mathrm{v}_d}C_h^c\right)\right] \tag{1}$$

$$B = B_a \sum_{i=1}^{N} \delta^{i-1} \tag{2}$$

where

B_a = annual user cost of detouring around the bridge, representing the disbenefit of removing the bridge from service;
T = average daily traffic (ADT) on the bridge;
τ = truck percent, expressed as a number between 0 and 100;
L = detour length in kilometers (NBI Item 19). If this parameter is missing or 0, then the default value of 1 is used;
C_l^t = distance-based detour cost for trucks (dollars per kilometer) specified by functional classification;
C_h^t = time-based detour cost for trucks (dollars per hour) specified by functional classification;
C_l^c = distance-based detour cost for autos (dollars per kilometer) specified by functional classification;
C_h^c = time-based detour cost for autos (dollars per hour) by functional classification;
v_d = detour speed (kilometers per hour) specified by functional classification;
B = total user cost of detour;
N = number of years in the user cost accrual period; and
δ = discount factor.

Once NBI data have been imported into the NBIAS database, the system uses a set of models called synthesis, quantity, and condition (SQC) models for translating NBI bridge-level data into an approximation of element information. The SQC models evaluate information about a bridge from the bridge's NBI record, and estimate (or "synthesize") which structural elements would exist based on a mix of engineering and stochastic rules. Once the likely elements are identified, the quantity of the element that one would expect to find on a bridge is estimated. Finally, given information about the NBI ratings for the bridge, the conditions can be estimated using an adaptation of the approach previously developed by the University of Colorado (*5*). Additional information on the SQC models has been published previously (*6*). When elements are synthesized, they are further categorized based on the climate zone of the bridge [specified by county using Highway Performance Monitoring System (HPMS) definitions]. National defaults for element costs and transition probabilities (which quantify deterioration and action effectiveness) have been specified by climate zone through a separate effort.

Although only NBI data are required for NBIAS, if an agency has element-level data for all or some of its bridges, users may import these data directly into the system. Element data can be imported directly from a Pontis database or through file import. Because NBIAS models only the AASHTO Commonly Recognized (CoRe) elements (*7*), one must specify how any non-CoRe elements map to the CoRe elements, or they are excluded from the analysis.

Preservation Models

The objective of the preservation models is to find the optimal preservation policy for each bridge element that minimizes the long-term cost of maintaining the element in a state of good repair. To understand the model, it is important to understand the implication of optimal policy. Delaying a recommended action under optimal policy is costly in the long run, as there is some probability that a more expensive action will be required in the future if action is deferred. The net benefit of taking the recommended action is calculated as the difference in the life-cycle cost of the element if action is deferred for one period (1 year) and if the optimal action is taken. In NBIAS the preservation decision-making process is treated as a Markov decision problem (MDP). The basic approach is identical to that used in Pontis (*8*).

The primary difference in the NBIAS approach compared to Pontis is that user costs are considered for deck elements, using models adapted from HERS. This change tends to have the effect of triggering more aggressive deck preservation actions on high-traffic bridges. This approach to introducing user costs into the preservation models has been detailed previously (*9*). Further, NBIAS includes a set of state-specific cost adjustment factors calculated based on reported bridge replacement costs. The preservation models are solved for each element and for each state using these adjustment factors.

Functional Improvement Models

NBIAS models four types of functional improvement actions:

- Widening of existing lanes and shoulders,
- Strengthening,
- Raising, and
- Replacement.

For each of these the system first evaluates whether there is a potential need for functional improvement, then determines the feasibility of improvement, calculates the improvement cost, calculates the improvement benefit, and finally evaluates the benefit–cost (B/C) ratio of the improvement to determine whether the improvement is economically justified.

In determining whether there is a potential need for an improvement, NBIAS applies two sets of business rules: a functional improvement policy and a set of rules derived from the process of calculating sufficiency rating and structural deficiency–functionally obsolete (SD/FO status. The user may choose to use one or both sets of rules. The functional improvement policy in NBIAS is similar to that in Pontis; the policy specifies threshold values for lane width, shoulder width, load rating, and vertical clearance by functional classification. The second set of business rules is unique to NBIAS. Essentially, any functional issue that results in a deduction in sufficiency rating, or that results in classification of a bridge as SD or FO, triggers a potential need.

Based on this second set of business rules, the following conditions cause identification of a potential widening need:

- Deck geometry rating (NBI Item 68) is less than 6;
- Underclearances rating (NBI Item 69) is less than 6 as a result of the horizontal underclearance; or
- There is a deduction in sufficiency rating resulting from a width of roadway deficiency calculated as described in Part 2B of the sufficiency rating calculation process (*3*).

If the rules derived from calculation of sufficiency rating and SD/FO status are applied, then a potential vertical clearance deficiency is identified in the following cases:

- Vertical clearance is less than 4.26 m;
- Vertical clearance is less than 4.87 m and the roadway is on the Strategic Highway Network (STRAHNET, NBI Item 100); or
- Underclearances rating (NBI Item 69) is less than 6 as a result of the vertical underclearance.

Further, based on these rules a potential bridge strengthening need is identified if the inventory rating (NBI Item 66) is less than 32.4 metric tons.

Once a potential need has been identified, the next step is to determine whether it is feasible to perform the improvement on the bridge. The feasibility of the potential improvement is also driven by a set of business rules. (i.e., it is assumed to be infeasible to widen a truss bridge). If the rules for a given improvement type dictate that the improvement is infeasible, NBIAS will consider replacing the bridge as a last resort measure to address the potential need.

Depending upon the physical condition of the bridge, it may be the case that the only feasible alternative for the bridge is replacement. The user may define rules that specify when a replacement should be triggered independently of other considerations. These rules may be specified based on SD/FO status, health index, sufficiency rating, and bridge age. Although conceptually similar to rules defined in Pontis (*10*), the replacement rules in NBIAS differ in that they may be used to trigger only one action, replacement, and can use a broader set of condition measures as triggers than those supported in Pontis.

The next step in the process is to identify the cost of the potential improvement. The cost is determined by multiplying the unit cost for the improvement by the deck area that will be improved, considering the change in bridge dimensions that may result from the improvement in the case of widening or replacing a bridge. For replacement, if a bridge replacement cost has been specified for the bridge (NBI Item 94), this cost will be used rather than the calculated cost. This calculation is performed in a similar fashion as that in Pontis, except that the state–specific cost adjustment factors are applied to the calculation.

Following calculation of the cost, the system identifies the benefits of the potential improvement. The benefits are based on the following concepts:

- Widening needs are evaluated based on the reduction of accidents on the bridge.
- Raising and strengthening needs are evaluated based on the reduction in the amount of truck traffic detoured around the bridge.
- It is assumed that when a bridge is widened, raised, or strengthened all preservation needs will be addressed, so these benefits are added to the accident and detour reduction benefits.
- Replacement needs are evaluated based on the sum of the above benefits, with an adjustment to raising and strengthening benefits to prevent double counting. Further, replacement is assumed to return all elements to the best condition state. The reduction in life-cycle cost resulting from this is included as a benefit.

NBIAS supports two different models for calculation of functional improvement benefits: Pontis models (*8*) and more recently developed Florida Department of Transportation (FDOT) models that provide replacements for the Pontis models for predicting accident cost reduction as a function of lane–shoulder width, as well as for the distribution of truck size and weights (*11, 12*). In applying the FDOT models, the piecewise polynomial and exponential functions used to describe the truck size and weight distributions were replaced with logistic functions. In calculating benefits NBIAS uses user cost parameters (e.g., value of time, vehicle operating costs, crash costs) adapted from the HERS model parameters.

Once the costs and benefits of a potential functional improvement need have been calculated, NBIAS evaluates whether the bridge is still a candidate for improvement. A bridge becomes a candidate for improvement or replacement only if the ratio of the sum of its discounted user benefits over the planning horizon to the project cost is greater than or equal to a specified threshold (1 by default). To screen out projects with marginal benefits, NBIAS evaluates the following expression for all potential functional improvement projects (for a 30-year period by default):

$$R = \frac{\overline{B}_a^u \sum_{i=1}^{30} \delta^{(i-1)}}{C} \qquad (3)$$

where

R = B/C ratio;
$\overline{B}_a^u$ = Average annual user benefit of improvement or replacement calculated using the ADT at the beginning of the planning horizon;
C = Cost of the improvement; and
δ = Discount factor.

If *R* is greater than or equal to the specified threshold, the project becomes a candidate for consideration in the program simulation. Note that functional needs and, by extension, this expression, are re-evaluated for each year of a program simulation. Once a project is considered as a candidate in the program simulation, its 1-year benefits and incremental B/C ratio are calculated and the candidate competes for funding with other project candidates through the incremental benefit cost (IBC) analysis described below. Further, note that the user specifies whether or not the threshold described above is applied to replacements triggered by user-specified rules.

Scenario Simulation

In the course of a scenario simulation NBIAS applies the recommendations of the preservation policy and functional improvement models to the bridge population, generates bridge needs, and evaluates the costs and benefits of meeting those needs. NBIAS uses an IBC approach to simulate selection of projects year by year with the objective of maximizing agency and user benefits. Within each 1-year simulation cycle, NBIAS performs the following sequence of operations, illustrated in Figure 1.

- NBIAS applies the recommendations of the preservation policy and the functional improvement models to generate preservation and improvement needs for all bridges.
- NBIAS applies heuristic replacement rules described above.
- For every bridge, NBIAS combines improvement needs and preservation needs to generate a set of project alternatives.
- NBIAS screens out the alternatives with a B/C ratio less than the specified threshold (1 by default).
- Valid project alternatives for all bridges are combined and the entire set is sorted by IBC.
- NBIAS allocates funds to project alternatives, stopping either when the budget is expended or the minimum threshold for IBCR is reached.
- NBIAS then simulates the effect of the selected projects on bridge condition.
- For the bridge network as a whole, NBIAS simulates the effect of project selection on over 200 measures of effectiveness.
- Bridge conditions are then rolled forward into the next year, following which NBIAS starts the simulation cycle again. The simulation period may be up to 50 years.

A distinctive feature of NBIAS is that for a given scenario, the system is capable of performing parametric runs, using either the annual budget or the cutoff B/C ratio as the parameterization variable. The system performs one run using a user-specified budget for each analysis period.

When the scenario is parameterized by annual budget, the user must specify lower and upper limits for the annual budget, and the number of parametric steps N. NBIAS then performs $N + 1$ simulations, in addition to the first simulation described above, increasing the annual budget from the minimum to the user-specified maximum in N equal increments.

When parameterization is done by the cutoff B/C ratio, the user specifies the upper and lower limits for the cut-off ratio, and the number of parametric steps. NBIAS then runs the simulation assuming an unlimited budget, and stops allocating funds upon reaching project alternatives with B/C ratios below the cutoff.

The output of the parameterization process is a four-dimensional array of simulation results. The dimensions of this array are

- Measure of effectiveness (206),
- Simulation year (up to 50),
- 26 bridge categories obtained from permutation of 13 functional classes and either on/off National Highway System or on/off the Strategic Highway Network.
- $N + 2$ parametric steps.

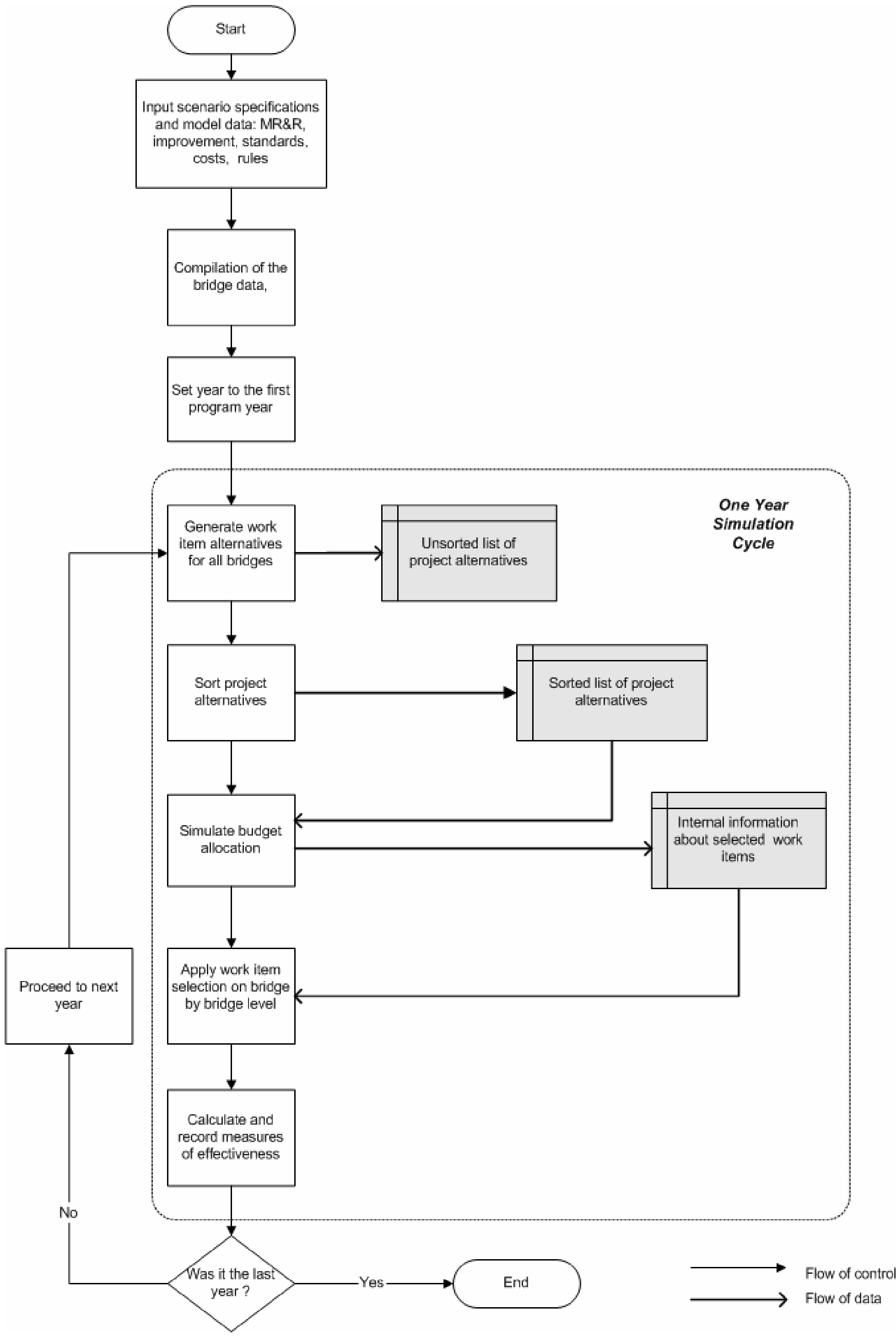

FIGURE 1 Program simulation flowchart.

The four-dimensional array of simulation results is loaded into the NBIAS database to support database queries and reporting, and saved as a binary file for use in generating the interactive views described below. One may save bridge-level results in the database, as well. Practically speaking, it is neither necessary nor feasible to store bridge-level results for all of the bridges in the United States. However, the detailed results may be useful as a diagnostic tool or to support state-specific analysis.

Measures of Effectiveness

NBIAS predicts 206 measures of effectiveness (MOEs). These are grouped into eight categories, as detailed below.

Needs

These represent work that the system would recommend if sufficient funding were available. Needs measures are reported by the type of work needed to meet the need. In calculating needs, the system sums the cost of all of the work that would be recommended on each bridge, given sufficient funding. In some cases there are multiple alternative projects defined for a particular bridge that may be justified, depending upon available funding. For instance, a bridge may have a preservation or improvement need, but there may be some additional benefit to replacing the bridge completely, given sufficient funding. In this case, the lower-cost alternative for the bridge would typically be defined as the structurally or functionally motivated need for the bridge, while the bridge replacement would be defined as an economically motivated need. NBIAS reports both of these types of needs.

Most needs are structurally/functionally motivated needs, and, generally speaking, a reference to “needs” in NBIAS refers to needs of this type. Economically motivated needs, which are exclusively bridge replacements, are reported for the sake of completeness. Needs measures are reported in units of currency (dollars) and numbers of bridges with a particular type of need.

Work and Backlog

These measures are closely related to needs. They represent the work that would actually be performed given a particular budget level. The backlog is the need that remains after the work is performed.

Work and backlog measures are reported either in units of currency (dollars) or numbers of bridges with a particular type of need. The backlog measures are split into two components: backlog incurred due to the budget constraints and backlog occurring because of the B/C ratio cut-off thresholds.

Benefits

A series of MOEs is defined for reporting benefits. NBIAS reports two types of benefits: obtained and potential. Obtained benefits are the benefits NBIAS simulates as occurring during a program simulation. Potential benefits are benefits that could be obtained if there were no budget constraints (or no B/C cutoff when parameterizing by B/C cutoff ratio).

Benefit–Cost Ratios

NBIAS calculates the average B/C ratio for work performed in each analysis period—overall and for different types of work. The average B/C ratios are weighted based on project cost. Also the system calculates the minimum B/C ratio (the B/C ratio cut-off) for each period.

NBI Condition Ratings

FHWA defines condition ratings for deck, superstructure, and substructure conditions in the NBI Coding Guide (*3*). These ratings, referred to in the system as NBI condition ratings, are based on a scale from 0 to 9, with 0 representing the worst condition and 9 representing the best. A series of MOEs is used to show the number of bridges with each rating.

Health Index

This MOE represents the overall condition of a bridge based on a weighted average of the condition of its elements, calculated using the approach originally developed by the California Department of Transportation (*13*). The health index ranges from 0 (the worst possible condition) to 100 (the best possible condition).

Sufficiency Rating

This is calculated based on FHWA guidelines. Sufficiency rating ranges from 0 (the worst possible condition) to 100 (the best possible condition). Generally speaking, the sufficiency rating is well-correlated with the health index. However, since the former considers functional issues and the latter does not, it is difficult to compare the two measures. A series of MOEs is used to report the average sufficiency rating and number of bridges within specified sufficiency rating ranges.

Structurally Deficient–Functionally Obsolete

NBIAS calculates the number of bridges and the percentage of deck area classified as SD or FO based on FHWA guidelines. Consistent with these guidelines, if a bridge meets the criteria for being both SD and FO, it is classified as SD.

SYSTEM IMPLEMENTATION

Figure 2 illustrates the NBIAS architecture, which consists of three main components: a database, the analytical module, and the what-if analysis module. The NBIAS database is a set of relational tables that contain data on a set of bridges, as well as additional information needed to perform an analysis. Import of NBI and element data into the database is depicted in Arrow 1 on the figure. The analytical module allows the user to define models and run budget scenarios

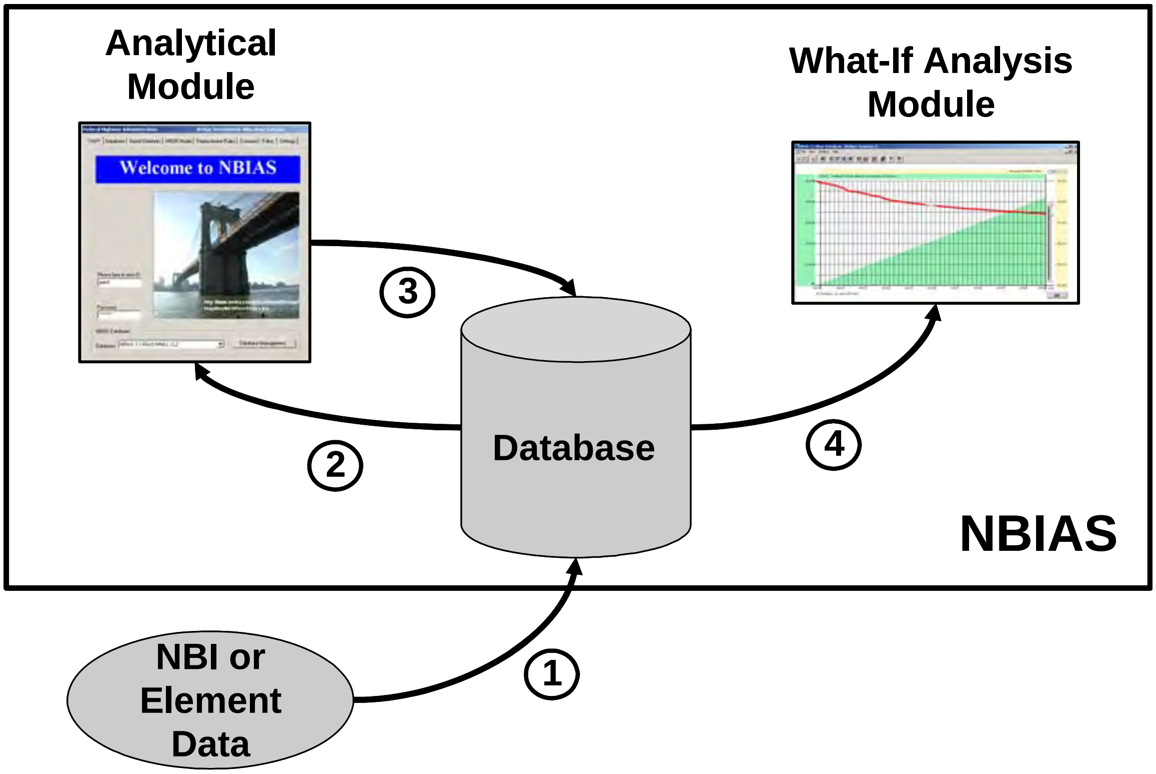

FIGURE 2 NBIAS architecture.

for analysis. It pulls data from the database (Arrow 2), generates a scenario, and stores the results back in the database (Arrow 3). A second module, the what-if analysis module, provides a variety of interactive views and reports which allow the user to see the results for a selected scenario, supporting access to the results in the database (Arrow 4).

Figures 3 and 4 show representative screens from the analytical module. Figure 3 illustrates definition of a preservation model. This example shows the model for Element 18 (concrete deck with a thin overlay) for Climate Zone 5 (intermediate/freeze–thaw), User Cost Group 4 (highest traffic bridges), and national average costs. The top portion of the screen defines what model is shown. The cost model, including user and agency costs for each defined state and feasible preservation action, is shown in the middle of the screen. Interpreting the actions (e.g., "Agency 1") requires reference to the CoRe Element Guide (*7*). Transition probabilities and the resulting model are shown at the bottom of the screen. Figure 4 illustrates the basic parameters defined when running a scenario. In this example, a 50-year simulation starting in 2007 has been defined, with 10 parametric steps to be run with annual budgets of $5 billion to $20 billion (budgets are shown in millions on the screen).

NBIAS results are displayed through a series of interactive views in the what-if analysis module. The views defined in the system, and the approach used to interpolating results between different parametric runs, have been described previously (*14*). Essentially, with the views one holds constant two or three of the dimensions of the results array described above, and the system displays the results in terms of the remaining dimensions. For example, one may specify a MOE, an asset group, and a time period, and view the results over time for different budgets.

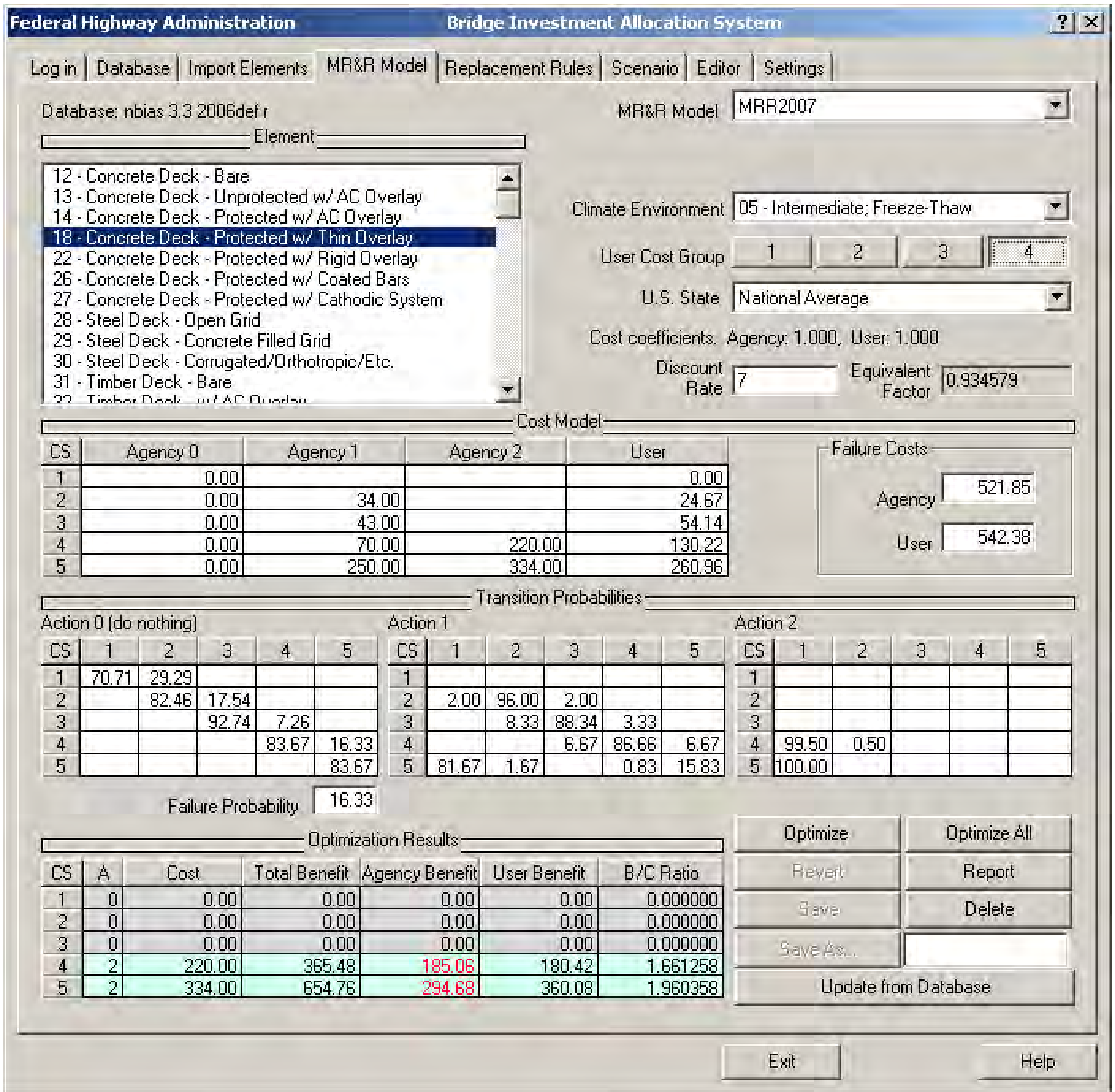

FIGURE 3 Preservation model in the NBIAS analytical module.

Figure 5 illustrates a multiple budget view with needs over time for four different annual budget levels. Generation of the views in the system typically requires less than a second as a result of the use of binary files for results retrieval and because the system interpolates network-level results between parametric steps.

As an alternative to the interactive views, one may generate tabular reports in the what-if analysis module. Figure 6 shows an example of report generation. Reports are saved as RTF and CSV files for viewing in Microsoft Word or Excel.

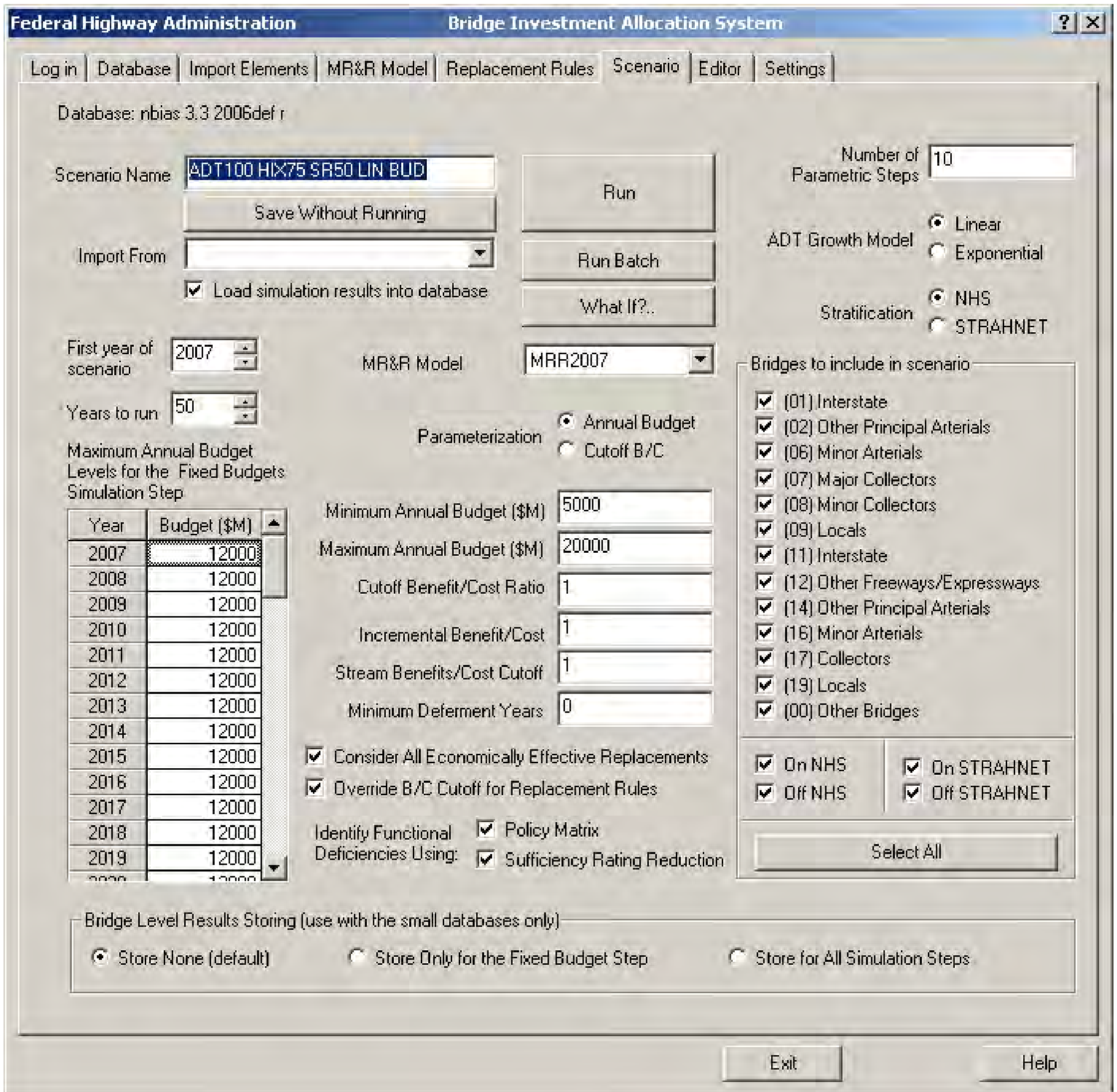

FIGURE 4 Scenario definition in the NBIAS analytical module.

CONCLUSIONS

In summary, NBIAS offers a comprehensive modeling approach for analyzing highway bridge investment needs at a network level. Key strengths of the system are its reliance on a fundamental approach first introduced by Pontis, its ability to perform analysis based solely upon NBI data, its use of a well-developed economic analysis approach designed to be consistent with that of HERS, and its use of interactive what-if views for analysis. Modeling changes made in recent versions of the system include: the classification of elements by HPMS climate zones (with default deterioration models for each zone), identification of potential functional improvement needs based on sufficiency rating and SD/FO status calculations; addition of FDOT functional improvement benefit models; support for heuristic replacement rules; specification of state-specific cost and benefit adjustment factors; and support for storing bridge-level details.

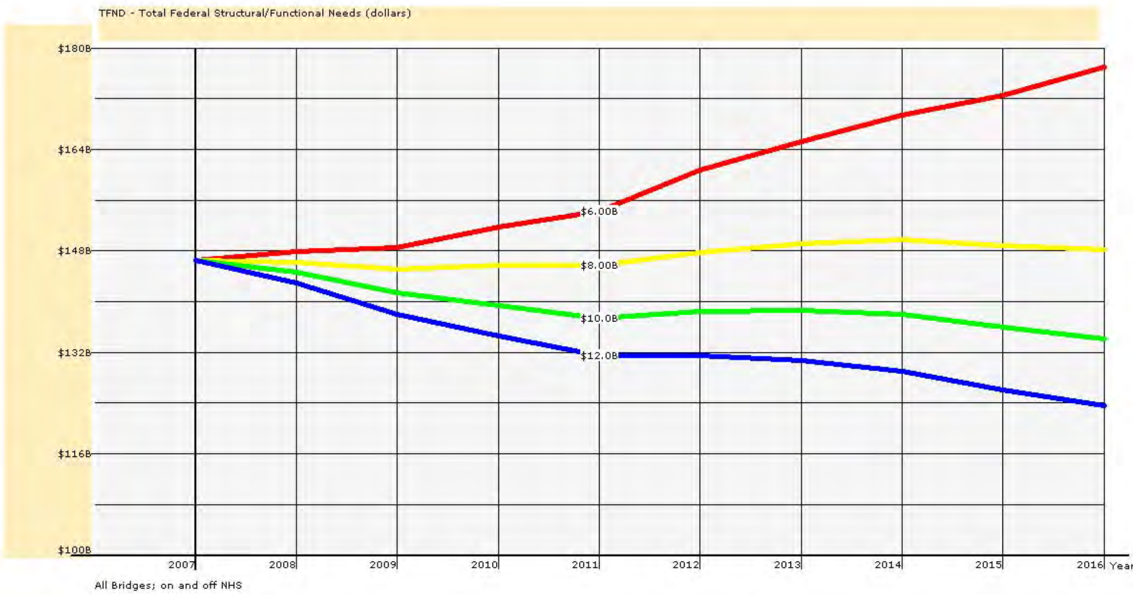

FIGURE 5 Sample multibudget view.

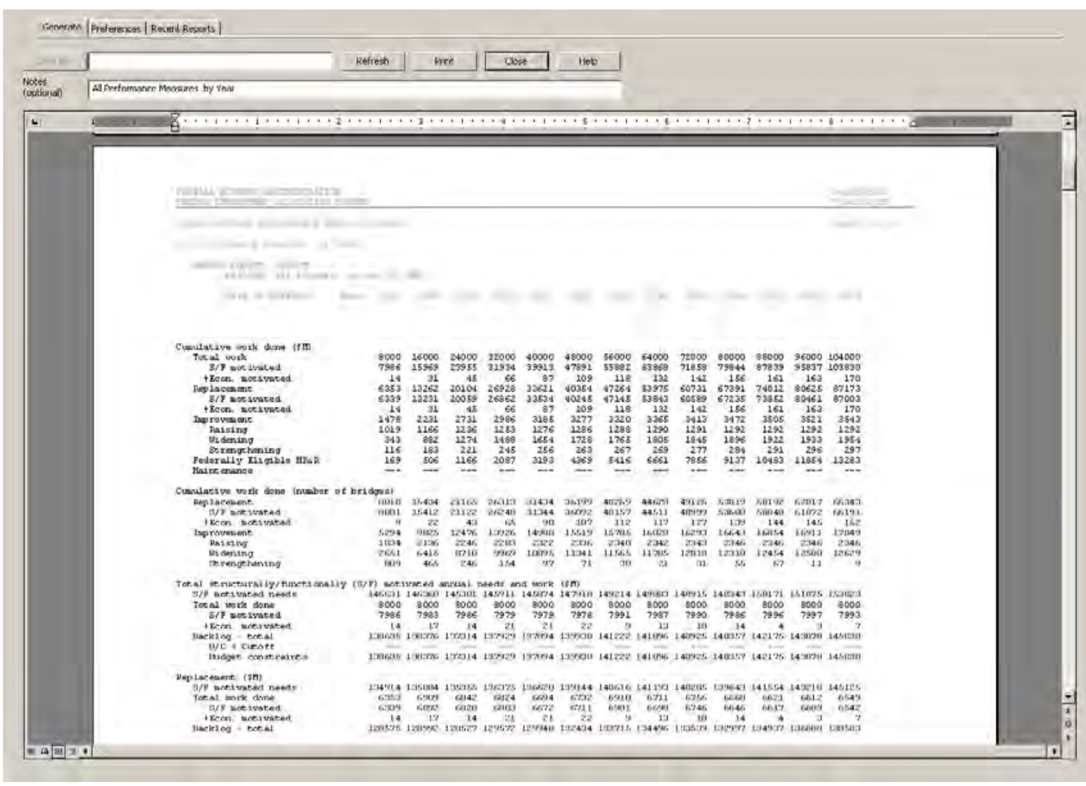

FIGURE 6 Sample report.

A number of U.S. states have used NBIAS for performing network-level needs analysis, and the tool is well suited for this purpose. Particularly if a state lacks element-level data needed for Pontis, or has not developed agency-specific cost and deterioration models, NBIAS may represent the most readily available system in the United States to support bridge investment analysis. Also, to the extent that agencies use HERS for analysis of highway investment needs, NBIAS is an obvious complement to such an analysis for bridge needs, as its costs and benefit models have been refined to support integration of HERS and NBIAS results. Despite these features, NBIAS is not a replacement for a BMS. It has no functionality for supporting the bridge inspection process, and though it can provide bridge-level results, is not designed to support development of detailed bridge-specific recommendations.

To help determine next steps for the system, FHWA has performed a peer review of the system pursuant to the Office of Management and Budget's Final Information Quality Bulletin for Peer Review, which stipulates that such reviews are required for any "highly influential scientific disseminations," a category that has been deemed to include NBIAS due to its use for helping develop the C&P Report. The peer review results are summarized in 2006 Status of the Nation's Highways, Bridges, and Transit: Conditions and Performance (*1*) and identified the following issues requiring further research.

- The system optimizes preservation actions based on the assumption that future decisions will be optimal. In cases where funding is constrained and an agency's practices differ from the optimal model, the solution identified by the model for any one year may be suboptimal.
- The system does not account for the effects of scour on bridges.
- It is unclear whether culverts should be included in the model, or if these are effectively handled by HERS.

Building on the peer review recommendations, a general area where further research is merited is in comparison of the NBIAS model recommendations to agency practice. Such research would inform further development of the modeling approach to better represent agency practice, including the characterization of costs and benefits that are not included in the system, but that may have a significant impact on agency decisions. For instance, anecdotal evidence suggests that many bridges are replaced as part of roadway expansion efforts (e.g., to add lanes) rather than as a result of physical condition. In such cases, one would ideally show that capacity expansion is justified both to provide mobility benefits not currently modeled by NBIAS and to address the preservation needs the system does now model, and that a preservation-only approach is suboptimal. A further area where more research is needed is the modeling of risk of bridge failure (conditions resulting in emergency repairs, posting of a bridge, or outright collapse). Currently the potential for failure is handled in only a rudimentary way, through modeling of the probability of failure in the worst condition state of an element.

ACKNOWLEDGMENT

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Bridge Information Modeling for the Life Cycle
Progress and Challenges

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Bridge management involves, to a large extent, a matter of bridge data management. At the project level, bridge project data management is complicated, unfortunately, by the proliferation of "stove-piped" software applications (and a variety of accompanying file formats) that have sprung up over the years without due consideration of functional interoperability that would accompany a well-thought-out leveraging of bridge data for multiple purposes through the entire bridge life cycle. This paper describes some of the potential means of leveraging bridge data from the design stage for more aspects of the life cycle than is typical in current practice. The intent is to demonstrate the viability of integrated bridge project delivery and life-cycle management via a prototype integrated system that illustrates representative data exchanges and applications throughout the bridge life cycle. The paper thus provides an update on current FHWA-funded research on this subject (Contract DTFH61-06-D-00037) being conducted by the authors.

Bridge management (BM) involves, to an increasing extent, management of bridge data. At the project level, bridge project data management is complicated, unfortunately, by the proliferation of "stove-piped" software applications (and the variety of accompanying file formats) that have sprung up over the years without due consideration of the advantages to be gained via functional interoperability that would accompany a principled leveraging of bridge data for multiple purposes throughout the bridge life cycle.

Advances in automation and communication technologies in recent years have been significant, but they have not yet been fully adapted and integrated with each other and then deployed to accommodate the unique requirements of the bridge management enterprise. This lack of deployment may be due in part to the lack of standards for representation and electronic exchange of bridge life-cycle data.

In the absence of industrywide nonproprietary standards for the representation and exchange of life-cycle bridge data, this paper describes and demonstrates some of the potential means and benefits of leveraging bridge data from the design stage for more aspects of the life cycle than is typical in current practice. The intent is to demonstrate the viability of integrated bridge project delivery and life-cycle management via a prototype integrated system that illustrates representative data exchanges and applications throughout the bridge life cycle. The paper will provide an important update on current FHWA-funded research on integrated bridge project delivery and life-cycle management (Contract DTFH61-06-D-00037) being conducted by the authors.

This project is motivated by the recognition that the current U.S. practice of information transfer during the bridge planning, design, fabrication, construction, operation, and maintenance processes involves repeated manual transcription of data that is error-prone, and time-consuming approvals (e.g., of shop drawings) and a lack of standardized formats hinder electronic information transfer. It is also recognized that without such standards, electronic information exchange is cumbersome at best, and often not possible. This paper presents current research to address this challenge under FHWA sponsorship to develop a program to explore the promise of parametric 3-D bridge information modeling (BrIM) as a technology that will enable not only acceleration of the bridge design and delivery (*1*) but also leveraging of that design-and-construction stage bridge data to enhance life-cycle management of the as-built bridge.

This paper thus provides a significant update to an earlier paper by Shirolé (*2*), that envisioned a confluence of various then-emerging automation and communication technologies to support bridge management functions.

PROJECT OBJECTIVES AND DEVELOPMENT

The purpose of this FHWA project is to explore the promise of parametric 3-D BrIM as an enabling technology not only for accelerating the bridge design and delivery, but also to enhance life-cycle management. It intends to articulate aspects of the envisioned accelerated bridge delivery process and to provide insight, partly via implemented demonstrations, to illustrate technical viability and potential benefits into the current technologies that are available to streamline the process of bridge delivery. It also explores the leveraging of bridge data generated thereby into “downstream” uses during the operations and maintenance aspects of the bridge life cycle, as illustrated in the context of a recently built, three-span tangent grade-separation structure. It is these aspects that are the principal focus of this paper.

Although at the present time there are no nonproprietary standards for electronic exchange of life-cycle bridge data, it is the vision of this project to facilitate the development of an integrated system for the entire bridge life cycle, i.e., from cradle to grave. This proposed project is the first step toward a streamlined approach to accomplish its overall program objective. In the future, a complete modeling of bridge information in a standardized format can facilitate integration of computer-aided design, computer-aided engineering, computer-integrated manufacturing, and a much more comprehensive BM that will enable not only rapid and better quality project delivery but also subsequent cost effectiveness of decisions through the life cycle. All three fundamental objectives of bridge delivery, namely higher quality, faster delivery, and more economical cost over the bridge life cycle, will then be attained.

Historically, in the development of various computational tools for supporting these various aspects (e.g., planning, design, detailing, estimating, fabrication, construction project management, bridge operations, and BM), individual aspects were typically addressed in stand-alone fashion without sufficient regard for complications arising from multiple data sources. Some of these complications involve the need for tedious, manual, error-prone re-entry of duplicate data into several software stovepiped applications. Instead if a coordinated shepherding of data supporting these individual applications were developed, bridge data integrity would be more easily maintained, and hand-off processes from one application to another would be streamlined if not made altogether seamless.

Just such an attempted coordinated shepherding of bridge data is shown in Figure 1, which shows a conceptual view of the organization of representative aspects of the bridge life cycle and how they each depend on bridge data represented in the center of the diagram. A coordinated handling and leveraging of that data could prevent the proliferation of problems resulting from multiple (potentially inconsistent) sources of data for either a given bridge or a population (network) of bridges and the routes they connect.

OPERATIONS AND MANAGEMENT ASPECTS OF THE BRIDGE LIFE CYCLE

The bridge data that is inherited by the bridge owner upon completion of the construction of the bridge is taken herein as the logical starting point for subsequent updating of the portions of the bridge project data that are relevant for the various aspects of bridge operations and management (e.g., bridge rating, posting, overweight vehicle permitting and routing, updating of bridge data for rating and posting based on inspection data management, and capital program planning and budget projections). These bridge operations and management aspects, as distinct from the design and construction aspects shown in Figure 1, are the principal focus of this paper.

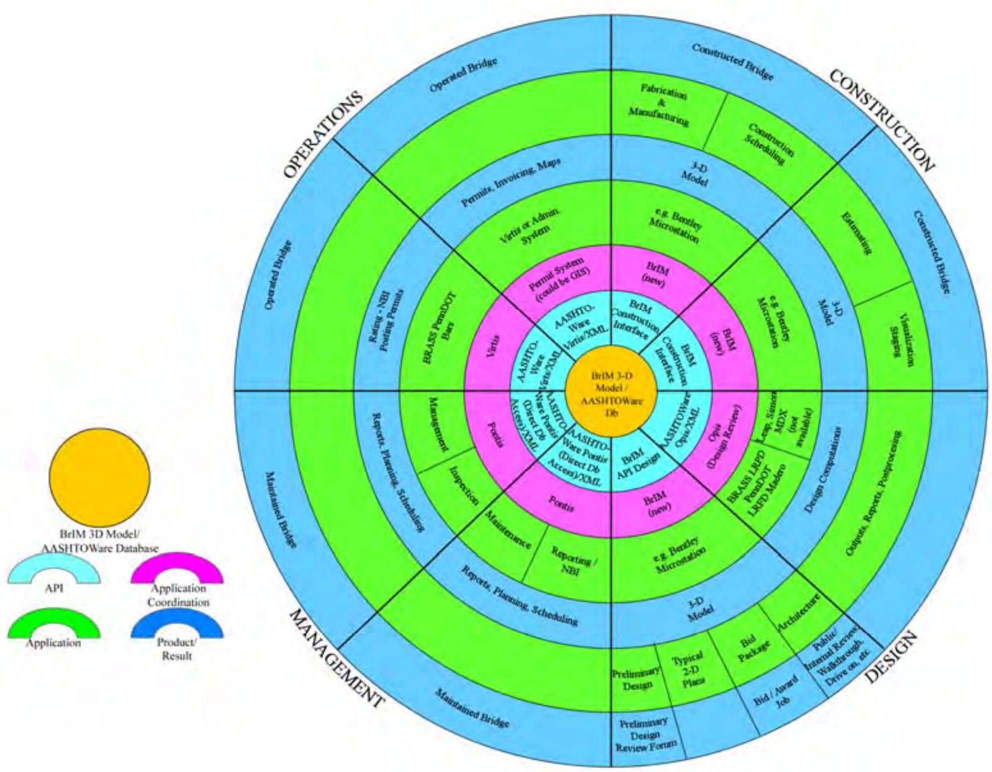

FIGURE 1 Data model-centric view of the bridge enterprise.

Bridge Data From Design and Construction Stage

Figure 2 shows a 3-D view of a portion of the computer bridge model that would have been used to generate design and construction information, including contract plans and shop drawings. That model has its geometry generated so that its geometry is entirely consistent with the roadway stationing, plan, and profile on the bridge site. Drawings in turn are merely extracted sections from such a model, an example of which is shown in Figure 3.

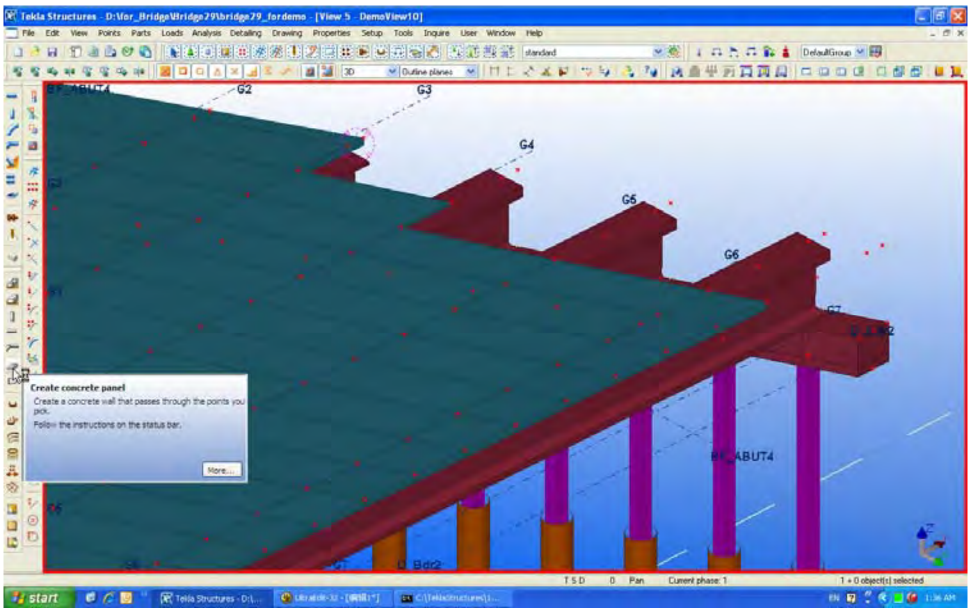

FIGURE 2 3-D view of a portion of the bridge computer model.

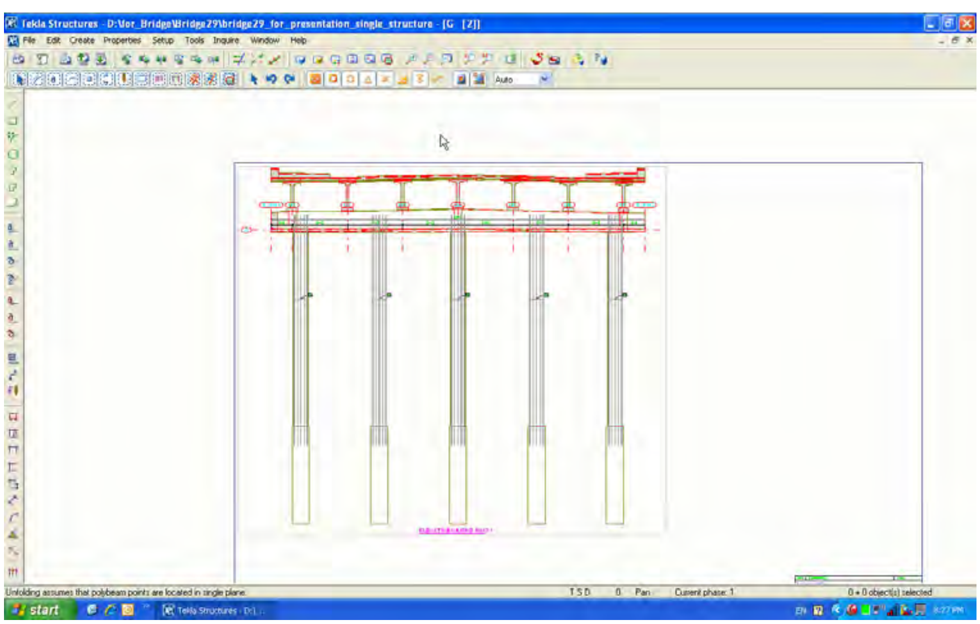

FIGURE 3 3-D bridge cross-section drawing extracted directly from 3-D model.

Bridge Operations: Load Rating

Load rating of bridges constitutes a central recurrent task during the operations phase of the bridge life cycle. The starting point for the bridge model suitable to drive load rating processes can be the bridge model inherited from the design and construction stages. But it is, of course, critically important that the bridge model data on which a load rating is based be updated based on recent inspection data to reflect current condition changes that may influence the load rating process. Section loss data and removal of questionable strands are examples of such updates.

In an integrated life-cycle environment envisioned herein, having the same model shared between design (checking) and load rating is a significant advantage. Bridge data (e.g., girder section data, material properties) thus need not be re-entered in order to conduct load rating, and multiple copies of such data do not proliferate and thereby require disambiguation. Updates (e.g., section loss) to the data have clearly defined access mechanisms and are reflected in the common AASHTOWare and BridgeWare database. For electronic data exchange purposes, that database is, in turn, accessible via either XML or the BridgeWare Application Programming Interface (API) which have been defined and designed to facilitate third-party access to the bridge data in order to leverage it for various purposes.

Figure 4 shows a view of one of the screens in Virtis used to verify the data already in the model defining the bridge girder geometry. Associated screens would enable adjustment of any data needed to reflect recently observed in situ conditions prior to executing a load rating operation.

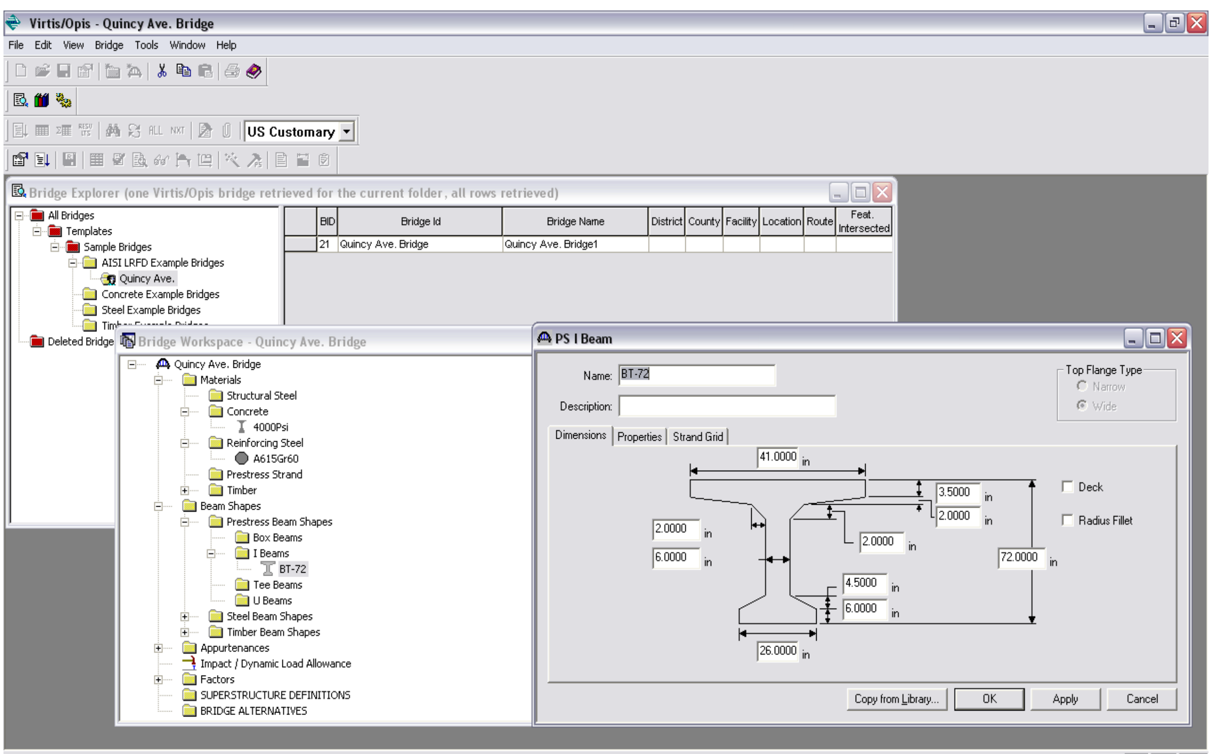

FIGURE 4 Checking and updating bridge beam definition in Virtis.

Figure 5 illustrates one of the screens illustrating an output from the load rating exercise. Having XML-based or BridgeWare API-based linkage software facilitates the execution of this load rating exercise by avoiding tedious time-consuming error-prone bridge data re-entry where such is not needed while in no way excluding the responsible bridge rating engineer from the work flow.

Bridge Operations: Routing and Permitting

Overweight and special vehicle routing and permitting requests constitute a significant and ongoing burden on surface transportation system owners. The software applications dealing with such requests rely not only upon the very same bridge data used for a bridge rating calculation but also upon the following:

- Software application capabilities to define custom loads (axle spacing, axle loads, etc.), and rate the bridge against such loads instead of the standard HS-20 or HL-93 type notional loads; and
- Interfacing to highway routing option data and the bridges located on those candidate routes, all of which will need to be rated in "batch" fashion for custom loads.

To the extent that BridgeWare-compatible data is defined for such bridges, there is no reason not to include the bridge network data and the applications utilizing such data in an integrated environment. By linking, such functionality would enable the following aspects of the routing–permitting work flow: online application for permit vehicle, bridge rating analyses and resulting travel routing, permit issuance, and secure permit payment processing. Figure 6 shows one of the software screens used in the permit application process (*3*), and Figure 7 shows resulting vehicle routing selection (courtesy of AASHTOWare).

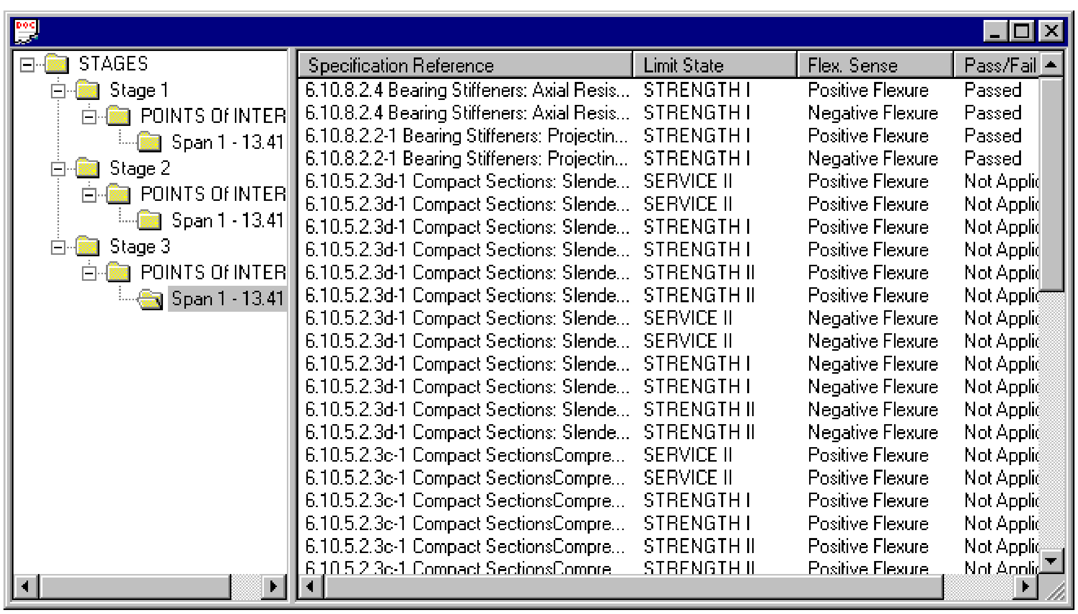

FIGURE 5 Detailed spec computation for bridge load rating report.

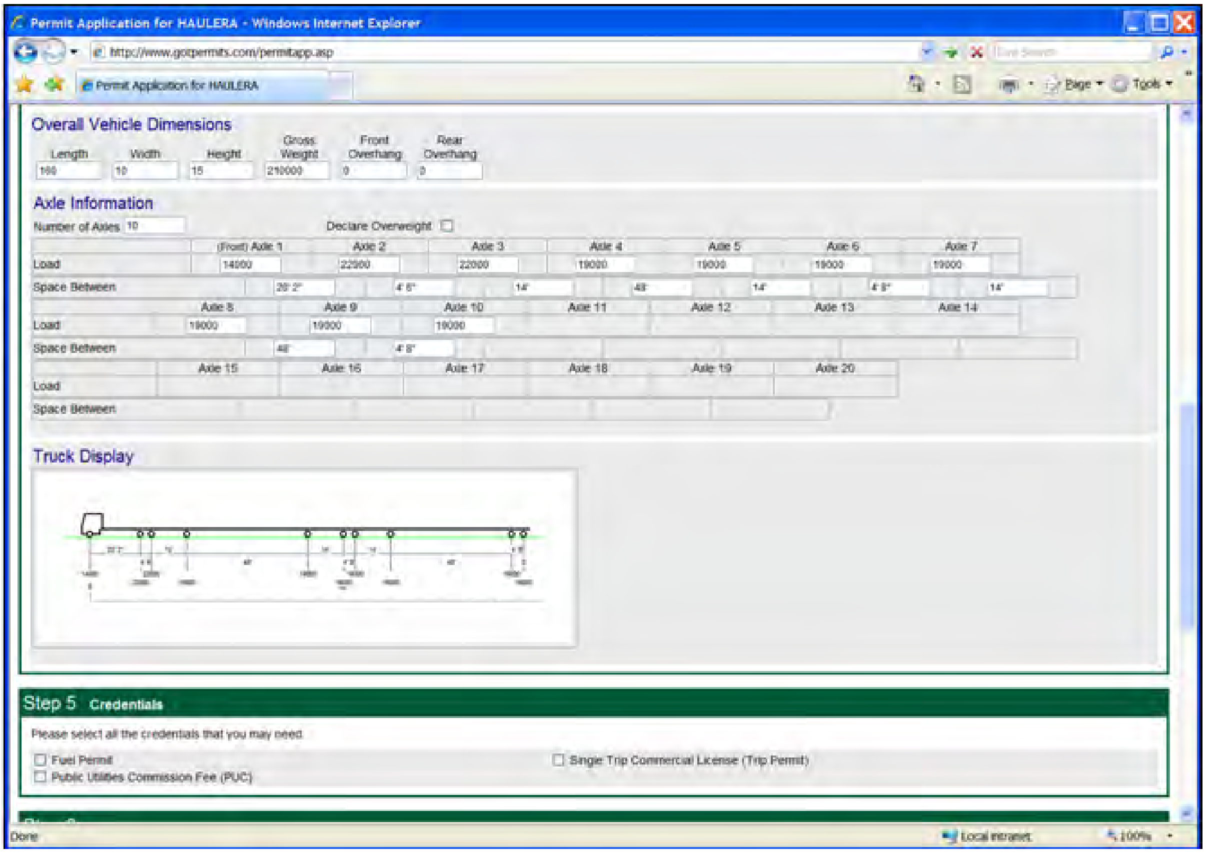

FIGURE 6 Permit application.

FIGURE 7 Route selection.

Bridge Management: Inspections

The records management burden associated with maintaining biennial bridge inspection data is significant. The Pontis software provides software support at the project level for these data via the BridgeWare database, whose ability to share with Opis (for design checks) and Virtis (for rating calculations) provides significant potential advantages to bridge owners. It makes sense for a life-cycle–integrated approach to bridge asset management to make use of this direct data-level support, which eliminates the need for software interoperability architectures in this particular portion of the envisioned integrated environment.

In this segment (project-level bridge inspection data management) would be found support for, e.g., National Bridge Inventory condition ratings report generation. As data requirements for such reports evolve over time, the overall bridge life cycle information management framework envisioned in Figure 1 would need to evolve with it. Figure 8 (courtesy of AASHTOWare) shows one of the bridge inspection data screens provided by Pontis.

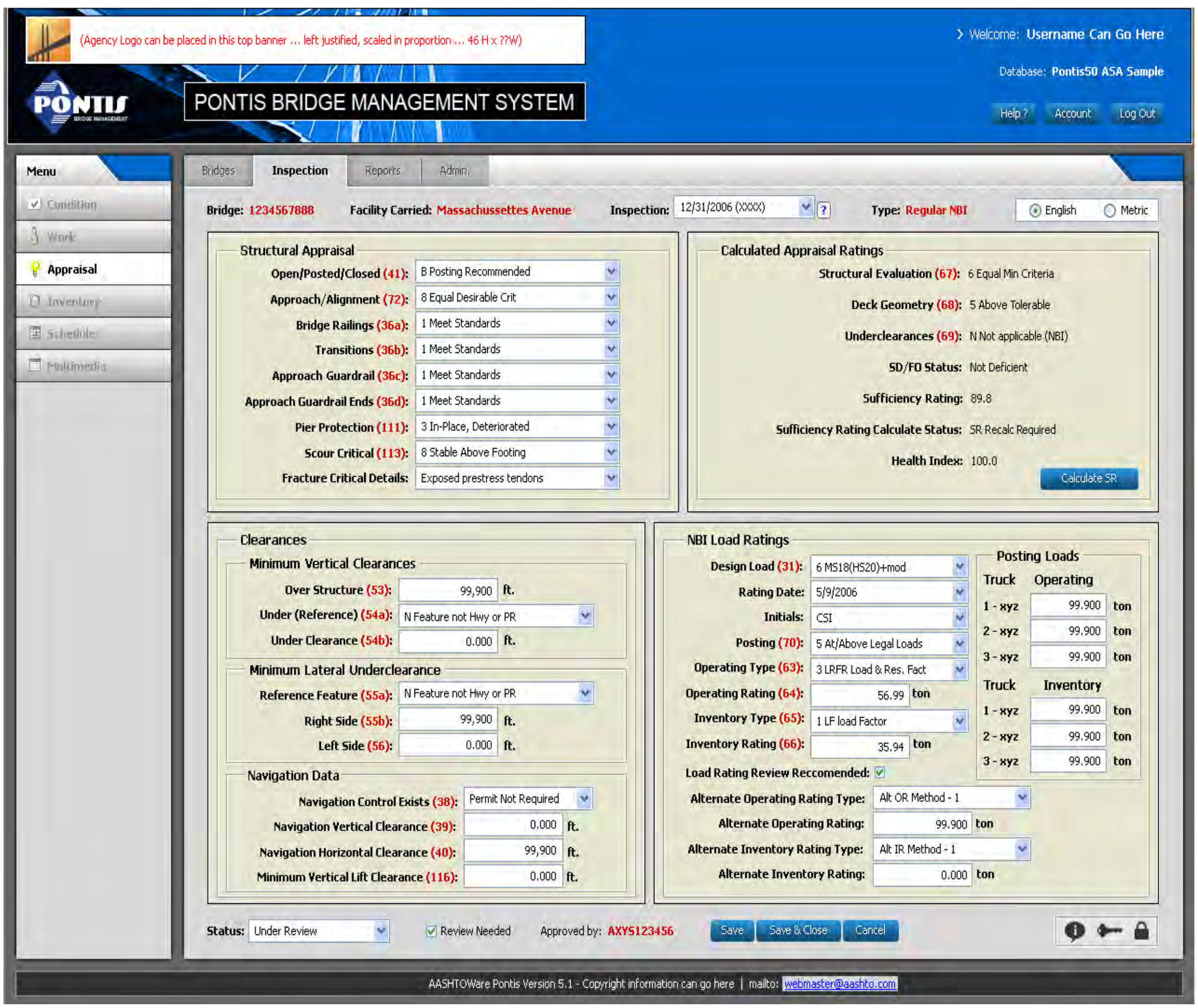

FIGURE 8 Illustrative bridge inspection data screen in Pontis.

Bridge Management: Programming

Although network-level BM functionality is typically considered distinct from the project level, access to both is provided by Pontis. To the extent that the bridge network is suitably defined in the BridgeWare database, further leveraging of that bridge data is provided by the fact that it is in a common format to support not only design and rating but also both project- and network-level BM support. As with the inspection data described above, this direct data-level support eliminates the need for software interoperability architectures in this particular portion of the envisioned integrated environment.

For example, access to the network-level bridge inspection (condition) and rating (load-carrying capacity) information enabled by Pontis would support the decision-making needs associated with exploration of performance–funding tradeoffs in planning capital program allocations. Benefit–cost investigations for various funding or program scenarios could then be pursued Figure 9 (*4*) envisions a graphical user interface for accessing such data for explorations of this sort, intended to explore trade-offs between performance and funding for either the entire bridge inventory or an appropriate subset of it.

A LOOK INTO THE FUTURE

During the past two decades, significant progress has been made in the area of bridge data collection, storage, and retrieval, techniques and methodologies for data analysis, and decision support. The project- and network-level bridge needs are being considered not just based upon condition, but also in terms of their vulnerabilities to catastrophic failure. As a result, the BM systems' capabilities are being enhanced.

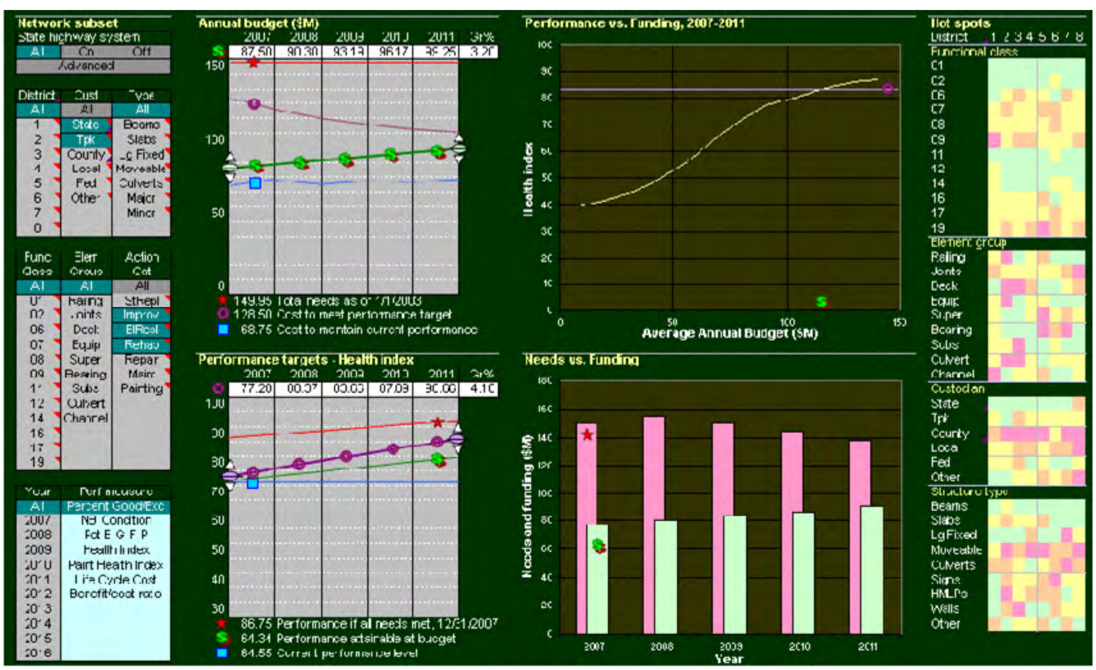

FIGURE 9 Dashboard mockup for network-level decision support.

The recently initiated FHWA Long-Term Bridge Performance Program and the ongoing NCHRP Project #14-15: Development of National Database for Bridge Maintenance Actions can be expected to provide very useful information for managing the bridge network. Specifically, one can expect these activities to provide quantitative information regarding to what extent different factors affect condition to help better model bridge deterioration as well as service life prediction of preservation and maintenance actions.

The rapid advances in the automation and communications technologies will continue their march. With the Internet access via mobile phones already widely used, real-time audio-visual communications and real-time monitoring capabilities will further aid timely management actions. These real-time capabilities will greatly benefit timely agency response to emergencies and in incident management.

To date, the seamless flow of design, construction, operations, and management information, which has been much needed, has not been possible. There is reason to believe, however, that sooner or later consensus on industrywide standards for interoperability will emerge. One can expect that this proof-of-concept FHWA project will have paved the way to a truly comprehensive and cost-effective approach to overall bridge life-cycle management.

SUMMARY AND CONCLUSIONS

With a focus primarily on bridge operation and management aspects, this paper has described an overview of current ongoing work to conceptualize and demonstrate key aspects of BrIM for the life cycle, a comprehensive shepherding of bridge design, construction, operation, and maintenance data to span the entire life cycle of the bridge enterprise. Highlights of this approach and accompanying software demonstration include the following:

- Use of a comprehensive cradle-to-grave view of the data is needed to support bridge life-cycle activities;
- Proliferation of tools and technologies (e.g., XML, APIs) which make possible reliable electronic exchange of bridge data in support of life-cycle applications, although resulting solutions are then necessarily ad-hoc; and
- Demonstrations of implemented software linkages in the context of two three-span straight grade separation bridges.

Further developments of integrated approaches for bridge life-cycle data management beyond the current demonstration stage will include

- Nationwide quantitative database information for maintenance actions,
- Accounting for vulnerabilities due to catastrophic events, as well as to deteriorating condition,
- Incorporation of real-time communication and monitoring capabilities, and
- Improved coordination that will be enabled by industrywide consensus standards in support of improved interoperability.

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BRIDGE MODELING AND NBI TRANSLATOR

A Discussion on the Efficiency of NBI Translator Algorithm

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The National Bridge Inventory (NBI) database is an extensive source of information on highway bridges in the United States. Among more than 100 NBI elements—deck, superstructure, substructure, and culverts—condition ratings are of special interest for bridge engineers and managers. The data for these condition ratings come from biannual bridge inspections in the field. As a part of their bridge management programs, many states have been collecting element-level condition data (mostly Pontis inspections) for more than 15 years. Element-level data provide more detailed condition data on sub-elements of the aforementioned general NBI element categories. Due to having such detailed condition data at hand, there has been an interest in developing algorithms that have the capability of estimating the NBI condition ratings from the Pontis element inspection data. If a sound estimation tool could be developed, the biannual NBI inspections done for these condition ratings would be deemed unnecessary. The NBI Translator is one of the algorithms that have been developed to achieve that goal and also works as a built in module within Pontis. Recently, there has been some concern on the degree of accuracy of this algorithm by users of both Pontis and the translator. This paper presents a literature review on bridge management systems and bridge inspections in the United States. In addition, background on NBI Translator algorithm and discussions on the efficiency of the tool are provided. A comparison study between the generated and actual values of the NBI ratings for the bridges in Iowa is also included. The paper concludes with a discussion on how to improve the algorithm and use the translated results in a simplified network-level tool for bridge management decision making.

In the past 40 years, there has been a shift from constructing new infrastructure to maintaining and managing the built infrastructure in the United States. Assessment of the deficiencies for the nation's infrastructure gained significant importance during this period. As the infrastructure gets older, more resources are required to maintain it at an acceptable level of service. Since the funds eligible for maintenance and rehabilitation activities are limited, effective resource allocation is now more critical than ever. Agencies are required to keep condition data on their pavements, bridges, and other infrastructure elements and justify their reasons for project selection, resource allocation, and funding requests.

As an important segment of the infrastructure system, bridges and their management have also been in the spotlight for the past four decades. Unlike pavements, the failure of bridge structures may result in disasters. Agencies in the United States learned from some incidents in the past and started implementing an extensive and comprehensive approach to bridge management.

Biannual National Bridge Inventory (NBI) rating is an effort to support bridge management and to form a basis for funding of bridge improvements in the United States. Agencies have also been collecting more detailed condition data for their bridge management systems (BMS). Modeling NBI ratings from the detailed element-level condition data has been a topic of interest because of the significant resource savings it will facilitate (*1*). There have been efforts but the degree of efficiency of the models is a discussion subject.

BRIDGE INSPECTIONS AND BMS IN THE UNITED STATES

On December 15, 1967, the Silver Bridge on U.S. Highway 35 suddenly collapsed into the Ohio River during rush hour (*2*). At the time of this tragic event, there were 37 vehicles crossing the bridge, and 31 of them fell down to the river. Forty-six lives were lost during this event, and nine people had severe injuries (*3*). In addition to the loss of life, an important road connecting West Virginia and Ohio was no longer in service. The catastrophe evoked concern over the reliability of the national network of bridges in the United States.

The 1968 Federal-Aid Highway Act put the states in action to collect and keep an inventory for federal-aid highway system bridges. In the early 1970s, the National Bridge Inspection Standards (NBIS) that form the basis of bridge inspection and inventory in the United States today were developed and implemented by FHWA. This legislation guided the data collection on bridge condition all over the nation. The failure of the Mianus River Bridge, Connecticut, in 1983 and Schoharie Creek Bridge, New York, in 1987 were other two unfortunate events after the collapse of the Silver Bridge that drew attention to the importance of keeping the nation’s bridges in sufficient condition and keeping up-to-date condition data (*4*).

In general, bridges are inspected every 2 years, and the condition ratings are reported to the FHWA. The inspection data are compiled by the FHWA into the NBI. After the analysis of the data, reports on bridge conditions are prepared and submitted to Congress. Decisions on the distribution of federal funding through programs such as Highway Bridge Replacement and Rehabilitation Program are based on these reports (*5*).

In addition to the biannual NBI inspections, many states also collect element-level bridge condition data for BMSs. Along with the Intermodal Surface Transportation Efficiency Act of 1991, which required the states to develop and implement BMSs, most of the states realized the importance and advantages of implementing BMSs. Although development of BMSs was made optional later in 1995 by the National Highway System Designation Act, many states decided to implement BMSs and took action (*6*). Forty-eight states were reported to be implementing a BMS as of September 1996 (*7*). Efforts to develop efficient national bridge management tools encouraged research in the area. A research project initiated by FHWA resulted in the development of Pontis BMS which later became the most popular bridge management tool in the United States. Forty-two states reported that they considered implementing the Pontis BMS. A few states preferred to develop their own BMSs (Pennsylvania, Alabama, New York, and North Carolina). The State of Maine implemented BRIDGIT which was developed as a result of a NCHRP Project (*6*).

PONTIS BMS

As previously stated, Pontis (*8*) is the most commonly used BMS in the United States that aims to help transportation agencies in the decision-making process regarding maintenance, rehabilitation, and replacement of bridge structures. Agencies are now aware that the aging highway system has considerable improvement needs; however, funding resources are limited.

Therefore, they need to make the best possible decisions for improvement, and these decisions should be based on facts. Pontis input data structure is a relational database that contains complete bridge inventory and inspection data. FHWA and AASHTO adopted Commonly Recognized (CoRe) Elements for Bridge Inspection in order to standardize element-level condition data collection within the United States. Bridges are presented by the CoRe elements in Pontis, and percentage of condition states for bridge elements are inspected and stored in the database. For each bridge element, specific condition states and related deterioration models were developed. Based on this detailed element inspection data, the program keeps track of current condition, simulates future condition, identifies bridge- and network-level needs and makes project recommendations in order to gain maximum benefits from scarce funds.

Although Pontis has been extensively used for maintaining bridge element condition data inventory, not all states benefit from the tool for resource allocation and identifying future projects literally for the time being. Implementing a BMS is a big organizational change, and it takes time to prepare the organization for such a strategic change and requires a substantial effort to customize Pontis parameters for the proper implementation of the cost, deterioration, and decision-support rules.

NBI CONDITION RATINGS AND BRIDGE ELEMENT CONDITION DATA

The FHWA Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges (Coding Guide) helps inspectors with the data collection process. States are encouraged to use the Coding Guide for standardization purpose (*5*). The Structure Inventory and Appraisal Sheet lists the NBI items necessary for inspecting individual structures, and these items can be divided into three main categories: inventory items, condition rating items, and appraisal rating items. NBI condition rating for an element is an evaluation of its current condition when compared to its new condition. In order to make the NBI condition ratings as objective as possible the inspectors are provided with the general condition rating guidelines listed in Table 2. NBI condition rating elements are different from BMS elements. Three subsystems of bridges and culverts receive overall condition ratings in NBI inspections (*5*):

- Item No. 58, Deck;
- Item No. 59, Superstructure;
- Item No. 60, Substructure; and
- Item No. 62, Culverts.

While the NBI condition ratings are assigned according to the 0 to 9 scale given in Table 1, element-level data collected for BMSs are assigned on a scale of 1 to 3, 1 to 4, or 1 to 5 based on the particular element. Of 106 CoRe bridge elements, 21 CoRe elements describe bridge decks, 35 CoRe elements describe superstructures, 20 CoRe elements describe substructures, and 4 CoRe elements describe culverts. In addition, smart flags are defined to describe special defects in miscellaneous bridge elements such as each beam, column, or girder. The rest of the CoRe elements are a variety of items such as bridge railings, joints, or bearings (*1*). Condition state 1 for an element is the best condition while condition states 3, 4, or 5 present the worst conditions for particular elements. In Table 2, condition state definitions of unprotected

TABLE 1 NBI General Condition Rating Guidelines (*5*)

Code	Description
N	Not applicable
9	Excellent condition
8	Very good condition (no problems noted)
7	Good condition (some minor problems)
6	Satisfactory condition (minor deterioration in structural elements)
5	Fair condition (sound structural elements with minor section loss)
4	Poor condition (advanced section loss)
3	Serious condition (affected structural elements from section loss)
2	Critical condition (advanced deterioration of structural elements)
1	"Imminent" failure condition (obvious movement affecting structural stability)
0	Failed condition (out of service)

TABLE 2 Condition State Definitions of Unprotected Concrete Deck

Code	Description
1	No damage
2	Distress ≤ 2%
3	2%–10% distress
4	10%–25% distress
5	Distress ≥ 25%

concrete deck from Pontis element configurations are provided as an example (*9*). The percentage of an element in each defined condition state is recorded during Pontis inspections.

The Pontis condition inspection data with extensive detail down to each individual element made agencies and experts in the field question the redundancy of NBI inspections for the same inspected bridges. Pontis inspection results provided agencies with much more detailed condition data for the aforementioned NBI items. Using the more detailed element-level data at hand for other bridge management requirements when possible is essential because data collection is a time- and resource-consuming process. For the year 1986, NBI costs were estimated to be approximately $150 to $180 million (*10*). Although NBI data and Pontis inspection data have discrepancies in item definition and rating scales, researchers have been trying to make a translation from bridge-element condition data to high-level NBI ratings to reduce the huge cost and time spent for data collection (*11, 12, 13*) and improve the consistency of the results. Hearn et al. (*11*) developed an estimator model for the purpose which was later developed as a software tool known as the NBI Translator or BMSNBI. The Pontis program has this software tool as a built-in module, and the tool can be used for the translation of NBI condition ratings from a defined set of element condition states for specified bridges in the Pontis environment.

NBI TRANSLATOR

The NBI Translator was developed at the University of Colorado at Boulder, with the collaboration of Colorado Department of Transportation (DOT) (*11, 13*). The translator generates condition ratings for deck (Item 58), superstructure (Item 59), substructure (Item 60), and culverts (Item 62) "by linking CoRe elements to corresponding NBI fields and mapping bridge management system condition states to NBI rating scale" (*11*). Bridge inspection data that contains both the NBI ratings and element-level condition state data of approximately 35,000 bridges were used to calibrate the NBI Translator (*13*).

Generation of NBI condition ratings is realized in four main steps (*13*). First, CoRe elements are grouped into matching NBI fields. Then, NBI condition ratings are generated for individual elements based on the quantities of that element in the different condition states. This table-driven procedure is shown in Table 3 (adapted from *13*).

Hearn, Cavallin, and Frangopol (*13*) describe the table-driven element NBI generation as follows. Percentages of element quantities in condition states are denoted by P_i and taken from element inspection records. Each row in Table 3 checks the sum of percentages for a minimum required sum. These minimum required sums, denoted by $M_{i,j}$, are called mapping constants. As previously mentioned, number and definition of condition states differ for CoRe elements for each material and use. For example, the condition states for steel deck are different from reinforced concrete deck. Overall, 20 different maps are required for generating NBI ratings. The four requirements for each NBI rating should be satisfied at the same time to assign that particular NBI rating to that particular element. The calibration process estimates these mapping

TABLE 3 Table for NBI Generation modify according to the guide

Requirements on Element Quantities			NBI Rating
P_1	≥	$M_{1,9}$	9
$P_1 + P_2$	≥	$M_{2,9}$	
$P_1 + P_2 + P_3$	≥	$M_{3,9}$	
$P_1 + P_2 + P_3 + P_4$	≥	$M_{4,9}$	
P_1	≥	$M_{1,8}$	8
$P_1 + P_2$	≥	$M_{2,8}$	
$P_1 + P_2 + P_3$	≥	$M_{3,8}$	
$P_1 + P_2 + P_3 + P_4$	≥	$M_{4,8}$	

constants. After assigning the NBI ratings for all elements, NBI ratings for each item (deck, superstructure, substructure, and culverts) are calculated by a weighted combination of element ratings. While the weights for deck and superstructure fields are based on relative quantity, the weights for substructure field are based on number of spans. Finally, NBI condition ratings are modified based on the smart flag condition reports. Smart flags may reduce the NBI ratings by a maximum of three points.

The objective of the calibration process is to find the mapping constants that will lead to the minimum difference between the NBI ratings given by inspectors and the generated NBI ratings from the element condition data.

Discussions on the NBI Translator Algorithm

Although the PC-based version of the NBI Translator algorithm has been available since 1994, the traditional NBI inspections for bridge subsystems are still being done since the translator results are not accepted as satisfactory by a majority of the states. In some of the states that have access to the NBI Translator through Pontis, bridge engineers reported that they have concerns regarding the efficiency of the tool. A recent study (*14*) on bridge management involving 17 state DOTs reported a general skepticism on the estimation accuracy of the NBI Translator. Among these states only Oklahoma has been using the translator for generating NBI ratings. However, due to the variance of the generated ratings they are in the process of stopping the use of the translator. In another study, Scherschligt (*15*) reports that Kansas DOT (KDOT) evaluated NBI Translator results as an alternative of performance measure for bridge priority evaluation. The coefficient of determination between generated and real deck condition ratings was only 25%. This implies that the translator was able to explain only 25% of the variation in the NBI deck condition ratings in this analysis by KDOT. KDOT decided that the translator results were statistically insufficient and inconsistent. Therefore, they eliminated the NBI Translator results from their alternatives of performance measures.

A study by Al-Wazeer et al. (*1*) proposes an alternative for NBI generation to improve the results of NBI Translator. Based on data from Wisconsin and Maryland, artificial neural network (ANN) models were developed, and results of ANN models were statistically compared with the NBI Translator results. The statistical comparison was based on the differences between the predicted and the actual observed NBI ratings. NBI error ranges were defined such as:

- NBI Error = 0 (the difference between the predicted and the actual observed NBI rating is zero);
- NBI Error = 1 (the difference is equal to the absolute value of one);
- NBI Error = 2 (the difference is equal to the absolute value of two); and
- NBI Error > 2 (the absolute value of the difference is greater than two).

Comparisons based on aforementioned error ranges showed that the ANN model had a higher estimation capability with respect to the NBI Translator model for a particular state when the data used for ANN training is from the same state. The superiority of the ANN model to the NBI Translator cannot be generalized since the statistical results are valid for only the data used in the study. However, the study drew attention to the importance of customizing the prediction model for each state.

STATISTICAL COMPARISON OF ACTUAL AND GENERATED RATINGS FOR IOWA BRIDGES

For the State of Iowa, NBI generation from the element-level condition data was performed using the built-in NBI Translator in Pontis software. Six hundred and eighty data points were used for the analysis of culvert ratings, and 3,038 data points were used for the analysis of substructure, superstructure, and deck ratings. Before using the NBI Translator for Iowa bridges, it was customized according to the element configuration of the Iowa BMS. This customization was done by modifying the driver file, Elements.prn, in the Pontis program folder which defines the elements to be included in the translation (*13*). First, the list of elements defined in original Elements.prn file in the program folder and Iowa elements defined in the Pontis inspection manual were compared to find the differences. Some elements that were included in the original Elements.prn file were not being used in the Iowa Pontis system; therefore, those elements were discarded in the modified Elements.prn file. Some elements had different numbers in the Iowa system, and they were also renumbered accordingly in the driver file.

The elements.prn file contains seven fields of information. These information fields are element ID (element number), element NBI field (deck, superstructure, e.g.), element material (unpainted steel, masonry, smart flag, e.g.), element type (slab, truss bottom chord, e.g.), element dimension (each, square feet, e.g.), and element name in both long and short forms (*13*). There were some elements in the Iowa Pontis setup which were not defined within the original Elements.prn file. In order to include these elements in the NBI generation, all seven fields of information for each element were coded into the modified Elements.prn file. The list of codes necessary for modifying Elements.prn file is provided by Hearn et al. (*11*). After making all the modifications to the driver file, the driver file in the Pontis program folder is replaced by the modified version and used in NBI condition rating generation.

Figures 1 through 4 summarize the findings of the comparison. For each rating item the percentage distribution of actual and generated ratings among the data set are presented as clustered column charts. Figure 1 shows that the NBI Translator estimates higher deck ratings than the actual observed deck ratings for Iowa bridges for ratings equal to or smaller than 7. While 34% of actual deck ratings have values of 8 and 9, the NBI Translator estimates no deck rating within this range. In fact, the translator was originally set up to estimate deck ratings as equal to or below 7, which corresponds to the “good” condition for bridge decks.

Figure 2 shows the comparisons for superstructure ratings. While 19% of the actual ratings are equal to 9, no observation equal to 9 appears in the generated ratings. The percentages of generated 5, 6, and 8 ratings are greater than the actual case, while the percentage of generated 7 ratings is lower than the actual case.

For the substructure ratings, the percentages of 4, 5, and 6 ratings for generated and actual ratings are very close for substructures (Figure 3). While approximately 45% of the actual ratings are equal to 8 and 9; only 1% of generated 8 rating is observed and no rating of 9 is generated again due to the way the translator was set up. The difference between the actual and generated ratings when the rating is equal to 7 is almost 48%.

For culverts, the algorithm generates 20% more 8 ratings, 22% less 7 ratings, and 9% fewer 6 ratings than the actual case. Once again, no rating equal to 9 was generated by the algorithm (Figure 4).

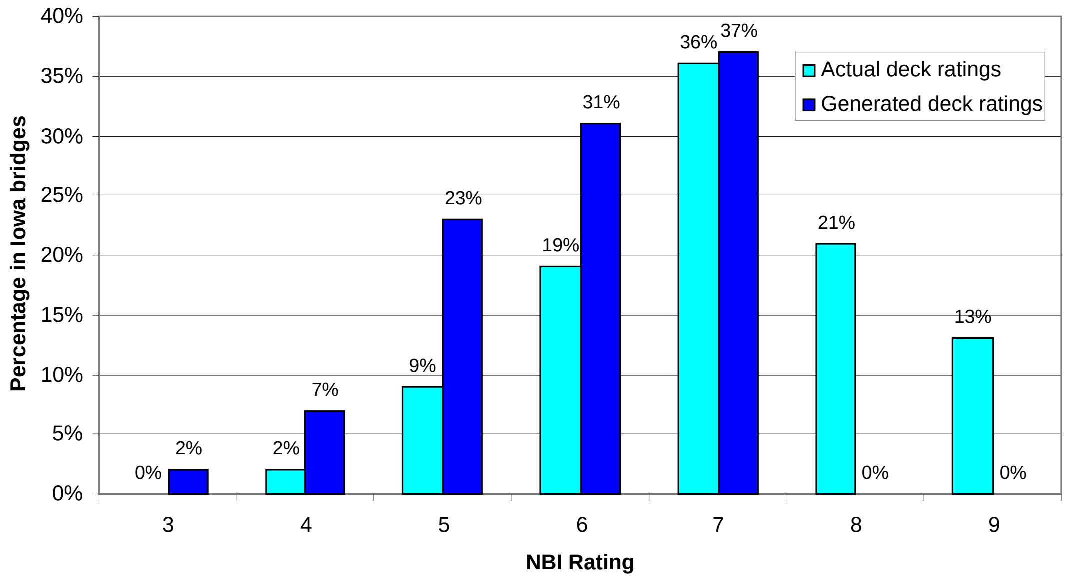

FIGURE 1 Comparison of actual and generated deck ratings.

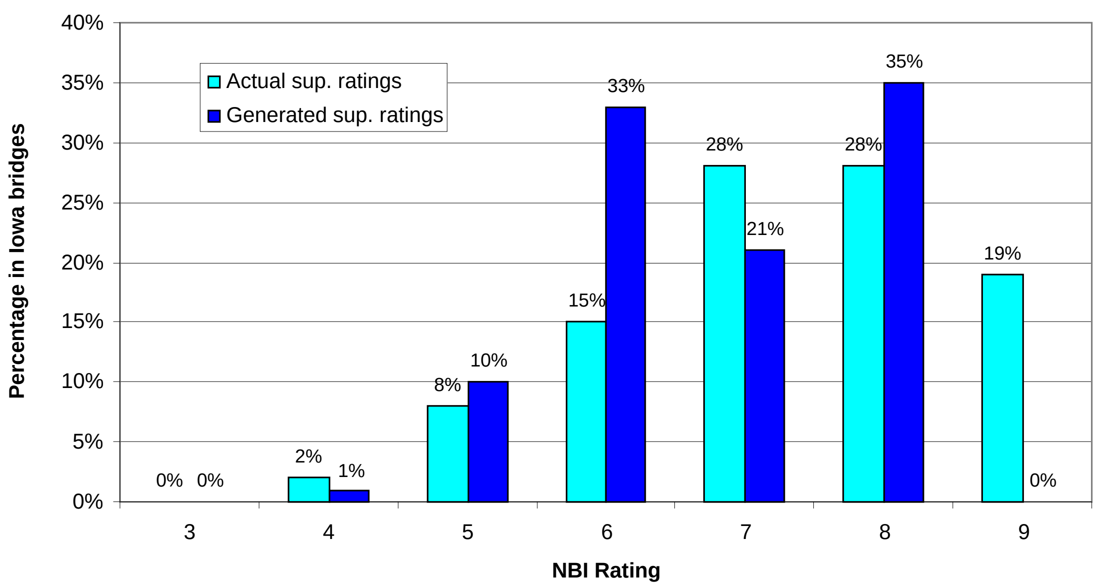

FIGURE 2 Comparison of actual and generated superstructure ratings.

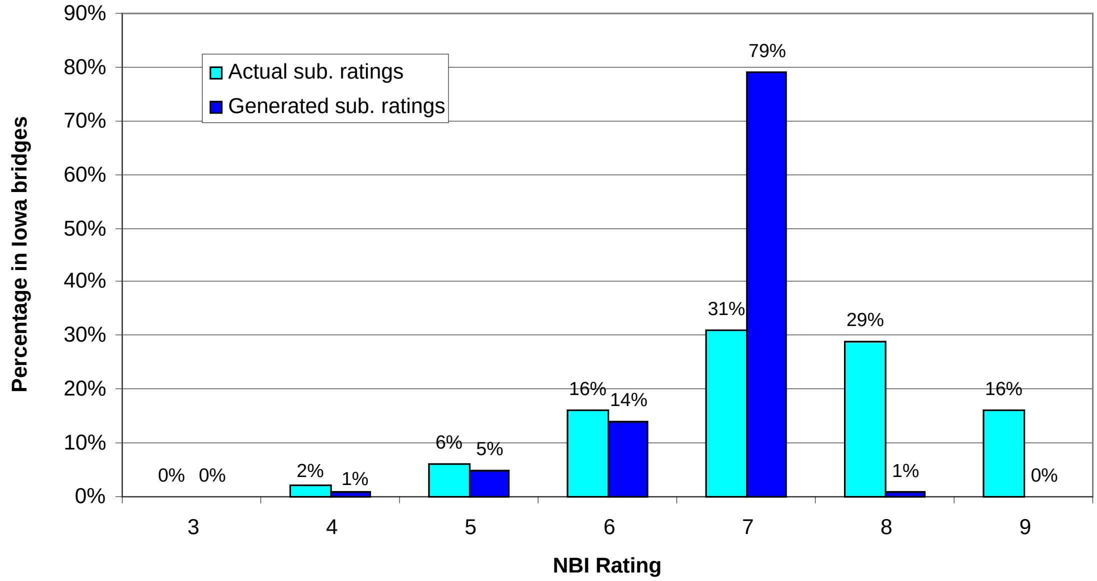

FIGURE 3 Comparison of actual and generated substructure ratings.

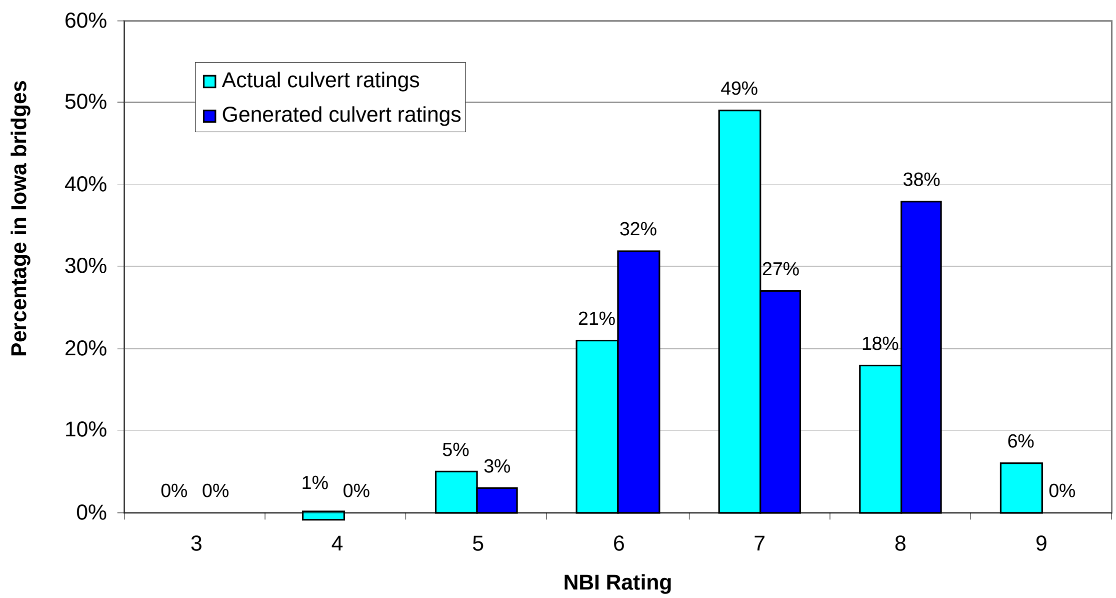

FIGURE 4 Comparison of actual and generated culvert ratings.

HOW TO IMPROVE THE TRANSLATOR

Ideally it would be desired to have an entirely objective system to collect bridge condition data. This would enable developing objective and standard performance indicators to evaluate bridge conditions which are efficient for all of the states. However routine bridge inspections are usually completed using only visual inspections and in this context they are considerably dependent on the subjective assessments of the bridge inspectors (*16*). Although national and local agencies provide guides and guidelines to assist bridge inspectors in the data collection process and make bridge inspections as objective as possible, the ratings have subjectivity. Visual inspection methods have been criticized due to their subjectivity in the literature (*17, 18, 19*) however they are still the most common bridge inspection methods due to budget constraints and lack of convenient and feasible alternative methods.

The current NBI Translator algorithm was developed based on element-level condition data and NBI ratings from 11 different states and from approximately 35,000 bridges. This extensive data was used to come up with a general translator algorithm that could be used in all states regardless of the location. Data from different states was banded together and used as the input data to develop this general algorithm. This approach is absolutely reasonable when the objective is to develop a general estimator, however, it is not useful in order to detect individual inspection practices of different organizations. A general estimator developed based on such an input structure may fail in sufficiently identifying the variability that comes from the custom practices of different states. As mentioned in an earlier section, in a recent study where an alternative algorithm (*1*) was proposed, it was reported that when custom input data was used for the same state this alternative algorithm had a higher estimation capability. Developing a customized estimator based on state-specific data may result in a more efficient and sufficient estimator and also motivate agencies to use such a tool to estimate the NBI ratings which will eventually have significant impact on bridge inspection costs.

The discrete characteristic of the NBI condition ratings make it impossible to use the ordinary least squares regression to develop an estimator where the NBI condition ratings are the dependent variables and element-level condition data are the independent variables. Also when evaluated from a statistical point of view the structure of potential predictor variables is complex. For each general NBI rating category there is a set of CoRe elements that are elements of that category and the condition data is presented as the percentages of those elements in different condition states. Because of these issues with the data there is not a straightforward statistical model to be used to develop an alternative algorithm but the data has potential to come up with a generalized linear model. Our current research focuses on developing an estimator based on a generalized linear model. The main challenge with the research is to define the most appropriate input structure for the model. After the model is developed it is planned to test the model with the data from other states and discuss its potential as an alternative algorithm.

CONCLUSION

This research paper reviewed bridge condition data and management in the United States and focused particularly on the estimation of NBI ratings from already-collected bridge-element condition data. The best-known algorithm for the purpose, which is also available within the most popular bridge management software in the United States was investigated and evaluated by a case study for Iowa bridge data.

The results of the statistical comparison for Iowa bridges showed that the generated ratings by NBI Translator algorithm with its current configuration are not representative of the actual NBI ratings. The results from this research support the concerns on the efficiency of the translator algorithm that have been previously reported. A more customized model for Iowa can lead to a more efficient model for estimating the NBI ratings. Using mapping constants specific to only Iowa bridge data instead of using the mapping constants calibrated with the data from 11 different states while creating the translator algorithm may be an option. An improved and more customized algorithm may yield better estimates of NBI ratings. A follow-up to another study in the literature, an ANN model can be developed for Iowa as another future research alternative. No matter what model is used. the current NBI rating system is prone to variation from the subjectivity of inspector decisions and might be hard to correlate to the more objective element-level condition data. Ultimately, an algorithm that calculates a 0 to 9 rating in an objective and consistent manner might be what is needed for improved bridge management data and decision-making tools.

Whichever rating system an agency use, the objective is to make consistent and objective decisions regarding bridge maintenance, rehabilitation, and/or replacement. Future research will cover the development of a simplified network-level tool utilizing consistent objective data to aid the decision makers and bridge managers in making resource allocation decisions and funding needs based on realistic and easy to use models.

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Structural Vulnerability and Weigh-in-Motion

Vulnerability Assessment of Individual Infrastructure Objects Subjected to Natural Hazards

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Current infrastructure management approaches have been developed to manage gradual deterioration (e.g., corrosion). Gradual deterioration risks are kept sufficiently low on assumption that preventative interventions will be performed before the probability of failure becomes unacceptable. This assumption is not applicable to potential sudden events (e.g., extreme floods). The proposed vulnerability approach for Pontis, the Hydraulic Vulnerability Assessment Program developed by the New York State Department of Transportation, is applied to evaluate three bridges. This case study demonstrates that this is a feasible method for identifying bridges exposed to hydraulic risks that warrant immediate or future vulnerability mitigations. This approach's limited scope, addressing only bridge components exposed to hydraulic hazards, and its qualitative nature render cross-component comparisons and estimation of mitigation intervention funds difficult. A more comprehensive vulnerability assessment approach employing recently developed hazard and component databases to quantitatively assess the vulnerability of a broader set of components (bridges, roadways, and culverts) to a wider range of hazards (avalanches, debris flows, floods, landslides, and rockfalls) is presented. This approach documents potential component hazard failure scenarios, identifies common component failure modes, and develops a structured methodology for assessing the potential component failure modes. The same case study is reanalyzed with the comprehensive approach to illuminate undocumented roadway and highway risks and to calculate the annual risks for the corresponding road link. This comprehensive vulnerability assessment approach is a quantitative, broader, and more transparent alternative in comparison to the Hydraulic Vulnerability Assessment Program for assessing the vulnerability of a given transportation infrastructure system.

Over the past 30 years numerous approaches have been developed and implemented to manage networks of structures with respect to gradual deterioration (e.g., due to corrosion). For roadway transportation systems these approaches have been formalized into computerized infrastructure management systems including Pontis (in the United States) and KUBA (in Switzerland) (*1*,*2*). These existing approaches consider risk (the probability of failure multiplied by the consequences of failure) indirectly by assuming that preventative interventions are always performed on the deteriorating infrastructure component before the probability of failure reaches an unacceptable level. With this assumption, these infrastructure management systems can neglect direct consideration of the risks of failure.

While such an assumption may be valid for well-managed infrastructure systems subjected to purely gradual deterioration, this simplification is not applicable to components subjected to sudden events (e.g., due to extreme flooding) because the uncertainty related to such sudden events is far greater than the uncertainty of gradual deterioration processes. Thus to manage an infrastructure system subjected to both potential deterioration processes and sudden events, an assessment approach for sudden events must be included in existing infrastructure management systems. In the recent proposed improvements to the Pontis bridge management system, a vulnerability assessment and prioritization tool based on the existing Bridge Safety Assurance Program developed by the New York State Department of Transportation (NYSDOT) was proposed (*3*, *4*).

BRIDGE SAFETY ASSURANCE PROGRAM

Program Overview

The primary aim of the Bridge Safety Assurance Program is to identify vulnerability-prone bridges so that funding for additional technical analysis and corrective actions can be allocated to address the most pressing needs. This vulnerability assessment program was initially developed to assess the potential occurrences and consequences of failures due to water hazards (predominately flooding) but has since been broadened to include steel fatigue, live load overload, road and maritime vehicle collision, concrete detail, and seismic-associated infrastructure vulnerabilities. Within the Bridge Safety Assurance Program, vulnerability is defined as an assessment of the likelihood of sudden failure and a relative quantification of the resulting consequences (*5*).

The hydraulic vulnerability assessment program starts with an initial screening of the existing bridges to determine which bridges are susceptible to hydraulic-induced failures. Then the vulnerability of the susceptible bridges is assessed with a two-stage assessment process in which (*a*) the bridge vulnerability to the given hazard (*LS*) and (*b*) the potential failure consequences (*FC*) are assessed. The *LS* and *FC* assessments are then arithmetically summed to produce a vulnerability rating score (*VS*) that is used to delegate bridges into urgency-based vulnerability mitigation programs (i.e., safety priority, capital program, no action).

$$VS = LS + FC \qquad (1)$$

Bridge Vulnerability to the Given Hazard

A *LS* to hydraulic hazards is composed of a hydraulic assessment of the bridge's location and a structural assessment of the bridge's foundation.

Hydraulic Assessment of the Bridge Site

The hydraulic assessment (*HA*) rates the bridge hydraulic vulnerability as a function of the following 10 categories:

- River slope–flow velocity (0–3),
- Channel configuration (0–2),
- Near a water body junction (0–1),
- Existing evidence of scour (0–5),
- Bridge opening capacity (0–2),
- Channel floor material quality (0–4),

- Debris–ice accumulation potential (0–4),
- Location in a backwater zone (0–1),
- Historical maximum flood depths (1–2), and
- Bridge hydraulic overflow potential (0–1).

Each assessment category is separated into subcategories and delegated numerically increasing values as a function of their contribution to the overall bridge vulnerability. For example the river slope–velocity is separated into three subcategories: flat [slope (s) ≤ 0.004 ft/ft], medium (0.0004 < s < 0.0015 ft/ft), and steep ($s \geq 0.0015$ ft/ft), with the respective numerical rating values of 1, 2 and 3. The *HA* ratings for the 10 categories are then totaled, resulting in a hydraulic vulnerability rating ranging from a maximum of 25 to a minimum of 1.

Structural Assessment of the Bridge Foundation

With the *HA* completed, the focus then shifts to assessing the vulnerability of the bridge foundation (*BF*). As with the *HA*, the *BF* vulnerability assessment is a category ranking system. This process separately assesses the vulnerability of each abutment and pier, with the most vulnerable element controlling the bridge vulnerability.

The bridge abutment vulnerability (*AV*) is quantified with the following five categories:

- Existing scour countermeasures (0–5),
- Abutment location on river bend (0–1),
- Embankment encroachment (0–4),
- Abutment foundation (0–10), and
- Angle of inclination (0–4).

And the bridge pier vulnerability (*PV*) is quantified with the following seven categories:

- Existing scour countermeasures (0–5),
- Angle of attack (0–4),
- Pier width (0–5),
- Multiple piers in floodplain (0–2),
- Pier foundation (0–10),
- Footing/pile bottom below streambed (0–1), and
- Simple spans (0–1).

As with the hydraulic vulnerability assessment, each of these 12 categories is subdivided into subcategories, with each subcategory being assigned a numerical rating value as a function of the subcategory's contribution to the overall element vulnerability. Thus the *AV* rating can range from a high of 24 to a low of 0, and the *PV* rating can range from a high of 32 to a low of 1. The structural assessment of the *BF* is then the maximum *AV* and *PV* rating.

$$BF = \max(AV, PV) \qquad (2)$$

With the *HA* and the *LS* assessments completed, the individual vulnerability scores are summed, resulting in a bridge classification score (*CS*).

$$CS = HA + BF \tag{3}$$

This *CS* is then employed to assign the various bridges to one of three vulnerability classes that are in turn assigned bridge event likelihood scores (*LS*):

- $CS > 35$ = high vulnerability → $LS = 10$;
- $CS < 25$ = low vulnerability → $LS = 2$; and
- $40 > CS > 20$ = medium vulnerability → $LS = 6$.

Failure Consequence Assessment

The potential *FC* are then assessed by assigning rating values as a function of the following three categories:

- Potential failure type (1–5),
- The exposed traffic volume (0–2), and
- The bridge contribution to the transportation network (0–3).

The failure type and exposure categories are then summed, producing a *FC* score ranging from a maximum of 10 to a minimum of 1.

Vulnerability Rating Score

The *VS* is then computed as shown in Equation 4 by summing the bridge *LS* and the *FC* score.

$$VS = LS + FC \tag{4}$$

This *VS*, ranging from a maximum of 20 to a minimum of 1, is used to separate the various bridges into urgency-based vulnerability mitigation programs with the following thresholds:

- Safety priority ($VS > 15$),
- Capital program ($9 < VS < 14$),
- No action ($VS < 9$),
- Safety program ($13 < VS < 16$),
- Inspection program ($VS < 15$), and
- Not applicable ($VS = 0$).

Within the NYSDOT system, bridges identified as safety priorities are failure-prone structures whose potential failure modes should be mitigated immediately or as part of the 5-year capital program. Bridges assigned to the safety program are potentially failure-prone bridges for which vulnerability reduction interventions, enhanced inspections, or addition to the long-term capital program may be considered in the future. Bridges in the capital program are bridges prone to failure under only extreme flooding events and thus the risk exposure can be tolerated until the implementation of a normally scheduled capital improvement project. Bridges included in the inspection program are nonfailure-prone bridges given that the current conditions remain constant. Thus the bridge and the surrounding environmental conditions should be reassessed during normally scheduled bridge inspections. Bridges identified as “no action” and “not applicable” are bridges with an extremely remote probability of failure and bridges not exposed to the given risk source, respectively.

Thus with such an approach an infrastructure manager can prioritize a group of bridges into urgency-based vulnerability mitigation programs.

BRIDGE SAFETY ASSURANCE PROGRAM CASE STUDY

The Zofingen Case Study

To help evaluate the strengths and limitations of the bridge safety vulnerability assessment approach, consider the case study of three bridges (designated bridges 1, 2, and 3) located in the Canton of Aargau in north-central Switzerland. These three bridges span the Wigger River and link the city of Zofingen to the adjacent towns Strengelbach and Brittnau as shown in Figure 1. Bridges 1 and 2, built in 1977, are three-span continuous bridges with respective lengths of 69.3 m (229 ft) and 73.6 m (243 ft) and cross not only the Wigger River but also the A2 national highway located just adjacent to the Wigger River. Bridge 3 is a 20.0-m (65-ft) simply supported bridge located just east of Brittnau, where it spans the Wigger River. At the time of the case study, this structure was being replaced as the existing structure was in a severely deteriorated state, thus the Wigger River was placed into bypass conduits.

FIGURE 1 Elevation and plan images of Bridges 1, 2, and 3 spanning the Wigger River.

Applying the Hydraulic Vulnerability Assessment

The vulnerability of the three bridges to potential flood-induced damage was evaluated using the hydraulic vulnerability assessment during an on-site inspection (Table 1). The hydraulic environment of all three bridges received ratings of 8 out of a potential score of 25. Additionally the foundations of the three bridges received ratings of 6, 6, and 1, respectively, out of potential ratings of 32, 32, and 24. When summed to determine the *CS*, it was found that bridges 1 and 2 had a *CS* of 14 and bridge 3 had a *CS* of 9. Thus all three of these structures were determined to have a "low vulnerability." On the consequence side, all three bridges are designed with horizontal shear keys and drilled piles at all support locations. Thus potential structural *FC* due to flooding are purely local structural damage. Within the network, bridges 1 and 3 are local roadways and bridge 2 is a collector roadway. Additionally bridges 1 and 2 carry between 4,000 and 25,000 vehicles per day, and bridge 3 carries less than 4,000 vehicles per day. Therefore, the three bridges respectively received vulnerability ratings of 4, 5, and 3 out of a potential score of 20. Thus, the Hydraulic Vulnerability Assessment manual would recommend that no action should be taken apart from the continuation of biannual on site hydraulic inspections.

The Bridge Safety Assurance Program: Review

The vulnerability of a transportation infrastructure component is then defined as an infrastructure component that can perform inadequately due to a set of potential natural hazard events. Additionally, in considering the most common source of hydraulic risks, the existing rivers and streams, bridges are the network components exposed to the highest hazard magnitudes (i.e., flood depth). Furthermore, a bridge is the infrastructure component with the largest potential post-failure interruption duration and reconstruction costs as the replacement of a single 45-m bridge can take multiple weeks and cost SFr 300,000 (US$300,000). But bridges commonly only comprise less than 2% of a given transportation system, and while the other components of a transportation infrastructure network (the roadways, galleries, and tunnels) are less prone to hydraulic hazards than bridges, their structural resistance to hydraulic hazards is far less. These potential additional vulnerability sources become even more apparent when the scope of the potential hazards is broadened from purely hydraulic hazards to include rockfall, torrent, landslide, and avalanche hazards, all common events in mountainous regions.

Additionally, during the past 5 years there have been significant advancements in formulating natural hazard and infrastructure component databases. A prime example of these recent developments is in Switzerland, where there are numerous national and regional programs currently underway to develop detailed geographic information system-based natural hazard maps and component infrastructure databases. In Switzerland, developments in the natural *HA* domain include the documentation of the maximum possible geographical reach of gravitational natural hazards (i.e., avalanche, landslide, rockfall, and torrent hazards) (*6*) and the formulation of detailed magnitude and return period natural hazard maps for avalanche, landslide, rockfall, static flooding, dynamic flooding, bank erosion, and torrent hazards (*7*). The key development within the infrastructure component documentation domain include the implementation of the KUBA infrastructure documentation and management program by 24 Swiss Cantonal and the Swiss federal roads authorities (*8*).

These recent developments provide a number of the inputs required to assess the vulnerability of infrastructure components to natural hazards, specifically, the location,

TABLE 1 Zofingen Case Study Hydraulic Vulnerability Assessments

Assessment Topic		Bridge 1		Bridge 2		Bridge 3	
Hydraulic Assessment	Streambed material	Cobbles	—	Cobbles	—	Cobbles	—
	River slope	Medium	1	Medium	1	Medium	1
	Channel bottom	Stable	1	Stable	1	Stable	1
	Channel configuration	Straight	0	Straight	0	Straight	0
	Debris/ice problems	None	0	None	0	None	0
	Near river confluence	No	0	No	0	No	0
	Affected by backwater	No	1	No	1	No	1
	History of scour	Small	1	Small	1	Small	1
	Historical max flood	> 10 ft	2	> 10 ft	2	> 10 ft	2
	Adequate opening	No	2	No	2	No	2
	Overflow relief available	Yes	0	Yes	0	Yes	0
	HA total	—	8	—	8	—	8
Abutment Assess.	Scour countermeasures	Sheet pile wall	0	Sheet pile wall	0	Sheet pile wall	0
	Abutment foundation	Long piles >20 ft	0	Long piles >20 ft	0	Long piles >20 ft	0
	Location on river bed	Straight	0	Straight	0	Straight	0
	Angle of inclination	20–45 degs.	1	20–45 degs.	1	20–45 degs.	1
	Embankment encroachment	Small	0	Small	0	Small	0
	AV total	—	1	—	1	—	1
Pier Assessment	Scour countermeasures	Sheet pile wall	0	Sheet pile wall	0	—	—
	Pier foundations	Concrete piles	0	Concrete piles	0	—	—
	Footing/pile below streambed	15–20 ft	1	15–20 ft	1	—	—
	Angle of attack	0–20 degs.	2	0–20 degs.	2	—	—
	Pier width	3–5 ft	2	3–5 ft	2	—	—
	Simple spans	Yes	1	Yes	1	—	—
	Multiple piers in floodplain	No	0	No	0	—	—
	PV total	—	6	—	6	—	—
	LS total	*CS* = 6+8 = 14	2	*CS* = 6+8 = 14	2	*CS* = 1 + 8 = 9	2
Conseq.	Failure type	Structural damage	1	Structural damage	1	Structural damage	1
	Exposure	4,000–25,000 ADT	1	4,000–25,000 ADT	1	< 4,000 ADT	0
	Functional class	Local road	0	Collector	1	Local road	0
	CS total	—	2	—	3	—	1
	VS	—	4	—	5	—	3

ADT = average daily traffic

magnitude, and probability of potential natural hazards as well as the location and key dimensions of the components. Unfortunately, historically there has been no comprehensive infrastructure failure assessment approach with which to estimate the implications these documented natural hazards may have on the built infrastructure. In the absence of such a comprehensive approach, qualitative assessment approaches like the Bridge Safety Program have been the only tools available for assessing vulnerability.

A current Swiss National Science Foundation funded research program entitled "Consideration of Vulnerability in the Management of Swiss Transportation Infrastructure" is working to develop such an infrastructure failure assessment framework by formulating potential hazard–component failure scenarios, identifying overarching component failure modes, and documenting component parameters that significantly influence the component resistance (*9*).

Therefore, in recognizing that network vulnerability can be caused by more than just hydraulic hazards and can affect more than just bridge infrastructure components, one can observe that a broader management approach must be developed.

LAYING THE FOUNDATION FOR A COMPREHENSIVE VULNERABILITY ASSESSMENT APPROACH

Quantitative Vulnerability Assessment Approach

Stepping back from a qualitative vulnerability assessment approach based on evaluating how prone the various components and environmental elements are to a given hazard and returning to the basis of risk assessment, the risk of failure of a given component i ($Risk_i$) can be computed by multiplying the probability of failure (P_{fi}) by the consequences of failure ($consequences_i$). Thus the risk of failure of component i takes the form of:

$$Risk_i = P_{fi} \cdot consequences_i \tag{5}$$

Within this approach, the vulnerability of a transportation infrastructure component includes the probability of inadequate performance and the related consequences due to a defined set of natural hazard events. The consequences of this inadequate performance can take two different forms: (*a*) direct consequences to the exposed component in the form of structural damage and (*b*) indirect consequences to the transportation traffic by restricting or completely denying the free flow of traffic. Immediate consequences to human life or well being are not directly considered by this approach (*10*). A vulnerable transportation infrastructure link is likewise defined as a link containing one or more vulnerable components.

The vulnerability of component i, is thus the probability of component i experiencing failure due to a given hazard event ($P_{fi|E}$) multiplied by the sum of the direct and indirect natural hazard induced consequences (CD_i, CI_i respectively).

$$Vulnerability = P_{fi|E} \cdot (CD_i + CI_i) \tag{6}$$

Delving further into this general perspective, the direct consequences of component i experiencing inadequate performance (CD_i) are the financial resources required to repair the component damage in the given failure mode caused by the natural hazard event magnitude. The indirect consequences of component i performing inadequately (CI_i) is the sum of the daily additional vehicular travel time resulting from the given component's failure, multiplied by the duration of the component failure and the average driver valuation of time.

Thus with information concerning the probability of potential hazardous events, one can calculate the component risk of failure.

Component Failure Assessment Steps

The processes of assessing the vulnerability of an infrastructure component is comprised of the following steps.

- Evaluate whether the given component is exposed to the potential hazard(s).
- Collect natural hazard event magnitude, location, and return period data.
- Collect component location and structural resistivity governing parameters.
- Consult (or develop if required) the infrastructure component failure assessment framework for the respective natural hazard and infrastructure component (an example framework is presented in Figure 2).
- Assess whether the given component has the structural resistance to withstand the potential natural hazard event magnitudes.
- If the structural resistance is exceeded, use the causality chains in the failure assessment framework to determine the specific failure mode.
- Model or estimate the potential failure durations, direct failure costs, and indirect failure costs.
- Calculate the component and associated link annual risk of failure.

Hazard–Component Failure Scenarios

The component failure assessment logic is developed by documenting the scenarios through which each hazard (avalanche, debris flows, floods, landslides and rockfalls) can cause a given component (roadway, bridge, or culvert) to fail. For example, in considering the scenarios in which a flood can cause partial or complete failure of a bridge, four individual hazard–component failure scenarios can be identified specifically:

- A flood can come in contact with a bridge superstructure, causing it to fail horizontally in shear or flexure.
- A flood can come in contact with a bridge superstructure and exceed the superstructure–substructure connection resistance either horizontally (sliding off bearings) or vertically (uplift).
- A flood can submerge the bridge roadway service, thereby denying public access to and usage of the structure.
- A flood can scour the material surrounding a pier, undermining the pier foundation.
- A flood can cause a bridge pier to fail horizontally in shear or flexure.

Therefore, as presented in Table 2, there are five different failure scenarios for the flood–bridge hazard–component combination.

In briefly looking through Table 2, one can observe that there are less than five different scenarios for the culvert, retaining wall, and tunnel components, while there are at least 19 different failure scenarios for the bridge and gallery components. The source of this difference is twofold—some of the components with a low number of failure scenarios are not exposed to all

TABLE 2 Hazard–Component Failure Scenarios

Component	Avalanche	Debris Flow	Flooding	Landslide	Rockfall	# Total	# Mode
Bridges	5	5	5	6	5	26	6
Culverts	1	1	1	1	1	5	2
Galleries	5	1	2	6	5	19	6
Retaining walls	0	0	1	3	0	4	3
Roadways	1	2	3	3	1	10	3
Tunnels	0	2	2	0	0	4	2
Total #	12	11	13	19	12	67	23

five hazards (i.e., a tunnel is assumed to not be exposed to avalanches, landslides, or rockfalls). Second, in cases where these components are exposed to a given hazard, the component resistance to the given hazard is limited or negligible (i.e., a culvert has a negligible resistance to being buried by an avalanche). Turning to the components with a high number of failure scenarios, bridges and galleries, this status is a direct result of the multiple levels of the component's resistance as seen in formulating the flood–bridge failure scenarios.

The remaining component, roadways, has an elevated number of failure scenarios, not only because it is exposed to all five hazards, but also because, for a roadway component to be operational, other supportive components—specifically, culverts and retaining walls—must also be functional. Thus, for instance, a landslide can cause a roadway to fail

1. Indirectly by damming a culvert passing underneath a roadway, thereby flooding the roadway;
2. Indirectly by compromising a supporting retaining wall;
3. Directly by undermining the roadway foundation; or
4. Directly by burying the roadway with debris.

Identifying Common Failure Modes and Structuring the Failure Assessment Logic

Roadway Failure Modes

The set of failure modes is developed by identifying the common failure modes or each component. For example, the five different hazards can cause a roadway component to partially or completely fail in 10 different failure scenarios, but when these hazard–component-specific failure scenarios are studied as a group, three common failure modes can be identified.

1. A roadway can fail due to the failure of supporting components (culverts and retaining walls).
2. A roadway foundation can be eroded or undermined.
3. A roadway can be buried in debris or liquid.

These failure modes can be confirmed and organized in a causality chain by observing that the first failure mode (a roadway can fail due to the failure of supporting components) can be the result of a flood discharge exceeding culvert capacity, a flood compromising the foundation of a retaining wall, a landslide compromising the foundation of a retaining wall or an avalanche, debris flow, landslide, or rockfall burying the culvert. If the supporting components are not affected by the hazard, a hazard can undermine the roadway foundation (failure mode and causality chain step 2) or bury the roadway in debris or liquid (failure mode and causality chain step 3).

This failure assessment process is presented in Figure 2 and determines the controlling failure mode by contrasting hazard and component data. Each failure scenario has different direct consequences, but all induce the same marginal indirect result: the closure of the infrastructure link.

Culvert Failure Modes

Turning to the culvert component, there are five hazard–component failure scenarios that can be simplified into two common failure modes:

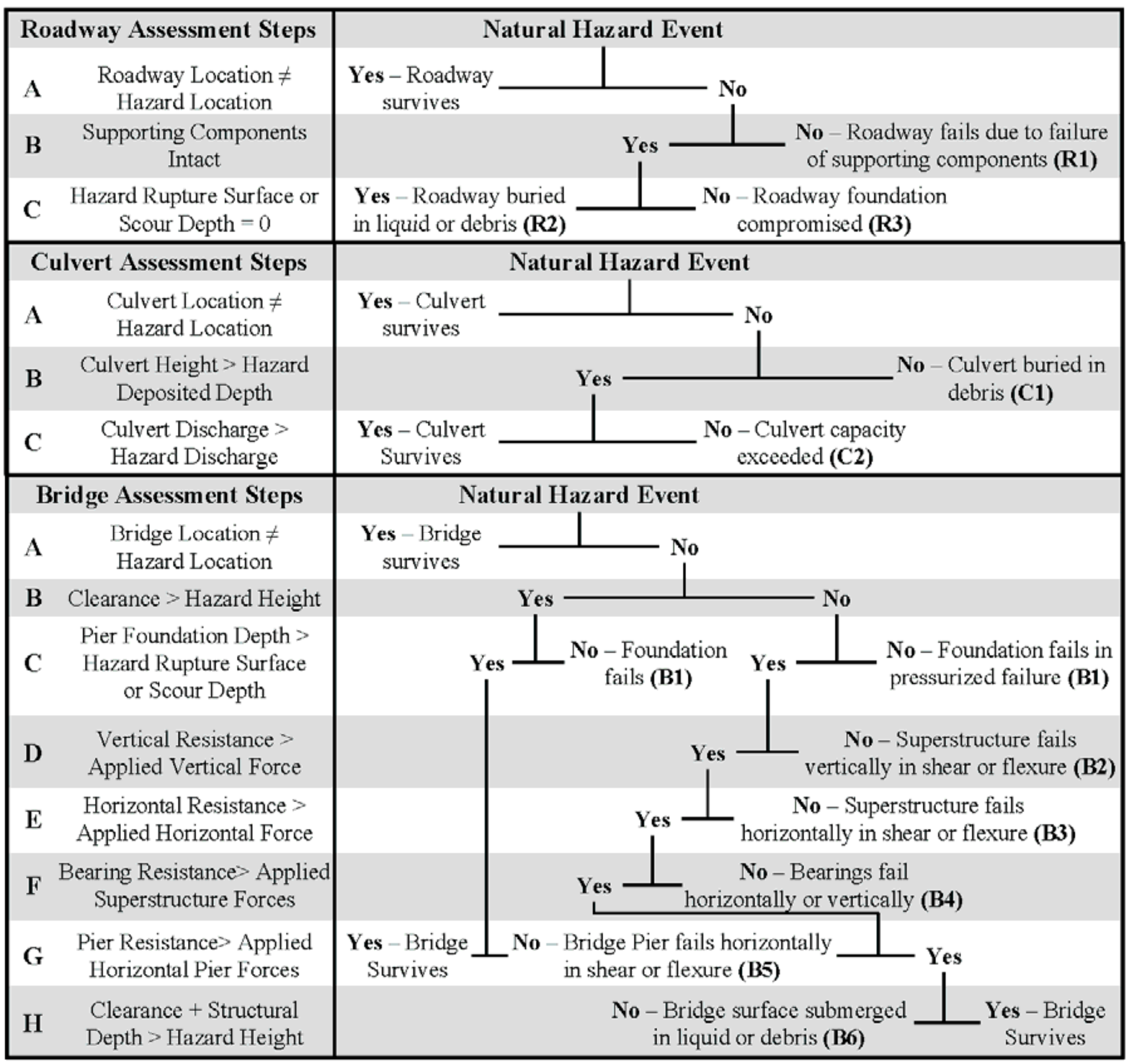

FIGURE 2 Structured component failure mode assessment approach.

1. A culvert can be clogged with debris.
2. A culvert discharge capacity can be exceeded.

The first culvert failure mode, a culvert becoming clogged with debris (i.e., avalanche, debris flow, landslide, or rockfall debris), is assessed in a similar manner as the first roadway failure mode, for it is assumed that the culvert has no independent resistance against these four hazards, save its location. The various hazards and hazard magnitudes can cause different direct failure costs and durations, but the final result is the same: the partial or complete failure of the culvert.

A different perspective is required to evaluate the second culvert failure mode, a culvert's discharge capacity can be exceeded, for this failure mode is a direct product of the culvert design capacity. If a culvert is designed for a 30-year return period discharge, it can be assumed that the culvert will remain functional for all discharge magnitudes up to and including the 30-year return period discharge, but will fail when the 31-year return period discharge occurs. Thus the vulnerability of a culvert to this second failure mode is a direct product of its resistance, and this resistance is a function of one component variable: the culvert design discharge capacity.

Bridge Failure Modes

Graduating from two of the components, each with 10 or fewer hazard–component failure scenarios, to the component with the highest number of hazard–component failure scenarios, the bridge component, one can observe that not only is this component exposed to all the hazards, but each hazard can induce a bridge component to fail in at least five different scenarios. When these 26 hazard–component failure scenarios are studied as a group, six common failure modes can be identified:

1. A bridge pier foundation can be compromised.
2. A bridge superstructure can fail vertically in shear or flexure.
3. A bridge superstructure can fail horizontally in shear or flexure.
4. A bridge superstructure–substructure connection resistance can be horizontally or vertically exceeded.
5. A bridge pier can transversely fail in shear or flexure.
6. A bridge roadway surface can be submerged in liquid or debris.

This relative large number of failure modes is the result of two key factors: a bridge is by definition an elevated structure (a bridge's primary form of structural resistance) and the superstructure, superstructure–substructure connection, and the substructure all have their own structural capacities (multiple additional forms of resistance). Thus when a hazard does exceed this primary form of resistance, the hazard can cause failure not only by burying the roadway surface in debris, but also by exceeding the superstructure structural resistance vertically, the superstructure structural resistance horizontally, the superstructure–substructure connection resistance, or the substructure resistance.

Structuring the Failure Assessment Process

As one can observe in Figure 2, the relative relationship between the component and hazard event parameters dictates the failure assessment process. Furthermore, this structured assessment process not only identifies which failure modes are mutually exclusive but also what component and hazard data are required to conduct each assessment step. Finally, with the failure mode identified, the failure durations, direct and indirect consequences for the given component–natural hazard scenario can be modeled or estimated.

COMPREHENSIVE VULNERABILITY ASSESSMENT APPROACH CASE STUDY

In the Bridge Safety Assurance Program, the case study scope was limited to the three bridges crossing the Wigger River. In applying the comprehensive vulnerability assessment approach, the scope of this case study can be increased to include the roadways leading onto and off of Bridges 1, 2, and 3 to the surrounding transportation system and the region highway, the A2, running parallel to the Wigger River. This broader scope provides a more comprehensive perspective of the relative performance of the various components and helps to ensure that the infrastructure network risks, and not only the bridge component risks, are actively considered.

Step A of the component failure assessment process, assessing potential geographic coincident between the infrastructure components and the natural hazards, is conducted by overlaying the hazard data within each return period over the component data. The area around Zofingen is prone to flooding, and flood hazard maps for 30-, 100-, 300-, and 10,000-year return period events have been developed. These hazard data have been overlaid on the infrastructure components that can carry through traffic, and the product of this analysis is presented in Figure 3. In Figure 3, one can observe that apart from the bridge components, the only other component coincident with the 30-year return period flood is a small portion of Roadway 2. For the 100-year flood, the scope of the components affected extends to include all of the Roadway links and a portion of the A2 Highway. This scope further increases for the 300- and 10,000-year floods. Continuing with the failure assessment of the roadway components, all known supporting components are assumed to be intact within all the links during all return period floods (Step B)

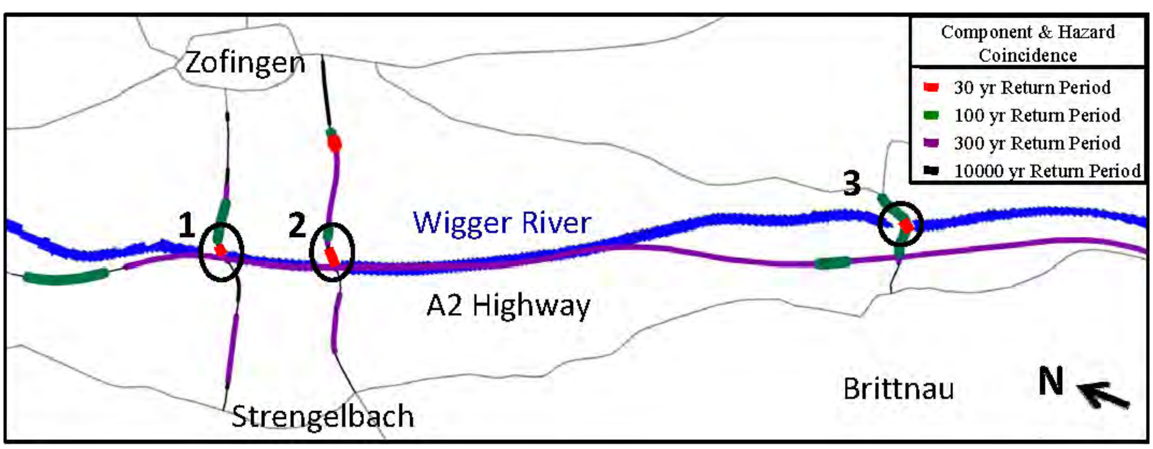

FIGURE 3 Component–hazard geographic coincident analysis.

and the depth of soil removed by the flood below the roadway surface is assumed to be zero (Step C). Thus, as shown in Table 3, the roadway failure mode for all the roadways is failure mode R2: roadway buried in liquid or debris.

Turning to the bridge components, all three bridges are geographically coincident with the flood waters for all return periods (Step A), but only at Bridge 3 during the 300- and 10,000-year return period floods does the flood depth exceed the bridge clearance (Step B). Bridge 3 has the superstructure and bearing vertical and horizontal structural resistance to withstand the applied flood forces (Steps C–E). Furthermore, all three bridges are set on pile foundations and thus have the structural resistance to withstand the flood applied forces and the flood induced local scour (Steps F and G). Finally, the flood water height at Bridge 3 does exceed the bridge clearance and structural depth (Step H). Thus Bridges 1 and 2 do not fail, but Bridge 3 experiences failure in mode B6: bridge surface submerged in liquid or debris.

The length of each component experiencing failure was determined from the geographic component–hazard coincident analysis. The direct component consequences and component closure durations were formulated from expert opinions, and the component daily detour times were modeled with the Swiss National Private Transportation Model (*11*). With these data, the component direct and indirect *FC* can be computed following the approach introduced earlier.

The total link consequences within each return period *i* (LC_i) are calculated from the individual roadway and bridge direct and indirect consequences ($CD_{R,i}$, $CI_{R,i}$, $CD_{B,i}$, $CI_{B,i}$, respectively) with Equation 7.

$$LC_i = CD_{R,i} + CD_{B,i} + \max(CI_{R,i}, CI_{B,i}) \tag{7}$$

The total annual link risk is then computed from the summation of the marginal risk between each return period threshold with Equation 8. Additionally it is assumed that the marginal risk for events with return periods less than 30 years is negligible.

$$Risk_i = \left(\frac{1}{30} - \frac{1}{100}\right) \cdot LC_{i,30} + \left(\frac{1}{100} - \frac{1}{300}\right) \cdot LC_{i,100} + \left(\frac{1}{300} - \frac{1}{10000}\right) \cdot LC_{i,300} + \frac{1}{10000} \cdot LC_{i,10000} \tag{8}$$

From Table 3 it can be seen that the annual risks of links 1, 2, and 3 are all under SFr. 14,000 per year. Link 2 is the link with the highest risk link of the three links with an annual risk of over SFr. 13,000, with over 75% contributed by potential traffic distributions. Link 3 is the second most risk prone link of the three links with the potential bridge failure risk contributing over 20% of the total link risk.

Turning to the potential risk facing the regional A2 Highway, the risks to flooding far exceed the other three link risks combined by a factor of 12. The closure of the A2 induces an additional daily travel time of 2,865 h and the financial value of this lost time accounts for almost 90% of the total A2 closure risk.

TABLE 3 Zofingen Case Study Comprehensive Vulnerability Assessments

Components	**Roadway 1**				**Roadway 2**				**Roadway 3**				**A2 Highway**			
Return Periods	**30**	**100**	**300**	**10000**	**30**	**100**	**300**	**10000**	**30**	**100**	**300**	**10000**	**30**	**100**	**300**	**10000**
Roadway location ≠ hazard location	N	Y	Y	Y	Y	Y	Y	Y	N	Y	Y	Y	N	Y	Y	Y
Supporting components intact	—	Y	Y	Y	Y	Y	Y	Y	—	Y	Y	Y	—	Y	Y	Y
Hazard depth = 0	—	Y	Y	Y	Y	Y	Y	Y	—	Y	Y	Y	—	Y	Y	Y
Failure mode	—	R2	R2	R2	R2	R2	R2	R2	—	R2	R2	R2	—	R2	R2	R2
Road failure length (m)	0	170	595	860	20	98	793	1123	0	292	310	311	0	492	5168	5168
Road direct consequences (SFr/30m)	250				250				250				500			
Road total direct consequences (SFr 1,000)	0	140	491	710	16.3	80.5	654	927	0	241	256	256	0	811	8528	8528
Road closure duration (days)	0	5	10	15	2	5	10	15	0	5	10	15	0	7	12	17
Daily road detour time (Hr/day)	50.00				74.62				40.67				2865.5			
Road indirect consequences (SFr 100,000)	0	3.0	6.0	9.0	1.8	4.5	9.0	13.4	0	2.4	4.9	7.3	0	240	413	584.5
	Bridge 1				**Bridge 2**				**Bridge 3**				—			
Bridge location ≠ hazard location	N	N	N	N	N	N	N	N	N	N	N	N	—	—	—	—
Clearance > flood depth	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	N	N	—	—	—	—
Pier foundation depth > scour depth	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	—	—	—	—
Vertical resistance > applied vertical force	—	—	—	—	—	—	—	—	Y	Y	Y	Y	—	—	—	—
Horizontal resistance > applied horizontal force	—	—	—	—	—	—	—	—	Y	Y	Y	Y	—	—	—	—
Bearing resistance > applied vertical force	—	—	—	—	—	—	—	—	Y	Y	Y	Y	—	—	—	—
Pier resistance > applied horizontal pier forces	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	—	—	—	—
Clearance+structural depth > hazard height	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	N	N	—	—	—	—
Bridge failure mode	—	—	—	—	—	—	—	—	—	—	B6	B6	—	—	—	—
Bridge failure length (m)	0	0	0	0	0	0	0	0	0	0	32	32	—	—	—	—
Bridge direct consequence (SFr/30 m)	4,000				4,000				4,000				—	—	—	—
Bridge total direct consequences (SFr1000)	0	0	0	0	0	0	0	0	0	0	416	416	—	—	—	—
Bridge closure duration (days)	0	0	0	0	0	0	0	0	0	0	12	17	—	—	—	—
Daily bridge detour time (h/day)	50.00				74.62				40.67							
Bridge indirect consequences (SFr 100,000)	0	0	0	0	0	0	0	0	0	0	5.9	8.3	—	—	—	—
Link total consequence (SFr 100,000)	0	4.4	10.9	16.1	2.0	5.3	15.5	22.7	0	4.9	12.6	15.0	0	249	498	669.8
Annual link risk (SFr)	**6,622**				**13,317**				**7,450**				**333,564**			

Evaluating the Comprehensive Vulnerability Assessment Approach

From this second case study, one can observe that the developed failure modes evaluated with the structured failure assessment logic is a viable method for transparently evaluating the potential infrastructure failure risks. Furthermore, the broad scope of this vulnerability assessment approach enables it to evaluate potential failure risks of many different types of components and component classes. The end product of this assessment, the equivalent annual financial link risk, facilitates one in illuminating, comparing, and contrasting the annual risk exposure of different components and in computing the annual link failure risk. Lastly, this method helps to ensure that components previously viewed as nonvulnerable, such as the regional highway discussed in the case study, can be evaluated alongside known vulnerable components.

CONCLUSIONS

Existing transportation infrastructure management systems have been developed to manage gradual deterioration (e.g., corrosion). The gradual deterioration risk is evaluated by multiplying the probability of failure by the consequence of failure and by assuming that preventative interventions are always performed before the probability of failure becomes unacceptable. This simplification is not applicable to sudden events (e.g., extreme flooding), for the uncertainty of an event resulting in failure is greatly increased.

The proposed vulnerability assessment approach, the Bridge Safety Assurance Program developed by NYSDOT, is reviewed and applied to assess the vulnerability of three bridges. This focused case study demonstrates that this qualitative assessment approach is a feasible approach for evaluating and ranking bridge components exposed to flood hazards. Unfortunately, the limited scope and qualitative nature of this approach causes a vulnerability assessment to be focused on bridge components exposed to floods while neglecting other hazards (avalanches, debris flows, landslides, rockfalls) or even other components (roadways, culverts, retaining walls).

A comprehensive vulnerability assessment program is developed by documenting the various hazard–component failure scenarios and identifying unique component failure modes. A structured and systematic assessment process employing hazard and component data is then employed to evaluate the potential of each failure mode.

The same case study is quantitatively reanalyzed with this comprehensive vulnerability assessment approach to illuminate previously undocumented highway and roadway risks that far exceed the previously documented bridge hydraulic-induced failure risks. Additionally, in applying this approach, one can quantify the annual failure risks for each transportation link.

This broader infrastructure component and hazard perspective and the ability to quantify in financial terms the risk facing each transportation link make the comprehensive vulnerability assessment approach a much more applicable vulnerability assessment approach than the proposed existing assessment method.

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Multihazard Applications in Bridge Management

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All components of bridge management have a potential for multihazard implications. Hence, accommodating multihazard strategy in bridge management can increase safety and security at reasonable costs. This paper first shows a simple and general quantitative method for computing multihazard effects on different bridge management tools and then discusses multihazard considerations as they apply to several bridge management tools, such as inspection, guides and manuals, and repair–retrofit.

Bridge management goals comprise increasing safety and security at reasonable costs. Safety considerations include, but are not limited to, ensuring adequate capacity for service loads and accommodating different hazard demands during the service life of the bridge. These two components of safety have multihazard implications. The costs of bridge ownership include, among others, cost of inspection, repair–retrofit costs, and operation–maintenance costs during the service life. All these cost components have multihazard implications. Figure 1 shows how different hazards affect bridge management objectives (*1*).

Since all components of bridge management have a potential for multihazard implications, one can ask how can we accommodate multihazard strategy in bridge management? We need a quantitative method in order to study this issue further. Ettouney, Alampalli, and Agrawal (*2*) presented a theory of multihazards that can be used in addressing this immediate task. The theory indicates that as hazards affect the bridge, they interact together through different aspects of the bridge. By identifying different ways such interactions occur—i.e., how the hazards affect the bridge—optimizing those issues can result in achieving management goals in improved safety and efficient utilization of resources.

In this paper, a general method for computing multihazard effects on different bridge management tools is developed (*1*). Then multihazard considerations as they apply to several bridge management tools, such as inspection, guides and manuals, and repair–retrofit are discussed. Examples in each case that show how to quantify the multihazard usage in bridge management and end with some recommendations for future efforts are offered.

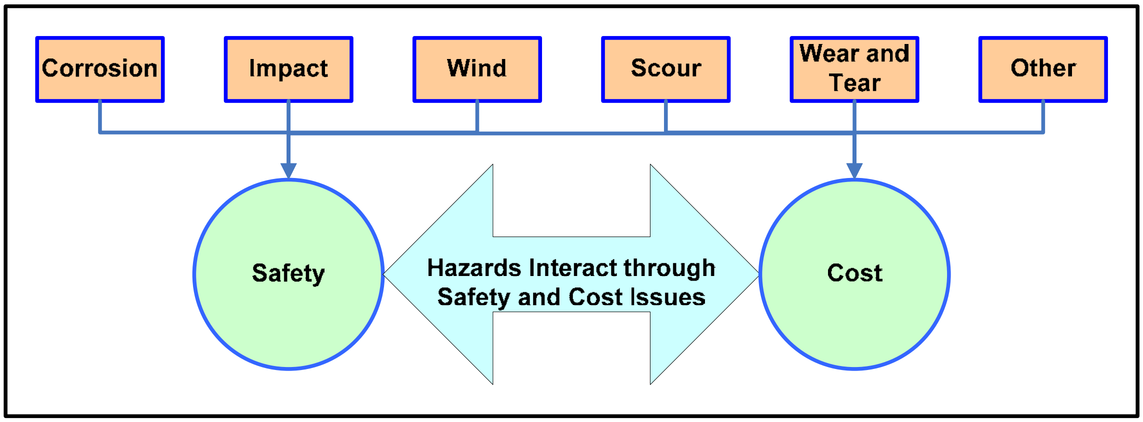

FIGURE 1 General multihazard approach in bridge management.

GENERAL METHOD FOR MULTIHAZARDS IN BRIDGE MANAGEMENT

In order to quantify management objectives (*MO*) as related to multihazard considerations, we define

$$MO = \sum_{i=1}^{i=N_{TOOLS}} MT_i \tag{1}$$

with MT_i as the ith management tool and N_{TOOLS} as the number of tools that are available to the manager. Management tools can be inspection, maintenance, or retrofit–rehabilitation. They can also be design, analysis, and operational costs. Equation 1 is a qualitative equation with arbitrary units. However, for generalization purposes, we can choose the units of Equation 1 to be monetary without any loss in accuracy. Thus, we can state formally that the management objective is to minimize Equation 1, subjected to the constraints

$$S_j \geq \overline{S}_j \tag{2}$$

The variables S_j and $\overline{S}_j$ represent the jth safety measure and the jth allowable safety measure, respectively. If we assume that the total number of safety constraints are N_{SAFETY}, then $j = 1,2,\ldots, N_{SAFETY}$. We note that Equations 1 and 2 formally represent *MO*. They are used in practice either subjectively by relying on the experience of the manager or objectively relying on computational tools such as Bridge Management Systems (BMS) (*3*) or other decision-making tools.

The summation in Equation 1 does not differentiate between different hazards; it only describes the different tools. We seek to describe the different hazards' effects in more detail. We can modify Equation 1 such that

$$MO = \sum_{i=1}^{i=N_{TOOLS}} \sum_{k=1}^{k=N_{HAZARD}} MT_{ik} \tag{3}$$

The number of hazards under consideration is N_{HAZARD}. The management tools now have double subscripts that indicate the dependence on the management tool itself as well as the type of hazard. The variable MT_{ik} is described as the cost of the ith management tool to accommodate the kth hazard. This is a more general form of MO. Unfortunately, it is not sufficient to address the multihazards issue. In order to do so, the double summation in Equation 1 can be generalized into a triple summation as shown below.

$$MO = \sum_{i=1}^{i=N_{TOOLS}} \sum_{k=1}^{k=N_{HAZARD}} \sum_{\ell=1}^{\ell=N_{HAZARD}} MT_{ik\ell} \tag{4}$$

Equation 4 will permit study of multihazard considerations for bridge management. Note that if

$$MT_{ik\ell}\big|_{k\neq\ell} = 0 \tag{5}$$

then Equation 4 reverts to Equation 3, which is the independent hazard condition. For quantitative study of multihazards in bridge management, we need to study the non-zero situations of $MT_{ik\ell}\big|_{k\neq\ell}$.

Perhaps a more intuitive form of Equation 4 is

$$MO = \sum_{i=1}^{i=N_{TOOLS}} \{CT\}_i^T \left[MHT\right]_i \{CT\}_i \tag{6}$$

where

$$MT_{ik\ell} = CT_{ik} \bullet MHT_{ik\ell} \bullet CT_{i\ell} \tag{7}$$

The size of the vector $\{CT\}_i$ is N_{HAZARD}. Similarly, the matrix $\left[MHT\right]_i$ is a square symmetric matrix of order N_{HAZARD}. The component in the kth row and ℓth column of $\left[MHT\right]_i$ is $MHT_{ik\ell}$. The kth and ℓth components of $\{CT\}_i$ are CT_{ik} and $CT_{i\ell}$, respectively. The cost of the kth hazard with regard to the ith management tool, if no other hazard is present is

$$Cost_{ik} = \left(CT_{ik}\right)^2 \tag{8}$$

The matrix is the multihazards matrix for the ith management tool. It becomes diagonal for no hazard interaction. The components of the matrix are dimensionless. We can, for simplicity, choose the components of the matrix such that

$$-1 \leq MHT_{ik\ell}\big|_{k\neq\ell} \leq 1 \tag{9}$$

If the ith management tool is pertinent to the kth hazard,

$$MHT_{ik\ell}\big|_{k=\ell} = 1 \tag{10}$$

and if the *i*th management tool is not pertinent to the *k*th hazard, then

$$MHT_{ik\ell}\big|_{k=\ell} = 0 \tag{11}$$

Hence for multihazards consideration, the manager should fill in the off diagonal components of $\left[MHT\right]_i$ with appropriate values. This can be accomplished qualitatively, based on the experiences of the manager, or quantitatively, using appropriate decision-making tools or structural health monitoring (SHM) experiments.

For the remainder of this paper, we study some management tools and present examples on how Equation 6 can be used to optimize the *MO*.

INSPECTION

Conventional bridge inspection is visual and is a major component of bridge management. It can be subdivided into three broad categories: scheduled, hazard specific, and special–unscheduled. Scheduled inspection covers several types of hazards, including wear and tear, corrosion, and fatigue. The hazard-specific inspections, before the event, cover hazards such as seismic and scour. The special or unscheduled inspections are conducted post-hazards, i.e., after a major earthquake or wind storm. Generally speaking, inspection practice does not follow a multihazard strategy. Because of this we ask can we change inspection procedures to accommodate multihazard considerations? Are there any benefits that can result from such a change? We use the analytical developments illustrated in the previous section to illustrate an objective method of developing an answer to these two questions.

Inspection is one of the management tools that is mentioned during the development of Equation 6, thus we have $i = 1$. Let us assume that an inspection process aims at only three hazards: normal deterioration (see Figure 2), earthquake resilience (see Figure 3), and scour resilience (see Figure 4). The manager estimates that the cost of inspection of each of these hazards can be estimated as (in relative terms) 1, 2, and 4 monetary units. Thus,

$$\left\{CT\right\}_{i=1} = \begin{Bmatrix} 1 \\ 1.41 \\ 2 \end{Bmatrix} \tag{12}$$

The inspection multihazard matrix is a square matrix of order of 3. The manager estimates its components to be

FIGURE 2 Corrosion damage.

FIGURE 3 Earthquake-type damage.

FIGURE 4 Scour damage.

$$[MHT]_{i=1} = \begin{bmatrix} 1 & -.3 & -.3 \\ -.3 & 1 & -.5 \\ -.3 & -.5 & 1 \end{bmatrix} \tag{13}$$

This indicates, in the opinion of the manager, that in the process of inspection, the scour inspection can also cover the seismic inspection with an added cost of about 50%. Similarly, the manager estimates that the wear-and-tear inspection can also cover the seismic and scour inspection with an additional cost of 30% each. If no multihazard considerations are taken, the multihazard matrix is

$$[MHT]_{i=1} = \begin{bmatrix} 1 & 0 & 0 \\ 0 & 1 & 0 \\ 0 & 0 & 1 \end{bmatrix} \tag{14}$$

Using Equations 6–13 with Equation 14 would reveal the relative cost savings for the inspection management tool if a multihazards approach was taken.

BRIDGE GUIDES AND MANUALS

There are currently numerous bridge rating or vulnerability manuals and guides for different hazards (*4*, *5*). These manuals and procedures form the basic tools of bridge management. We note that there are some interactions among the hazards that are the subjects of those manuals in the manner that they affect the bridge. Thus, these rating guides or manuals include some measure of interaction between the hazard effects on the bridge. A close inspection of the contents of the manuals or guides reveals that many of the contents are fairly similar, which indicates some level of redundancy in the evaluation process. If such redundancies are reduced or eliminated, then the process can become much more efficient. This is exactly what multihazards consideration aims to achieve. The concept is shown in Figure 5.

Consider the possibility that the interaction among the hazards in different guides or manuals is accounted for; then there would be a net increase in efficiency in the bridge cost of ownership, without any loss of safety measures. Of course accounting for the hazard interaction (removing redundancies) among the manuals or guides would entail additional costs, such as cost of modifying the manuals, cost of retraining, and cost of implementation of the modified manuals. Note that those costs are one-time costs; they are not recurring costs. On the other hand, the benefits of multihazard considerations in the manuals or guides include more efficient and less redundant procedures, safer results since all hazard interactions are accommodated, and less overall training costs. In addition, those benefits are recurring benefits. Thus it is obvious that there is a value for including multihazard considerations for bridge manuals and guides.

REPAIR AND RETROFIT

Figure 6 shows how multihazards can affect a given bridge structure. Because of this, we discuss the questions: can we employ a multihazard strategy for bridge repair–retrofit efforts? Also, can we employ a SHM strategy to utilize the potential benefits of multihazard considerations in the repair–retrofit effort?

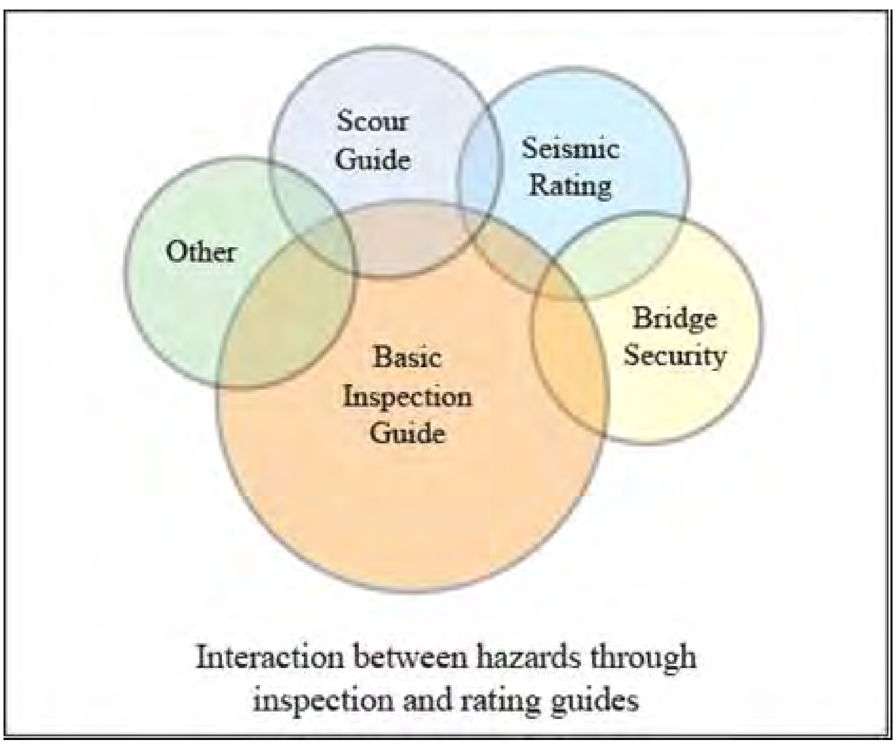

FIGURE 5 Interactions between hazards through inspection and rating guides.

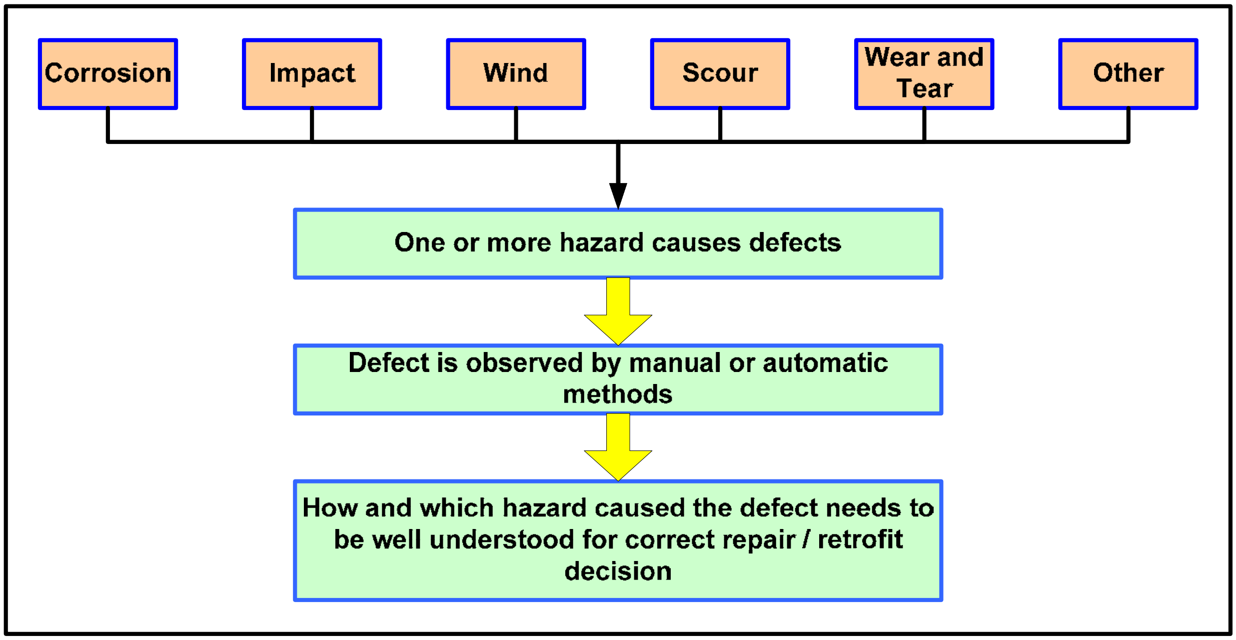

FIGURE 6 Logic of considering multihazards in repair–retrofit efforts.

Steps of Retrofit Efforts

In order to study multihazard effects on the repair–retrofit of bridge management, we observe that there are four components: identifying needs–cause, identifying level of repair–retrofit, securing budget, and performing the actual work. Each of these steps includes the potential of multihazard considerations. We discuss the first three components below. Discussion of the fourth component is beyond the scope of this paper.

Need and Cause

The interrelationships between the needs and causes of different hazards are shown in Figure 6. The extra costs of considering multihazards versus additional benefits must be weighed for the most efficient management approach. Figure 7 shows the disadvantages of a single-hazard approach to retrofit–repair situations. Figure 8 shows specific examples of how observed damage can be approached in a multihazard manner.

Level of Retrofit

In deciding on the level of retrofit for a particular situation, it is prudent to consider other potential hazards while making the decision. In many cases, expending additional financial resources can help save future costs and improve performance of the system. See Figure 9 for an illustration of this concept in a scour and seismic situation.

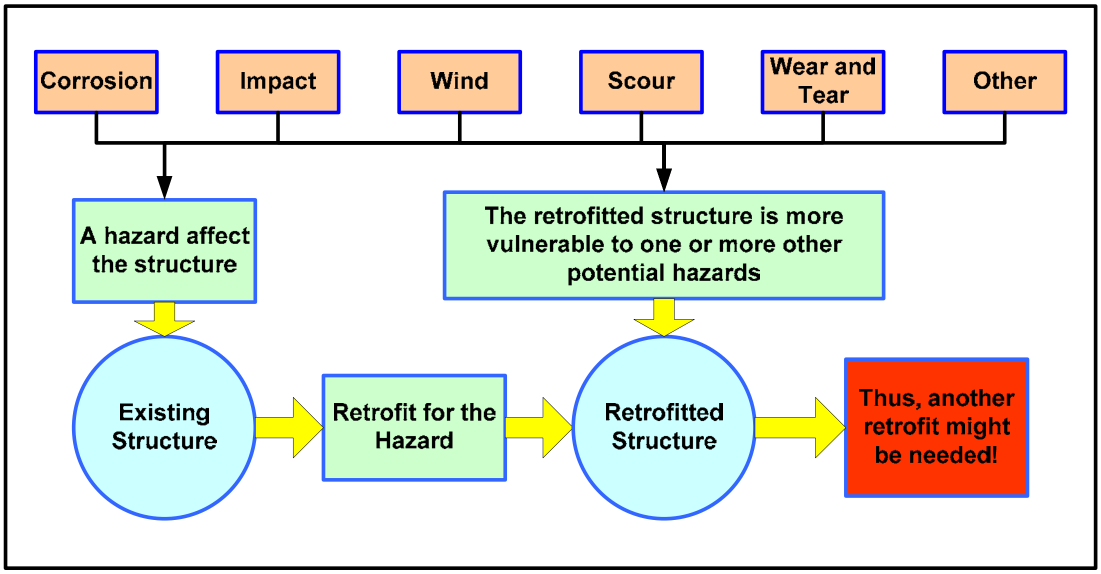

FIGURE 7 Disadvantages of a single-hazard approach to repair–retrofit.

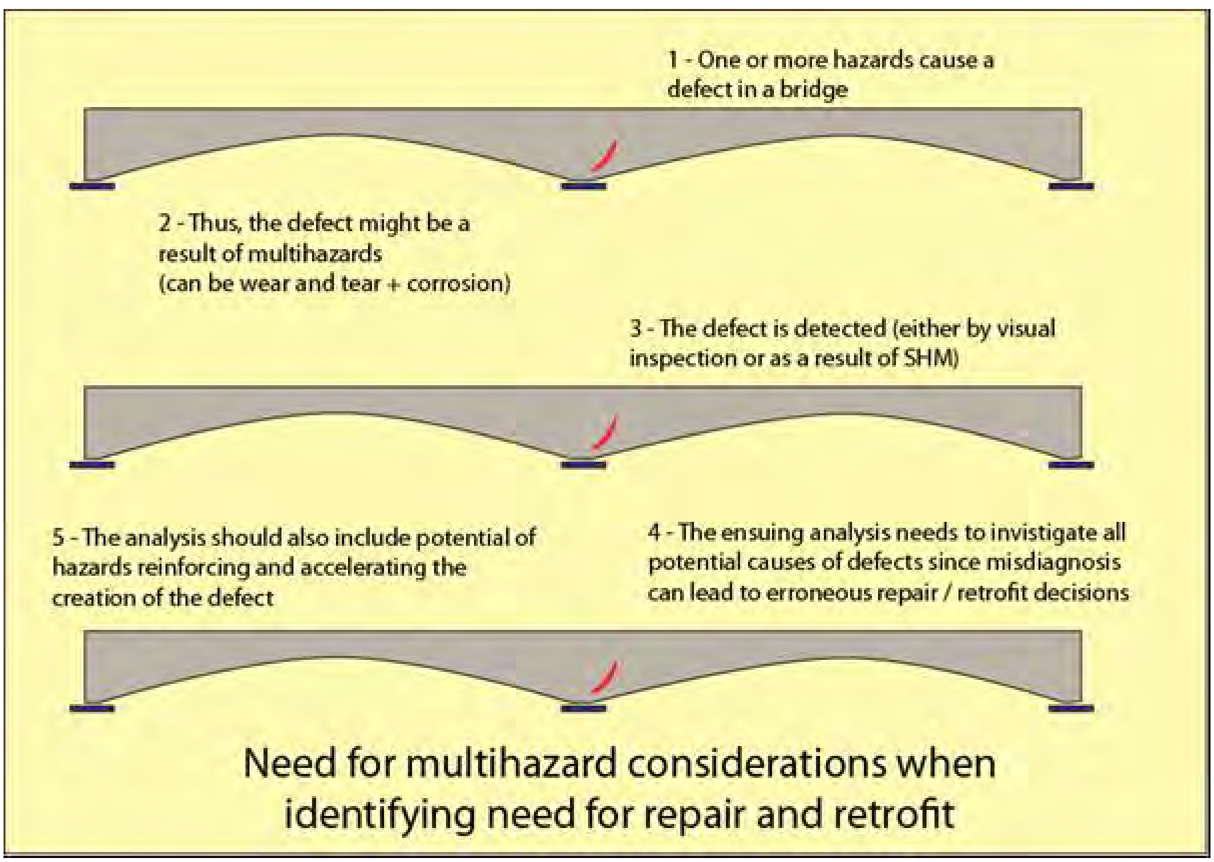

FIGURE 8 Interactions of hazards through damage.

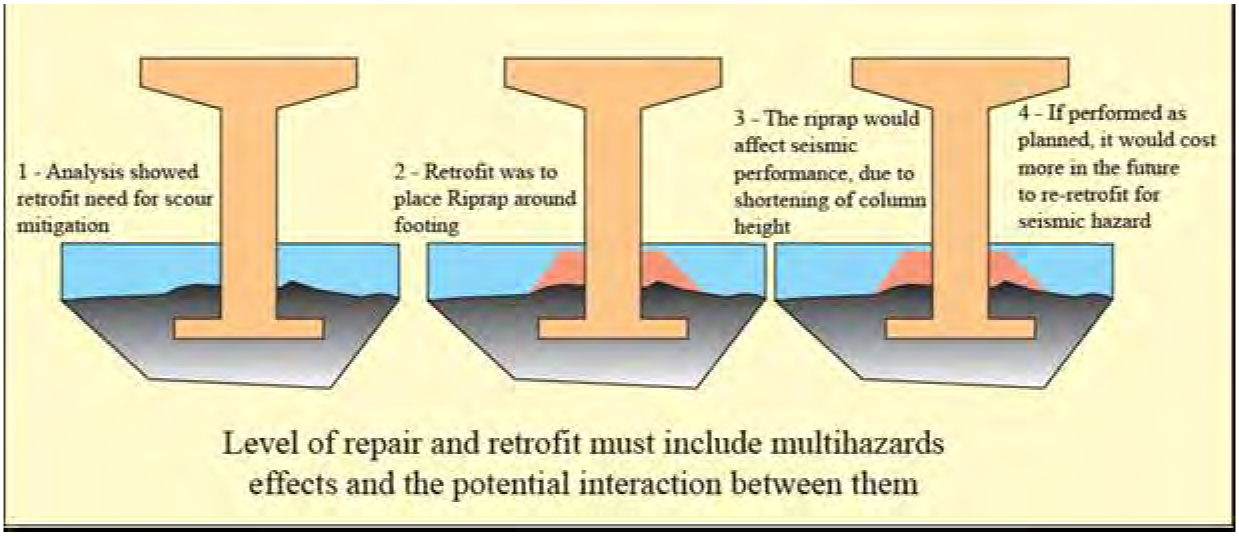

FIGURE 9 Multihazards and level of retrofits.

STRUCTURAL HEALTH MONITORING

The potential SHM role in bridge management considerations of multihazards is shown in Table 1.

EXAMPLES

Net Value of Multihazard Considerations

We note that Equation 6 utilized the cost and cost reductions in evaluating the net value of multihazard considerations. There is a different approach for evaluating the net value. It is based on direct evaluation of the cost of pursuing a multihazard approach as

$$C_{multihazards} = \sum_{i=1}^{i=N_{HAZARDS}} C_i \tag{15}$$

Budget

There are three main components of the budget of any repair–retrofit: initial costs, discount rate, and service life of both the bridge and the repair–retrofit itself. We note that all of these budget components are dependent on multihazard considerations: they all interact through the different hazards as shown in Figure 10.

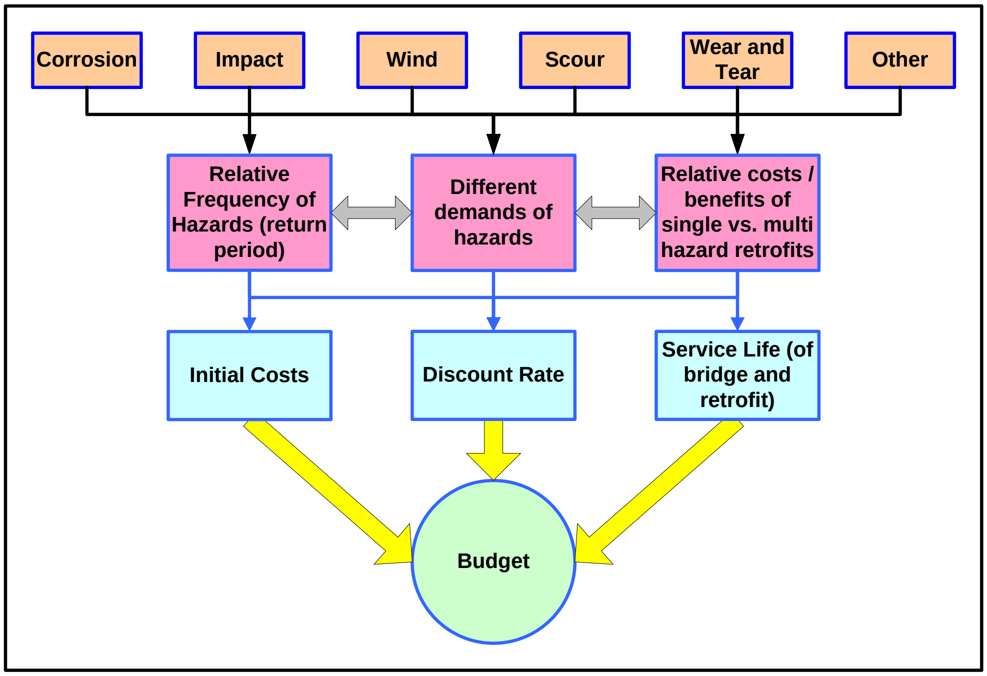

FIGURE 10 Multihazard considerations and budgeting efforts.

TABLE 1 SHM Role in Bridge Management and Multihazards Considerations

Issue	SHM Role
Structures	– Structural behavior – Failure mechanisms – Material behavior – Design methodology – Analysis methods – Common sense
Hazards definition	– Loading descriptions – Long-term effects on structure – Interaction with other hazards – Influence on analysis – Influence on design – Mitigation methods
Sensing, measurements and instrumentations (monitoring)	– Sensors – Instrumentation – Data collection methods – Data mining – Data analysis – Long-term durability aspects
Decision-making tools	– Financial – Reliability – Financial analysis - Estimating techniques - Benefit–cost analysis

Also, the net benefits of pursuing a multihazards approach is

$$B_{Multihazards} = \sum_{i=1}^{i=N_{HAZARDS}} B_i \tag{16}$$

where C_i and B_i are the net cost increase and the net benefit for the *i*th hazard. The net value for pursuing a multihazards approach is

$$V_{Multihazards} = B_{Multihazards} - C_{Multihazards} \tag{17}$$

Another form of computing the value of multihazards considerations is the benefit-to-cost ratio (*BCR*)

$$BCR = \frac{B_{Multihazards}}{C_{Multihazards}} \tag{18}$$

Using Equation 18, if $BCR > 1$, then multihazard considerations is the optimum management procedure.

Monte Carlo Simulation

A potential method for estimating the multihazard matrix $\left[MHT\right]_i$ for a given management tool is the Monte Carlo simulation tool. This can be accomplished by first estimating the applicable hazards for the *i*th management tool, e.g., particular retrofit option (say a suggested hardening scheme that might increase column resistance to blast loading). The manager knows that this particular retrofit should have an effect on the seismic behavior of the column and the corrosion resistance of that column. Because of this, the order of $\left[MHT\right]_i$ is three. In order to accurately estimate the off-diagonal components of $\left[MHT\right]_i$, the manager performs a Monte Carlo simulation effort using the range of hazard space for the three hazards as inputs. The proposed retrofit is used as the basis of the analysis, and the computed structural responses are used to compute the off-diagonal terms of $\left[MHT\right]_i$, i.e., how the proposed retrofit would affect the behavior of the column for the other hazards. Armed with the completed matrix, the manager can now use Equation 6 to estimate the cost of the retrofit that accurately accounts for the desired multihazard effects. Obviously, the use of Monte Carlo simulation is time consuming and is only justified in important and large-size projects. The complete coverage of this example is beyond the scope of this paper.

CONCLUDING REMARKS

In this paper, we discussed several aspects of bridge management as they relate to multihazard considerations. We offered simple quantitative techniques that can quantify the potential benefits of this important issue. There are numerous knowledge gaps that are needed in order to fulfill the promise of multihazards usage in bridge management. These include improved automated design, analysis, and decision-making tools for processing the multihazard approach; integrating tools such as structural health monitoring into a comprehensive multihazard approach; and modifying manuals and guides to accommodate multihazard strategies. Multihazard considerations in bridge management can achieve increased safety at a better value for a given and limited financial resource.

ACKNOWLEDGMENT

All of the views presented in this paper are those of the authors and not necessarily of their organizations.

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Innovative Bridge Weigh-in-Motion System for Bridge Maintenance

A Case Study with Bridge on Highway I-59

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One of the primary causes of bridge failure is overloaded commercial vehicles. In order to prevent deterioration of a bridge it becomes important not only to design and build bridges according to the regulations but also enforce commercial vehicle maximum weight standards. For these reasons, a bridge weigh-in-motion (B-WIM) system should be a key asset in maintaining the balance sought. The B-WIM device placed on the soffit of a bridge includes strain transducers and, usually, separate axle detectors placed on the deck, to determine axle loadings, axle spacing, speed, and gross vehicle weight as a truck moves across the bridge. B-WIM devices have become an important means to better enforcement and better bridge design in many international countries. The objective of this paper is to gather as much information as possible about existing and emerging B-WIM systems in order to determine its potential for implementation in the United States. The information sought includes the benefits of using B-WIM, the steps involved in implementing its technology in the United States, and how to evolve the current systems. The paper will summarize an assessment study that was carried out on an Interstate 59 (I-59) bridge structure in Alabama with a B-WIM (Bridge Weigh-In-Motion) system called SiWIM, developed in Slovenia by ZAG and CESTEL. The main objective of this study was to assess the performances of the system based on the preliminary data, and to acquire experience on its implementation, in order to elaborate technical rules for the choice of bridge types suitable for B-WIM, and how to design their instrumentation. The accuracy classes of the system with respect to the specifications will be assessed. The data was analyzed with regard to overloading on axles (single axle or axle group) or gross weight overloading and to calculate what proportion of the loaded vehicles was overloaded, either on axles or relative to the vehicle's gross weight. The effect on the road structure was also analyzed, because dimensioning for the road structure is based on knowledge of the traffic or on an ability to assess the volume of traffic accurately. Moreover, the damaging impact of an overloaded vehicle may vary, in that the number of standard axle loads per vehicle factor and the incidence of overloading are not covariants. Therefore, measurements were carried out to gain a picture of the make-up of the heavy traffic in terms of loads and the incidence of overloading. By gaining a picture of the make-up of the heavy traffic in terms of loads and the incidence of overloading we can establish a foundation in fact for (*a*) appropriate actions to achieve better compliance with regulations (*b*) planning of road maintenance, and (*c*) dimensioning the road structure in road building and road maintenance.

Transportation infrastructures play an essential role in the strength and growth of the nation's economy due to their high cost and direct impact of public safety. However, many issues are likely to accelerate the deterioration of the existing bridge structures, for instance, the aging of bridges, increased traffic volume, and low-level maintenance decision-making. Of particular concern is the increase in the number, size and weight of heavy commercial vehicles on the nation's highways, which has made administration and legislation difficult for the authorities. The global economy is expanding rapidly, and the result has been a significant expansion of freight shipments on a highway system network that is already overloaded along certain shipping corridors and at key network hubs. In United States a large majority of transported freight to be trucked is required on the nation's highways. Also, the constantly increasing volumes of heavy vehicles caused an urgent necessity to protect transportation infrastructures from fatigue and enforce strict truck weigh limits.

Illegally overloaded and oversized vehicles damage bridges and pavements and create safety hazards. This hastens the need to inspect heavy vehicle flow on highways and thereby reduce deterioration of our roads and bridges and improve traffic operations. Because of the complexity and size of the highway network and the limited resources available to enforcement agencies, an effective program of highway maintenance and safety could benefit substantially from an affordable sampling and enforcement program that is not manpower intensive. A portable dynamic sampling methodology capable of reliably delivering accurate measurements of vehicle type, size, and weight from moving vehicles would be very attractive.

In United States, of the approximately 600,000 bridges built before 1940, 220,000 (40%) of the total bridges are considered deficient or of obsolete design, and are eligible candidates for Highway Bridge Replacement and Rehabilitation program (HBRRP) funding. FHWA has estimated over $51.0 billion is needed to keep roads and bridges in their current state of repair (Robert 2000). In order to preserve the bridge performance in a safe and serviceable condition for success of the infrastructure system, the comprehensive bridge management system (BMS) has been established to determine optimal strategies within budgetary constraints for maintenance, rehabilitation, and replacement (MR&R), based on the bridge performance evaluation results. It is obvious that one of the major components of BMS is the knowledge of bridges, also namely data collection and interpretation on the truck type, axle weight, gross weight, space of axles and speed provides valuable information to state highway agencies and greatly assist in determination of planning, administration and enforcement techniques required to reduce the number of overweighed trucks on the roads.

To weigh heavy vehicles on a static scale at low speed or on portable pads is the common practice. Although these processes are the most accurate way, they cost much on labor and can just give sample of the traffic data on the highways. Also, they can not provide vehicle classification and axle spacing. The most undesirable issue of these procedures is delays to trucks.

However, these limitations can be overcome if vehicles can be weighed while moving on highways not having to deviate from their original route, which is the most attracting advantage of weigh-in-motion (WIM) systems. WIM systems allow for the unobtrusive and continuous collection of vehicle weight information. WIM system can provide increased highway efficiency and vehicle classification and axle spacing. With the survey that the commercial truck traffic is likely to increase greatly over the next 10 years, these systems should be used nowadays for a wider range of tasks, which includes the management and maintenance of highways and other infrastructures, bridge, culvert and etc. (Yiannis 2005).

In recent years advanced weigh-in-motion systems applied to bridges (B-WIM) have shown enormous potential in detecting oversized and overweight commercial trucks in Europe.

Interest within the United States has grown as evidenced by the recent UTCA B-WIM technology Request for Proposals.

LITERATURE SURVEY ON DIFFERENT WEIGH-IN-MOTION

The most common technologies of sensor types in WIM system in practice include piezoelectric sensors, capacitive mats, bending plates scale, double bending plate scale, single load cell systems (deep pit load cell), optical WIM, and Bridge-WIM systems (Center, 1997; Yiannis, 2005).

For weighing with static scales, it is very easy for these overloaded or oversized trucks to avoid these peak hours. While with high-speed and imperceptible WIM systems, a continuous unbiased weighing data of all vehicles can be provided when the vehicles pass the system.

WIM systems are to obtain accurate axle load and gross weight to support transportation planning and decision making purposes. Also, bridge WIM can offset the dynamic effects as bridges are long compared to the pavement WIM system, which will have less effect on accuracy of bridge WIM systems than they have on the pavement ones (Zag, 2003).

Requirements of WIM System

WIM system is greatly influenced by its surrounding sites. The American Society for Testing and Materials (ASTM) gave some guidelines for the installation of effective WIM system. Most important factors are the road geometry, pavement characteristics including not only the longitudinal evenness but also deterioration. The geometric features of the site influence the accuracy of the dynamic and static load measured at the site (Center, 1997).

The major concern of environmental requirements is climatic conditions, traffic conditions and the facilities needed to install and operate the WIM system will be taken into consideration as well. As for sensors, first of all, they have these temperature requirements; Second, they must be insensitive to water and salt exposure. Except for this, the traffic conditions and mechanical resistance, electronics, facilities and other must also be considered (Cost, 1999).

WIM System Evaluation and Comparison

The most important issue in WIM installation is how to select an appropriate sensor technology. The balance between cost and accuracy should be considered. Bushman and Pratt analyzed three basic types of WIM sensor technology (piezo-electric, bending plate and single load cell) in terms of accuracy and cost (Bushman, 1998; Yannis, 2005).

Taylor and Bergan did reseach on WIM system between different level of accuracy and different system and maintenance costs (Taylor and Bergen, 1993). The comparison of WIM system is summarized in Table 1.

The main advantages of WIM systems over their static counterparts include the following: (*a*) free traffic flow is ensured; (*b*) better rates of processing are provided; (*c*) more truthful data is provided; (*d*) the sensors in WIM systems are less influenced by dynamic effects; (*e*) dynamic weights of heavy vehicles are provided; and (*f*) more sites can be monitored with WIM systems when compared to static weighing stations, which are expensive to set up and operate.

TABLE 1 Comparison of WIM System (Center 1997)

	Piezoelectric Sensor	**Bending Plate Scale**	**Double Bending Plate Scale**	**Single Load cell (Deep Pit Load Cell)**	**B-WIM System**
Accuracy (95% confidence)	±15%	±10%	/	±6%	±5%
Performance (percent error on GVW at highway speeds)	±10%	±5%	±3%-5%	±3%	±5%
Expected life	4 years	6 years	/	12 years	/
Estimated Initial Cost per Lane (Equipment and Installation)	$9,500	$18,900	$35,700	$52,500	/
Initial installation cost	$9,000	$21,500	/	$48,700	/
Estimated average cost per lane (12-year life span including maintenance)	$4,224	$ 4,990	$7,709	$7,296	/
Annual life-cycle cost	$4,750	$6,400	/	$8,300	/

BRIDGE WEIGH-IN-MOTION SYSTEM

Bridge WIM is a WIM system using an instrumented bridge as a large sensor, and the strains measured in some of the bridge elements are used to determine the gross weights and axle loads of vehicles crossing the bridge (Cost, 1999). Apart from collecting data about the vehicle parameters, the B-WIM system also supplies information about the impact factor, lateral distribution factor and strain records which can be further used for the analysis of bridges. Due to its unique features, B-WIM is now a very established technology of traffic data collection in many countries around the world. Nowadays, Bridge WIM system replaces traditional one using axle detectors on the pavement with NOR (Nothing On the Road) or FAD (Free-of-Axle Detector) (Zag, 2003). Measurements during the entire vehicle passing over the structure provide spare data, which facilitates solving difficulties due to dynamic effects from vehicle-bridge interaction. This is a major advantage over the pavement WIM systems where measurements of an axle last only a few milliseconds.

Bridge WIM method was developed by Moses and his team in 1979. They proposed and implemented a new idea to use existing instrumented bridges from the road network to weigh vehicles in motion. Bridge WIM is particularly appropriate for short term measurements as it can be easily installed and detached from the bridge and is the only WIM system, where fully portable and permanent installations provide equal accuracies of results (provided that the same calibration procedure is used). Additionally, bridge assessment by supplying additional ‘structural’ data can be obtained by the B-WIM system. B-WIM systems can be further enhanced by adding video technology to provide visual data to the user (Zag, 2003).

The main advantages of a portable B-WIM system are: (*a*) it can be employed to monitor truck weight and size without interfering with traffic flow; (*b*) portable installations are not visible to truck traffic as it crosses the instrumented bridge; (*c*) can be installed without damaging the pavement or interfering with the traffic; (*d*) can be moved from one location to another without influencing accuracy of the results and (*e*) enhanced data quantity and quality to support pavement design, bridge–structural design, transportation planning, and traffic safety.

At this stage of development many of the potential benefits of B-WIM in terms of traffic safety and maintenance are largely anecdotal. The elements of this project are important steps to take if the full range of potential applications of the technology is to be realized. B-WIM technology research has been identified as a 2008 PRIORITY by FHWA, and UAB won the UTCA applied research project to install the latest generation Bridge WIM system.

Particular Requirements for Bridge-WIM System

According to COST 323, the denser the traffic, the shorter is the optimal length of the structure. During the inverse calculation of the weight, an influence line based on actual strain readings not on theoretical analysis can prove more accuracy (Cost, 1999).

O'Brien and Žnidarič (2001) extended applicability of modern bridge WIM systems to other types of bridges, such as short slabs and long span bridges. On the long-span ones measurements are taken on parts of the spans between the secondary structural elements in the lateral direction, e.g. stiffeners. Typical such structures are steel box girders and steel orthotropic deck bridges (Zag, 2003).

Table 2 summarizes optimal, acceptable criteria for bridge selection and the information about bridge B007239 on I-59.

SIWIM System

SiWIM does not only provide the same traffic data as the pavement WIM systems, but also some additional, measured structural parameters that can be used for optimized bridge assessment. SIWIM system also use NOR or traditionally FAD for detecting the axle loads.

If we calculate the influence line based on the strain records acquired at the site, we can ensure the right way to represent bridge behavior in a B-WIM algorithm. During the calibration test, SiWIM system can do this without requiring any detailed dimensions or structural information of the bridge (Zag 2003).

The WIM analysis is an inverse-type problem in that the structural response (bending moment) is measured, but the live loads causing this moment must be calculated. The basic principles of bridge WIM algorithm is based on the comparison of measured and modeled bending moments of a span due to a passing vehicle (Figure 1). The calibration factor can be obtained during the calibration with trucks of known weight and dimensions. And the sketch of SiWIM system was listed in Figure 2.

$$M_E(t) = A_1 I(x) + A_2 I(x - L_1) + A_3 I(x - L_1 - L_2) \tag{1}$$

TABLE 2 Ideal and Acceptable Criteria for Some of the Basic Bridge Characteristics (Cost 1999, Zag 2003)

Criteria	**Ideal**	**Acceptable**	**Bridge B007239**
Structural material	Reinforced concrete, prestressed concrete, steel	Concrete, masonry, stone	Reinforced concrete
Superstructure or bridge type	Slab, beam/deck systems (Steel/prestressed concrete/reinforced concrete girders), culvert, steel orthotropic decks	Arches	Slab, beam systems
Traffic density	Free traffic—no congestion		Free traffic—no congestion
Span	More than 1 or 2 spans	More than 2 spans	9 spans
Span length	length 16 to 32 ft 6 to 16 ft for AD only	Length 3 to 16 ft Length 15 to 50 ft	Length = 34 ft = 10.36 m
Skewness	0° to 20°	20° to 45°	0°
Supports	Fixed (integral bridge)	Simply supported	Simple supported
Vibration	<10% of static values	<30% of static values	<10% of static values
Pavement	Smooth, no bumps	Small bump	Smooth on the centre of the bridge, pavement on the approach to the bridge is uneven.

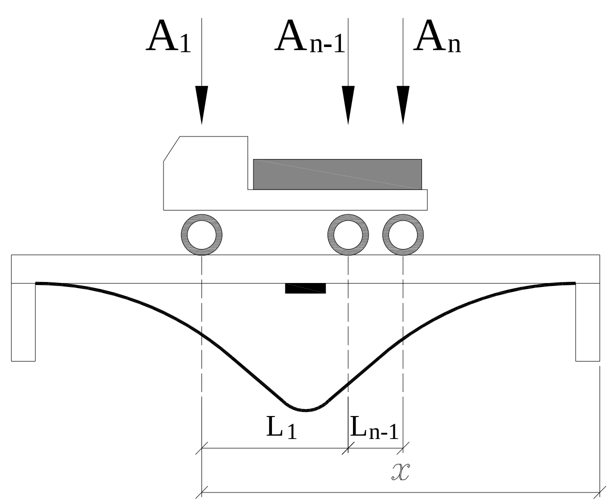


FIGURE 1 Combination of individual axle contributions to the bending moment at the point of measurement (Zag, 2003).

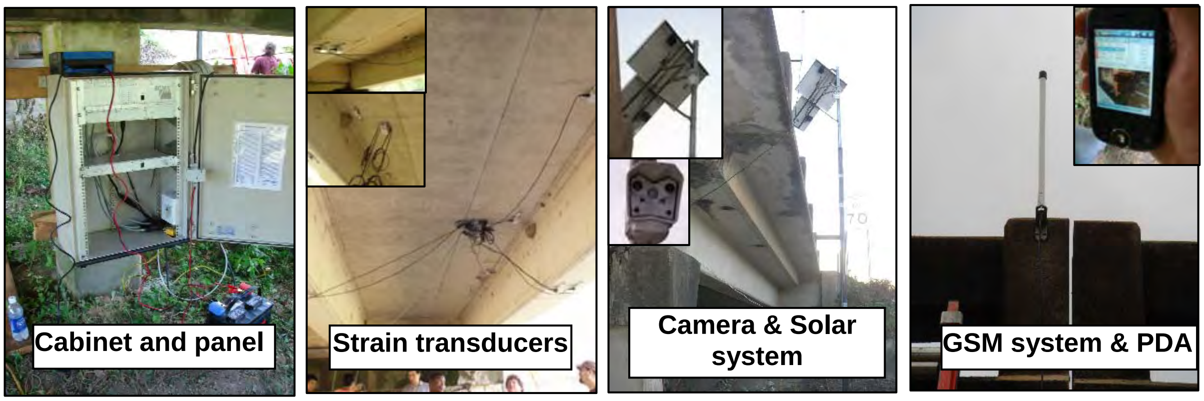


FIGURE 2 Figures of SiWIM system.

Influence Lines And Bridge Calibration

Among all the parameters of a B-WIM installation, the influence lines (IL) of the bridge are the most important one. In SiWIM system, the influence line is defined as the IL of bending moment at the point of measurement (strain transducer location). The more accurate of the IL, the more accurate of the axle weights can be obtained. Poor IL can provide reasonably good estimate of the gross weight, but the axle weight would be redistributed very differently with the actual one (Zag, 2003).

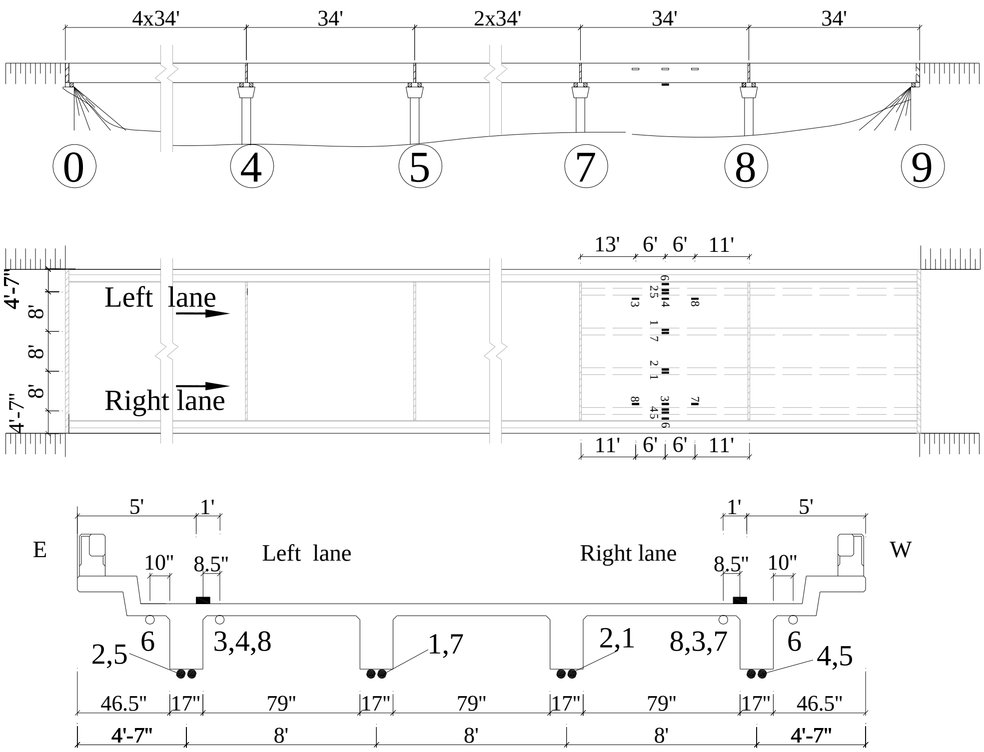
Trucks of known weight are used to calibrate all bridge WIM systems. It is recommended to use a test plan according to the European WIM specification. And when the vehicles are selected for calibration test, it is better to select those which can represent the main traffic classification. The appropriate test program is selected mainly based on the trade-off between cost and target accuracy. The more accurate the results, the more elaborate and expensive test plan is needed (Zag, 2003).

Accuracy is evaluated according to the European specifications for WIM by comparing WIM results to the values obtained on a more accurate static scale. "COST 323 specifications define an accuracy class with a letter and a number in the parentheses. Class A(5) is the most accurate one and is followed by classes B+(7), B(10), C(15), D+(20), D(25) and E(30). The number in parentheses is the confidence interval δ at the confidence level π of approximately 95 % of all results. The exact confidence level depends on number of test vehicles, on type of the check (initial calibration or subsequent validation) as well as on the test and the environmental conditions" (Cost, 1999).

CALIBRATION TEST FOR BRIDGE B007239 ON I-59

Bridge Information

Bridge B007239 lies on Highway I-59 near exit 166 over Muckleroy Creek in St. Clair County in Alabama. The bridge is a nine span simple supported T- beam bridge with span 9 × 34 ft = 306 ft. Figure 3 shows the elevation, plan and cross section of the bridge.



Note:

1. "▬", "●"----8 strain transducers ST500 for weighing;
2. "▭", "○"----8 stain transucers ST500 for axle detection

FIGURE 3 The position of the weighing and FAD sensors.

For SiWIM instrumentation eight strain transducers were mounted on the soffit of the beam at the eighth span from the north of the structure. To detect axles and speed, four additional strain transducers (left lane: No. 3 and 8; right lane: No. 8 and 7) were placed just beneath the slab and 12 ft apart. Additionally, there are four extra strain transducers which are mounted on the slab for detecting the axles.

Calibration Test Plan and Result

SiWIM was used on the B007239 Bridge which was instrumented and calibrated for this purpose between October 18 and 23, 2007. Bridge B007239 was calibrated according to limited reproducibility (R1) under environmental repeatability (I) which stands for 2 to 10 different trucks and the calibration test was short measurements in mostly constant environmental conditions. As we found that the representative vehicle of I-59 would be semi-trailer, hence, five-axle semi-trailer trucks from traffic flow were used for calibration as preweighed trucks. The accuracy of the calibration was in Table 3.

TABLE 3 Accuracy Result

Criteria	No.	Identif-ication	Mean	St. dev.	π_o	Class	δ	δ_{min}	$\delta_{criteria}$	δ_{class}	π	π_c	Accepted Class
		(%)	(%)	(%)	(%)		(%)	(%)	(%)		(%)	(%)	
Gross weight	6	100.0%	0.00	1.98	73.1	B(10)	10.0	8.0	8.0	10	97.0	99.0	C(15)
Group of axles	12	100.0%	–0.60	2.64	87.1	B(10)	13.0	13.0	10.0	10	99.9	99.9	
Single axles	6	100.0%	0.00	4.68	73.1	C(15)	20.0	16.0	11.0	15	93.6	97.7	

Attained accuracy class and is generally higher than π_0 and δ = confidence interval.

TEST RESULT

Traffic Analysis

The truck vehicles of I-59 are illustrated in Figure 4. At the same time, the gross weight histograms of heavy vehicles in both lanes could be obtained for calculating the probability functions of traffic loading in both lanes. All these data can be used for accurate traffic planning and reconstruction of roads.

System can also be used for selecting of potentially overloaded trucks from traffic flow and also to tell us, when overloaded traffic is to be expected. With WIM measurements we can get information about places, date and time when and where overloaded trucks appear. With WIM system we can also monitor number of overloaded trucks in particular road sector. And

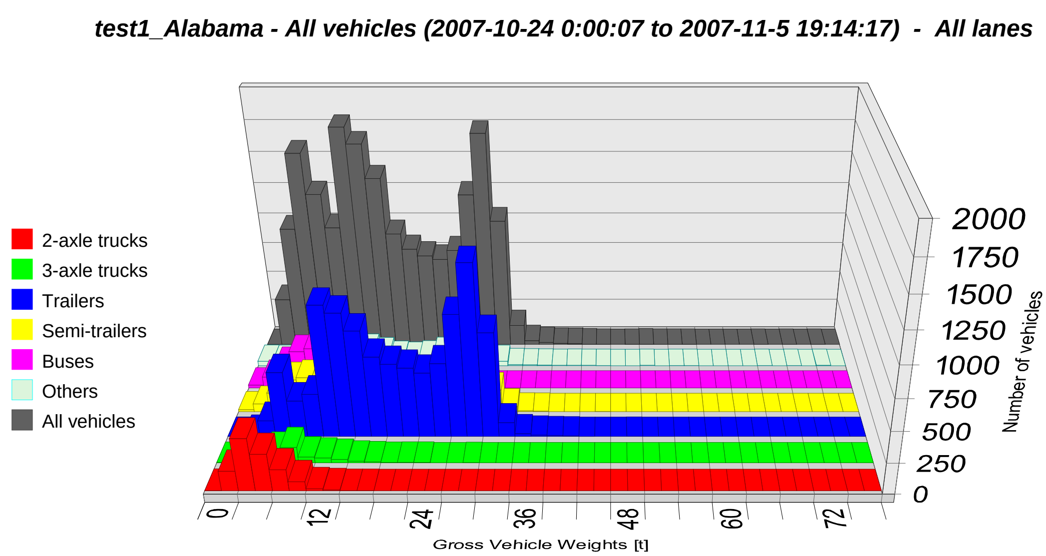


FIGURE 4 Heavy vehicles according to GVW.

from SiWIM, density diagrams are shown can be obtained. And on this basis, we can see the specific time that density of heavy vehicles decrease and increase.

During the test, two vehicles were noted with GVW over 120 kips were captured during that period and they were inspected by SiWIM.

Characteristics of Traffic on Highway I-59

Traffic loading was calculated with converting axle loads into the equivalent single axle loads (ESALs). Method used the reference axle loads according to the Swedish code, which specifies single, double and triple axle loads limits (10 metric tons for single, 18 metric tons for double and 24 metric tons for triple axles). These results were compared to the current procedure, which in absence of WIM data, assumes 1.3 ESALs per heavy vehicle to calculate cumulative ESAL values on a road section. As expected, the measured results differ from the assumed values.

Table 4 list the Cumulative ESAL factors and overloading information during the time interval 10:47 a.m. (October 23) to 15:16 p.m. (October 25).

Bridge Assessment for Maintenance

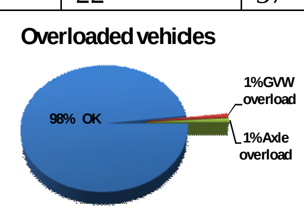
When optimizing assessment of existing bridges, which are most likely heavily deteriorated and thus suffer from reduced carrying capacity. However, many heavily deteriorated bridges can still

TABLE 4 Cumulative ESAL Factors and Overloading

Total	Lane 1				Lane 2			
	All Axles	ESAL	Overloaded Axles	ESAL	All Axles	ESAL	Overloaded Axles	ESAL
Single	3934	2615.9	30	145.8	3010	1252.5	19	89.6
Double	3283	2554.2	12	30.6	2394	1850.1	8	8.1
Triple	11	19.4	1	0.6	14	11.5	1	0.3
More	1	0.2	0	0.0	0	0.0	0	0.0
Total	7229	5189.7	43	177.0	5418	3114.1	28	98.1

	Lane 1			Lane 2		
	All Vehicles	Overloaded		All Vehicles	Overloaded	
		GVW	GVW+AL		GVW	GVW+AL
Trucks	2356	22	47	1818	22	37

44 of 4,174 vehicles were overloaded only by GVW, equivalent to 1.0%.
40 of 4,174 vehicles had at least one axle overloaded, equivalent to 1.0%.
Together, 84 of 4,174 vehicles were overloaded, equivalent to 2.0%.



NOTES: Traffic loads from number of heavy vehicle $ESAL_{spec}$ = 2,713 ESAL/day.
Traffic loads from SiWIM data $ESAL_{WIM}$ = 4,289 ESAL/day.
Calculated factor from WIM data is 2.06 per vehicle.

carry safely the normal traffic. Yet, this sufficient safety can only be proved by the optimized structural safety assessment procedures. There, it is essential to use in the assessments as accurate realistic data as possible. While the most adequate resistance and dead load data are obtained from the in-depth inspection of the structure, the best technique for the reliable traffic load acquisition is the use of a WIM system. WIM results are used either to model the site-specific loading schemes or to develop the appropriate rating (assessment) loading schemes. As a result, uncertainties of the calculation models can be considerably decreased which generally results in less severe or even abolished actions on bridges.

Similarly, without the WIM data the highway asset owners and managers are not aware of and cannot properly respond to the problems of overloading. In other words, they cannot perform efficient enforcement and take measures needed for optimal maintenance and reconstruction of road infrastructure.

Load Modeling

Figure 5 presents the gross weight histograms of 4,174 heavy vehicles in both lanes, which were used to calculate the probability functions of traffic loading in both lanes on I-59 south bridge.

Using the convolution method, which assumes that maximum loading effect is achieved due to two vehicles side-by-side on a span a maximum expected gross weight of 200 kips was obtained for a period of 20 years (Figure 6*a*). The long 20-year period was used as the evaluated structure was expected to last for at least this period of time.

When the WIM measurement is performed, the following live load model on the bridge can be modeled using the formula (Mores, 1987):

$$Q = a \times W_{0.95} \times H \times m \times I \times g \tag{2}$$

where Q = predicted maximum live load effect; a = deterministic value relating the load effect to a reference loading scheme; $W_{0.95}$ = characteristic vehicle weight; H = headway factor; m = factor reflecting the variations of load effects; I = impact factor; and g = lateral girder distribution factor. All parameters, except a, are random variables and are evaluated from the WIM data.

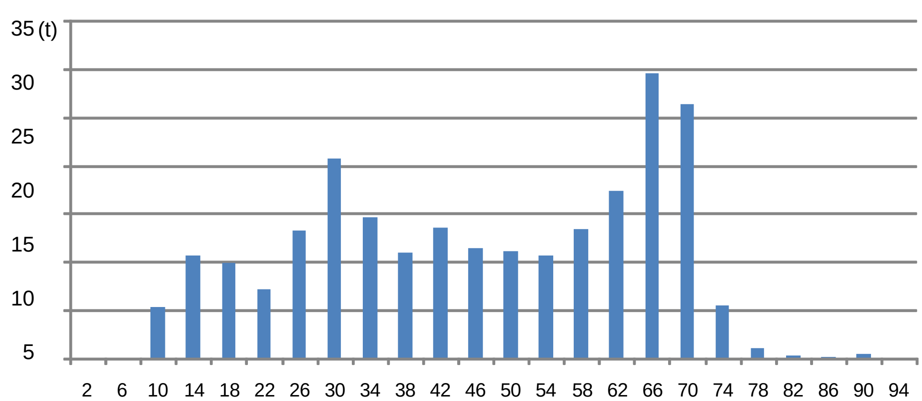
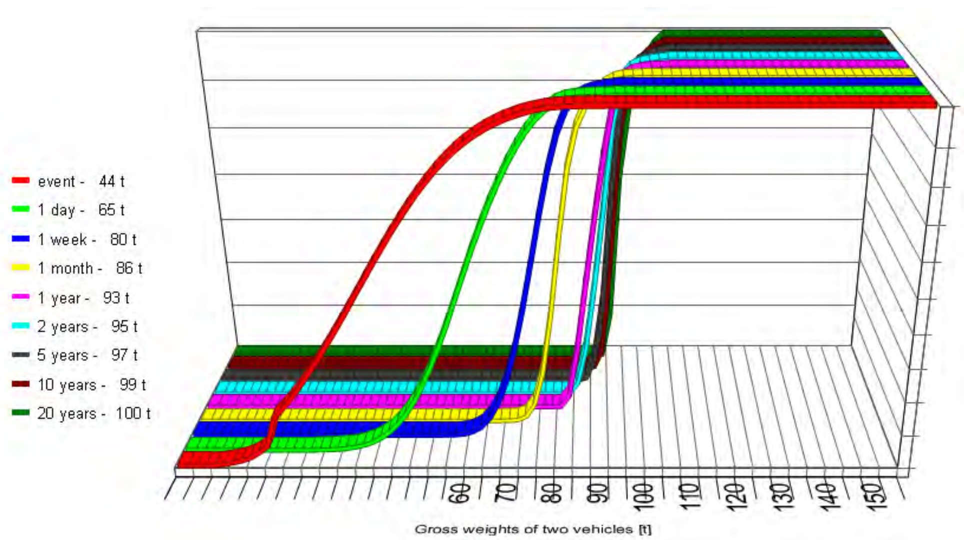


FIGURE 5 Gross weight histogram—all trucks.

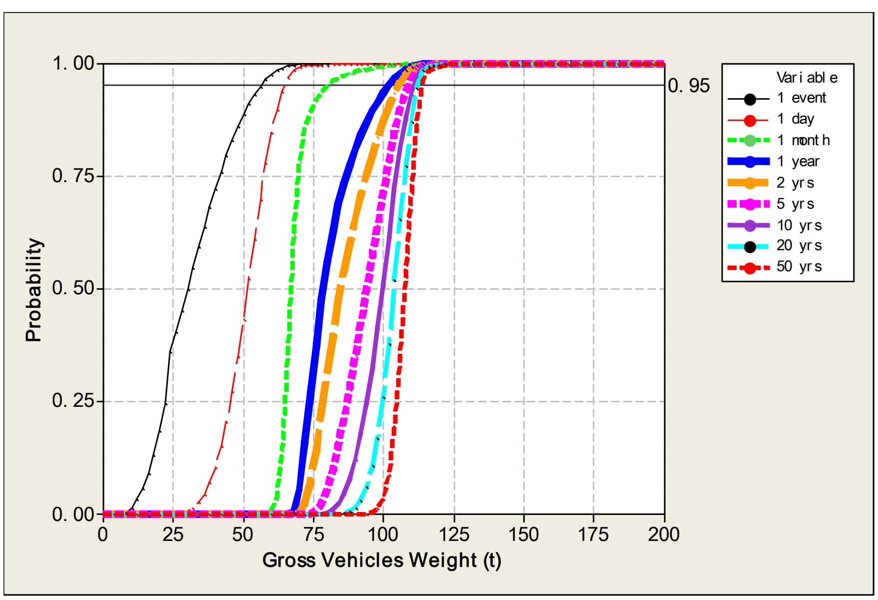
While a very basic statistical evaluation is applied to determine $W_{0.95}$ (Figure 6*b*), *m*, *I*, and *g*, the convolution method was used to simulate the headway factor *H*.

Soft Load Testing

The objective of any load testing is to measure response of the bridge (structure) in order to optimize bridge assessment (use measured rather than theoretical structural parameters) and thus to find reserves in structural safety. Generally, two levels of load testing are known: diagnostic load testing and proof load testing.



(a)



(b)

FIGURE 6 Expected maximum GVW of a 2-vehicle event and $W_{0.95}$ determination.

The diagnostic load tests are used widely in many countries, while the proof load tests are used rarely, mainly because of the danger of damaging the structure being tested. The major disadvantage of both types of load testing is their cost, mainly because of the high number of rented loaded vehicles necessary and because traffic needs to be closed while the test is performed which makes them inappropriate for assessment of existing bridges.

A novel concept of soft load testing was introduced through the implementation of SiWIM systems. The method can be efficiently applied on a large number of bridges without interrupting the traffic. It corresponds to the lowest level of load application. The test is again aimed to optimize the structural model used for safety assessment of a particular bridge.

The main focus of the work done was on improving the reliability of methods for determining traffic loading on bridges. This is being done by determining the actual traffic loading from SiWIM data. Bridges are unusual in that, especially for longer spans, a high proportion of the total loading is due to the dead and superimposed dead load. The further research will discuss material properties and their on-site measurements and definition of dead load for probability-based assessment. For bridge monitoring, bridge WIM systems was investigated to provide synchronized data on the traffic load effects and the induced stresses and strains, and therefore become a part of “Intelligent Bridge Monitoring Systems” to optimize the allocation of funds for bridge repair or replacement and to reduce the corresponding traffic disruption.

Using the measured influence line combined with the true load distribution factors (how load distributes over the main structural elements) lead to safety factor, so we can make some conclusions about the existing bridges to do correct dictions for the maintenance and retrofitting of the existing bridge. Once the structural model is optimized with the true influence lines and load distribution factors, it can be used to calculate instantly the safety level under any traffic loading, including the special transports.

And a bridge must always fulfill the following criteria:

$$R \geq G \times \gamma_G + Q \times \gamma_Q + A \times \gamma_A \tag{3}$$

or

Capacity ≥ Dead loads + traffic loads + other loads

Safety is then expressed as:

$$Safety = \frac{Capacity - Dead\,loading}{Traffic\,loading} \geq 1 \tag{4}$$

In a bridge management system, structural safety is estimated by the rating factor *RF*. In other words, if the value of rating factor *RF* is greater than 0.95, it will be okay.

$$RF = \frac{\Phi \times R_d - \gamma_G \times G_n}{\gamma_Q \times Q_n} \tag{5}$$

Where R_d = the bearing capacity of cross section; Φ = the capacity reduction factor; G_n and Q_n = the dead and live load effects and γ_G and γ_Q = the respective safety factors. Bridge WIM gives the essential information to calculate the realistic traffic loading Q, and allows for reducing the live load safety factor γ_Q, which both increase the safety of the structure.

CONCLUSION

The conventional several types of WIM systems on highways are compared. The static method is time-consuming and cause undesirable delay although it is the most accurate and common one. And it can just obtain some sample data of the traffic flow. While WIM systems can overcome the shortcomings and provide necessary results.

A newly developed Bridge WIM system provide a new concept of heavy vehicles weighing. Bridge WIM is a WIM system using an instrumented bridge as a large sensor and used to determine the gross weights and axle loads of vehicles when they pass the bridge. As bridges are long compared to the pavement WIM sensors, the dynamic effects have less effect on accuracy of bridge WIM systems than they have on the pavement ones. And the FAD systems are promising and provide a strong reason for preferring B-WIM against pavement sensors. Specifications that involve only traffic volume do not give satisfactory estimates on real traffic loading. And SiWIM system is a powerful instrument for collection of traffic loading data that can be used for numerous applications. Bridge safety can be calculated more efficiently with SiWIM system

A novel concept of load testing was introduced in B-WIM systems which can provide not only traffic data, but also can measure important structural parameters for the instrumented bridges, such as real influence lines and load distribution factors. And the result can be used for calculation in the live load model, the rating factor and safety evaluation in optimized bridge assessment and maintenance.

The results of the research will establish the foundation necessary to further advance the implementation of Bridge WIM technology in highway safety, maintenance and design. The availability of real-time information collected at Bridge WIM can be beneficial not only for road users but also for the overall management of efficiency, safety and environmental protection on the road network. The research what we are doing and the further research on B-WIM system will provide good benefits and suggestion and enforcement for the bridge management and maitenance system.

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