1184

TRANSPORTATION RESEARCH RECORD

Structures Maintenance

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
WASHINGTON, D.C. 1988

Transportation Research Record 1184
Price: $9.00
Editor: Alison Tobias

mode
1 highway transportation

subject areas
25 structures design and performance
40 maintenance

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Printed in the United States of America

Library of Congress Cataloging-in-Publication Data
National Research Council. Transportation Research Board.

Structures maintenance, 1988

(Transportation research record, ISSN 0361-1981 ; 1184)
Reports for the Transportation Research Board 67th Annual Meeting, held in Washington, D.C., 1988.
1. Bridges—Maintenance and repair. 2. Bridges—Inspection. I. National Research Council (U.S.). Transportation Research Board. Meeting (67th : 1988 : Washington, D.C.) II. Series.
TE7.H5 no. 1184 [TG315] 380.5 s 89-12464
ISBN 0-309-04724-2 [624'.2]

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Contents

Foreword

Bridge management, in its broadest sense, includes not only such planning considerations as expected life, rate of deterioration, effect of maintenance on extending the life of structures, and user costs, but also inspections, repair techniques, materials, monitoring systems, and numerous other activities. Both planning and work-related topics are included in this Record.

Designing a structure is relatively simple compared with analyzing a structure after it is built; a designer can assume materials characteristics, construction control, and so on, but the closeness of fit between design assumptions about expected loading and the as-built structure may be quite different. Investigators are addressing this problem from both the theoretical and the pragmatic level and both approaches are included in this Record. To examine the problem of risk analysis to address uncertainties in material properties and traffic loading, Tee, Bowman, and Sinha describe the application of fuzzy logic for assessing the condition of concrete slab bridges and offer an example problem to illustrate use of the methodology.

The Pennsylvania Department of Transportation has been a leader in the development of bridge management systems and two papers herein address the development of the process used to ensure that bridge inspections performed by different sections in the department provide accurate information with consistent interpretation statewide. In another paper, Kurt demonstrated that microcomputers can provide a good computing base for managing local bridge systems, based on his work in Kansas. Weyers, Cady, and Hunter describe how they used the expertise of a group of knowledgeable individuals to develop an economic decision tree presenting the least cost solution to 20 bridge maintenance and rehabilitation areas.

The other two papers in the Record provide a solution to the problem of checking pins in bridge pin and hanger types of bridges and deep monomer impregnation and in situ polymerization of a bridge deck.

Failure of a pin in a pin and hanger type of bridge in Connecticut some years ago caused all agencies owning bridges with that type of design to make special inspections to ensure the safety of their own bridges; however, checking the pins in situ or removing the pins for checking has proved difficult in the past. Authors Carroll, Martin, and McDonald offer solutions to both problems.

Weyers and Cady demonstrate that the deep monomer impregnation and in situ polymerization of a bridge deck using the grooving technique is commercially feasible and can be successfully completed to a set of specifications by contractors with no previous experience with the process.

A Bridge Field Inspection Procedure To Check the Integrity of Pins in a Pin and Hanger Strap Connection

Frank L. Carroll, Fred A. Martin, and Steven A. McDonald

Following the near collapse of an Interstate bridge in the City of St. Louis, Missouri, a procedure using ultrasound has been developed to check the integrity of the pin in a pin and hanger strap connection. Ultrasound is transmitted into the ends of pins and reflections from defects are displayed on the scope of the ultrasound instrument indicating the presence and location of cracks or wear. Using this procedure, 90 bridges containing 675 pin and hanger strap connections have been checked. Four percent of the pins have been found to be defective. The pins from the Interstate bridge in St. Louis were not removed intact. However, the predictions obtained with the ultrasound procedure correlated closely with the information obtained from the pins and pin pieces after they had been removed by burning. A pin pusher has since been developed, built, and used to remove pins intact. Eight pins were removed intact from one bridge and good correlation between predictions and actual conditions was found. Of the eight pins, it was predicted that one would be completely severed, one would be cracked, and six would be sound. After removal of the pins, the predictions were verified as accurate by visual examination. A limitation of the procedure is the inability to precisely predict the defect size. However, the threshold depth where a crack can be identified has been determined to be approximately ⅛ in.

The vulnerability of bridges with pin and hanger strap connections came into sharp focus in March 1987 following the near collapse of a bridge on Interstate 55 in the City of St. Louis, Missouri. At the time of the incident, the bridge was under contract for widening and rehabilitation. A contractor's workman discovered that one span carrying the northbound lane of I-55 had dropped about 1¾ in. at the finger plate expansion device. Further investigation revealed that 4 of the 12 pins in the hanger strap connections at this joint had failed. Total collapse of the span did not occur because the expansion joint was completely closed and some of the finger plate support steel came to rest on an abutting stringer.

The design parameters for this pin and hanger strap connection (Figure 1) as an expansion device assumed that the pin would be free to rotate in the web as the girders expanded and contracted in response to temperature changes. The pin was designed to resist the shear and bending forces resulting from dead load, live load, and impact forces. Torsional forces were not considered. This design assumption has been proven to be invalid because it has been found that the joint was frozen as a result of corrosion between the pin, the straps, and the web. Consequently, torsion forces exceeding the pin strength were developed.

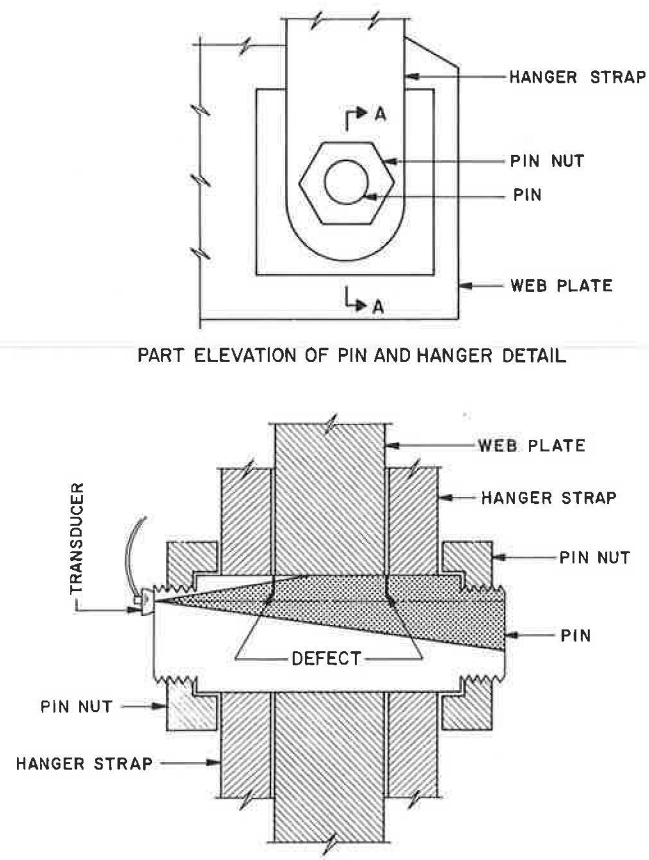

FIGURE 1 Part elevation of pin-and-hanger detail with enlarged section *A-A*.

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This near collapse of a bridge carrying 38,000 vehicles/day prompted the development of a procedure for nondestructive testing of pins because a visual inspection would give no indication of possible or imminent failure.

The options available for inspection of pins were quite limited. Visual inspection was totally unreliable and disassembly was not practical. Of the other nondestructive tests such as magnetic particle testing, dye-penetrant testing, radiography, and ultrasound, only ultrasound showed some promise of being feasible and practical. The only parts of the pins that were accessible for testing purposes were the ends.

The initial attempt to perfect an ultrasound testing procedure involved fabricating a pin of the same length as the pins that failed. The test pin differed from the pins in the bridge in that it did not have threaded ends. This was not considered to be a problem. In order to determine whether defects could be detected and located from the application of ultrasound at the end of pins, cuts of ¼ and ½ in. were made in the sample pin. After calibration of the ultrasound instrument, it was possible to identify defects and determine their lateral distance from the end of a pin. This was further verified by scanning from both ends of the pin. The initial testing was done with a 1-in.-diam transducer and the gain (or amplification of the reflected sound) required to identify the sample defects was noted.

With this very limited experience, the procedure was taken to the field and the remaining pins in the I-55 bridge were checked. The procedure failed to produce any indication of defects in the remainder of the pins in the bridge. This did not seem to be reasonable so the procedure was re-evaluated. It was concluded that the beam spread from the 1-in.-diam transducer was not reaching the body of the pins where defects would occur. To remedy this problem, the 1-in.-diam transducer (Figure 2) was changed to a ½-in.-diam transducer (Figure 3) to take advantage of the increased beam spread. After scanning the pins from both ends, indications of defects were found in most of the remaining pins on the I-55 structure. The contractor immediately placed falsework under the bridge to prevent collapse.

Verification of the predictions made with the ultrasound procedure was considered necessary because there were almost 100 bridges in the state with pin and hanger strap connections. To verify the predictions, the pins had to be removed intact. However, the contractor for the I-55 bridge

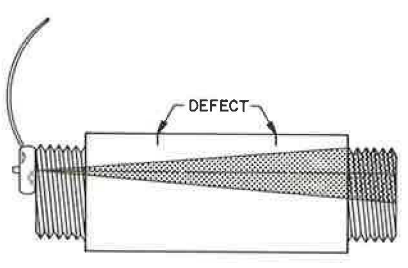

FIGURE 2 Beam spread with 1-in.-diam 3.5 MHz transducer.

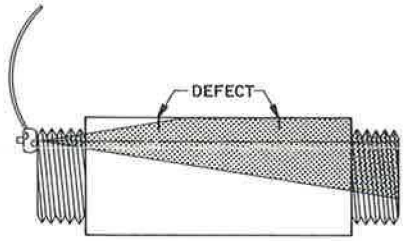

FIGURE 3 Beam spread with ½-in.-diam 3.5 MHz transducer.

was unable to push, pull, hammer, or otherwise force the pins from the girder. He resorted to the use of an oxygen-burning wand that burned a hole about ½-in. in diam through each pin. This relieved the pressure so that the pins could be forced out with a pneumatic hammer. The predictions obtained with the ultrasound procedure on this bridge correlated closely with a visual inspection obtained from the pins and pin pieces after they had been removed. The removal method of burning made it impossible to verify the predictions with the use of ultrasound.

The rudimentary procedure used on the I-55 bridge was adopted to begin checking all of the bridges in Missouri that had pin and hanger strap connections. It was very important to be able to verify that the testing procedure was in fact identifying as defective those pins that were truly defective and not identifying as defective those pins that were good. Consequently with the development of the procedure to test pins, a device later called the "pin pusher" (see Figure 4) was designed to permit recovery of the pins intact. To gain some idea of the force required to extract a frozen pin from a web, a hanger strap with part of a broken pin was put in a press in the laboratory and the pin removed. A force of 40 tons was required to press a

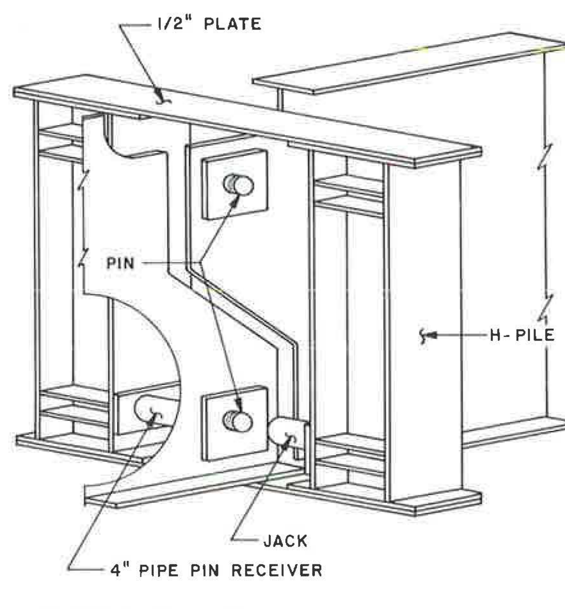

FIGURE 4 Pin pusher.

2-in. pin from a 2-in. thick hanger strap without the use of heat.

Based on this information, the pin pusher was designed and built to apply a force of 50 tons. The pin pusher has been used to remove 24 pins intact from two bridges that were found to have defective pins. Of the 8 pins from one bridge, it was predicted that 1 would be completely severed, 1 would be cracked, and 6 would be sound. After removal of the pins, it was found that 1 was severed, 2 were cracked and 5 were sound. After finding that one crack was missed in the original prediction, the test procedure was modified by increasing the amplification of the reflected sound that was transmitted through the pin. The increased amplification permitted the identification of smaller defects.

Of the 16 pins in the other bridge, it was predicted that 8 would be cracked and 8 would be sound. After removal of the 16 pins, it was found that 8 were cracked, 6 were sound, and 2 had corrosion grooves about ⅛ in. deep.

A typical inspection of a bridge with pin and hanger strap connections requires a minimum crew of four, a snooper truck capable of operating from the deck, and a supply truck containing signs for traffic control and miscellaneous tools and equipment. Initially, a 110-volt power supply generator was required to operate the ultrasonic testing instrument, however, a portable nicad battery pack is now being used. A power source is required to operate a portable grinder. The ultrasonic instrument weighs about 12 lb. and is carried in the snooper bucket by the inspector.

After reaching a pin, the inspector's first operation is to remove accumulated paint, rust, and scale from the end of the pin. This is done with a small hand-held grinder. It has not been necessary to have an extremely smooth finish on the end of the pin. A couplant such as glycerin is then applied to the end of the pin to facilitate the transfer of the sound waves from the transducer to the pin and back. The ultrasonic testing machine is calibrated to the length of the pin. The inspection consists of positioning the transducer on the end of the pin and slowly moving it over the entire area of the pin, while observing the scope of the ultrasonic instrument to determine if defects are present.

The reading on the scope of the machine for a good pin (Figure 5) shows an initial spike, a spike representing the far shoulder at the threaded end of the pin, a spike representing the end of the pin, and a flat line between the initial spike and the shoulder spike.

The reading for a defective pin (Figure 6) shows an initial spike, a spike on the scope at the location of the defect, a spike representing the far shoulder at the threaded end of the pin, and a spike representing the end of the pin. The reading for a pin with total failure shows an initial spike, a tall spike at the defect location, and that the spikes representing the far shoulder and the end of pin are gone. The location of the spike on the scope that indicates a defect will correspond to the distance of the defect from the end of the pin. The height of the spike is related to the size of the defect. A very shallow defect will produce a very short spike whereas a deeper defect will produce a higher spike.

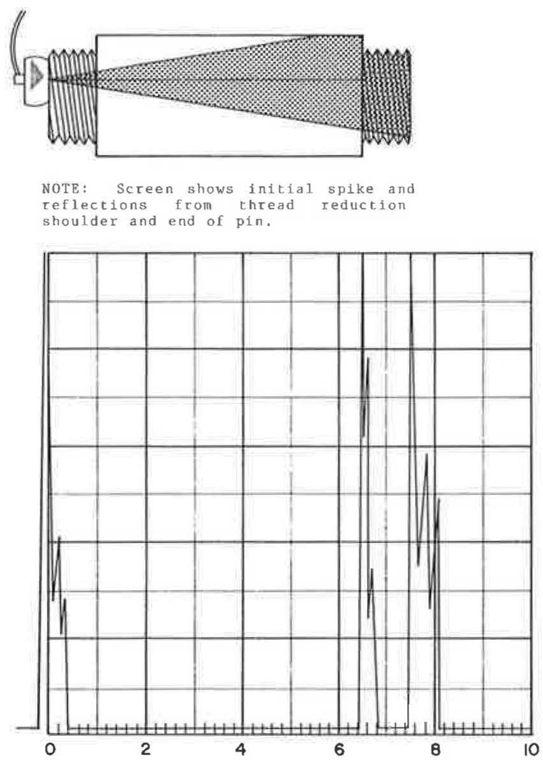

FIGURE 5 Screen of good pin.

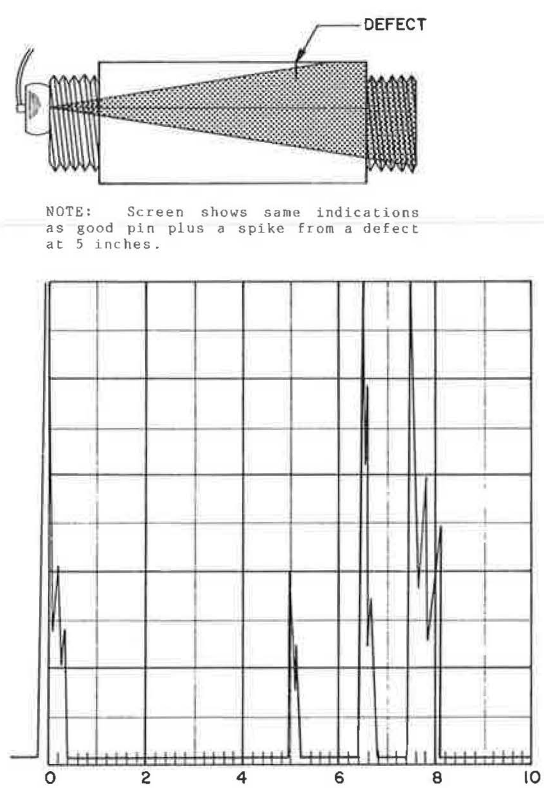

FIGURE 6 Screen of pin with defect.

Pins are checked from each end. This allows full coverage of the body of the pin and allows the location of the defect to be measured from each end of the pin. This method helps to verify the exact location of a defect.

Cracks normal to the length of the pin can be identified but seams or other defects parallel to the length of the pin cannot normally be detected. Due to their shape, it has been difficult to detect shallow corrosion grooves with the straight beam ½-in. diam transducer. To solve this problem, a ½-in. diam 18° angle beam transducer has been used to direct the sound beam more nearly normal to the defect. The application of the sound beam more normal to the defect results in more reflected sound being displayed on the scope. Consequently, the presence of the shallow corrosion grooves can be identified.

For each pin inspection, a written record is made indicating the date and condition of the pin. This information will be used as historical data for future inspections. If defects are found, an assessment is made about how serious they are and a decision is then made on whether to restrict traffic on the bridge, provide shoring, or allow traffic to continue until the pin can be replaced or the connection modified.

On completion of the inspection, the pin ends are repainted with a primer to protect them from rusting and to serve as a quick indicator that the pin has been inspected. A color-coding system is used to indicate the year of the inspection.

The ultrasonic equipment in this inspection procedure had an initial cost of $6,228 in 1987. If the inspection crew is based in the area of the bridges to be inspected, they can inspect approximately 20 pins during a 10-hr work day. This includes setting up and removing traffic control devices. Most of the bridges inspected up to the present (January 1988) are approximately 20-yr old. Defects have been found ranging from those ⅛ in. deep to total failure of the pins.

Based on experience with this procedure, the authors have concluded that the procedure has been valuable in determining defective pins in an early stage of distress. It has been successful in determining the location on the perimeter of the pin where the defect occurs. It is limited in that the readings do not specifically indicate the exact depth of the defect, however, the relative size of defects is proportional to the amount of signal reflected. It is also difficult to determine from the inspection whether the defect is caused by wear or corrosion or is in fact a crack. In order to maintain confidence concerning the integrity of pin connections, the authors have concluded that bridges with no indications should be inspected at 2-yr intervals; bridges with only slight indications should be inspected annually; and bridges with several slight or moderate indications should be scheduled for pin replacement or retrofit.

Publication of this paper sponsored by Committee on Structures Maintenance.

Inspection of Fracture Critical Bridge Members

RON L. PURVIS

Beginning in 1987, the Federal Highway Administration sponsored a 2-day training course entitled "Inspection of Fracture Critical Bridge Members." The course was developed and taught by Byrd, Tallamy, MacDonald and Lewis Consulting Engineers. It was attended by Federal Highway Administration state and local bridge inspectors who have responsibility for on-site inspection of highway bridges. All state and most local agencies responsible for existing bridges have inspection programs in place. Certain modifications may be necessary, in addition to inspector training, if fracture critical members are to be inspected in accordance with the guidelines provided in the course. The guidelines require that each fracture critical member receives a hands-on, close-up 360° inspection. Additional nondestructive testing may be appropriate if a potential fracture is identified. Additional resources may be required to provide this level of inspection. The inspector is often not in a position to budget and schedule these resources. Fracture critical members should be first identified by a qualified bridge engineer. It is recommended that each agency include in its program a procedure for documenting and flagging each fracture critical member to ensure that it receives appropriate priority when the bridge is inspected. An inspection plan is formulated for each bridge with fracture critical members that include equipment, inspection technique, and staffing. The potential for fatigue cracks is evaluated and locations identified. The plan is then discussed with the inspector to ensure that the priorities are understood.

The following provides an overview of the implementation of an inspection program for fracture critical bridge members.

BACKGROUND

National Bridge Inspection Program

The National Bridge Inspection Standards (NBIS) administered by the U.S. Department of Transportation, Federal Highway Administration (FHWA) are almost 20 yr old. It is the law that all highway bridges open to the public must be inspected at 2-yr intervals for safety. Bridges with restricted capacity or certain structural problems are normally inspected more frequently. Some agencies require that the inspector be a registered professional engineer. Minimum requirements are that the team leader have 5 yr of related experience and successfully complete a training course.

Quality of Inspection

In most situations the only method available to detect flaws in a bridge member is visual inspection. It is important to identify the flaws early in the typical crack-development scenario. If the defect is identified as soon as it can be seen by the inspector, the service life of the member often has been reduced by more than 80 percent. Fractures have occurred on bridges that have been open to traffic for relatively short periods of time. On such a structure, there may be only one opportunity for the inspector to identify the flaw and prevent the fracture. If the fracture is likely to cause a sudden failure of all or part of the bridge, it is extremely important that the defect be identified in time to prevent a possible catastrophe.

The flaw is often very small. The inspector has to be close, to know where to look, and to recognize the crack when it first becomes visible. A supplement to the FHWA Bridge Inspector's Training Manual has been developed along with a 2-day advance training course to provide instructions in this area. The manual and course were developed by the Byrd, Tallamy, MacDonald and Lewis Division of Wilbur Smith Associates, consulting engineers, under the sponsorship of the FHWA.

IDENTIFYING FRACTURE CRITICAL BRIDGES

Definition of a Fracture Critical Member

A fracture critical bridge must have one or more fracture critical members (FCMs). An FCM is a tension member or component whose failure will produce a sudden collapse of the structure. The training course is developed on a level to include the non-engineer inspector and the theory is presented accordingly. The participant is taught how to identify a tension member and to determine whether its failure will result in an immediate bridge collapse. The portion of a member in tension is being pulled apart. This

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causes cracks to grow and leads to fracture. A member in axial tension is stressed uniformly throughout the cross section for the total length between connections.

Hangers, suspension cables, and some truss members normally are stressed in axial tension. The stress varies throughout members that are in bending. The inspector must be aware of tension zones in such members. For example, on a simple beam, maximum tension is in the bottom flange at midspan. An equally important location on a continuous span is the top flange over the support. High stress may also be concentrated at locations along a member where the cross section changes or where there is a discontinuity.

Redundancy

For the inspector to determine whether a sudden collapse will occur when a member fractures, it is necessary to understand the term redundancy as it applies to primary bridge members or connections. Redundancy is the ability of other members to help carry the load when a member becomes weak or fails. Three different types of redundancy are possible depending on the design. These are load path, structural, and internal.

Load path redundancy relates to the minimum number of members required to support the deck under traffic. A bridge with fewer than three girders or trusses is considered nonredundant and therefore fracture critical. Bridges with three or more girders are considered redundant, because if one girder becomes weak the others will help carry the load. There are degrees of redundancy that should be considered depending on the girder spacing, stiffness of the deck, and framing system. A capacity analysis by a structural engineer may be necessary to predict the failure scenario on some bridges.

Structural redundancy relates to the support provided by the cantilever created after a continuous member is weakened. This occurs only on interior spans with members continuous across supports on both ends. There must be a minimum of three continuous spans to have a structurally redundant span that is located in the center.

Internal redundancy relates to crack propagation through the cross section of a member. Many members are composed of several parts. A crack must reinitiate in each part on internally redundant members. Built-up members with plates attached by rivets or bolts have internal redundancy. Others are reinforced concrete, cables, and members composed of several separate sections. Rolled steel members have no internal redundancy, nor do built-up welded members. As fatigue and fracture of steel members are studied, it is found that cracks not only propagate freely through welds, they often start because of the weld.

Many agencies define FCMs in terms of load path redundancy. Structural and internal redundancy, however, are also considered in evaluating problems. Examples of fracture critical spans are spans supported by two or fewer single web girders, box girders, trusses, suspension cables, cross girders, caps, and tie members on tied arch spans. Spans supported by four or fewer pin-and-hanger assemblies also qualify.

FATIGUE AND FRACTURE

Predicting Fracture

It is important that the inspector identify a crack or flaw before the member fractures. Physical characteristics make certain members more susceptible to fracture. The magnitude of the total stress or the number of times a member is stressed, or both, contribute to the fracture. Also, design details have an important influence on crack initiation. The remaining factor is flaws in the member. The inspector's efficiency at identifying FCM problems is significantly enhanced by an understanding of fatigue and fracture.

Fractures require a driving force. Normally this force is produced by the load on the structure. The force on a particular cross section of the member is called stress. The stress may take the form of compression, tension, or shear. Compression squeezes or pushes down on a member. Cracks normally do not cause problems in compression members because the material is not being pulled apart. If a crack exists in a compression member (which is rare), there is no force to make it grow. Tension stretches or pulls a member apart. Cracks are of concern in tension members because the stress causes the member to fracture. The cracks grow perpendicular to the direction of the tension stress. Shear is similar to tension, but rather than pulling the member apart it tends to tear or slice the material. Some cracks grow as a result of shear. The direction of a shear crack is at a 45 degree angle to the force. Bridge members may be subjected to only one or a combination of these stresses.

The fracture may be the result of an overload in which the member is stressed beyond its capacity or yield point. This rarely occurs on bridges designed to carry standard legal loads. More often cracks are caused by repeated loads that do not exceed the legal limit. Fatigue is the term used to describe the process of material damage caused by repeated loads. One load is a cycle. A cycle must subject the member to a certain magnitude of change in stress or stress range before it is significant in causing fatigue cracks. Bridges that carry a large volume of heavy loads are more likely to experience fatigue problems.

Fatigue crack initiation is not only related to the number and size of the stress cycle, it is also related to design details. Stress concentrates at locations where the rigidity of the member changes. Fatigue occurs at points of stress concentration. Details that cause changes in the rigidity of the member have been categorized to help the designer avoid cracking problems. These categories are used by the inspector to predict crack initiation in existing bridges.

All bridge members have flaws. Their size and location influence crack initiation and propagation. Flaws provide a focus of crack initiation. It may be in the base metal of the member or in the weld metal. Many flaws are not visible. Nondestructive testing (NDT) is used to identify these flaws during the shop inspection. On older bridges NDT was not always required. Field welds and repair welds often do not receive NDT. Flaws in the base metal may be caused by fabrication, transportation, erection, or in-service damage. Such flaws include bolt and rivet holes, notches, grinding marks, copes, and flame cuts. Service flaws include collision damage, damage from improper straightening, or section loss caused by corrosion.

Material Consideration

There are two types of fracture: ductile and brittle. When ductile fracture occurs, the material stretches before it separates into two parts. The fracture is slow and there is often time to prevent a disaster. A brittle fracture occurs very rapidly. The brittle fracture is of particular concern to the bridge inspector. Certain members are more likely to fail by brittle fracture. Members composed of thick plates are more likely to have a brittle fracture than members made of thinner plates. They tend to break rather than bend. Also, the colder the temperature, the more likely the occurrence of a brittle fracture. The tougher the steel, the less likely it is to fracture. Bridges designed today contain steel meeting these specifications, but they can be tested to determine the steel toughness. Information on the steel toughness is of benefit to the inspector in predicting potential problems.

Design Consideration

Fatigue cracks start at locations in steel members where the rigidity of the member changes. These locations are created by designers attempting to save material. Cover plates are added to beams to avoid using a larger size. Stiffeners permit the use of very thin webs on members. As the member bends under a load, stress is concentrated at areas where the rigidity in the member changes. Cracks begin at these locations.

Fatigue cracks may be a result of either in-plane or out-of-plane bending. In-plane bending is a result of load distributed from the floor directly to the member. Out-of-plane bending is usually the result of the load's being transferred to the member through secondary members. This force tends to twist the member, and may be transmitted into thin parts of the members, such as the web, that were not designed to resist the stress. A crack may begin in the web in the space between the connection plate or stiffener and the flange. Often the crack is not perpendicular to the primary stress, therefore it does not represent as immediate a problem as the crack caused by in-plane bending. Inspectors, however, are cautioned to bring any cracks to the attention of a qualified structural engineer for evaluation.

Loads on the Structure

Another factor that influences whether the fracture is brittle or ductile is the loading rate. Static loading is least likely to produce brittle fracture, whereas dynamic loading often results in a brittle or sudden fracture. Bridges experience a combination of static and dynamic loading. Inspectors should be aware of situations in which the dynamic loading is exceptionally high. Examples are bridges that receive heavy pounding loads, which might be caused by low approaches or poor vertical alignment.

Fatigue cracking is caused by repeated loads that produce stress cycles. Larger loads create stress cycles that cause fatigue damage. A certain design detail may be capable of carrying a limited number of stress cycles that are created by the larger loads using the structure. When the number of cycles exceeds the limit, cracking occurs at predictable locations. The inspector should know about the loading history of a bridge before conducting an evaluation.

Crack Initiation and Propagation

Most cracks in steel bridges occur at predictable locations. Cracks occur at areas of stress concentration. They normally originate at a flaw. The flaw is often associated with a weld. When a fatigue crack caused by in-plane bending grows to a size visible to the inspector, at least 80 percent of the service life of the member has already expired. The small crack has been growing beneath the surface in a semi-elliptical pattern. After the crack reaches the surface it must penetrate through the paint before it is visible to the inspector. Occasionally the visibility is accentuated by rust stains that are associated with the crack.

NDT is available to help verify the existence of a crack. After the crack has been found, these tests will locate its boundaries and measure the size. In general inspection, however, NDT is not very effective in helping to find cracks that have not been identified.

IMPLEMENTING A FRACTURE CRITICAL BRIDGE INSPECTION PROGRAM

Modifying Existing Program

The purpose of fracture critical inspection training is to help prevent sudden bridge collapse. For the most effective use of resources, the level of inspection should be appropriate to the criticality of the member. Agencies normally do not have the resources to enable inspectors to perform hands-on, close-up, 360° inspections of all the components of all their bridges once every 2 yr. FCMs, however, should receive this level of inspection. To implement necessary fracture critical inspection procedures into an existing program, the following are needed:

1. Inspectors trained for required level of inspection,
2. Identification of fracture critical members,
3. Budgeting of adequate resources and planning for implementation of activities,
4. Inclusion of quality control procedures to monitor the program.

FCM Inspection Training

Many bridge inspectors are not graduate engineers. For them to be effective, it is important that they understand where to look for problems. They should also know how to look, what to look for, and what to do if they find a potential problem. An ongoing FCM inspection training program is necessary for the required results to be achieved. Bridge inspection training must also extend beyond the classroom. Bridge inspectors normally work independently. A new inspector should demonstrate an understanding of the job before working alone. Certain personal traits should also be demonstrated. The job requires an attitude of perseverance and diligence. An inspector may often look at several hundred details having no problem before a flaw is found. The temptation to take short cuts is great. Inspectors should be continually monitored to ensure that a proper attitude is maintained.

Identification of FCMs

The inventory and condition data are stored in a file for each bridge. The file contains previous inspection reports, as-built drawings, repair and maintenance work recommended and performed, and current load rating information. It is recommended that each file be flagged to indicate if this is a fracture critical bridge. Otherwise the inspector may be at the site and ready to work before he realizes that a special level of inspection is necessary. This level of inspection often will require special equipment that must be planned in advance. A qualified structural engineer should identify FCMs. At times a structural analysis may be necessary. The documentation should include critical locations and critical details. Special concerns such as previous damage and repairs should also be noted if these areas warrant special attention. Some FCM bridges are more critical than others because of the number of cycles, type of details, or material. The file should contain all this relevant information so that the level of inspection is appropriate for the bridge.

Resource Requirements and Implementation

Fracture critical bridge inspections are expensive if done properly. Because the inspector concentrates on critical areas where cracking is most likely to occur, this approach is the most efficient use of resources. Special equipment, such as hydraulic man-lifts or platform devices, may be required to access the critical areas. To supplement this equipment, special manpower support and traffic control may also be required. The bridges should be studied carefully to ensure that the necessary resources are provided. It is also important to ensure that an adequate amount of time is available for the inspection. The quality of inspection must have priority over the quantity, particularly for a fracture critical bridge. It may be helpful to combine inspection teams for optimum efficiency in using the equipment. Special equipment may also be necessary such as NDT devices, special lights, or ventilation devices, particularly for closed boxed girder bridges.

Quality Control

The final consideration in implementing a fracture critical bridge inspection program is quality control. Quality control is often performed on a hit-or-miss basis. There is rarely a well-defined program. To be useful, it must consist of clearly defined procedures that result in a quantitative measurement of the quality. These procedures should be performed in a standardized way so that they can be compared from team to team and from year to year. Quality control should monitor overall and individual levels of conformance.

THE FCM BRIDGE INSPECTION

Preparation

Fracture critical bridge inspection begins before the team arrives at the bridge. The team should study the file carefully while it is still in the office. It is important for each inspector to understand which members are fracture critical and where the fracture critical zones are located. The loading history of the structure is helpful. Fatigue-prone details should also be identified. Records of damage to the structure because of collision or corrosion and repairs are also important. In addition to access equipment, the team may need special tools such as magnifying glasses, spotlights, or dye-penetrate testing kits.

Assignment of Duties

When more than one person is making the inspection, it is important to coordinate the activities. Considerations should be given to the skills of the individuals making up the team. One person must be in charge. It is the team leader's responsibility to ensure that duplications are minimized and that there are no omissions in inspection of FCMs. Data collection should also be coordinated so that it can be efficiently incorporated into a report.

Hands-on Inspection

A hands-on inspection should be performed on all of the FCMs. All details identified as prone to cracking must be checked closely. The inspector's eye should be within 24 in. of the surface. The member is viewed from all sides and all angles. The inspector should use additional light and magnification to evaluate the member if necessary. The inspection should begin with a general evaluation of the structure and fracture critical member. It is important to look for things such as misalignment of spans, either horizontally or vertically. Unusual movement or noise might also indicate serious problems. During the overall evaluation, inspectors should also look for distortions or damage created by traffic, flooding, and so on. After the overall evaluation, each member and each detail should be checked closely. The inspector should focus on tension zones of fracture critical members and fracture critical connections. Details that create stress concentration should receive special attention. Examples of details that should be checked closely are

- Intermittent welds between the web and tension flange;
- Areas of sudden change or cross section near the ends or cover plates;
- Locations of stress risers such as nicks, scars, flaws, and holes that have plug welds, irregular weld profiles, and areas where the base metal has been undercut;
- Locations where stiff bracing members of horizontal connection plates are attached to thin webs and girder flanges;
- The web adjacent to a floor beam connection plate;
- Gusset plates, improperly coped members re-entering corners, and the gap between web stiffeners and flanges;
- Longitudinal and vertical stiffener intersections;
- Longitudinal stiffeners that have been connected together with butt welds;
- Location of welds at gusset-transverse-web intersections;
- Flanges that pass through a web, such as girder flange passing through a box girder pier cap;
- Box-beam-to-column intersection; and
- Eyebars.

Discontinuities resulting from in-service problems should also be scrutinized. Examples of these are corrosion, flaws, and welded repairs. Areas where corrosion is likely to give problems are as follows:

- Under deck joints;
- Areas around scuppers and drain pipes;
- Under open steel grates;
- On flat surfaces where debris accumulates;
- On exposed surfaces of fascia members;
- On steel in contact with concrete;
- At overlapping steel plates; and
- At corners of steel angles and channels.

Other special details that should be given attention during the FCM inspection are

- Shear connectors in the negative moment region;
- Pin and hanger assemblies;
- Tack welds on bolted or riveted connections;
- Unfilled holes or holes filled with weld metal;
- Field welds in tension zones; and
- Suspicious attachments making tension zones, such as utility attachments.

FCM Inspection Report

By definition, fracture critical bridges are those prone to failure that may result in a catastrophe. It is important that the inspection of a fracture critical bridge be documented thoroughly and accurately. This should include a narrative description of all FCMs, whether there are serious problems or not. Photographs and sketches should be included. In cases in which there are many details and findings, tables and charts are also necessary. The data should be organized and cross referenced for efficiency in interpreting the report. The report should provide information on why problems occurred. Repairs are not likely to be effective unless they begin with the cause of the problem. The report should also include conclusions and a summary of the findings. Along with communicating the existing condition, the inspection report should provide an ongoing record of the condition of the bridge and verification of the thoroughness of the inspection activities. Bridges are unquestionably safer because they are inspected. Most deficiencies can then be identified and appropriate remedial actions taken. Occasionally there will be serious flaws that cannot be seen by the inspector. If a fracture occurs, the report can be used to verify that a proper inspection was made.

What To Do if a Flaw or Crack is Found

It would be difficult to defend a situation in which a bridge failed after the defect had been identified. It is therefore important that the inspector communicates the findings in a timely manner. Ordinarily the inspector would prepare the report and forward it to a supervisor for review. If the supervisor is busy, this may take a week or more. Flaws on fracture critical members should not wait that long for evaluation by an engineer. Some flaws such as a visible crack in a tension flange of a two-girder bridge should be reported immediately. The inspector should go to a phone and call a supervisor. The agency should have an approved procedure for immediate closure if this is warranted. Other problems such as a flaw in a web may be reported when the inspector returns to the office. It is better for the inspector to err on the side of safety. If there is a question about the significance of a finding, an engineer should be contacted as soon as possible. When problems are identified, it is a good idea to go back and look at similar details throughout the bridge. Often inspectors have found cracks at other locations that had already been inspected after finding the first. This demonstrates that it helps to know exactly where to look and what to look for on the other details. After a flaw or crack has been identified, it may be helpful to do additional evaluation with NDT such as dye penetrate, magnetic particles, or ultrasonic or radiographic procedures.

Publication of this paper sponsored by Committee on Structures Maintenance.

Bridge Safety Inspection Quality Assurance: Pennsylvania Department of Transportation

Ron L. Purvis and Heinz P. Koretzky*

In December 1986, the Pennsylvania Department of Transportation inaugurated its enhanced Bridge Management System. This system is the resource for district and statewide management decisions involving bridges. Much of the data base for the Bridge Management System is provided by inspectors assigned to decentralized district bridge safety inspection units. It is vital that the system have accurate information with consistent interpretation statewide. Each district has the responsibility for quality control within its inspection units. The Bridge Management System Division's Bureau of Bridge and Roadway Technology, located in the Central Office, monitors the overall quality of bridge safety inspection quality assurance activities. In 1985, Wilbur Smith Associates' Byrd, Tallamy, MacDonald and Lewis Division was selected to develop a quality assurance manual and implement the procedure for a 3-yr period beginning in 1986. The manual defines the quality assurance procedure and details the steps necessary to make it operational. This entails a review of the inspection activities in each district and statistical analysis of the findings. Various aspects of the findings are compared statewide. An annual report is also provided each year as part of the quality assurance implementation. This document contains a summary of the district reports and a comparison of the findings statewide. Although bridge safety inspection quality control and quality assurance are important, few if any state or local transportation agencies have developed standard procedures. The activities are often performed on a hit-or-miss basis, and the results may be difficult to interpret. This paper, in which the bridge safety inspection quality assurance activities developed and implemented for PennDOT are described, should be of interest to other practitioners involved in bridge management and inspection.

The Commonwealth of Pennsylvania Department of Transportation (PennDOT) has a decentralized bridge safety inspection program managed by each of the 11 districts. The inspection activities comply with federal requirements as contained in the National Bridge Inspection Standards (NBIS). The bridge safety inspection provides PennDOT with information on each bridge that is used to complete and update the bridge-inspection-related data base for the Bridge Management System (BMS). This portion of the system accepts, stores, updates, and reports physical and operating characteristics for all public bridges in Pennsylvania. It includes the federal Structure Inventory and Appraisal (SI & A) information. In addition, BMS is the resource for district and statewide management decisions, such as prioritization, rehabilitation, replacement and maintenance needs, costs, future needs, and predictions, and thus becomes a tool for budgeting. It provides about 20 standardized monthly management reports based on data contained in the system.

Much of the BMS information related to bridge inventory structural condition and load capacity determinations of the bridges is provided by inspectors assigned to district bridge safety inspection units, or consultants. Damage or deterioration is reported. Timely remedial actions are programmed by the district and traffic is restricted until appropriate repair or replacement is effected.

Responsibility for Quality Inspections

The accuracy and consistency of the inspection and documentation is vital, not only because it affects programming and funding appropriations but also because it will initiate responsive corrective actions to ensure that bridges remain safe for public use. The department therefore addresses this need with quality control (QC) and quality assurance (QA) procedures. QC is the responsibility of each district. The district develops and enforces bridge inspection QC procedures, which are updated regularly. An outline of the procedures is submitted to the BMS Division, Bureau of Bridge and Roadway Technology (BART). The BMS Division functions as a technical resource to coordinate and standardize the bridge inspection program and disperse appropriate information. This division is also responsible for controlling the overall state compliance with the NBIS. Bridge inspection QA is an independent central office function performed by the BMS Division to ensure that the districts are operating in accordance with the approved QC plans and hence with the NBIS.

Definition of QC and QA

The distinction between QC and QA should first be clarified as the terms are applied to this project. QC is the

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enforcement, by supervisor, of procedures that are intended to maintain the quality of a product or service at or about a specified acceptable level. QC of the inspection of PennDOT's bridges is a daily operational function performed within each district for designated staff members under the supervision of the district engineer.

QA is the verification or measurement of the level of quality of a sample product or service generally by a third party organization. The sampling must be sufficiently representative to permit a statistical correlation with the whole group. The findings are compared against accepted standards to determine whether specified procedures are followed. QA must be performed by an organization external to the operational QC function in order to be objective and unbiased. Statewide bridge inspection QA activities are the responsibility of the BMS Division, Bureau of Bridge and Roadway Technology.

Formalization of QA Activities

Because of the ever-growing demand for quality BMS data, a need was identified by the BMS Division to develop and implement a formalized QA program. Early in 1985, it was decided to engage a consultant to supplement the Bridge Management System Division staff in the execution of the existing QA activities, formalize the procedures, and expand the program. Proposals were evaluated and the Byrd, Tallamy, MacDonald and Lewis (BTML) Division of Wilbur Smith Associates (WSA) Consulting Engineers was selected to perform these activities. The 3-yr contract was executed in January of 1986. The work was to be accomplished in three phases, with each phase completed in a 1-yr time frame.

The work plan for Phase I consisted of developing and analyzing the merits of various QA concepts. A concept was then recommended. After the department's concurrence, the next task was to develop a manual that translated the QA concept into clearly defined procedures and to implement the procedures in four districts on a total of 120 bridge inspections. QA procedures were implemented in the remaining 7 districts in Phase II on 210 representative bridge inspections. The QA procedures and manual were refined during Phase II. Phase III consisted of QA evaluations on representative inspections of state bridges in all 11 districts. Also included in Phase III were QA evaluations for local inspection programs and bridges less than 20 ft long. Approximately 1,000 QA evaluations were performed during the 3-yr program.

QA CONCEPT

Initial Study

Earlier studies by the Operations Review Group within the department indicated the need to improve the uniformity of the inspection procedures statewide. In response to these studies, the BMS Division had bridge safety inspection verification checks in place and operational before this project began. The procedures were, however, performed by individuals with a number of other responsibilities. It was necessary to schedule the quality assurance activities around their other duties. The QA activities were performed on an "as time available" basis while holding documentation to a minimum. The previous program was helpful in monitoring the quality of the statewide inspection program; however, because of overriding priorities, there was a problem staying on schedule. Also, because of those priorities, it became difficult to apply the findings statewide.

During QA development interviews, the department identified several priorities for evaluating the bridge safety inspection program. The first priority is quality. The inspection should be thorough and in accordance with PennDOT guidelines and NBIS. Secondly, there should be a uniform level of inspection and documentation between the districts.

The QA evaluation should not be subjective. The QA manual should provide procedures that permit an objective, quantitative measure of the quality and uniformity achieved by each district bridge inspection program, as well as providing an operational guideline that is to be followed by the department's QA staff that will also inform the districts how their performance is to be measured.

Selecting the Samples

Many possible approaches for selecting sample inspections for QA were identified during the initial studies. The study was reported in a "Letter Report" entitled "Various Concepts Suitable for Use in Implementing a Statewide High Quality Bridge Safety Inspection Quality Assurance Program," dated November 1986, revised December 1986. This study was authored by Ron L. Purvis, Heinz P. Koretzky, and Leonard E. Schwartz. Some of these concepts or approaches are:

Concentrate on worst sources: This approach involves identifying a mean quality level and classifying district bridge inspection programs as above or below that level. The evaluations would concentrate on the bottom x percent of districts.

Concentrate on selected critical bridge types: This approach would involve identifying and classifying critical bridge types, then concentrating on the worst of those types, with priorities based on the degree of redundancy. For example, top priority would be two girder types with pin and hanger details.

Concentrate on condition and deficiencies: This approach would identify bridges by condition, or other BMS deficiencies or FHWA sufficiency points, and apply QA to the worst x percent.

Concentrate on coding items: This approach would classify BMS coding items by order of importance and QA those in order, with priority given to the federal SI&A items.

Concentrate on sensitive items: This approach would classify those items that are identified as critical through sensitive analysis and assign various ranges of acceptable deviation based on sensitivity.

Concentrate on bridge types: This approach would classify bridge types based on overall conditions and QA in order of urgency.

Concentrate on posted bridges and candidates for posting: This approach would require QA on all items of the previously mentioned bridges, but only carry out *x* percent of the items on any bridge on a rotating basis.

Concentrate on routes: This approach would QA bridges as they appear on a selected segment of certain highway routes.

Concentrate on statistical distribution: This approach would identify the bridge population as it exists in every district and inspection *x* percent sample that best represents the existing bridges.

The last method was chosen for selecting the bridges for QA inspections because it best suited the requirements of the initial program with limited funds. A relatively small sample size of 5 percent was considered adequate (2.5 percent/yr) because the objective of QA is not to supplement the existing inspections but to measure the quality of the overall program. Most of the other concepts already listed could have accomplished similar objectives but were found unsuitable at this time because the samples selected would not represent the overall bridge population on the PennDOT highway system. As QA data are collected and analyzed statewide for one or two years, the QA procedures can be logically refined by incorporating some of these other concepts.

Conceptual Procedure

After the method to be used in selecting the sample bridges was determined, the procedure for performing the QA on each bridge was studied. Potential procedures for conducting the evaluation are as follows:

A. Accompany the inspection team in the field (by PennDot);

B. Conduct independent inspection and complete documentation (by consultant);

C. Conduct independent inspection but document only deviations (by consultant);

D. Conduct partial inspection based on report review but document only deviations;

E. Spot-check certain documents based on sensitivity;

F. Spot-check certain items or districts based on past performance;

G. Review inspection file;

H. Question individuals involved in supervising district inspection programs; and

I. Accompany district personnel during district's quality control visits.

To properly evaluate the potential concepts, a detailed listing of objectives (work items) was necessary. The objectives identified were: evaluate thoroughness of inspection; evaluate judgment of inspector; evaluate adequacy of documentation; evaluate follow-up to the inspection; evaluate load rating; evaluate posting document; monitor compliance with district's quality control plan; identify differences in quality compared with a statewide norm; identify differences in quality between teams; identify need to improve existing guidelines; identify need to improve inspector certification training programs; identify need to alter resource commitment; obtain representative QA results; obtain accurate QA results; and make effective use of QA resources.

Because some of the work items and objectives were considered more important than others, a weighted number from 1 to 3 was given to each. Each potential item was then given a rating from 0 to 5 to reflect how it met each objective. A rated effectiveness is totaled for each item. The overall objectives recommended were a combination of several of the items evaluated in Figure 1. A discussion at the end of this paper lists the concepts (*A* to *J*) with the advantages and disadvantages of each.

QA Intensity Level

Most of the activities included in the initial QA concept were refined into four levels of intensity. The reason for the four levels was to permit adjustment of the procedures to match the anticipated current and future needs and available resources of the department's BMS Division. Each level was also costed in relative terms and the results presented to the department in the previously referenced letter report.

- Level 1 is the minimum acceptable approach. This provides a verification of limited field condition and appraisal ratings. There is no check of the inventory and inspection documentation other than the ratings. This is, of course, the least expensive level.
- Level 2 provides a quality verification of the sensitive inspection items and inspection file check for completion. A few more basic BMS inventory items are also verified. On Level 2 there is also no detail check of file data. The cost of Level 2 was estimated to be 1.6 times greater than that of Level 1.
- Level 3 provides an independent quality review of sensitive inspection and inventory items related to items considered most important, sufficiency, and a check of file documentation. As with Levels 1 and 2, the BMS data are available to the QA team and only out-of-tolerance deviations are documented. Level 3 is estimated to cost 2.6 times that of Level 1.
- Level 4 is the most thorough. An independent check is made of all meaningful inspection data. The inspection and inventory documentation is recreated by the QA team. This approach is estimated to cost 6.6 times that of Level 1.

OBJECTIVES	WEIGHT FACTOR	A	B4	B3	C	D	E	F	G	H	I	J
		Q.A. CONCEPT RATING*										
Evaluate thoroughness of inspection	2.00	3	5	4	3	2	2	3	1	0	1	2
Evaluate judgement of inspector	2.00	5	5	4	2	3	3	3	2	1	3	2
Evaluate adequacy of documentation	2.00	4	5	3	3	3	2	3	3	1	2	2
Evaluate follow-up to inspection	2.00	1	0	0	3	2	3	3	5	4	4	1
Evaluate load rating/posting	2.00	2	2	2	2	3	3	3	5	3	3	1
Monitor compliance with quality control plan	2.00	4	4	4	1	3	2	2	2	4	4	5
Identify differences in quality between districts	1.50	4	4	4	2	3	3	0	4	3	3	2
Identify differences in quality between teams	1.50	4	5	4	2	3	3	0	4	2	3	3
Identify need to alter guidelines	1.00	4	4	4	3	3	4	4	4	3	4	2
Identify need to alter training	1.00	5	5	4	2	3	3	4	3	3	4	2
Identify need to alter resource commitment	1.00	5	5	5	1	4	3	3	2	2	3	1
Obtain representative QA results	3.00	1	5	4	3	3	3	0	4	2	2	1
Obtain accurate QA results	3.00	3	5	4	3	3	3	2	4	2	2	2
Conservative use of QA resources	2.50	5	1	3	5	4	5	4	3	3	3	2
TOTAL WEIGHTED EFFECTIVENESS		92	107	94	70	82	82	64	90	62	76	54

* Level of Intensity Rating Scale

5 = maximum   3 = fair   1 = minimal
4 = good   2 = marginal   0 = none

This table is of use in determining relative importance of various QA items. For example, "B4" with a total of 107 would be the most important item while "J" with a total weight effectiveness of 54 would be the least important item using the combination of weight and level of intensity rating scale. Those factors could be updated to reflect new QA program objectives.

FIGURE 1 QA concept evaluation chart.

Figure 2 contains the approved Phase I QA concept. Note that the levels are partially developed. Since this was developed, the Structure Inventory Record System (SIRS) has been merged into the BMS. The appropriate codings are described in detail in the Coding Manual, PennDOT Publication #100A, dated December 1986.

QA MANUAL

Outline

The QA concept developed and approved under Phase I was then expanded into detailed procedures, which are

LEVEL I	II	III	IV		APPROVED PHASE I QA CONCEPT - All approved activities marked with an "X" - Activities not desired are noted accordingly.
					FIELD EVALUATION
			X	A4	Accompany District Inspection Team in field, evaluate procedures, log, tools, crane and traffic continuity
		X		A3	Accompany District Inspection Team in field, evaluate procedures, log, tools
	X			A2	Review inspection log and tools. Some field visits one team a year.
X				A1	Review inspection tools only
			X	B4	Independent field inspection and complete documentation (previous ratings not available)
X	X	X		B3	Independent field inspection document only deviations (SIRS printout available)
X	X	X	X	C	Computer edit of District's BMS data.
Not	desired			D	Partial inspection based on Office Report Review Document only deviations (inspection file available). (No field work.)
					SIRS DATA EVALUATION MOSTLY FIELD
			X	E4	Verify all SIRS items that can be obtained in the field.
		X		E3	Verify only sensitive SIRS items that can be obtained in the field.
	X			E2	Verify only SIRS items affecting Sufficiency Rating.
X				E1	Verify only SIRS items identifying bridge.
Not	desired			F	Spot check certain Teams or Districts based upon performance history.
					OFFICE EVALUATION
			X	G4	District file evaluation including independent checking of rating analysis computations, and timely implementation of appropriate repair and posting procedures.
		X		G3	District file evaluation for essential documentation related to repairs, capacity analysis, and posting
	X			G2	Cursory evaluation of files
X				G1	Spot check file
			X	H4	Evaluation back-up procedures including Bridge Collapse Board of Inquiry, crane and underwater scheduling and QC plan activities
		X		H3	Evaluate back-up procedures including QA plan activities
		X	X	I	Verbally quiz individuals involved in District Inspection Program to verify appropriate knowledge. Evaluate staffing and valid certifications
Not		practical		J	Accompanying District during QA activities
					BRIDGE MAINTENANCE EVALUATION
			X	K4	Bridge Maintenance, review coding and paper trails and simple repairs
		X		K3	Bridge Maintenance, review coding and simple repairs
	X			K2	Bridge Maintenance, review simple repairs
	X	X	X	L	Close-out meetings.

FIGURE 2 Approved Phase I QA concept.

described in the QA Manual (1989 edition). The procedures are divided into the following sections:

1. Planning the evaluation,
2. QA at the bridge site,
3. QA at the district office,
4. Computer edit of BMS data,
5. Bridge maintenance evaluation,
6. District findings, and
7. Annual report.

Planning the Evaluation

Planning the QA evaluation involves (a) selecting and approving the level of QA to be performed at the beginning of the year, (b) visiting the districts each year in a different sequence that must be also determined, and (c) selecting sample bridges for each district consistent with the distribution of bridge types in the district.

The QA Manual contains detailed procedures for selecting the sample bridge inspections. The recommendation is for 5 percent of the bridges to be inspected by the district teams during that year. The selection process is designed to provide a sampling that is representative of all the bridges inspected that year. A profile of all the bridges in the district is first developed for use in selecting the samples. The features that are considered most important in the sample selection process are: type of superstructure, total length, sufficiency rating, and district team performing the inspection. Figure 3 is an example of a district structure profile.

Because the QA review includes a field evaluation to assess the quality of the district inspection, it is important that it be performed soon after the district inspection is completed. Therefore, the sample bridges must be selected from those inspected within the last few months. The objective is to match the district's bridge population profile as closely as possible, selecting only from the group that was recently inspected. Beyond that, the selection is made at random. Difficulty of access to the bridge because of size or location should not disqualify a bridge from inclusion in the sample group.

QA at the Bridge Site

The QA at the bridge consists of an independent verification of certain sensitive condition/appraisal items prescribed in the QA Manual based on the intensity level of QA review (see Table 1 for the activities included in the field review). The QA procedure for each activity described in the manual contains a range of requirements that increase with the QA intensity level.

Assessing the quality of the field inspection is an important function of QA because deficiencies in this part of the program could affect the safety of the state's bridge system. A hands-on, close-up inspection of the bridge is therefore included in all QA intensity levels. The levels differ, however, in the information available to the QA team when it performs the evaluation and documentation required to describe the condition finding. Level 4 is a totally independent inspection without benefit of any previous inspection reports. Complete independent inspection documentation is provided at this level. In Levels 1 to 3 the QA team has the previous data and verifies them at the site. If the condition rating given by the district is not more than one number different from that of the QA team's

Structure	Type	Code	County A	B	C	D	E	F	Total	%	AVAILABLE Total	Samp	%
Steel	Beams	A	78	74	34	119	59	33	397	19.7	11	6	1.5
	Box Beams	B	-	-	-	-	-	-	-	-	-	-	-
	Girder, Flr Bm	C	14	9	1	5	15	7	51	2.5	1	-	-
	Truss	D	4	11	4	4	6	16	45	2.3	1	1	2.0
Concrete	Arch	E	-	-	-	-	1	-	1	n	-	-	-
	Slabs	F	44	70	15	76	43	36	284	14.1	12	5	2
	Tee Beams	G	25	49	11	37	48	24	194	9.6	8	3	1.5
	Arch	H	8	3	1	6	-	1	19	1.0	n	-	-
Prestressed Concrete	Channel Beams	I	1	-	-	-	-	1	2	n	1	1	5.0
	"T" & "I" Beams	J	18	28	3	1	3	-	53	2.6	6	2	4.0
	Box Beams	K	64	89	41	43	33	16	286	14.2	13	3	1.0
Timber		L	-	-	-	-	-	-	-	-	-	-	-
Masonry		M	2	61	3	6	14	7	93	4.6	1	-	-
Wrought Iron		N	-	-	-	-	-	-	-	-	-	-	-
Concrete Encased		O	10	8	1	6	-	2	27	1.4	2	-	-
Culverts		P	95	154	65	117	81	53	565	28.0	24	9	1.8
Other			1	1	-	-	-	-	2	n	-		-
Unknown			1	-	-	-	-	-	1	n	-		
TOTAL			365	557	179	420	303	196	2020	100	80*	30	1.5
Length													
20' - 70'			205	320	100	240	170	110	1145	57	47	17	57
70 ' - 150'			74	109	35	83	62	41	404	20	16	6	20
Over 150'			86	128	44	97	71	45	471	23	17	7	23
Sufficiency Rating													
Less than 50			30	58	15	45	32	21	201	10	7	3	10
50 - 80			89	188	60	140	109	69	658	33	26	10	33
Over 80			243	311	104	235	162	106	1161	57	47	17	57
Inspection Team													
A			365		179		303	196	1043	52	49	17	57
B				557		420			977	48	31	13	43

FIGURE 3 Example of district structure profile.

TABLE 1 ACTIVITIES INCLUDED IN FIELD REVIEW

Field Q.A. Review Activities	QA Intensity Level I	II	III	IV
Verify and identify the structure	X	X	X	X
Photograph the structure		X	X	X
Verify inventory data based on QA intensity level	X	X	X	X
Take measurements for load rating check			X	X
Verify "safety" features and load posting signs	X	X	X	X
Perform independent condition and appraisal	X	X	X	X
Compare with district ratings and reconcile, if possible	X	X	X	
Document findings based on QA level	X	X	X	X
List and prioritize maintenance and repair needs		X	X	X

rating, then it is within "tolerance" and no further documentation is required. The QA team also has to collect inventory data details required to perform load rating and posting information. QA for these activities varies with the level selected.

QA at the District Office

QA at the district office consists of verifying the availability and accuracy of the documentation on file (see Table 2). The evaluation of each varies from cursory (*C*), to standard (*S*), to in-depth (*I*), depending on the level. The QA Manual includes details describing each level. The levels of the office review are designed to coordinate with the level of the field review. The details obtained in the field are confirmed in the office. The file is also evaluated to determine how the inspection is used. For example, were recommendations implemented, or was a new load rating analysis necessary? The QA teams use the form shown in Figure 4 to rate each item and comment as necessary.

A questionnaire is also completed during the office visit to monitor the district procedures. Because the districts are decentralized, there are no standardized procedural requirements as long as overall standards are met. However, it is helpful in evaluating the results to relate level of conformance to the unique organizational structure of the district under review.

Computer Edit of BMS Data

The BMS system is programmed to flag certain data and items for consistency and conformance with guidelines. It identifies certain erroneous entries; for example, codes that do not apply. It identifies inspections that are overdue. If teams are omitted this is also flagged. Because the function is performed within the BMS Division, it was not highlighted during the initial development of the manual. It is a part of the total QA effort, and is therefore included in the manual.

It is anticipated that at some time in the future the BMS system could be enhanced with additional indicators to select on command the appropriate sample bridges to receive QA. These are currently being selected from a computer printout containing the recent inspections in the district.

Bridge Maintenance Evaluation

An important purpose of the bridge safety inspection is to identify maintenance and repair needs and priorities. PennDOT has standardized this process in the December 1986 Coding Manual. This part of the QA evaluation focuses on the accuracy of the maintenance and repair needs identified by the districts and the procedures and paper trail for implementing the work. This portion of the QA effort was included in Phase III of the current QA program. An

TABLE 2 CHECKLIST FOR VERIFYING AVAILABILITY AND ACCURACY OF DOCUMENTATION ON FILE

Office QA Items	QA Level I	II	III	IV
General file contents	C	S	S	I
Inventory documentation	C	S	S	I
Inspection documentation	C	S	S	I
Proposed improvements		C	S	I
Load rating analysis		C	S	I
Compliance with posting policy		C	S	I

Note: C = cursory, S = standard, and I = in-depth.

Rating scale for QA evaluation of bridge file.

5	4	3	2	1
Exceeds Standard		Meets Standard		Below Standard

Bridge ID Number ____________ QA Team _____ Date________

Following are the significant findings of the QA file review organized by topic:

1. General file contents
   Rating for completeness ________ Rating for accuracy ________
   Remarks:

2. Inventory documentation
   Rating for completeness ________ Rating for accuracy ________
   Remarks:

3. Inspection documentation
   Rating for completeness ________ Rating for accuracy ________
   Remarks:

4. Proposed improvements
   Rating for completeness ________ Rating for accuracy ________
   Remarks:

5. Load rating analysis
   Rating for completeness ________ Rating for accuracy ________
   Remarks:

6. Compliance with posting policy
   Rating for completeness ________ Rating for accuracy ________
   Remarks:

FIGURE 4 PennDOT bridge safety inspection QA program bridge file evaluation form.

example of the QA levels developed for this activity is as follows:

- Level 1. Structural elements requiring repairs within 6 months are identified.
- Level 2. Same as Level 1 except that the recommended repair is included for the elements identified.
- Level 3. Same as Level 2 except that all maintenance and repair needs are listed. This list includes the repairs necessary to return or preserve structure at the original condition.
- Level 4. Same as Level 3 except that priorities are included for the maintenance and repair.

Ideally, the inspection documentation identifies immediate problems, potential problems, and necessary maintenance to avoid future problems. It is expected that the bridge safety inspection file will include a paper trail that, in combination with BMS data, indicates the recommended improvements, a priority for each, and the dates that the work is scheduled and completed. The QA evaluates this based on the required QA level.

District Findings

A report is submitted for each district QA evaluation that provides the details of the findings. After this report is reviewed, the findings are discussed with the district in a close-out meeting. The district report is designed to provide a quantitative measurement of the quality consistent with the original QA objectives. The same items are evaluated in the same order on each bridge review. The report provides a statistical correlation of the findings. The data are organized so that areas where the district consistently differs with the judgment of the QA team may be readily identified. See Figures 5 and 6 for examples of how this material is displayed in this report. Unique findings are also listed. The report contains a section for the summary and conclusion. After the report is submitted and reviewed, a close-out meeting is held with the district and BMS Division staff to discuss the findings and resolve any problems.

Annual Report

The annual report contains a summary of all QA activities performed for a given year and a comparison of these findings statewide. In this report, the bar charts for each inspection item are arranged so that all the district results are listed side by side. This format is helpful in identifying inspection items that have received a wide range of ratings for a given condition (see Figures 7 and 8). This information is helpful in identifying possible needed enhancements in the inspector's training information, or in the guidelines contained in the BMS coding. If deviations are experienced for a particular item in just one district, it is more likely an internal problem.

	STRUCTURE	CODE	TYPE	APP RDWY E15 Q S	DECK WS E16 Q S	DECK E17 Q S	SUP STR E18 Q S	SUB STR E20 Q S	CHAN E21 Q S	STR COND E24 Q S	DECK GEOM E25 Q S	WATERWAY E27 Q S	APP ALGN E28 Q S	SAFE FEAT A24 Q S
1	4013	G	21103	7 7	7 7	6 4	5 6	5 4	5 6	5 5	5 4	5 6	6 6	4222 4222
2	4015	F	21101	7 7	7 7	7 7	6 7	6 7	6 6	7 6	5 6	7 6	8 8	4222 4222
3	5011	P	21931	N N	N N	N N	N N	N N	8 8	8 8	N N	7 6	N N	8888 8888
4	2044	J	42207	6 7	5 7	6 6	6 8	5 6	7 7	8 8	7 7	N N	7 7	6688 6688
5	2045	K	42204	5 7	7 6	7 6	7 7	7 7	6 7	6 7	6 6	7 6	5 6	6688 6688
6	6007	J	42206	6 6	7 7	5 6	5 6	6 6	7 6	7 5	7 7	7 6	7 5	6644 6668
7	5010	P	21931	6 8	7 7	N 4	7 6	8 8	6 7	6 6	6 8	6 6	7 7	4222 4222
8	8001	D	19118	5 6	6 4	8 8	7 6	8 8	5 7	7 7	6 7	7 5	6 7	8688 8888
9	5018	A	16104	7 7	6 7	7 7	7 6	7 7	6 6	7 7	7 7	6 7	6 7	8884 8888
10	8001	P	21931	8 8	6 6	7 7	N N	5 6	7 8	6 8	7 7	5 6	7 5	4222 4222
11	5084	O	86104	7 7	7 6	7 7	6 7	6 7	6 6	7 6	5 6	7 5	8 8	6222 6222
12	9014	A	16104	7 5	5 6	N 8	7 7	7 7	7 7	7 7	7 5	7 7	6 8	3388 4488
13	9011	D	16118	7 7	7 5	6 7	6 7	6 6	8 8	5 6	6 7	7 6	6 6	3222 3222
14	9027	A	16104	6 7	8 8	5 7	7 7	7 6	6 7	6 6	5 7	6 7	6 7	4222 4222
15	9021	A	16104	7 7	7 7	8 8	N 8	8 8	7 8	5 7	6 6	7 5	7 5	4644 4888
16	2004	A	16104	6 7	8 8	5 6	6 6	8 8	7 7	6 7	7 6	6 7	5 7	4643 4884
17	7005	K	42204	7 7	7 7	8 8	7 7	8 8	7 7	N 7	7 7	6 7	7 7	8884 8888
18	4001	A	16104	6 6	N 7	7 7	7 7	5 6	7 6	5 7	6 7	6 6	4 6	4222 6222
19	2008	J	42206	4 7	7 7	6 7	5 6	7 5	N N	7 6	N N	7 5	7 N	4333 4333
20	4001	C	18114	7 7	6 7	6 7	7 5	8 8	6 7	4 6	6 7	7 6	7 7	3222 3222
	OUT-OF-TOLERANCE + 2			4	3	2	2	1	0	5	3	4	5	

The numbers across the top are BMS item numbers. There are two ratings below each item number for each bridge. The QA rating first then the district rating. The code letter is related to the STRUCTURE TYPE PROFILE.

FIGURE 5 Rating comparison of bridges selected for QA review.

There is also a section on recommendations for the next year. This section proposes modifications in the program based on the annual findings. A recommendation is made for the QA level for the next year. If there are improvements warranted in the QA procedures, these are also recommended. This section might also contain suggestions for improvements in the statewide BMS coding guidelines or inspector's training. When accepted by the department, the recommendations are implemented either by BTML, under Phase II and III of the existing contract, or by other agents of PennDOT.

IMPLEMENTING THE QA PROCEDURES

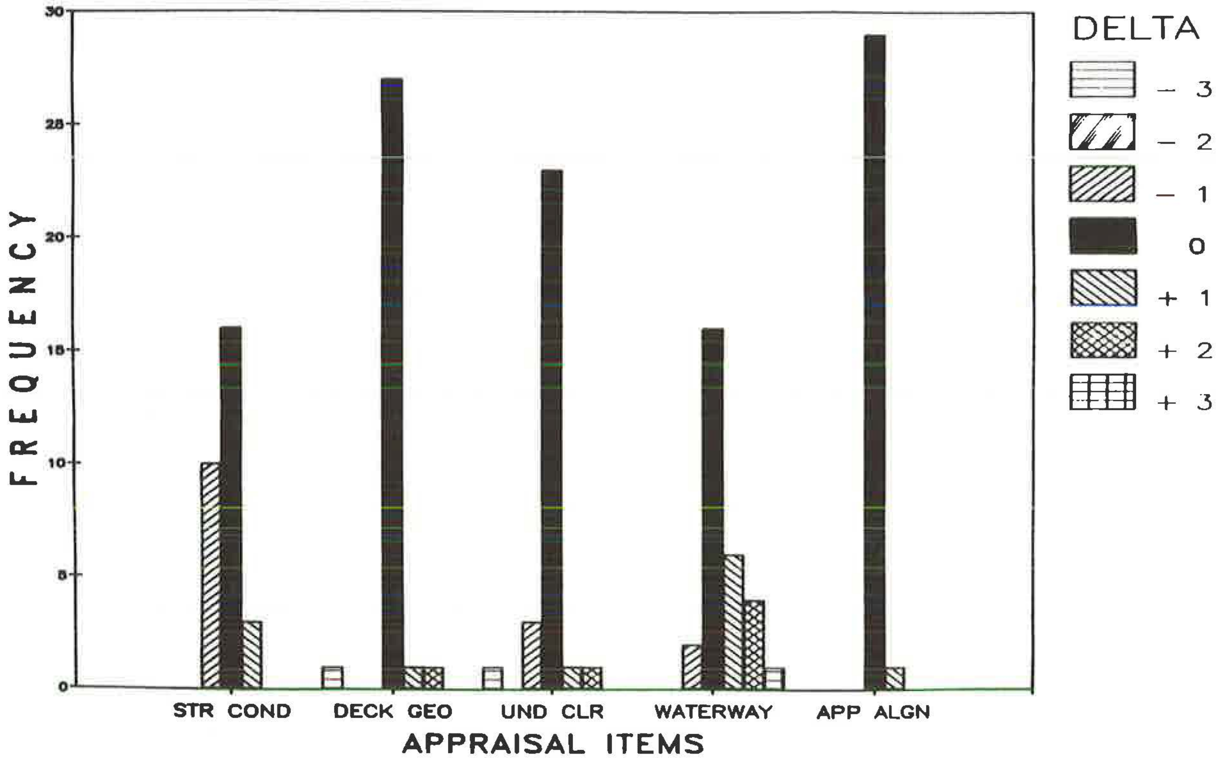
Start-up

Phase I QA evaluations were performed during the development of the manual using interim procedures. This meant that the districts did not receive specific QA procedural information before the review results that explained the focus of the Phase I evaluation. Some procedures were modified as the evaluation was in progress. In Phase II, the procedures were in accordance with the draft manual that was given to the districts for review and comments before the QA evaluations. The manual was refined again for Phase III.

The QA Team

The QA team leader must be approved by the chief of the BMS Division. The Phase I team leader is a registered professional engineer, and has attended the department's bridge safety inspection training course. The team is normally composed of two inspectors, the second member being a graduate engineer with 2 yr of bridge inspection experience. Occasionally the team was accompanied by the principal investigator, who was involved in defining the QA concept and developing the manual.

Because all the district's inspection ratings are compared with the QA ratings, the judgment should be the same on



NOTE: + INDICATES DISTRICT RATING GREATER THAN REVIEW

FIGURE 6 Difference between QA review and District *C* ratings.

each evaluation. Therefore, changes in QA team members were minimized while implementing the evaluations.

Time Requirements

The 3-yr QA implementation began in early June 1986 and was completed in December 1988. This does not represent a continuous effort because procedures were being developed, modified, and approved by the BMS Division. During the first 2 yr, 4 to 6 weeks were required for each district review to plan, evaluate, and report on 30 sample inspections and to complete the close-out requirements. Some overlapping of district reviews was possible during the report review and close-out scheduling. In Phase III (1988), QA evaluations were performed in all 11 districts. The number per district increased with the addition of local bridges, less than 20-ft long bridges, and special emphasis bridges. An additional QA team was added during Phase III to keep the evaluations on schedule.

	DISTRICTS					
CONDITION ITEMS	A	B	C	D	Sum	%
Approach Slab	30	30	30	29	119	99.2
Approach Roadway	23	26	29	26	104	86.7
Deck	30	28	29	27	114	95.0
Superstructure	29	30	30	30	119	99.2
Paint	30	30	30	30	120	100
Substructure	30	30	30	30	120	100
Channel	29	30	27	25	111	92.5
Culverts	30	30	30	30	120	100
Condition Sub Total	231	234	235	227	927	96.6
APPRAISAL ITEMS						
Structure Condition	30	30	30	30	120	100
Deck Geometry	29	27	27	25	108	90.0
Underclearance	30	30	27	30	117	97.5
Waterway	28	27	24	26	105	87.5
Approach Alignment	23	27	30	27	107	89.2
Appraisal Subtotal	140	141	138	138	557	92.8
TOTAL	371	375	373	365	1484	95.1

FIGURE 7 Summary ratings within tolerance.

Findings

The QA level performed for the 3-yr implementation was generally at or above Level II. The findings will be more meaningful as the program generates sufficient results to define reasonable expectations. The department was pleased with the correlation between the condition and appraisal ratings of the QA team and the district inspection teams. The overall correlation of the ratings within tolerance was 94.9 percent for 3 yr.

Most deviations seemed to be caused by the individual interpretation of the guidelines by the different district teams rather than a deficiency in the inspection procedures. The lowest correlation of the ratings within tolerance was for Approach Roadway at 92.0 percent, Deck

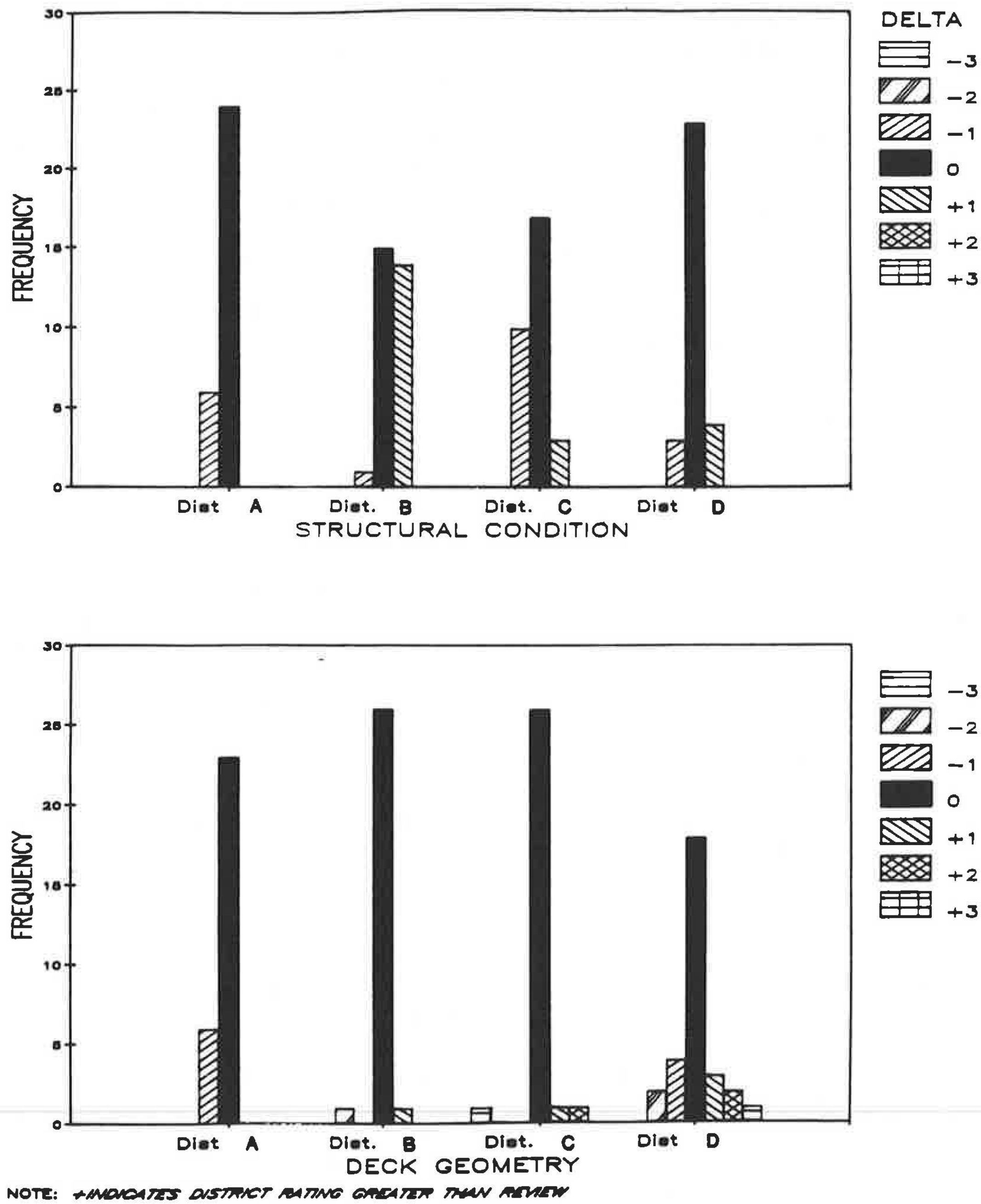


FIGURE 8 Difference between QA review and Districts *A, B, C,* and *D* ratings.

Wearing Surface at 91.5 percent, Approach Alignment at 92.7 percent, Deck Geometry at 91.4 percent, and Channel at 91.5 percent. Generally, the correlation was better on the condition rather than appraisal item. It was also better on the state rather than local inspections.

Details of the findings are contained in the district reports and summarized in the Phase I, II, and III annual reports. The annual report also contains the resolution of problems that were reported and discussed at the close-out meetings. Some common topics were load rating procedures, inspection documentation, and posting policy conformance.

Recommendations

The following recommendations were made for future QA implementation:

- Ensure implementation of Level IV QA procedures;
- Improve the instructional guidelines for Approach Roadway, Deck Wearing Surface, Structural Condition, Approach Alignment, Deck Geometry, and Channel;
- Develop a standard load rating and posting form to include in all bridge files;

- Improve guidelines for updating load ratings;
- Expand QA procedures involving bridge maintenance follow-up;
- Ensure that QA scope includes inspections of bridges owned by localities and railroads, bridges inspected by consultants and authorities, and bridges with 8- to 20-ft openings; and
- Put additional emphasis on proper inspection of scour, fracture critical details and underwater structure components during QA evaluations.

DISCUSSION OF QA ITEM EVALUATION

A. Accompany inspection teams in field: The major flaw with this method is that the presence of the QA review team probably will influence the inspector's attitude and performance. This would not be representative of the day-to-day operations. The major advantage is that the QA evaluation can be made with fewer resources than can an independent review, and there is the opportunity to test the inspector's knowledge by asking questions. The method is thought to be more appropriate for district office quality control than central office quality assurance and is appropriate to identify performance differences between districts.

*B*4. Independent field inspection and complete documentation (previous documentation not available): This is considered to be the most effective method of verifying the inspection and field documentation; however, it is also the most expensive. Not only will it take considerably more time and resources to recreate all the inspection and inventory documentation but an additional trip to the bridge will often be required to resolve differences. QA Level 4 uses this approach.

*B*3. Independent field inspection—document only deviations (previous ratings available): This is an acceptable alternative to the previous method. The QA team takes the current BMS printout to the bridge, but to avoid being influenced team members do not look at the ratings until after completing a separate condition evaluation. They provide documentation only on the ratings that differ significantly from the district's ratings. The approach is part of Levels I, II, and III, the difference being the number of inspection items subject to QA.

C. Computer edits districts' BMS data: A carefully conceived computer edit of inventory and inspection data entered by the district for each bridge is a relatively inexpensive method of identifying erroneous and contradictory information. It is currently performed by BMS Division but requires modifications to fit the desired level. This method identifies contradictions in entered data but does not determine or verify the actual situation in the field.

D. Partial inspection based on report review—document only deviations (previous report available): This is a method used by some districts for quality control. They look for unusual condition changes or very low ratings when reviewing the reports. The items are then evaluated in the field to verify the rating. This is not practical for QA because it requires an initial review of all the reports. The QA field evaluations would then be scattered throughout the state. Other objections are that the evaluations would be slanted toward problem bridges and the evaluation would not determine the thoroughness of the inspections. An inspector could get by with a poor job as long as the ratings did not change. This approach is not recommended.

E. Evaluate certain items based on sensitivity: Resources do not permit a complete check of all the information contained in the inspection and inventory file for each QA evaluation. Some information is more sensitive than others in considerations such as sufficiency or load rating. QA Levels II, III, and IV include this method for selecting inspection items in the inventory evaluation.

F. Evaluate certain teams or districts based on performance history: An important objective of the QA program is to provide an accurate picture of the overall bridge safety inspection program; the sampling technique therefore should provide a representative group of bridges. This method would not do that, and it is not recommended.

G. Evaluate inspection file: The file normally contains backup data for the load rating analysis, posting recommendation, and maintenance work orders. The file also contains detailed reports of the periodic inspection findings. QA Levels I, II, III, and IV include a different level of file quality evaluation of all the bridges selected for the field review.

H. Evaluate backup procedures: It is difficult to evaluate the various data in the file without understanding the procedures that generate and use the information. QA Levels II and IV include an evaluation of the office planning and follow-up procedures related to the bridge safety inspection program.

I. Verbally question individuals involved in supervising district inspection program: The districts are unique and have special requirements of their inspection program. A standardized questionnaire is helpful to document the organizational structure, procedures, and personnel capability found in each district. This and the previous method will often overlap. It is also included in QA Levels III and IV.

J. Accompany district personnel during quality control visits: Each district has an approved QC plan for bridge safety inspection. An objective of QA is to monitor compliance with the QC plan. The district QC plans include field visits by the different levels of supervisors responsible for the program. The visits are often spontaneous or combined with other responsibilities. Although it might be useful, it is not practical to include on-site monitoring of these visits as part of the QA concept. The review of this activity is, therefore, restricted to verification by asking questions during office interviews included in QA Levels III and IV. The effectiveness of the district QC efforts will be evidenced by the results of the other QA activities. Therefore, this is not a practical QA work item.

Table 2 includes items *K* and *L*, which were added after this appraisal was made.

ACKNOWLEDGMENTS

Other members of the project team contributed substantially to the development of information contained in this report. Notable are Leonard E. Schwartz, Project Manager, Bridge Maintenance Systems Division, Bureau of Bridge and Roadway Technology, Pennsylvania Department of Transportation; and Thomas H. Suthers, QA Team Leader, Wilbur Smith Associates, BTML Division (WSA/BTML). The following individuals contributed in an advisory capacity: Roland Berger, WSA/BTML; Martin Rissel, WSA/BTML; Robert Nickerson, Federal Highway Administration; William Moyer, Gary Hoffman, Steve Simco, Joseph O'Melia, Charles Mergl, Roland Cope, and Robert Paul, all of PennDot; and Neal Wood, Pennsylvania Turnpike Commission.

Publication of this paper sponsored by Committee on Structures Maintenance.

Application of Fuzzy Logic to Condition Assessment of Concrete Slab Bridges

A. B. TEE, M. D. BOWMAN, AND K. C. SINHA

There is presently no well-established procedure for a bridge inspector to follow when assessing the combined effects of multiple flaws or imperfections on a bridge. Consequently, the evaluation of an existing bridge is based on a highly subjective procedure, and usually suffers from imprecision and personal bias. Different bridge inspectors may assess a given bridge differently. The purpose of this article is to examine the application of fuzzy logic for assessing the condition of concrete slab bridges. A number of factors that affect the condition of a bridge deck are examined. An example problem is presented to illustrate use of the proposed methodology.

Bridge structures of today reflect the engineering experience and research developments that have evolved over many centuries. An impressive amount of research has been conducted in the development of new technology and materials for the design and construction of bridges during the last five decades. The use of welding, high strength structural bolts, epoxy-coated reinforcing bars, and prestressed concrete are examples of recent technological advances in the field of bridge engineering (*1*). Nevertheless, these technological advances have not precluded a number of tragic bridge failures.

After a bridge has been built, it must be kept in a serviceable state through regular inspection and maintenance. Unfortunately, it took the collapse of the Silver Bridge in Ohio in 1967, which claimed 46 lives, to arouse public interest and awareness of the importance of inspection and maintenance of bridges (*2*).

Unlike the design and construction stages, bridge inspection is usually performed by a much smaller team; commonly by a single bridge inspector. The problems encountered in this type of work are numerous and often complex. The inspector must be thoroughly familiar with the various bridge design and construction features to be able to recognize and properly interpret any structural deficiencies and evaluate their seriousness before making any appropriate improvement recommendation (*3*).

During a bridge inspection, information collected and perceived by the inspector can be divided into objective and subjective parts, respectively (*4*). The objective portion involves measurable information such as the remaining diameter of corroded reinforcement bars or the width of cracks at specific locations, whereas the subjective portion involves the wisdom and experience of the inspector, who must evaluate the severity of the deficiencies and their combined impact on the overall structural integrity. Although the importance of the subjective information is recognized, the present inspection procedure does not have the capability to incorporate systematically the subjective information into the rating process.

Consequently, a method that combines both the objective knowledge and the imprecise subjective wisdom of bridge inspectors to make logical and systematic evaluations would be very useful. One such technique that uses this wisdom is the fuzzy logic approach.

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OBJECTIVE

The primary aim of this paper is to present a fuzzy logic approach for assessing the combined effects of imperfections on the overall condition of a bridge.

The imperfections discussed in this paper are the corrosion of steel reinforcement, and the scaling, cracking, and spalling of concrete. Membership functions describing the various states of structural condition for these imperfections are presented. A simple example is also included to illustrate the application of the fuzzy logic methodology and to highlight the advantages and limitations of the proposed method for bridge inspection.

BRIDGE DECK DETERIORATION

The deterioration of concrete bridge decks along the nation's highways is a major problem for many states. The common structural deficiencies associated with the deterioration of bridge decks are the corrosion of steel reinforcement and the cracking, scaling, and spalling of concrete.

The effect of these imperfections on a bridge deck is imprecise and subjective knowledge and can best be handled by employing the fuzzy logic via the membership functions. The central feature of the fuzzy logic approach is the membership function, which represents numerically the degree to which an element belongs to a set. Instead of using only 0 and 1 when dealing with objective information, the degree of membership can take on values between 0 and 1 to fully describe subjective concepts. The membership functions describe the various states of structural condition as a result of the structural imperfection. This new system is a closer representation of the subjective information of the human cognitive process (*5*).

The membership functions presented in this paper are developed on the basis of structural analyses and information extracted from the literature. It should be emphasized that the membership functions, albeit reasonable, are subjective in nature and can be further enhanced through expert opinion or availability of additional information if necessary.

Although corrosion, scaling, spalling, and cracking are not the only types of imperfections that occur on a concrete bridge deck, they represent some of the most common and severe problems. Thus, the development of membership functions described herein applies to corrosion, scaling, spalling, and cracking only.

CORROSION OF REINFORCING STEEL

Although several factors contribute to the deterioration of concrete structures, one of the primary causes is corrosion of the reinforcing steel. The repeated applications of deicing chemicals on bridge decks and roadways during the winter months release large quantities of chloride ions that penetrate to the reinforcing steel level. In the presence of moisture, an electrical potential difference occurs and the corrosion process is initiated (*6, 7*). The corrosion process is accelerated by the presence of oxygen. The corrosion products increase the volume of the reinforcing steel, thereby creating tensile stresses in the surrounding concrete. When these stresses exceed the tensile strength of the concrete, the concrete cracks and eventually spalls or delaminates (*8*).

Corrosion can also cause loss of bond between concrete and the reinforcing bars. When this happens, the tensile stress is decreased in the region of bond loss and the bond stresses are increased in the remaining bonded regions. If this process continues, a loss of concrete cover will occur and the available strength of the bridge section will be reduced (*9*).

Development of the membership functions for evaluating the effect of corrosion on the structural condition is based on the ratio of the ultimate moment capacity to the service load moment at critical points of the bridge (*10*). For the positive steel of a continuous slab bridge, the critical locations are assumed to be approximately at the 4/10 point of the exterior span and the center of the interior span; for a simply supported slab bridge, the critical location is at the center of the span. The critical point of the negative steel in a continuous slab bridge is at the interior support. In short, these critical locations are the points of maximum positive and negative moment along the bridge deck. The ratio of the ultimate moment capacity to the service load moment is taken to be the factor of safety. It should be noted that the reduction in reinforcement cross-sectional area is taken as the critical corrosion parameter in the present study. The effect of bond loss caused by corrosion was not included in the analysis for development of the membership functions. Consequently, a summary is presented in Tables 1 and 2 of the computed factor of safety corresponding to the percent reduction in the area of positive and negative steel, respectively. Different slab thicknesses were examined to illustrate the influence of slab thickness on the corresponding safety factors.

From these tables, it can be observed that an increase in the slab thickness yields a slight increase in the factor of safety. It should be noted that in the analyses a thicker slab permits a reduction in the required area of steel for a given loading condition. However, a thicker slab results in a higher dead load moment, and the reduction in the required area of steel is not directly proportional to the thickness increment.

It should also be pointed out that the area of steel provided is slightly more than the area of steel required, depending on the bar size and spacing selected. For example, if the required area of steel per foot of slab width

TABLE 1 FACTOR OF SAFETY CORRESPONDING TO CORROSION OF POSITIVE STEEL

Bridge[a] Type	Slab Thickness (in.)	Percent Reduction in Area of Steel					
		0	10	20	30	40	50
A	12½	1.81	1.64	1.48	1.31	1.14	0.96
	14½	1.87	1.70	1.52	1.35	1.16	0.97
	16½	1.90	1.72	1.55	1.36	1.17	0.98
B	12	1.87	1.70	1.52	1.34	1.16	0.98
	14	1.90	1.73	1.54	1.36	1.17	0.99
	16	1.93	1.75	1.56	1.38	1.18	1.00
C	12	1.77	1.62	1.46	1.29	1.13	0.95
	14	1.84	1.67	1.50	1.33	1.15	0.97
	16	1.90	1.72	1.54	1.37	1.17	0.99
	18	1.92	1.73	1.55	1.37	1.18	0.99

[a]A = Simple-span reinforced concrete slab bridge (span length, 20 ft); B = Simple-span reinforced concrete slab bridge (span length, 15 ft); C = Continuous reinforced concrete slab bridge (3 spans 27-34-27 ft).

TABLE 2 FACTOR OF SAFETY CORRESPONDING TO CORROSION OF NEGATIVE STEEL

Bridge Type[a]	Thickness (in.)	Percent Reduction in Area of Steel[b]					
		0	10	20	30	40	50
	12	1.58	1.42	1.27	1.10	0.95	0.79
A	14	1.71	1.54	1.37	1.20	1.02	0.85
	16	1.79	1.61	1.42	1.25	1.07	0.90

[a] A = Continuous reinforced concrete slab bridge (3 spans 27-34-27 ft).
[b]Reduction in area of steel at the first interior support.

computed is 1.56 in^2, the selected steel will most probably be No.8 bars at a 6-in. spacing, providing an area of steel of 1.58 in^2. The area of steel provided per foot of slab width in this case is 0.02 in^2 more than that required. Such minor variations in reinforcement provided cause the computed factor of safety to fluctuate slightly in these tables.

The variation of the factor of safety with respect to the percent reduction in the area of steel for different slab thicknesses can be represented by a relatively narrow band or envelope, as shown in Figure 1. Membership functions describing the various states of structural condition with respect to the reduction in area of steel for slab bridges are presented in Figure 2. The safety factors shown in Tables 1 and 2 and the corrosion envelope depicted in Figure 1 were used as guidelines in the development of the various membership functions. It should be emphasized that the position and magnitude of the membership functions, albeit reasonable, are subjective in nature. This can be further fine-tuned through the availability of additional information or expert opinion.

CONCRETE DETERIORATION FACTORS

Characteristics of Cracking

Cracking is defined as an incomplete separation of concrete into one or more parts, with or without a space between them (*11*). A comprehensive review of the common causes and characteristics of cracks in concrete can be found in the report by Manning and Bye (*12*).

Cracking was once viewed as a fault of design or workmanship (*13*). However, it can be readily shown that under normal and reasonable stress conditions, reinforced concrete members are already cracked and will generally perform satisfactorily with respect to their load-carrying capacity. Cracks appearing at the time of construction because of shrinkage or settlement of the falsework are usually fine and do not adversely affect the performance of the bridge deck (*13*). Conversely, pattern cracking resulting from the use of reactive aggregates may occur several years after construction, increase in magnitude,

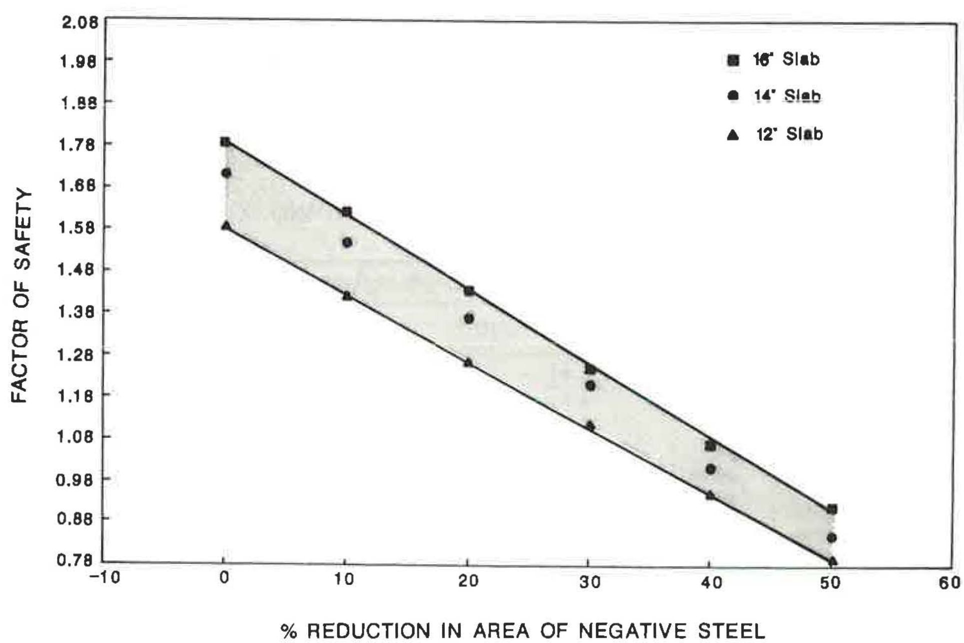

FIGURE 1 Factor of safety envelope for reinforcing steel corrosion.

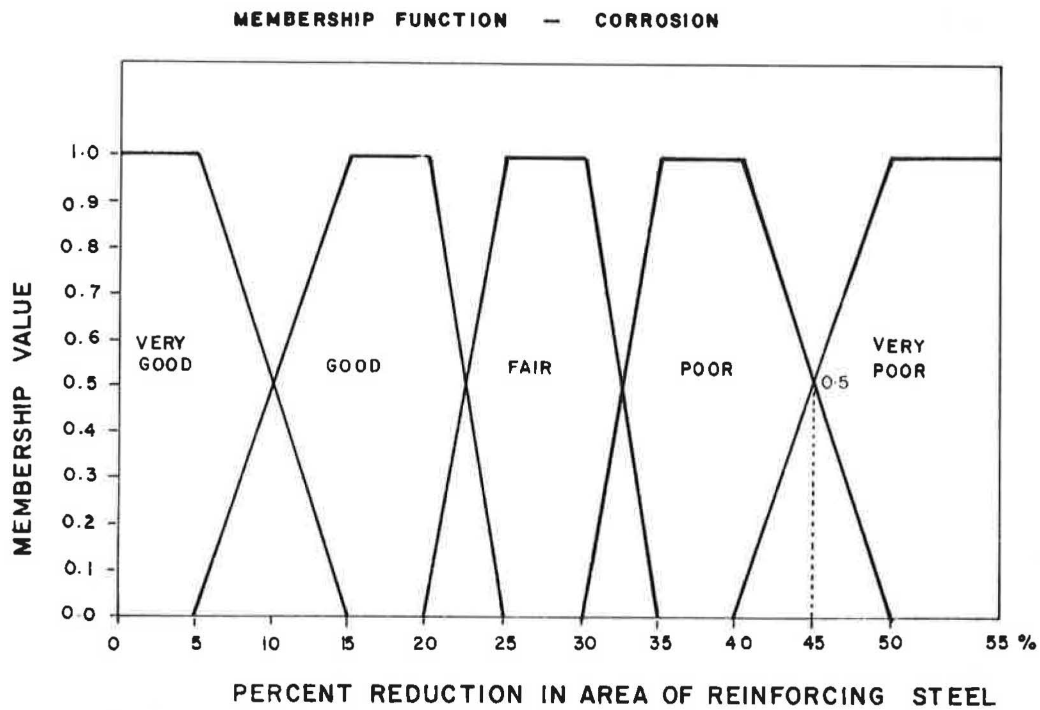

FIGURE 2 Structural condition membership functions for reinforcing steel corrosion.

and eventually result in complete disintegration of the concrete.

It is desirable to limit crack widths for corrosion protection, leakage prevention, and aesthetic reasons. The concrete crack width classification proposed by Ryell and Richardson (*14*) is given as follows:

Description	*Crack Width*
Hairline	Less than 0.004 in.
Narrow	0.004 to 0.01 in.
Medium	0.010 to 0.030 in.
Wide	More than 0.030 in.

Although the depth of a crack can be an important classification parameter, it is generally neglected in most studies. This is because the depth of a crack can be determined only by coring, except in those cases in which the crack is visible on the opposite surface of the member. Because coring samples are usually not taken during routine inspection, the depth of cracks in bridge decks or structural members is not readily available. Thus, cracks are generally categorized using crack width as the only parameter (*15, 16*).

Membership Functions for Concrete Cracks

Based on the classification of crack width previously described, it can be safely assumed that a concrete member having an average crack width falling within the 0.013- to 0.020-in. range is in fair condition, provided that there are no other flaws. From the values given in the concrete crack width classification, it can be inferred that concrete members having average crack widths greater than 0.030 in. are in very poor condition. Similarly, a concrete member can be said to be in very good condition if it contains average crack widths of less than 0.004 in. The intermediate ranges between very good and fair, and from fair to very poor are described using the linguistic variables "good" and "poor," respectively.

Hence, the structural condition membership functions for concrete cracks were formulated using the crack width classifications. These are shown graphically in Figure 3.

Characteristics of Scaling

Scaling is the flaking of surface mortar often accompanied by the loosening of surface aggregates. Scaling is believed to be caused by freezing and thawing, poor workmanship, or inadequate curing of the concrete (*17, 18*). When concrete cools below the freezing point of water, there is an initial period of super-cooling during which ice crystals form in the large capillaries. Because water in the cement paste is in the form of a weak alkali solution, the alkali content in the unfrozen portion of the solution in these capillaries increases. An osmotic pressure is created, and water migrates from the unfrozen pores to the frozen cavities. The combination of pressures caused by ice accretion and osmosis causes the paste to crack.

Since the introduction of air-entrained concrete, the incidence of scaling has been reduced to minimal proportions (*18*). Heavy and severe scaling, where it occurs, may be corrected by using a thin epoxy mortar patch to waterproof the area and prevent penetration of water to the reinforcing steel.

Membership Functions for Scaling

Scaling is described qualitatively in terms of its depth, as reported in a number of studies (*2, 15, 16, 18*). A classification of scaling as a function of its depth, which was reported in a cooperative study by the Bureau of Public Roads and the Portland Cement Association (*18*), is given as follows:

Description	*Depth*
Light	0.0 to 0.25 in.
Medium	0.25 to 0.50 in.
Heavy	0.50 to 1.0 in.
Severe	More than 1.0 in.

Membership functions describing the various structural condition states caused by scaling were developed using the values in this scaling classification table as general guidelines. Because the average depth of medium-to-heavy scaling is approximately 0.5 to 0.6 in., the structural condition that corresponds to scaling having a depth of between 0.5 to 0.6 in. can be described using the linguistic variable "fair." On the other hand, the structural condition for scaling with a depth in excess of 1 in. can be described as very poor. The poor classification is, of course, between the fair and very poor range. Similarly, the structural condition for scaling less than 0.25 in. in depth is classified as very good. The classification "good" falls between the classifications "very good" and "fair." The shape, position, and magnitude of the membership functions for structural inadequacy when scaling is present are shown graphically in Figure 4.

Characteristics of Spalling

Spalling is the breaking loose of pieces of concrete, and often occurs initially near the top reinforcing steel (*9, 16, 18*). Spalling results from large tensile stresses within the concrete that are usually caused by corrosion of reinforcing bars and freezing of the concrete member. The products of corrosion exert stresses within the concrete that cannot be supported by the limited plastic deformation of the concrete, thereby causing the concrete to disintegrate. Also, when a structural member is frozen, separation of cement and the reinforcing bars can occur and lead to the formation of cracks and spalls.

Membership Functions for Spalling

The classification of spalling, as reported in the U.S. Army Corps of Engineers bridge inspection brochure (*16*) is given as follows:

Size	*Description*
Small	A roughly circular or oval depression no more than 1 in. deep and 6 in. in diam.
Large	A roughly circular or oval depression more than 1 in. deep and 6 in. in diam.

The membership functions describing the various structural condition states for spalling are expressed as a function of the spalling width, as shown in Figure 5.

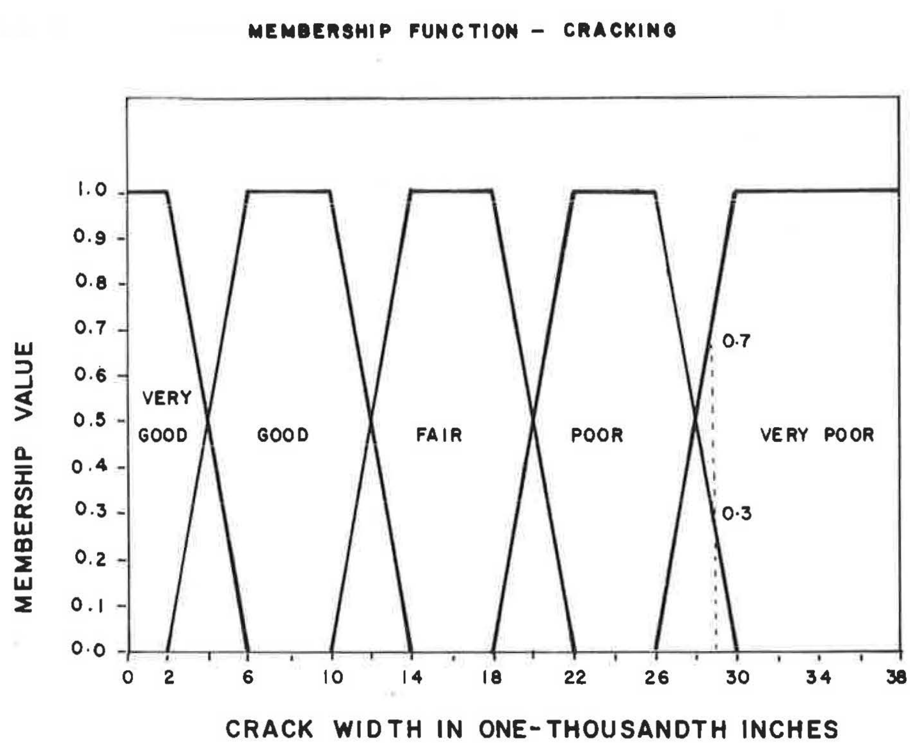

FIGURE 3 Structural condition membership functions for concrete cracks.

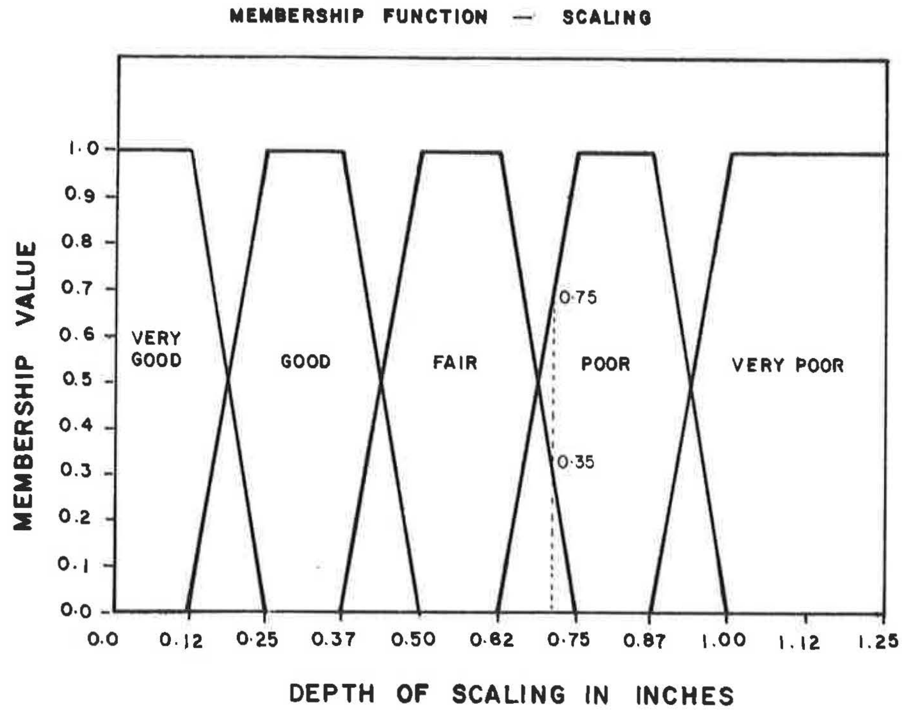

FIGURE 4 Structural condition membership functions for concrete scaling.

It can be noted in the spalling classification table that 6 in. is the dividing width between small and large spalls. In other words, the structural condition of concrete members with spall widths of 6 in. can be described as fair. The other two extremes, the very good and very poor structural condition classifications, were assumed to correspond to spall widths of less than 2 in. and greater than 10 in., respectively. Similarly, the intermediate stages, the good and the poor classifications, fall between the very good and fair, and fair to very poor classifications, respectively. Obviously, considerable extrapolation of the results in the spalling classification table was used; additional information and expert opinion on spalling can be used to improve these membership functions.

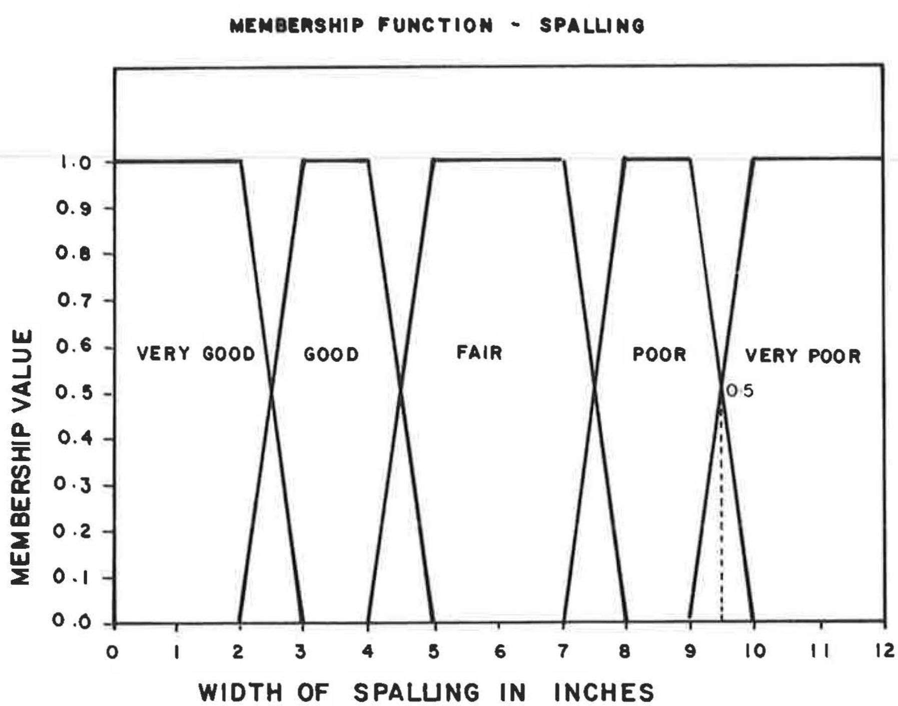

FIGURE 5 Structural condition membership functions for concrete spalling.

ILLUSTRATIVE EXAMPLE

In general, the condition of a reinforced concrete deck can be reasonably estimated by evaluating the severity of various imperfections such as corrosion of the reinforcing bars, and cracking, spalling, or scaling of the concrete and assessing their combined seriousness. The combined effect of these deteriorations on the deck is generally difficult to assess objectively and consistently. However, with the development of fuzzy logic, there is now a method to handle this problem.

As an illustrative example to demonstrate the use of fuzzy logic as a potential tool in modeling the interaction between concrete quality and corrosion, the following imperfections are assumed present on a concrete bridge deck:

Concrete quality: Average crack width = 0.029 in., Average scaling depth = 0.70 in., and Average spalling width = 9.50 in.
Steel quality: Average degree of corrosion = 45 percent.

The overall reinforced concrete quality in this hypothetical example can be modeled based on a combination of the characteristics for the flaws. The method for combining the flaw evaluations is to examine a range in the characteristics that corresponds to either no interaction or complete interaction of the flaws. This fuzzy logic method has been suggested for evaluating metals fatigue by Bowman et al.*(20)*.

Let *A*, *B*, *C*, and *D* stand for the fuzzy sets representing cracks, scaling, spalling, and corrosion, respectively. The effect of each imperfection acting separately is obtained by the union of fuzzy sets *A*, *B*, *C* and *D*, and the effect of all flaws acting jointly is given by the algebraic sum of *A*, *B*, *C*, and *D*. Thus, the grade of membership in a particular structural quality level, represented by fuzzy set *E*, can be evaluated as follows:

$$\mu_{A \cup B \cup C \cup D} \leq \mu_E \leq \mu_{A+B+C+D}$$

where μ_A, μ_B, μ_C, μ_D, and μ_E are the grades of membership in fuzzy sets *A*, *B*, *C*, *D*, and *E*, respectively.

By using the corrosion and concrete quality parameters in conjunction with the membership functions, the overall deck condition resulting from the combined effect of all parameters can be obtained. To illustrate this procedure, consider the "very poor" structural condition. From Figure 3 it can be observed that a 0.70 membership grade for "very poor" structural condition is indicated for a 0.029-in. crack width. (Note that the same 0.029-in. crack width gives a 0.30 membership grade for "poor" structural condition and a 0.0 membership grade for "fair," "good," and "very good" structural conditions.) Proceeding in this manner, the "very poor" condition can be evaluated for all of the imperfection severities as follows:

$$\mu_{vp}(A) = 0.70$$

$$\mu_{vp}(B) = 0.00$$

$$\mu_{vp}(C) = 0.50$$

$$\mu_{vp}(D) = 0.50$$

Using these values, the very poor structural condition evaluation can be bounded as follows:

$$\begin{aligned}\mu_{A \cup B \cup C \cup D} &= \max[\mu_{vp}(A), \mu_{vp}(B), \mu_{vp}(C), \mu_{vp}(D)] \\ &= \max[0.7, 0, 0.5, 0.5] = 0.7 \\ \mu_{A+B+C+D} &= 1 - [1 - \mu_{vp}(A)][1 - \mu_{vp}(B)] \\ &\quad \times [1 - \mu_{vp}(C)][1 - \mu_{vp}(D)] \\ &= 1 - (1 - 0.7)(1 - 0)(1 - 0.5)(1 - 0.5) \\ &= 0.925\end{aligned}$$

Consequently, the membership value for very poor structural condition classification falls in the range:

$$0.7 < \mu_{vp}(E) < 0.925$$

The lower and upper bound in this range can be viewed as the degree of "belief" that the overall structural condition is very poor when the effects of flaws are acting separately and when they are acting jointly, respectively.

The same procedure is repeated to define the upper and lower limits of the remaining structural condition classifications. The grades of membership for fuzzy set *A*, *B*, *C*, and *D*, and the upper and lower limits of fuzzy set *E* are shown in Table 3. Based on the tabulated results, it can be concluded that the strongest membership for the condition of the reinforced concrete deck caused by the combined effect of the various flaws is most closely associated with the "poor" classification.

It should be noted that in this fictitious example it is assumed that each flaw has equal impact or importance

TABLE 3 STRUCTURAL CONDITION GRADES OF MEMBERSHIP

Membership	Cracks 0.029 in.	Scaling 0.70 in.	Spalling 9.5 in.	Corrosion 45%	Union	Algebraic Sum
Very good	0	0	0	0	0	0
Good	0	0	0	0	0	0
Fair	0	0.35	0	0	0.35	0.35
Poor	0.3	0.75	0.5	0.5	0.75	0.956
Very poor	0.7	0	0.5	0.5	0.7	0.925

when computing the overall deck condition. However, in reality, this may not be true because certain flaws may be more important than others. If a flaw were to influence the overall deck condition differently, then an importance coefficient denoted as alpha (α), a number between 0.0 and 1.0, must be assigned to the flaw to reflect its influence on the overall structural integrity. For example, a small alpha value would be assigned to the flaw that is relatively unimportant. Conversely, for "important" flaws, their alpha values would be nearly equal to 1. If all flaws are to have equal importance (as in the example given), then the alpha values would be equal to 1.

The alpha value merely modifies the grades of the membership of a flaw. The alpha value of each flaw may be obtained through expert opinion survey or from structural analyses. The remaining computational steps will still be the same as in the algorithm previously mentioned. The limitation here is that the alpha value cannot be a fuzzy number (a number described by a fuzzy set).

The proposed approach selects the condition classification that has the highest membership range as the overall condition rating. The remaining condition classifications, which have lower membership ranges, were ignored. In some instances, this approach may not yield satisfactory results. For example, if the severity of corrosion shown in Table 3 were to be 5 percent instead of 45 percent, then the strongest membership for the overall condition of the bridge deck would be associated with the "very good" classification, even though the characteristics of all other flaws remain the same. (Note that 5 percent corrosion level gives a membership value of 1 for the "very good" condition classification and a 0.0 membership grade for the rest of the condition classifications.)

A more desirable approach would be to consider the membership limits of all condition classifications in determining the final condition rating. The lower and upper membership limits in Table 3 can be graphically represented as intervals, as shown in Figure 6. These intervals can be further depicted in the form of a modified histogram with unit cells, as shown in the same figure. The resultant condition classification can thus be obtained by computing the first central moment of area of this histogram. Using this approach, the final condition assessment resulting from the combined effect of the various flaws is found to be closely associated with the "poor" classification.

The major limitation of this approach is that it is not suitable for computation by hand. However, with the advent of the computer age, this limitation should not prevent the application of this approach to real-world inspection problems.

CONCLUSIONS

The procedure for rating an existing bridge structure requires a careful evaluation of many complex and often conflicting factors. Such evaluation is frequently based on the personal judgment, intuition, and perhaps experience of each inspector. As a result, different inspectors may assess a given bridge differently. Hence, a logical assessment procedure capable of incorporating both objective knowledge and engineering judgment systematically would be desirable. The theory of fuzzy mathematics offers a technique that can be employed to formulate such an assessment procedure.

A number of factors are known to affect the overall quality of a reinforced concrete member. The factors reported herein are cracking, scaling, and spalling of concrete and corrosion of reinforcing steel. Because of imprecise knowledge concerning the severity of these imperfections, linguistic instead of numerical rating variables are used to describe their condition. Successful use of this

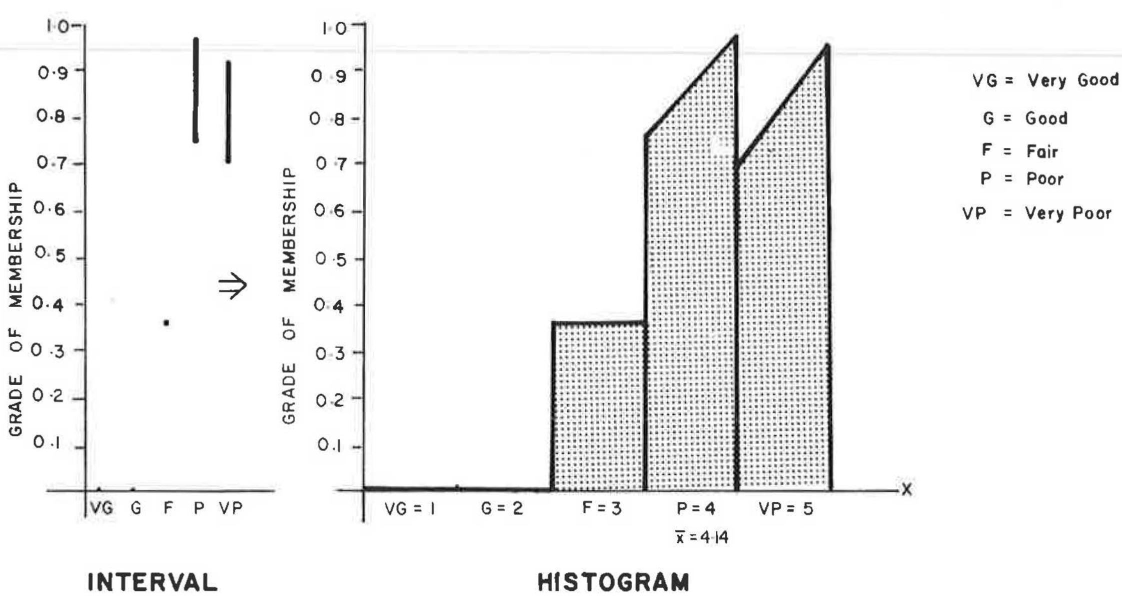

FIGURE 6 Condition classification histogram.

procedure depends on the development of adequate membership functions. The membership functions presented herein are developed on the basis of information extracted from the literature and structural analysis. They can also be formulated or improved through expert opinion.

ACKNOWLEDGMENT

The authors sincerely appreciate the financial support given by the Joint Highway Research Project of Purdue University, the Indiana Department of Highways, and the Federal Highway Administration.

REFERENCES

1. Robert G. Bartlett. What Direction In Bridge Building? In *Special Report 132: Systems Building for Bridges.* HRB, National Academy of Sciences, Washington, D.C., 1972, pp. 3–5.
2. *Bridge Inspector's Training Manual.* Federal Highway Administration, U.S. Department of Transportation, Washington, D.C., 1970.
3. *AASHTO Manual for Bridge Maintenance.* American Association of State Highway and Transportation Officials, Washington, D.C. 1976.
4. J. L. A. Chameau, A. Altschaeffl, H. L. Michael, and J. T. P. Yao. Potential Applications of Fuzzy Sets in Civil Engineering. *Journal of Man-Machine Studies,* Vol. 19, 1983, pp. 9–18.
5. L. A. Zadeh. *Fuzzy Sets. Information and Control.* Vol.8, Academic Press, New York, 1965, pp. 338–353.
6. J. E. Slater. *Corrosion of Metals in Association with Concrete.* ASTM 818, American Society for Testing and Materials, Philadelphia, Pa., 1983.
7. *Symposium on Bond and Crack Formation in Reinforced Concrete.* Réunion Internationale des Laboratoires d'Essais et de Recherches sur les Matériaux et les Constructions (RILEM), Paris, France, 1957.
8. *Guide to Durable Concrete.* American Concrete Institute, ACI Committee 201, Vol.74, American Concrete Institute, Detroit, Mich., 1977, pp. 573–609.
9. *Analysis and Design of Reinforced Concrete Bridge Structures.* ACI Committee 443 Report, American Concrete Institute, Detroit, Mich., 1977.
10. A. B. Tee, and M. D. Bowman. *Corrosion Assessment of Reinforcing Steel in Bridge Decks.* Technical Report No. CE-STR-86-25, School of Civil Engineering, Purdue University, West Lafayette, Ind. 1986.
11. *Cracking, Deflection, and Ultimate Load of Concrete Slab Systems.* ACI Publication SP-30, American Concrete Institute, Detroit, Mich. 1971, pp. 55–75.
12. P. G. Manning, and D. H. Bye. *Bridge Deck Rehabilitation Manual, Part One: Condition Surveys.* Ontario Ministry of Transportation and Communications, Downsview, Ontario, Canada, 1983.
13. B. Bresler. *Reinforced Concrete Engineering.* Vol. I, John Wiley and Sons, Inc., New York, 1974.
14. J. Ryell and B. S. Richardson. *Cracks in Concrete Bridge Decks and Their Contribution to Corrosion of Reinforcing Steel and Prestressing Cables.* IR51, Ontario Ministry of Transport and Communications, Downsview, Ontario, Canada, 1972.
15. *Bridge Inspector Training Course - Inspection Checklists.* U.S. Department of Transportation, Washington, D.C., 1971.
16. *Bridge Inspection Brochure.* Facilities Engineering Support Agency, U.S. Army Corps of Engineers, May 1983.
17. *NCHRP Synthesis of Highway Practice 57: Durability of Concrete Bridge Decks.* TRB, National Research Council, Washington, D.C., May 1979.
18. *Durability of Concrete Bridge Decks. A Cooperative Study of Ten States.* Report 1-6, Bureau of Public Roads, Washington, D.C., and Portland Cement Association, Skokie, Ill., 1965–1970.
19. *Strength and Serviceability Criteria, Reinforced Concrete Bridges, Ultimate Design.* Bureau of Public Roads, Washington, D.C., Oct. 1969.
20. M. D. Bowman, G. E. Nordmark, and J. T. P. Yao. Fuzzy Logic Approach in Metals Fatigue. *International Journal of Approximate Reasoning,* Vol. 1, 1987, pp. 197–219.

Publication of this paper sponsored by Committee on Structures Maintenance.

Cost-Effective Bridge Maintenance and Rehabilitation Procedures

RICHARD E. WEYERS, PHILIP D. CADY, AND JOHN M. HUNTER

Twenty bridge maintenance and rehabilitation areas were identified and repair procedures were compiled for each area. Initial cost and life for each procedure was determined by expert opinion expressed during a group encounter session, for which guidelines and procedures are presented. An engineering economic evaluation of the alternative procedures for each of the 20 bridge maintenance and rehabilitation areas was performed considering the time value of money, sensitivity of least cost parameters, and economic intangibles. An economic decision tree presents the least cost solution to the identified bridge maintenance and rehabilitation areas for various field conditions.

The deterioration of the highway transportation system is a national trend. As a result, highway maintenance expenditures are increasing at a rate of $300 million per year (*1, 2*). By 1990, maintenance could account for more than one-half of all highway expenditures. The lack of sufficient funds being allocated to bridge maintenance and rehabilitation, coupled with past revenue crunches related to the fuel crisis and recessionary periods, has resulted in a large backlog of bridge maintenance and betterment needs. Reflective of this national trend is Pennsylvania's bridge problem.

Pennsylvania currently has approximately 22,500 bridges longer than 20 ft. Thirty-five percent of these bridges are classified as structurally deficient or functionally obsolete (*3*). The estimated improvement cost to bring the presently classified structurally deficient or functionally obsolete bridges up to a minimum acceptable condition is $2.5 billion. Pennsylvania's reaction to the bridge problem was the enactment of Billion Dollar Bridge Programs I and II and the development of a bridge management system (*4*). The latest program, Billion Dollar Bridge II (Act 100), was signed into law on July 9, 1986. The act identified approximately 3,300 bridges for replacement and rehabilitation over a 10- to 12-yr period at a total cost of $1.6 billion.

The bridge management system (BMS), which was phased into service from December 24, 1986, to April 30, 1987, contains an enhanced structural inventory record system (SIRS), a bridge replacement and rehabilitation system that is able to determine present needs and project future conditions, a bridge maintenance system for present and future needs, and an integrator that links the BMS with other Pennsylvania Department of Transportation (PennDOT) computer management systems. The objective of the BMS is to make the best use of available funds in an overall program of maintenance, rehabilitation, and replacement, while keeping the bridge system operating at the demand level of service and ensuring public safety. Thus, there is an urgent need to identify cost-effective bridge maintenance and rehabilitation procedures. Recognizing this, PennDOT instituted Research Project 84-11, "Cost-Effectiveness of Bridge Repair Details and Procedures," on September 21, 1985, part of which is reported here. The objectives of the study (*5, 6*) were to

1. Identify approximately 20 common bridge maintenance and rehabilitation problem areas,
2. Compile procedures used to address the identified areas, and
3. Determine the least-common cost solution to the 20 identified bridge maintenance and rehabilitation problem areas.

IDENTIFICATION

Pennsylvania's 11 engineering districts were visited and each district bridge engineer and bridge maintenance coordinator was interviewed. During the interviews, common bridge substructure, superstructure, deck, and appurtenance problem areas were identified and selected sites were visited and photographed. The results of the 11 interviews were compiled and a frequency of occurrence number (1 to 11), potential cost savings (small, moderate, large, very large), and effect on safety (small, moderate, extreme) term was assigned to each problem area. The potential cost savings may result from employing a standard method rather than doing nothing at all or from selecting the most cost-effective solution. For candidates to be included in the final identification list, they had to meet the selection criteria of two of the conditions; that is, each final candidate presented in Table 1 meets the selection criteria of frequency of occurrence of five or greater and cost-savings

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TABLE 1 BRIDGE PROBLEM AREAS

Item No.	Bridge Element	Activity	Problem Cause
Substructure			
1	Pier	Deterioration of concrete	Corrosion of reinforcing steel (salt water from leaking joints)
2	Pier caps	Deterioration of concrete	Corrosion of reinforcing steel (salt water from leaking joints)
3	Back walls	Structural fracture of concrete	Pavement migration
4	Block covered slope walls	Erosion of soil	Deck drainage or failed drainage systems
5	Soil slope walls	Erosion of soil	Deck drainage or failed drainage systems
6	Hammerhead piers	Structural cracking	Inadequate design
7	Diaphragms	Deterioration of concrete or steel member	Corrosion of steel (salt water from leaking joints)
8	Prestressed box beam	Longitudinal cracking, breaking of strands	Moisture trapped in box beam, low cover, failure of shear keys
9	Prestressed beams	Spalling of concrete cover and breaking of strands	Collision damage
10	Steel beams	Fatigue cracking	Weld detail
11	Patching with asphaltic concrete	Spalling of concrete deck	Corrosion of reinforcing steel (deicer salts)
12	Patching with portland cement concrete deck	Spalling of concrete	Corrosion of reinforcing steel (deicer salts)
13	Deck overlay with latex-modified concrete	Spalling of original concrete deck	Corrosion of reinforcing steel (deicer salts)
14	Expansion joints all types	Leaking or failed	Improper design or construction methods
15	Deck replacement for steel superstructure	Deterioration of concrete deck	Corrosion of reinforcing steel (deicer salts)
16	Deck replacement for prestressed I-superstructure	Deterioration of concrete deck	Corrosion of reinforcing steel (deicer salts)
17	Deck replacement for prestressed adjacent box beam superstructure	Deterioration of concrete deck	Corrosion of reinforcing steel (deicer salts)
18	Deck replacement for prestressed spread box beam superstructure	Deterioration of concrete deck	Corrosion of reinforcing steel (deicer salts)
19	Drainage-scuppers	Clogged	Improper design
Appurtenances			
20	Parapet	Geometry	Improper design
21	Approach slab—all types	Horizontal and vertical movement	Improper design
22	Drainage-drainpipe	Clogged	Improper design
23	Stress relief joints	Rough riding surface	Pavement migration

potential large or very large, or frequency five or greater and effect on safety of moderate or extreme, or cost-saving potential large or very large and effect on safety of moderate or extreme. Presented in Table 1, in addition to problem areas (bridge element and activity), are the probable causes of the 23 Pennsylvania bridge candidate problem areas. The four most severe problem areas identified are decks, drainage, joints, and piers.

COMPILATION OF PROCEDURES

Six of the 11 Pennsylvania engineering districts compiled and submitted common practices and innovative maintenance and rehabilitation procedures for the 23 areas presented in Table 1. In addition, methods were compiled from the literature. Procedures were selected to maximize the number of alternative approaches to the solution of the identified problem areas. During the selection of alternative solutions, deterioration of concrete diaphragms was combined with concrete beams. Also, fatigue cracking of steel beams was excluded from the selection list because this item does not lend itself to an economic evaluation (repair details are site specific, and expediency is of primary concern because of public safety). Thus, a total of 21 bridge maintenance and rehabilitation areas were represented in the economic evaluation of 49 procedures. The repair alternatives for the 21 bridge-maintenance and rehabilitation activities are presented in Table 2.

TABLE 2 ALTERNATIVE SOLUTIONS FOR BRIDGE MAINTENANCE AND REHABILITATION ACTIVITIES

Bridge Element	Alternatives
Pier and pier cap	Repair, depth
	< 1 in.
	1 to 3 in.
	> 3 in.
	Replacement
Pier	Encasement
	Replacement
Structurally cracked hammerhead piers	Post-tension
	Epoxy grout reinforcement repair
Block or concrete slope walls	Replace deteriorated area with stone
	Completely replace with stone
Soil slope walls	Partial protection with stone
	Complete protection with stone
Back walls	Repair
Box beams	Repair
	Replace
Prestressed I-beams	Repair
	Replaced
Scuppers	New square box design
	Straight drop pipe
Expansion joints	Repair open armored
	Repair sliding plate
	Replace, < 2-in. movement with armored compression seal
	Replace, < 2-in. movement with unarmored compression seal
	Replace, 2- to 4-in. movement with neoprene strip seal
	Replace, > 4-in. movement with tooth joint without trough
	Replace, > 4-in. movement with tooth joint with trough
Decks	Patch
	Asphalt concrete (temporary)
	Type 1, HES concrete
	Type 1, Mg-phosphate concrete
	Type 1, polymer concrete
	Type 2, HES concrete
	Type 2, Mg-phosphate concrete
	Type 3, HES concrete
	Type 3, Mg-phosphate
	Overlay
	Asphalt concrete
	Latex-modified concrete
	Replacement
	Steel superstructure,
	cast-in-place
	precast
	Prestressed I-beams,
	cast-in-place
	precast
	Adjacent box beams,
	partial depth
	full depth
	Spread box beams,
	cast-in-place
Drainpipes	Remove and replace
Parapets	Modify existing
	Replace with precast
	Replace with cast-in-place
Approach slabs	Replace
Pavement relief joint	Install

COST-EFFECTIVE ANALYSIS

Cost-effectiveness can be achieved through a standardized methodology of comparison of all costs incurred over the service life of a structure considering the time value of money. This is the meaning of cost-effectiveness. Decisions based on initial costs or individual events will generally not result in a least cost solution. Cost-effective decision models for maintenance, rehabilitation, and replacement of bridges have been developed (*7*). The life-cycle models require the initial costs and service life of all bridge activities over the life of a structure. Before the phased implementation of the BMS, Pennsylvania engineering districts were not tracking the initial cost and service life of bridge-maintenance and rehabilitation activities (*5*). Because the BMS data-gathering system is part of the bridge-inspection program, it will take a 2-yr inspection cycle to complete the data base. Thus, a method to determine the cost and lives of bridge maintenance and rehabilitation procedures is needed.

Costs and Lives by Expert Opinion

Bridge experts from PennDOT staff were asked to participate in a group encounter session in which they would express their opinions on the initial cost and service life of various bridge maintenance and rehabilitation procedures. From the list of experts who were willing to participate in the group encounter session, 11 were chosen to represent the range of geographic, economic, climatic, and demographic conditions throughout Pennsylvania that affected bridges. The objective of the encounter session was to collect the opinions of individual experts that were free from influence exerted by a member or members of the group or by observers of the encounter session.

Guidelines developed to minimize member or observer influences included:

1. Encounter session should be conducted by an individual familiar with group dynamics;
2. Observers should be limited and should not be perceived as authorities on the subject;
3. Observers should not discuss activities or results nor interject their opinions during, before, or after the encounter session;
4. The observer's function is only to answer technical questions during the question period;
5. The encounter session must be structured and schedules should be maintained;
6. Group member input that may influence the work of the group should be considered and acted upon; and
7. A sense of accomplishment must be promoted among the participants.

Variability within a group of experts is expected; extreme variability limits the usefulness of global economic decisions. Therefore, cost and life were clearly defined to exclude

TABLE 3 CANDIDATE MAINTENANCE AND REHABILITATION PROCEDURES

Procedure	Description	Procedure	Description
1	Pier and pier cap patching: depth of repair < 1 in.	26	Bridge deck overlays: asphalt concrete
2	Pier and pier cap patching: depth of repair 1 to 3 in.	27	Bridge deck overlays: latex-modified concrete
3	Pier and pier cap patching: depth of repair > 3 in.	28	Repair of expansion joints (open armored)
4	Pier encasement	29	Repair of expansion joints (sliding plate)
5	Pier and pier cap replacement	30	Expansion joint replacement, replace with unarmored compression seal
6	Pier replacement	31	Expansion joint replacement, replace with armored compression seal
7	Back wall repair	32	Expansion joint replacement, replace with armored neoprene strip seal
8	Rehabilitation of block and concrete slope walls (partial repair with stone)	33	Expansion joint replacement, replace with tooth joint (without trough)
9	Rehabilitation of block and concrete slope walls (replacement with stone)	34	Expansion joint replacement, replace with tooth joint (with trough)
10	Soil slope wall protection, partial protection using stone	35	Replacement of deck on steel superstructure, cast-in-place concrete
11	Soil slope wall protection, full protection using stone	36	Replacement of deck on steel superstructure, precast deck
12	Post-tensioning of cracked hammerhead piers	37	Deck replacement, prestressed I-beam superstructure, cast-in-place deck
13	Epoxy-grouted reinforcement for repair of cracked hammerhead piers	38	Deck replacement, prestressed I-beam superstructure, precast deck
14	Repair of cracked prestressed box beams	39	Deck replacement, adjacent box beams, partial removal of existing deck, cast-in-place concrete
15	Replacement of longitudinally cracked box beams	40	Deck replacement, adjacent box beams, full deck removal, cast-in-place concrete
16	Prestressed I-beams, patch and restore prestressing	41	Deck replacement, spread box beam superstructure, full deck removal, cast-in-place concrete
17	Prestressed I-beams, replace	42	Drainage scuppers, replace with new square box design
18	Bridge deck patching: asphalt concrete (temporary)	43	Drainage scuppers, replace with straight drop pipe
19	Type 1 deck removal: Patch with high early-strength portland cement concrete	44	Remove and replace deck drain pipe
20	Type 1 deck removal: patch with rapid-setting magnesium phosphate concrete	45	Parapets, modify existing
21	Type 1 deck removal: patch with polymer concrete	46	Parapets, replace with precast units
22	Type 2 deck removal: patch with high early-strength portland cement concrete	47	Parapets, replace with cast-in-place units
23	Type 2 deck removal: patch with rapid-setting magnesium phosphate concrete	48	Approach slab replacement
24	Type 3 deck removal: patch with high early-strength portland cement concrete	49	Install pavement relief joints
25	Type 3 deck removal: patch with rapid-setting magnesium phosphate concrete		

any highly variable component. However, these highly variable components should be considered by a case-by-case comparison. Accordingly, the definitions of cost and life formulated for the group encounter session was as follows:

Cost: expressed as a single value per specified unit and consisting of labor, materials, equipment, and overhead (labor fringe rates; insurance; administration, including engineering; and inspection). Traffic control, profit, user costs, and economic impacts on the service area are all site specific and thus were not included in the definition of cost.

Service life: the period of time over which the maintenance activity is expected to be effective, assuming that appropriate modifications and repairs are made to the other bridge elements that contribute to the problem. (For example, in repairing piers damaged by the corrosion of reinforcing steel caused by a leaking expansion joint, it is assumed that the expansion joint would be repaired.)

Site difficulties were taken into consideration by formulating the cost and life questions to produce a range of costs (reasonably lowest, most frequent, reasonably highest) and service lives (reasonably shortest, most frequent, reasonably longest). In addition, intangible economic factors were considered by asking the experts to rate the procedure, in words, as poor, good, very good, or excellent, and using a number rating of 1 to 10 to define their meaning of poor, good, very good, and excellent.

The type of information presented for each procedure was also considered. The information has to be of such detail that the experts can reasonably estimate the cost and life of a general application of the procedure rather than a specific application. The descriptions and procedure numbers for the 49 candidate maintenance and rehabilitation procedures that were evaluated in the 2-day group encounter session are presented in Table 3.

Data Reduction

The nature of the data obtained from the encounter session, largely opinion, would be expected to be highly var-

TABLE 4 DATA REDUCTION SUMMARY

Procedure No.	Letter Rating[a]	No. Rating: Value	No. Rating: Coefficients of Variation	No. Rating: Points Deleted	Most Frequent Life (yr): Value	Most Frequent Life (yr): Coefficients of Variation	Most Frequent Life (yr): Points Deleted	Most Frequent Cost: Value	Most Frequent Cost: Coefficients of Variation	Most Frequent Cost: Units	Most Frequent Cost: Points Deleted
1	1.9	4.9	51	0	9.8	77	0	30.0	83	\$/ft^2	0
2	2.3	6.5	20	1	13.0	52	0	39.1	30	\$/ft^2	0
3	2.7	6.6	27	0	18.0	30	0	52.2	15	\$/ft^2	1
4	2.7	6.7	24	0	23.9	21	1	232.0	33	\$/yd^3	0
5	3.4	7.9	25	0	42.0	33	0	1,155.0	54	\$/yd^3	0
6	1.5	3.7	84	0	36.0	37	0	1,244.4	59	\$/yd^3	0
7	2.5	6.5	37	0	22.7	44	0	702.5	42	\$/yd^3	0
8	2.1	4.6	60	0	18.3	69	0	120.7	48	\$/yd^3	1
9	2.7	7.1	23	0	28.5	48	0	113.3	28	\$/yd^3	1
10	2.2	5.4	26	0	20.1	50	0	173.5	26	\$/yd^3	0
11	2.5	7.3	12	1	27.7	45	0	164.5	28	\$/yd^3	0
12	2.4	6.4	28	0	25.2	42	0	28,530.0	75	\$/pier	0
13	1.8	4.1	66	0	20.8	51	0	23,339.0	66	\$/pier	1
14	2.2	5.0	54	0	16.5	39	0	35.3	42	\$/ft	2
15	3.2	8.4	11	0	44.1	20	0	390.0	53	\$/ft	1
16	2.0	4.8	58	0	19.9	44	0	14,845.0	57	\$/beam	0
17	3.3	8.5	9	1	42.8	8	2	727.9	61	\$/ft	0
18	1.1	2.4	108	0	0.1	90	2	7.28	134	\$/ft^2	1
19	1.8	4.4	61	0	4.3	51	0	16.6	43	\$/ft^2	0
20	1.7	3.7	65	0	3.8	61	2	23.7	49	\$/ft^2	0
21	1.7	3.7	68	0	5.5	78	0	24.9	51	\$/ft^2	1
22	2.5	5.6	38	0	7.5	37	1	26.7	42	\$/ft^2	0
23	2.0	4.2	45	0	6.8	40	1	33.9	55	\$/ft^2	0
24	2.8	7.0	26	0	16.1	48	0	52.9	34	\$/ft^2	0
25	2.0	3.9	77	0	12.5	69	0	69.2	36	\$/ft^2	0
26	1.5	4.3	61	0	3.9	39	0	4.1	54	\$/ft^2	0
27	2.5	7.1	10	2	13.6	52	1	29.0	82	\$/ft^2	0
28	1.4	3.2	72	0	3.9	74	0	71.4	63	\$/ft^2	1
29	1.4	3.9	74	0	3.5	46	2	70.5	46	\$/ft^2	1
30	2.3	6.0	33	0	15.7	46	0	198.6	69	\$/ft^2	0
31	3.1	7.9	13	0	24.5	29	0	299.5	49	\$/ft	0
32	3.0	7.7	16	0	22.7	27	0	326.8	50	\$/ft	0
33	2.5	5.7	42	0	26.7	9	2	604.5	45	\$/ft	1
34	3.3	8.3	13	0	26.5	20	1	608.0	53	\$/ft	1
35	3.6	9.0	7	0	35.0	22	0	42.9	49	\$/ft^2	0
36	2.4	5.6	25	0	26.1	36	0	82.9	72	\$/ft^2	0
37	3.7	9.1	4	1	35.9	30	0	52.1	45	\$/ft^2	0
38	2.5	6.2	32	0	24.5	22	1	88.9	70	\$/ft^2	0
39	2.1	5.4	52	0	23.0	24	1	32.2	31	\$/ft^2	2
40	3.2	8.1	17	0	30.5	24	0	48.0	55	\$/ft^2	0
41	3.5	9.0	6	1	30.5	24	0	54.0	47	\$/ft^2	0
42	2.4	5.6	57	0	32.7	26	0	1,295.0	44	\$/each	1
43	2.3	5.5	47	0	27.9	32	0	430.9	57	\$/each	0
44	2.5	6.4	42	0	22.5	36	0	37.3	58	\$/ft	0
45	2.4	6.0	42	0	30.3	36	0	114.8	33	\$/ft	0
46	1.9	4.3	51	0	24.5	23	0	96.8	40	\$/ft	0
47	3.2	7.2	42	0	34.5	22	0	103.6	35	\$/ft	0
48	3.4	8.2	15	0	23.1	20	0	15.4	27	\$/ft^2	2
49	3.2	8.4	8	1	19.7	37	0	98.5	42	\$/ft	1

[a]Meanings of the word rating were poor (A) = 1, good (B) = 2, very good (C) = 3, and excellent (D) = 4.

iable. Furthermore, it would be expected to be highly subject to "outliers," or data points that are obvious errors relative to the mainstream of the group. The primary potential sources of such errors are deliberate instances of inclusion of data that are too high or too low in an effort to favor or disfavor a particular procedure (bias) and mistakes based on misinterpretation of the procedures being evaluated. The outlier elimination procedure described in the *National Bureau of Standards Handbook 91* was used (*8*). The 95 percent confidence level was used to eliminate outliers. Outlier elimination was not applied to the word rating because that type of data is not readily amenable to the process.

The results of the data reduction for the most frequent life, most frequent cost, and work and number ratings are presented in Table 4. The values shown are the arithmetic means for the encounter group after the elimination of outliers. The numerical values for word ratings are based on poor (A) = 1, good (B) = 2, very good (C) = 3, and excellent (D) = 4, to show a relative mean position (i.e., 1.5 is equivalent to a poor-to-good rating). Also shown in Table 4 are the coefficients of variation, indicating the degree of variability of the data, and the number of outliers eliminated in each instance.

Comparison with Data from Another Source

Cost data obtained from the Pennsylvania Bureau of Design, Contract Management Division (R. Harley, personal communication, September 9, 1986), are compared with the means of the encounter session cost data for 20 of the 49 procedures (see Table 5). More than half (55 percent) of the Contract Management Division's figures do not fall within the ranges obtained from the conference session. Most (70 percent) of the values from the Contract Management Division fall below the mean "most frequent" values obtained in the encounter session. Note that the most widely disparate results invariably involve lower costs from the Contract Management Division data on items that require considerable engineering (pier, beam, and deck replacement items). The explanation of these differences most certainly lies in the definition of cost used in the encounter session, which specified the inclusion of engineering and inspection costs. It was verified with the Contract Management Division (R. Harley, personal communication, September 25, 1986) that their cost figures do not include these items. It is believed, therefore, that the cost figures generated by the encounter session are reasonably representative of actual costs.

TABLE 5 COMPARISON OF COST DATA FROM ENCOUNTER SESSIONS WITH COSTS FROM PENNDOT CONTRACT MANAGEMENT DIVISION

Procedure No.	Mean Cost Values from Encounter Sessions ($) Lowest	Highest	Most Frequent	Data from Contract Management Division ($)	Units
1	16.6	65.2	30.0		ft^2
2	24.5	61.1	39.2	46.88[a]	ft^2
3	33.6	88.0	52.2		ft^2
5	932.5	1,670.0	1,155.0	482.44[b]	yd^3
6	933.3	1,855.6	1,244.4	482.44[b]	yd^3
7	458.3	852.8	702.5	600 to 1,000[c]	yd^3
17	517.3	733.3	727.9	193 to 368[d]	ft
18	3.71	11.20	7.28	2.91[e]	ft^2
19	10.6	30.1	16.6	27.28	ft^2
22	16.5	39.8	26.7	21.40	ft^2
24	36.5	87.0	52.9	46.41	ft^2
26	3.0	7.4	4.1	1.00[f]	ft^2
27	24.4	36.7	29.0	5.09 to 5.62[g]	ft^2
35	33.9	59.5	42.9	23.73 to 24.73[h]	ft^2
37	32.4	74.5	52.1	24.23 to 25.73[i]	ft^2
40	37.1	64.0	48.0	20.20 to 21.70[i]	ft^2
41	40.6	73.2	54.0	24.23 to 25.73[j]	ft^2
47	81.9	140.5	103.6	68.52	ft
48	13.9	26.0	15.4	16.49 to 16.80[f]	ft^2
49	78.5	125.0	98.5	110.7	ft

[a]Depth not defined.
[b]Class AA concrete—large work area.
[c]Class AA concrete—small work area.
[d]Based on beam at $175 to $350/ft and 3 ft^2 of deck removal (partial) ft of beam at $6/ft^2$.
[e]Based on $155.34/ton and 3-in. average patch depth.
[f]Based on $3.99/$yd^2$ ($80/ton for 2-in.-thick overlay).
[g]1¼- to 1½-in. depth, including scarification.
[h]Includes $4.00 to $5.00/$ft^2$ for deck removal.
[i]Includes $4.50 to $6.00/$ft^2$ for deck removal.
[j]Includes $40.00 to $50.00/$yd^3$ for slab removal, assuming 10-in. thick slab.

Economic Analysis

In practice, cost-effective analyses of periodic bridge maintenance and rehabilitation events should be performed on a case-by-case basis in an overall bridge maintenance, rehabilitation, and replacement program. The periodicity of bridge maintenance and rehabilitation procedures presented in the paper is generally undefined and is limited in number. These constraints precluded the effort required for an overall economic evaluation. The chosen economic engineering method is based on selecting the least equivalent uniform annual cost (EUAC) alternative. The EUAC for the 49 procedures was calculated from the following equation using the mean most frequent cost and life.

$$\text{EUAC} = P(A/P, i, n) \quad (1)$$

TABLE 6 EQUIVALENT UNIFORM ANNUAL COSTS

Procedure No.	Life (yr)	$(A/P, i, n)$	First Cost ($)	EUAC (yr^{-1}) ($)
1	9.8	0.13155	30.0/ft^2	3.95/ft^2
2	13.0	0.10646	39.2/ft^2	4.17/ft^2
3	18.0	0.08555	52.2/ft^2	4.47/ft^2
4	23.9	0.07263	232/yd^3	16.95/yd^3
5	42.0	0.05739	1,155.0/yd^3	66.29/yd^3
6	36.0	0.06043	1,244.4/yd^3	75.20/yd^3
7	22.7	0.07467	702.5/yd^3	52.46/yd^3
8	18.3	0.08467	120.7/yd^3	10.22/yd^3
9	28.5	0.06657	133.3/yd^3	7.54/yd^3
10	20.1	0.08001	173.5/yd^3	13.88/yd^3
11	27.7	0.06746	164.5/yd^3	11.10/yd^3
12	25.2	0.07066	28,530/pier	2,015.93/pier
13	20.8	0.07843	23,339/pier	1,830.48/pier
14	16.5	0.09043	35.3/ft	3.19/ft
15	44.1	0.05658	390.0/ft	22.07/ft
16	19.9	0.08048	14,845/beam	1,194.72/beam
17	42.8	0.05707	727.9/ft	41.54/ft
18	0.1	10.27495	7.28/ft^2	74.79/ft^2
19	4.3	0.26419	16.6/ft^2	4.39/ft^2
20	3.8	0.29545	23.7/ft^2	7.00/ft^2
21	5.5	0.21244	24.9/ft^2	5.29/ft^2
22	7.5	0.16316	26.7/ft^2	4.36/ft^2
23	6.8	0.17708	33.9/ft^2	6.00/ft^2
24	16.1	0.09189	52.9/ft^2	4.86/ft^2
25	12.5	0.10951	69.2/ft^2	7.58/ft^2
26	3.9	0.28855	4.1/ft^2	1.18/ft^2
27	13.6	0.10310	29.0/ft^2	2.99/ft^2
28	3.9	0.28855	71.4/ft	20.60/ft
29	3.5	0.31850	70.5/ft	22.45/ft
30	15.7	0.09343	198.6/ft	18.56/ft
31	24.5	0.07169	229.5/ft	16.45/ft
32	22.7	0.07467	326.8/ft	24.40/ft
33	26.7	0.06866	604.5/ft	41.51/ft
34	26.5	0.06891	608.0/ft	41.90/ft
35	35.0	0.06107	42.9/ft^2	2.62/ft^2
36	26.1	0.06943	82.9/ft^2	5.76/ft^2
37	35.9	0.06050	52.1/ft^2	3.15/ft^2
38	24.5	0.07169	88.9/ft^2	6.37/ft^2
39	23.0	0.07414	32.2/ft^2	2.39/ft^2
40	30.5	0.06458	48.0/ft^2	3.10/ft^2
41	30.5	0.06458	54.0/ft^2	3.49/ft^2
42	32.7	0.06272	1,295.0/each	81.22/each
43	29.7	0.06723	430.9/each	28.97/each
44	22.5	0.07503	37.3/ft	2.80/ft
45	30.3	0.06477	114.8/ft	7.44/ft
46	24.5	0.07169	98.6/ft	6.94/ft
47	34.5	0.06141	103.6/ft	6.36/ft
48	23.1	0.07396	15.4/ft^2	1.14/ft^2
49	19.7	0.08496	98.5/ft	7.97/ft

where

P = most frequent first cost,
$(A/P, i, n)$ = capital recovery factor,
i = interest rate (in decimal form), and
n = most frequent life (yr).

An interest rate of 5 percent was based on the observation that the true (inflation-adjusted) time value of money is 4 to 6 percent on a long-term basis (*9*). The computer EUAC values are presented in Table 6.

In addition to the selections based on EUAC, the ratings of the alternatives were examined to take intangibles into account. In order to incorporate both the word and number ratings into this process, these two variables were first subjected to linear regression analysis to establish the relationship between them. The results are shown in Figure 1. It was decided, a priori, that the cutoff period should lie midway between a word rating of "poor" (A) = 1 and "good" (B) = 2, or at a word rating of 1.5. From the regression line in Figure 1 this results in a number rating cut-off value of between 3.3 and 3.4. Therefore, all maintenance procedures with a number rating of less than 3.4 were considered unacceptable. Examination of Table 6 reveals that only two procedures were eliminated by this process: Number 18 (asphalt patching of bridge decks) and Number 28 (repair of open armored expansion joints). Both of these were also eliminated in the economic analyses. A decision matrix summarizing the selected procedures, based on the economic analysis, is shown in Figure 2.

Sensitivity Analysis

The economic analysis presented is based on the most probable cost and life values. However, the data for cost and life are, not unexpectedly, highly variable for most of the alternative strategies. Thus, the effects of variability or sensitivity analysis were performed on the economic decisions rendered. The sensitivity analysis was performed using a procedure sometimes called minimin-maximax in the technical literature. The objective function (EUAC) used in the economic decision making is the product of the initial cost and the capital recovery factor that, in turn, is a function of the life of the alternative. It is intuitively clear that low initial costs or long service lives will lead to low annual costs. Likewise, high initial costs or short lives will lead to high annual costs. Therefore, the combination of the shortest life with the highest first cost gives the highest annual cost (maximax), while the combination of the longest life with the lowest first cost gives the lowest annual cost (minimin). Because the individual elements (high and low cost, long and short life) are, in themselves, extreme values (i.e., low probability of occurrence), their products in maximum or minimum represent values having an infinitesimal probability of occurrence. They do, however, define ranges of values that are representative of the sensitivity of equivalent uniform annual cost to expected variability in first cost and service life.

Because the minimin and maximax values represent extremes of very low probability of occurrence, it is generally not appropriate to use them for evaluating economic decisions rendered on the basis of most probable values. Rather, the most probable value within each minimin-maximax range should be used, midpoint range (arithmetic mean) being the most logical choice. If the mean values for the maximin-minimax ranges are then substituted for the most frequent values in the economic calculations, the effect of sensitivity on the economic decisions becomes evident. The results are summarized in Table 7. In general, the decisions rendered in the economic analysis are not significantly affected by the expected variations in first cost and service life. Notice that even in those few instances that show a different decision (break-even point and, in one case, procedure), no changes should be made in the decision matrix developed using "most frequent" values (see Figure 2). Rather, the sensitivity analysis results merely flag those items that display tendencies to be sensitive to the variability of the input data.

CONCLUSIONS AND DISCUSSION

Bridge maintenance and rehabilitation problem areas and least cost repair solutions have been identified. In addition, a method to determine costs and lives of bridge activities using expert opinion has been developed. It must be rec-

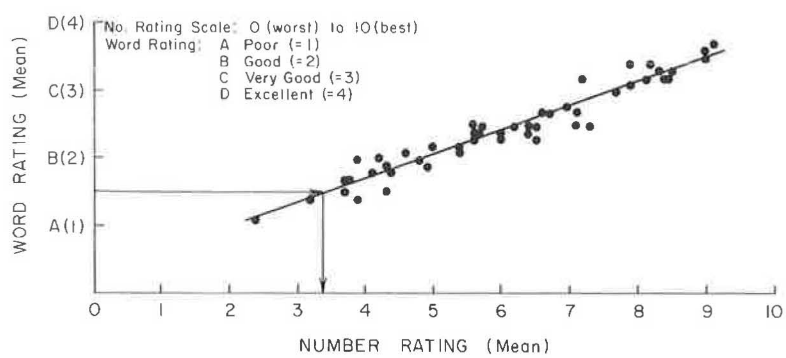

FIGURE 1 Correlation of word rating and numbering system.

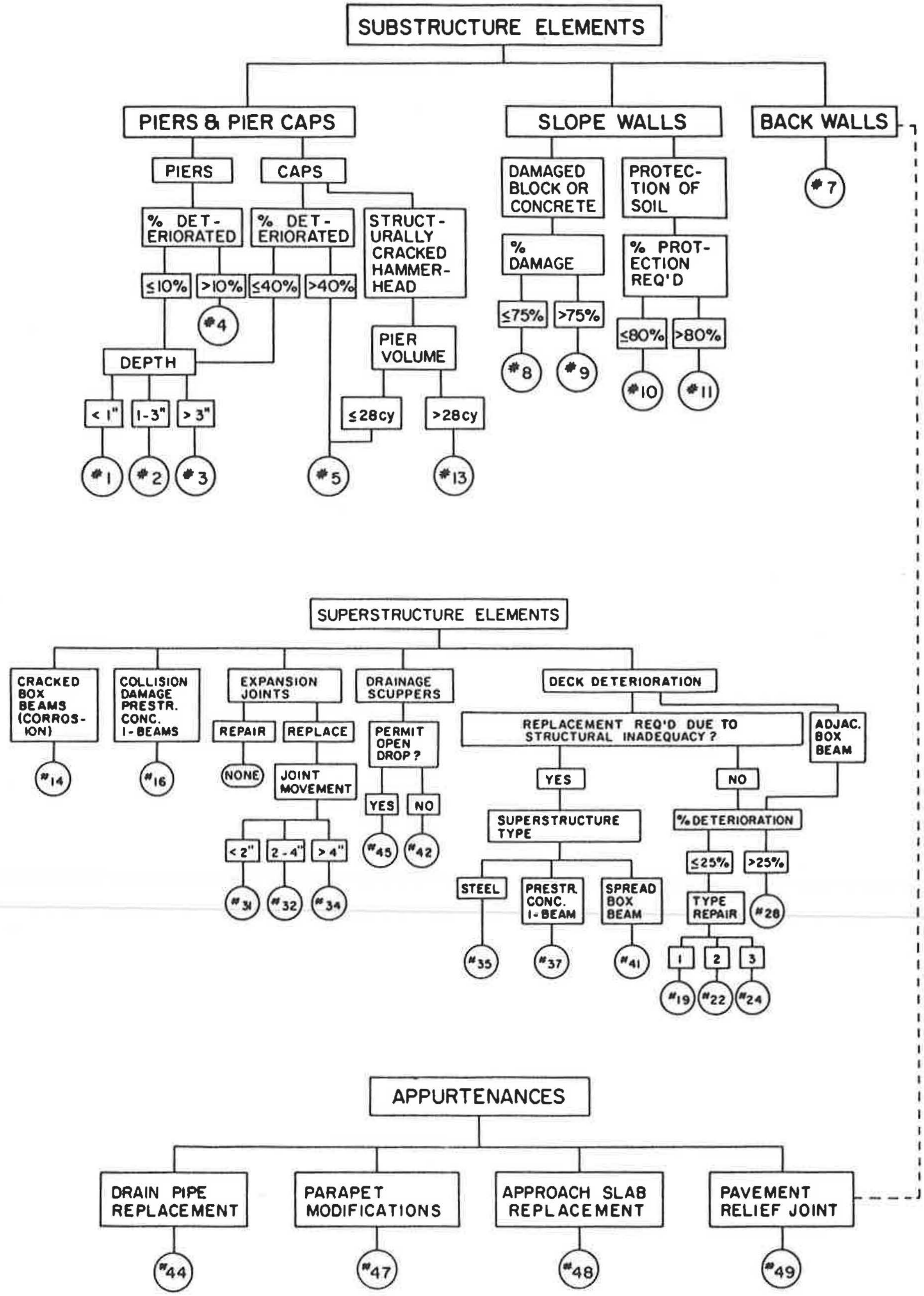

FIGURE 2 Summary of selected procedures.

TABLE 7 EFFECTS OF SENSITIVITY ON ECONOMIC DECISIONS

Element	Sensitivity Effect on Economic Decision
Piers	None
Pier caps (deteriorated)	Average breakeven point reduced from 40 percent to 20 percent deterioration
Hammerhead piers (structurally cracked)	Breakeven point increased from 28 yd^3 to 38 yd^3
Slope walls (deteriorated block or concrete)	None
Slope walls (protect soil)	Breakeven point decreased slightly (80 percent to 75 percent)
Back walls	Not applicable (only one choice)
Cracked box beams	None
Collision damage to prestressed I-beams	None (assuming beams > 43-ft length)
Drainage scuppers	None
Expansion joints	None
Decks	None
Drain pipe replacement	Not applicable (only one choice)
Parapets	None
Approach slab replacement	Not applicable (only one choice)
Pavement relief joint	Not applicable (only one choice)

ognized that in such a dynamic field as bridge maintenance, rehabilitation, and replacement, three categories of information will always exist: massive numerical data from tracked past experience, limited data from newly applied technological developments, and vague data from emerging technologies. The expert opinion method that includes economic intangibles and a sensitivity analysis presents a solution to the problem of identifying emerging bridge technologies that may be least cost solutions to existing problems.

ACKNOWLEDGMENTS

The authors wish to acknowledge the support of PennDOT and the Federal Highway Administration in funding the research described in this paper. Heinz Koretzky, of PennDOT's Bureau of Bridge and Roadway Technology, was the project manager. The authors wish to thank him for his help and many good suggestions throughout the project.

REFERENCES

1. *Highway Maintenance Research Needs*. Report No. FHWA-RD-511. Federal Highway Administration, U.S. Department of Transportation, Washington, D.C., 1975.
2. Howard M. Raiken. Federal Role in Supporting Research and Development for Reducing Transportation Maintenance Costs. In *Transportation Research Record 598*, TRB, National Research Council, Washington, D.C., pp. 5–11, 1976.
3. *The Pennsylvania Bridge Management System: Executive Summary*. Report No. FHWA/PA-85-045 + 84-28A. Pennsylvania Department of Transportation, Harrisburg, Pa., April 1986.
4. Ronald C. Arner, John M. Krugler, Richard M. McClure and Kantilal R. Patel. The Pennsylvania Bridge Management System. In *Transportation Research Record 1083*, TRB, National Research Council, Washington, D.C., pp. 25–34, 1986.
5. Richard E. Weyers, Philip D. Cady, and John M. Hunter. *Cost-Effectiveness of Bridge Repair and Procedures, Part I: Final Report*. Report No. FHWA-PA-86-025+84-11, Pennsylvania Department of Transportation, Harrisburg, Pa., May 1987.
6. Richard E. Weyers, Philip D. Cady, and John M. Hunter. *Cost-Effectiveness of Bridge Repair Details and Procedures, Part II: Implementation Report*. Report No. FHWA-PA-86-026+84-11, Pennsylvania Department of Transportation, Harrisburg, Pa., May 1987.
7. Richard E. Weyers, Philip D. Cady, and Richard M. McClure. Cost-Effective Decision Models for Maintenance, Rehabilitation, and Replacement of Bridges. In *Transportation Research Record 950*, Volume 1, TRB, National Research Council, Washington, D.C., pp. 28–33, 1984.
8. M. G. Natrella. The Treatment of Outliers. *Experimental Statistics*, National Bureau of Standards Handbook 91, National Bureau of Standards, U.S. Department of Commerce, Washington, D.C., 1963.
9. J. R. Craig, D. S. O'Connor and J. J. Ashlskog. *Economics of Bridge Deck Protection Systems*. International Corrosion Forum, National Association of Corrosion Engineers, Houston, Tex., March 1982.

Publication of this paper sponsored by Committee on Structures Maintenance.

Deep Impregnation of Concrete Bridge Decks

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The deep monomer impregnation (depth of impregnation 3 to 4 in.) and in situ polymerization of a bridge deck using the grooving technique is presented. The study shows that the process is commercially feasible and the work can be successfully completed to a set of specifications by contractors with no experience with the monomer impregnation and in situ polymerization process. Laboratory estimates of operation times are compared with field performances. Field operation times were significantly less for the impregnation time and the polymerization time but slightly greater for drying times. Safety procedures and cost estimates are also presented. The deep impregnation process is shown to be cost competitive with cathodic protection.

The nation's bridges continue to deteriorate at an alarming rate. In June of 1985, the Federal Highway Administration reported that about 75,000 bridges on the federal aid system and about 184,000 bridges off the federal aid system were deficient (*1*). Essentially, there has been no reduction in the backlog of deficient bridges despite significant increases in bridge rehabilitation and replacement efforts by the states. The 1986 rehabilitation or replacement upgrade cost for all the deficient bridges was about $48.3 billion, about $3 billion more than the 1984 estimate. Approximately one-half of the deterioration cost is related to concrete bridge decks with much of the deterioration related to chloride deicer salts penetrating the concrete and corroding the reinforcing steel (*2*).

The average bridge deck in the snow belt constructed with uncoated reinforcing steel with a 2-in. average cover depth will begin to spall about 7 yr after construction and will require rehabilitation at an age of 22 yr (*3*). This implies that one-half of the bridges constructed with uncoated reinforcing steel and 2 in. of cover will deteriorate at a more rapid rate.

In 1973, the first bridge deck to be constructed with epoxy-coated reinforcing steel was built in West Conshohocken, Pennsylvania. To date, it appears that epoxy-coated reinforcing steel will significantly increase bridge deck life (*4, 5*). However, even in Pennsylvania, the pioneer in the use of epoxy-coated reinforcing steel, acceptance was slow. Of 625 new bridge decks built in Pennsylvania from 1973 to 1978, 468 were built with uncoated reinforcing steel, 90 with galvanized reinforcing steel and only 67 with epoxy-coated reinforcing steel (*5*). More than half (36) of the new bridges built between 1973 and 1978 in Pennsylvania using epoxy-coated reinforcing steel in the deck were built in 1977 (22) and 1978 (14). Thus, presently there exists a significant number of bridges built with uncoated reinforcing steel that are still in sound condition, but these will begin to deteriorate in the near future.

From 1967 to 1975, extensive laboratory testing clearly demonstrated the capability of deep impregnation to combat the bridge deck problem (*6–12*). Deep impregnation consists of drying the concrete, using propane fired infrared heaters, to the desired depth of impregnation, soak-impregnating the concrete with a monomer, and thermally polymerizing the monomer in situ. The monomer system is a mixture of 100:10:0.5 parts of methyl methacrylate, trimethylolpropane trimethacrylate (promotor and cross-linking monomer) and 2, 2-azobisisobutyronitrile initiator (MMA-TMPTMA-AZO). Deep impregnation stops corrosion by encapsulating the chloride, replacing the corrosion cell electrolyte (concrete pore water solution) with a dielectric material (polymer), and restricting the ingress of moisture and oxygen needed in an active corrosion cell by partially filling the capillary void system.

In 1975, a small test section (3.5 ft by 11.5 ft) on an 8-yr-old heavily trafficked bridge deck near Bethlehem, Pennsylvania, was impregnated to a depth of 3 to 4 in. (*13*). At the time of impregnation no spalls or patches existed on the deck. However, the deck was critically contaminated with chlorides at the depth of the top reinforcing steel. In 1984, 9 yr after the impregnation, the deck had numerous spalls and delamination planes but there was no evidence of spalling or delamination in the test area (*14*). Spalling was adjacent to, and delamination planes extended to the borders of, the impregnated area, but was not within it. In addition, the surface wear of the impregnated area was 65 percent less than the surrounding nonimpregnated area and the chloride content within the impregnated area was significantly less at the 99 percent confidence level. A microscopic examination revealed the most significant finding, a preexisting corrosion cell that had been arrested by the impregnation process, and the deep impregnation

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significantly retarded the ingress rate of chloride at all depths even though the shrinkage or thermal cracking, or both, was not filled with polymer.

Although the deep impregnation by soaking was shown to be capable of stopping reinforcement corrosion, two problems remained. First, the time required for the 3- to 4-in. deep impregnation was too long, about 4 days. Second, large, nonhorizontal deck cracks required large excesses of monomer in order to pond their surface. Also, problems of containment of the monomer and potential hazards of having a large area of monomer, a highly volatile and flammable material, exposed during the impregnation process had to be addressed. The grooving technique (*15*, *16*) alleviated these problems. Grooves cut along lines of equal elevation act as vessels for the monomer and minimize the amount of monomer while reducing the exposed monomer surface area. Because the impregnation takes place through the sides and bottoms of the grooves, 1- and ½-in. deep grooves reduced the 4-in. depth impregnation time from about 4 days to about 16 hr. The grooves are cut to a depth of ½ in. above the top reinforcing steel and the width and spacing are sized to accommodate the total volume of monomer required to impregnate the concrete to the desired depth.

However, small-scale laboratory tests and field trials of deep impregnation were not sufficient to resolve a number of significant questions that had to be addressed before the technique could become a commercially feasible field procedure. The questions included the effects of heating large areas of the deck to the temperature required for rapid and adequate drying, potential problems of bridge geometry on groove cutting, ability of the grooves to provide adequate containment after drying, means of providing effective weather protection during drying and impregnation, and potential problems in providing uniform groove-filling in the field. Also, there is a question of whether a typical bridge contractor, unfamiliar with the process, would be capable of impregnating a bridge deck to a given set of specifications.

The following presents the results of a full-scale deep impregnation of a bridge deck using the grooving technique to determine the commercial feasibility of deep impregnation and to compare laboratory results with field results.

TEST BRIDGE

The test bridge is a three-span multigirder bridge with simply supported steel plate girders, permanent steel deck forms, and composite design. The end spans are 42 ft and 38 ft and the center span is 131 ft. The deck width, curb to curb, is 44 ft (two 12-ft traffic lanes and two 10-ft aprons). The deck concrete was placed in April 1972 and the first live load (construction equipment) application occurred on May 12, 1972. The bridge is located on the Mt. Nittany Expressway (US-322) over Pennsylvania Route 45 near Boalsburg, Pennsylvania. The bridge is on a skew, 7 degrees, 40 minutes, 03 seconds, essentially on a tangent, and is on a slight upgrade of about 1.4 percent. According to the deck plans, the traffic lanes are cross sloped ⅛ in./ft and the aprons are cross sloped ⅜ in./ft. The design deck thickness is 8 in., with a 2-in. minimum cover depth. The main reinforcement (transverse direction) is made up of No. 5 bars on 6-in. centers, top and bottom. The top longitudinal bars are No. 4 bars 12 in. on center, and the bottom bars are No. 5 bars 9 in. on center. The concrete mixture was Pennsylvania Class AA concrete using No. 57 crushed limestone and a natural bank sand with a 28-day compressive strength of 3,750 psi. The design slump and air content values were 2.5 in. and 6.5 percent. Measured slump values averaged 2.25 in. and the air content varied from 5.4 to 8.0 percent. Averages of two concrete compressive strength cylinders were 3,440 psi at 6 days and 3,643 psi at 10 days.

Sixty ft, or approximately one-half of the center span, was selected for the deep impregnation trial installation. The remainder of the span is to serve as a control for future performance reference purposes. The bridge had been open to traffic for 13 yr before the trial deep impregnation.

PRELIMINARY TEST WORK

Precise leveling survey was performed on the test area to establish the equal elevation groove cut lines. The leveling survey elevations and mean directions are presented in Figure 1. The determined groove orientations were subsequently verified using a 6-ft spirit level.

A hand-held pachometer was used to take rebar depth of cover measurements at a sufficient number of points to determine the distribution of the rebar depth at a statistical significance level comparable to the reported accuracy of the instrument (*17*). The average cover is 2.86 in., with a range of 2.3 to 3.3 and a standard deviation of 0.22 in. Thus, there is a probability of about 1 in 20,000 of having any steel in the deck with a cover depth of less than 2 in. A rolling *R*-meter (pachometer) set at a cover depth of 2 in. verified the hand-held results by showing no reinforcement with less than a 2-in. cover depth.

The groove width, depth, and spacing are interrelated functions of reinforcement depth and impregnation rate and time. Three 4-in. diam by approximately 6-in. deep cores were taken to determine the rate of impregnation and percent by weight of polymer loading. The cores were dried in an oven at 230°F ± 5°F for 72 hr, allowed to cool and be soak impregnated from the top surface only for 16 hr using the MMA-TMPTMA-AZO monomer system, and polymerized in a hot water bath. The results of the depth of impregnation for the three cores are presented in Table 1. The average 16-hr impregnation was 2.9 in., unit weight of the unimpregnated concrete was 141 lb/ft^3 and the monomer loading was 3.5 percent by weight.

Using previously developed procedures (*18*), various combinations of groove dimensions and spacing and impregnation times were evaluated. However, the primary consideration for this deep impregnation test trial was to evaluate a combination of factors that are representative

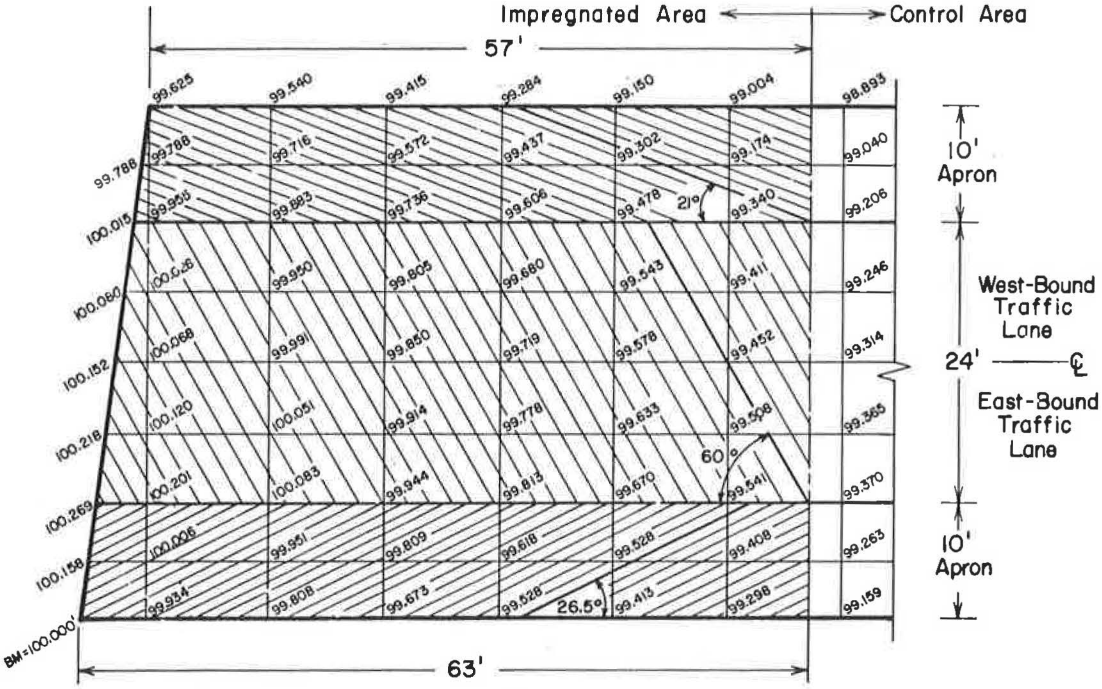

FIGURE 1 Results of precise leveling survey on deck surface and resulting groove orientations (indicated by direction of cross-hatching).

of typical bridge decks. Therefore, the groove width, depth, and spacing were determined for a typical cover depth of 2 in., depth of impregnation of 3.63 in., and the impregnation time based on the rate of impregnation of the test cores. Given these conditions, the following groove-impregnation characteristics were selected:

Groove width = 0.75 in.,
Groove depth = 1.50 in.,
Groove spacing = 3.00 in. center to center, and
Impregnation time = 16 hr.

The estimated quantity of monomer was 3,950 lb, based on an average depth of penetration of 4 in. and 3.5 percent by weight monomer loading determined from the cores.

GROOVE-CUTTING OPERATION

Approximately 11,000 lineal ft of grooves had to be cut to cover the 2,640-ft^2 test area. The specifications required that the grooves extend to within 1 ft of the curb lines, and meet the following tolerances:

1. Groove spacings:
   (a) ± 0.25 in. between any two adjacent grooves,
   (b) number of whole groove widths (including the equivalent of partial width at ends) over any 10-ft length measured perpendicular to grooves = 40 ± 1,
2. Groove width: ± 0.0625 in.,
3. Groove depth: from a straight edge resting on the pavement surface to all points vertically below on the groove root: ± 0.125 in.

The contractor used a standard water-cooled concrete saw with two diamond set blades sandwiching a smaller diameter abrasive cut-off wheel to cut the groove width in one pass. The grooves were cut one at a time with snap lines set about every 5 ft for controlling the groove orientations. A wheel and guide on the front of the machine assisted in maintaining proper groove spacing between adjacent grooves. The groove-cutting operation is illustrated in Figure 2.

Some early problems were experienced by the contractor's forces in maintaining the direction and spacing of the grooves. However, after cutting about five grooves (about 60 lineal ft), they became accustomed to the operation and were producing acceptable work at a rate of 120 ft/hr. The groove spacing and depth were within specifications for the entire job. However, the groove width was generally 0.125 in. narrower than the 0.75 specified width, or about 0.6 in. narrower than the lower specification limit. This

TABLE 1 IMPREGNATION OF PRELIMINARY TEST CORES

Core No.	Length (in.)	16-hr Impregnation Depth (in.)
1	4.7	3.0
2	4.5	2.8
3	2.8	2.8[a]
Average		2.9[b]

[a]Complete penetration.
[b]Omitting Core 3.

FIGURE 2 Groove-cutting operation.

deviation was considered acceptable because the depth of the grooves was 0.1 in. greater than 1.50 in. (the depth and width deviations offset each other).

There were no significant problems with the groove-cutting operation. Only minor learning difficulties were experienced. This was also true for the small-scale laboratory trial impregnations. The task was time consuming but improvements can be made by using larger equipment with gang saws. Also, the removal of the sediments is a problem if they are allowed to dry out in the grooves. Any equipment development should include a tailings vacuum system.

WEATHER PROTECTION

For the drying and impregnation phase of the deep impregnation process, decks need to be protected from precipitation and surface runoff. A tent arrangement was developed consisting of heavy plastic tarpaulin supported on half-arch pipe sections attached to the parapets and railings and supported by cables. The tent was subjected to several periods of moderately heavy rainfall up to ½ in. and 20 mph winds. Water collected in sags of the tent and threatened to collapse it. The problem was eliminated by using lollipop support props in the tent.

Surface runoff was collected by two diversion dams constructed with asphalt cold mix and sealed with asphalt emulsion. Four-in. diam holes through the deck in front of the second dam on each side of the deck drained the water from the deck.

The performance of the weather protection devices was exceptional. The deck remained dry during the drying and impregnation phases.

DRYING

The drying equipment was specially designed and built for the contractor. The drying train consisted of six units, 36 in. deep, 60 in. wide, and 86 in. long. The train formed by bolting the six units together was able to dry a 43-ft long section (the full width of the 44-ft wide deck) covering 5 ft of the bridge at a time. Each unit housed three 120,000 BTU/hr, propane-fired, infrared radiant heaters operating at 3 psi pressure. Pressure regulators were installed in the fuel line of each heating element and permitted individual heating adjustments for the 18 heater elements.

TABLE 2 SURFACE HEATING RATE SPECIFICATIONS

Time (min)	Surface Temperature °F
Start	Ambient
15	375 ± 25
30	475 ± 25
45	575 ± 25
Until dry	575 ± 25

To minimize thermal gradients and thus thermal stresses, the heating rate was controlled by surface temperatures in accordance with the specifications presented in Table 2. In addition, the dried areas were covered with R-19 glass wool insulation immediately after the heaters were removed to reduce thermal gradients during cool down. A 24-in.-wide strip of R-19 glass wool insulation was placed on the deck in front of the heater to reduce heating losses and to reduce thermal gradients in front of the heating train. The front side of the heating train is shown in the photograph in Figure 3.

Small scale laboratory drying trials with a 600°F surface temperature showed that drying to a depth of 4 in. below the surface took about 3.5 hr at an ambient temperature of 75°F (*18*). The drying times on the trial deck impregnation took somewhat longer and ranged from 3.9 hr to 6.0 hr, with an average of 4.6 hr for the 14 drying operations (4.5-ft advance with 0.5 ft overlap per setup). The mean ambient temperature was somewhat lower than 75°F and ranged from 57°F to 82°F, with a mean ambient temperature of 60°F.

The increased drying times were most likely related to lower temperature experienced in the field and wind veloc-

FIGURE 3 Front side of heating train during drying.

ities not experienced in the laboratory. To determine when the concrete is dry at the desired depth of impregnation, it appears necessary to measure the temperature of the concrete at the desired depth of impregnation. The concrete is to be considered dry at a temperature of 220°F. For thermocouples set from the top of the deck to measure the temperature at the desired depth of impregnation, a correction factor must be applied to account for the false high temperatures caused by the conduction of heat to the junction of the thermocouple. Laboratory experiments indicated the correction factor to be about 50°F. However, field measures indicate the correction factor to be about 25°F.

Laboratory and small-scale field trials showed that the high surface temperatures caused shrinkage or thermal cracking, or both. Generally, these cracks were minor and extended to a depth of about 1 in. Fine drying shrinkage or thermal cracks were also observed in the field trial. These cracks are generally oriented perpendicular to the groove directions. A typical shrinkage or thermal crack is shown in Figure 4.

One week after the field impregnation trial was completed (i.e., backfilling of the grooves), 12 4-in. diam cores by approximately 6 in. in depth were taken. Three were taken from the control section and 9 from the impregnated section. The shrinkage or thermal cracks were observed in the impregnated cores and generally ranged in depth from 0.10 in. to 1.35 in. and were not filled with polymer. Only in one case did a shrinkage or thermal crack exceed the depth of the groove (1.5 in.). That crack depth was 2.98 in. However, a core taken from the control area also contained shrinkage cracks to a depth of about 0.60 in. and there was no significant difference in the cracking between the impregnated area and the control (unimpregnated) area. A microscopic examination of the other two cores taken from the control area was not performed because these two cores were taken for compressive strength tests.

The shrinkage or thermal cracks were visible to the unaided eye during the heating phase of the drying cycle for both laboratory and field trials. These cracks were not visible on cooling and presented no problems during impregnation for either the laboratory or field trial impregnations.

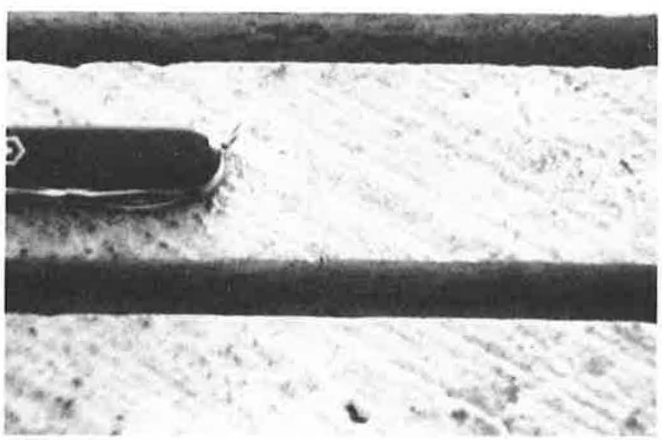

FIGURE 4 Typical minor drying shrinkage/thermal crack.

FIGURE 5 Two-man crew filling individual grooves.

IMPREGNATION

Laboratory experiments indicated that 0.75-in.-wide by 1.5-in.-deep grooves cut to impregnate to a depth of about 4 in. would empty in about 16 hr (*18*). The filling of the grooves with monomer was carried out by three 2-man crews working simultaneously. Groove filling was done at the ends of the grooves; polyethylene sheets covering the deck were folded back just enough to expose the groove ends, thus minimizing direct exposure of the monomer to the atmosphere. All the grooves were filled in about 4 hr. The grooves were refilled as the concrete absorbed the monomer. Refilling continued until all 14 drums (5,600 lb) of monomer were used (4,000 lb or 10 drums was the estimated amount required to impregnate to a depth of 4 in.). The entire process, from mixing of the first drum until the last drum was emptied, took about 6.5 hr. A two-man crew filling the grooves is shown in Figure 5. The monomer was allowed to soak for an additional 15 hr. However, it appeared that all of the monomer that was going to soak in did so within the first 4 hr.

The reduction in the field impregnation time from the estimate of 16 hr based on laboratory results to about 4 hr is most likely related to the higher field drying temperatures (600°F field surface temperature, 450°F at 1 in., 380°F at 2 in., 300°F at 3 in. and 220°F at 4 in. at the end of the heating cycle compared with a 230°F oven-drying temperature).

As previously stated, the estimate of monomer needed to impregnate the deck test area to an average depth of 4 in. was 3,950 lb based on the laboratory loading of cores of 3.5 percent by weight. A total of 5,600 lb of monomer was placed in the grooves. Approximately 1,000 lb of excess monomer was vacuumed from the grooves after 21.5 hr of soak impregnation time. It is difficult to estimate vaporization losses, but it appears that about 4,000 lb of monomer soaked into the deck. Therefore, grooves should be only filled once and the polymerization process begun immediately once the grooves are empty.

POLYMERIZATION

Hot water ponding polymerization in the laboratory on full-depth simulated 6 ft^2 deck slabs indicated whether the water was maintained at about 205°F; the concrete temperature at a depth of 4 in. reached a steady state temperature at 122°F in about 16 hr. The polymerization time for impregnated concrete at 122°F is about 4.5 hr. Therefore, the estimated total polymerization time is about 21 hr.

Precast concrete barriers placed across the ends of the test section and the parapets acted as the lateral supports for the bridge hot water polymerization pond. A vinyl tarpaulin was used to cover the deck and act as the hot water pond containment vessel. The weatherproofing tent was spread over the deck surface to protect the vinyl tarpaulin. Two distribution heaters, one on each of the two 200-hp portable boilers, injected live steam into the 30,000 gallon polymerization hot water pond. The minimum depth of 10 in. was maintained at the highest elevation point within the test area. The surface of the hot water polymerization pond was open to the atmosphere during the polymerization process.

Except for leakage and evaporation losses, the heating system was a closed loop. The boiler feed was drawn continuously from the water bath. Boiler No. 1 was fired and boiler No. 2 came on about 2 hr later. The temperature of the pond was slightly less than 200°F 6 hr after boiler No. 2 came on.

The temperature at a depth of 4 in. in the concrete reached a steady level of 135°F (123°F actual temperature, corrected for thermal conductivity). The temperature of the pond was difficult to maintain at 200°F because of water leakages and evaporation losses, which had to be replaced. Pond and concrete temperatures throughout the polymerization process are presented in Figure 6.

The polymerization process took about 17 hr or about 4 hr less than the estimated time of 21 hr. This occurred in spite of the adverse weather (temperatures of 45°F to 60°F, sporadic light rain, and a steady northwest wind at about 20 mph) and equipment malfunctions and water loss that kept the temperature 10°F below the desired 205°F. Thus, it appears that hot water polymerization of large areas is more efficient than small laboratory test slabs.

GROOVE FILLING

The grooves were backfilled with a latex-modified mortar with a 10-in. slump using rubber-edged squeegees to distribute and compact the mortar. The grooves were easily filled in 1 working day. The groove-filling operation is shown in Figure 7 and a close-up of the surface after 1 day is shown in Figure 8.

Sections of cores 1, 2, 3, and 6 were subjected to 300 cycles of rapid freezing and thawing in water (ASTM C 666, Procedure A). The primary purpose of freeze-thaw testing was to evaluate the performance of the latex-mod-

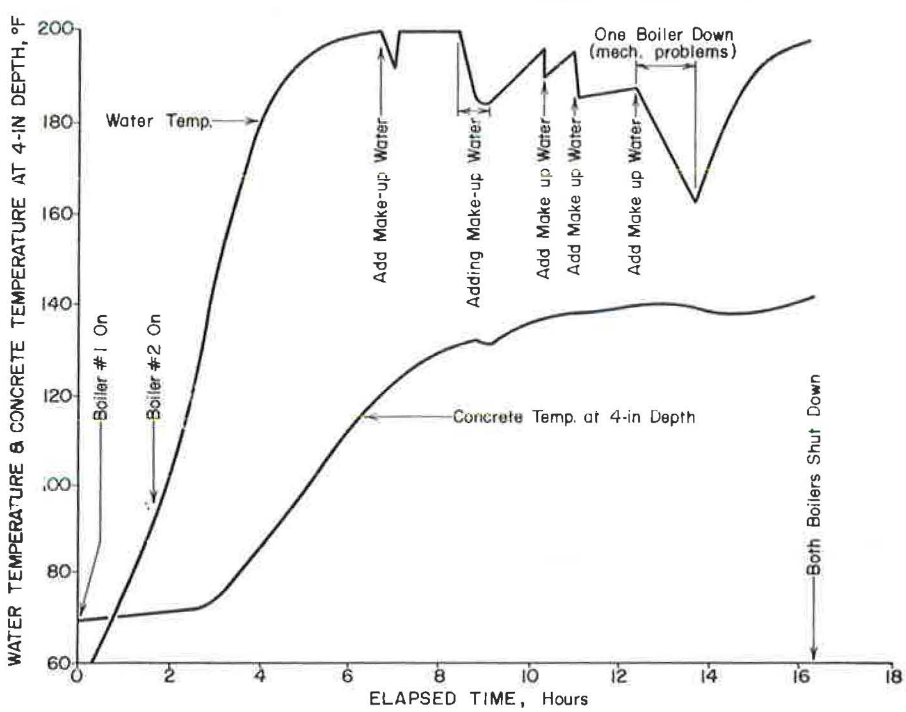

FIGURE 6 Deck polymerization temperatures.

FIGURE 7 Groove-filling operation.

ified mortar groove filler. The range of results is presented in Figure 9. As shown, the latex-modified mortar groove filling fared well. In most instances, nil to very light scaling of the groove filling occurred and the groove filling remained intact. The photographs presented in Figure 9 also illustrate the expected superior performance of polymer-impregnated concrete. The dashed lines indicate the approximate depths of impregnation and the arrows the groove filling.

SAFETY PROCEDURES

Potential safety hazards inherent in the process of deep monomer impregnation of bridge decks are related to the nature of the chemicals used. The monomer is volatile and flammable, and its vapor is explosive (explosion limits of 2.12 to 12.5 percent). Therefore, the prevention of sources of ignition, the minimization of monomer exposure to the atmosphere, and the provision of emergency facilities must be thoughtfully provided for.

Fire protection was provided during the period beginning with the mixing of the monomer until the completion of the polymerization. The fire-fighting facilities were staged

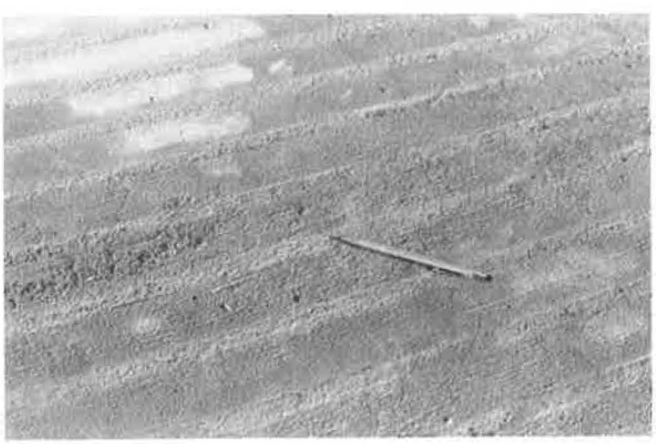

FIGURE 8 Deck surface 1 day after groove filling.

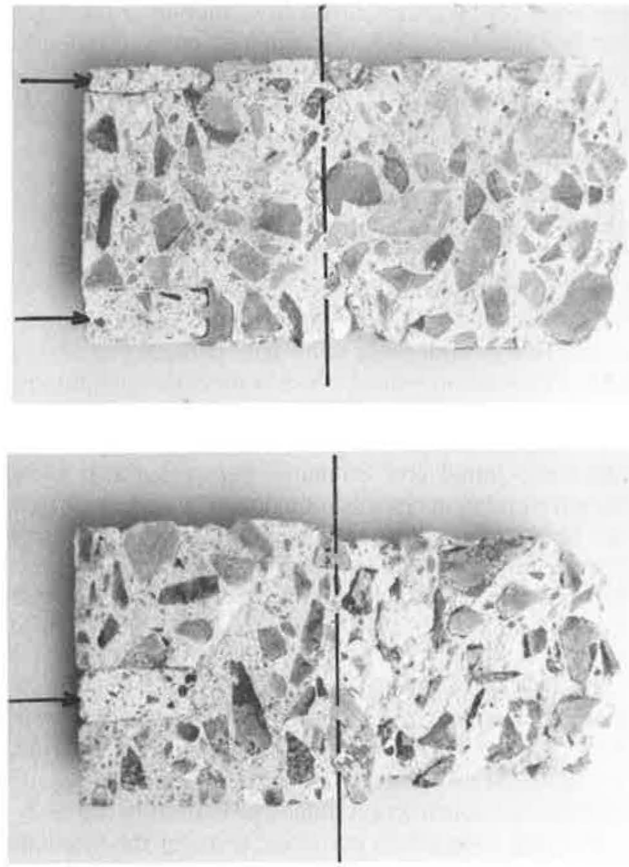

FIGURE 9 Condition of core remnants after 300 freeze-thaw cycles in water.

upwind, beyond the monomer mixing and distribution area. Water and foam facilities were provided. The catalyst was added to the monomer and mixed in electrically grounded 55-gallon drums with air-driven, propeller-type stirrers. Polyethylene sheets covered the deck during the monomer groove-filling and impregnation operation and minimized monomer evaporation. The air in the weather protection tent and below the bridge was checked at frequent intervals for monomer vapor concentrations. Concentrations remained well below the lower explosive limit (2.12 percent) throughout the groove-filling and impregnation operations. At the end of the soaking period, the polyethylene sheeting was removed and the excess monomer remaining in the grooves was vacuumed up using an air-motor-driven, explosion-proof industrial vacuum unit. This step proved to be the most potentially dangerous activity of the entire operation. Monomer vapors in the atmosphere within a radius of about 1½ ft from the vacuum exhaust showed concentrations typically in the range of 1.5 to 1.75 percent, but at times exceeded the lower explosive limit of 2.12 percent.

In addition to fire and explosion hazards, the chemicals used are toxic to varying degrees. The monomer components are considered to be moderately toxic (primarily irritants). Therefore, personnel protection against skin contact and breathing high vapor concentrations must be provided for. Workers involved in monomer mixing and distribution wore one-piece hooded coveralls, goggles, long rubber gloves, and dust masks. Those distributing the monomer to the grooves inside the tent wore canister masks as protection against organic vapors.

COST

Pilot projects, such as the one being reported here, always have associated with them extraordinary high costs related to the lack of contractor familiarity (risk factor) and suitable, efficient equipment. With respect to deep monomer impregnation and in situ polymerization, based on the experience of this project, process inefficiencies were identified and initial cost estimates were calculated using a capital equipment amortization rate of 10 percent. Because the amortization costs of large capital equipment are an inverse function of the square footage of bridge deck to be treated by a contractor per year, costs were determined for 1, 4, and 10 bridges per year using a typical bridge deck size of 44 ft wide, curb to curb, by 120 ft long. Obviously, larger bridges at a given location will result in lower unit costs. The total initial cost per ft^2 in 1985 dollars for 1, 4, and 10 bridges treated in a year by a contractor is \$16.98, \$13.05 and \$11.96, respectively. Unit cost per process and construction item is presented in Table 3.

For cost comparison purposes, costs for the installation of a cathodic protection system, the only other process capable of arresting the corrosion of black steel in concrete, were obtained for four 10-yr-old bridge decks. The installation of the cathodic protection systems was performed under one contract. The cathodic protection system used was a platinized wire primary anode with secondary carbon-strands anodes placed 1 ft on centers with transverse locations for redundancy and covered with a 1¼ in.-thick latex-modified concrete overlay. The initial 1985 cost per ft^2 for the four bridge decks, not including deck repairs carried out preliminary to the installation of the cathodic protection system, is \$13.34 (monomer impregnation and in situ impregnation work did not require preliminary deck repairs). For a valid comparison between deep impregnation and cathodic protection, it is necessary to compare life-cycle cost rather than initial cost because cathodic protection has additional future costs of electrical power, system maintenance, and periodic monitoring. These costs total, in 1985 dollars, \$0.13/ft^2. Using an average true (inflation-adjusted) interest factor of 5 percent *(19)*, the break-even point for cathodic protection and deep impregnation based on life-cycle costing is \$15.57/ft^2. This would occur at about two bridges/yr/contractor. However, it needs to be pointed out that the cost of cathodic protection was based on four bridges under a single contractor with both systems, cathodic protection and deep impregnation, having a 40-yr service life. Thus, on an equivalent comparison life-cost basis, deep impregnation of concrete bridge decks would be a least cost solution to corrosion protection over cathodic protection or no less than a cost-competitive solution.

TABLE 3 ESTIMATED INITIAL COSTS BASED ON VOLUME APPLICATION (\$/ft^2)

Item	No. of Bridges/Yr/Contractor 1	4	10
Grooving	1.82	1.75	1.57
Drying	2.15	1.29	1.08
Weather protection	2.76	1.49	1.23
Impregnation	2.67	2.35	2.29
Polymerization	1.16	0.56	0.40
Groove filling	0.71	0.71	0.71
Monitoring (process control)	0.33	0.25	0.23
Fire protection	0.45	0.45	0.45
Lightning and electric power	0.54	0.54	0.54
Construction superintendent	1.24	1.24	1.24
Mobilization	0.93	0.72	0.66
Traffic maintenance and protection	0.39	0.30	0.28
Surety bonds	0.08	0.07	0.06
Profit	1.05	0.81	0.74
General overhead	0.70	0.52	0.48
Total	16.98	13.05	11.96

RESULTS AND DISCUSSIONS

Nine 4-in. diam cores were taken from the impregnated area as stated previously. Four were along the center line at the joint of two heaters, three within the interiors of heating units, one in a heater overlap area and one next to the parapet. The field and laboratory impregnation depth measures for the nine cores are presented in Table 4. Cores 5, 7, and 12 were taken from areas under heating units and thus should represent typical condition. The depth of impregnation of about 3.5 in., which agrees with laboratory estimates, is indicated in Table 4.

The project clearly demonstrated the technical feasibility of deep impregnation of bridge decks and that it can be done on a commercial basis. A contractor who had no experience with deep impregnation was able to successfully impregnate a deck area of about 2,600 ft^2 to the depth of 3 to 4 in. Although the drying times were greater than laboratory estimates, impregnation times appear to be significantly less and field polymerization times also appear to be less than laboratory estimates.

ACKNOWLEDGMENTS

The authors wish to acknowledge the support of the National Cooperative Highway Research Program of the Transportation Research Board in funding the laboratory development and application work described here as part of a larger project. Also, the authors acknowledge the funding support of the Pennsylvania Department of Transportation and the Federal Highway Administration of the field test impregnation trial reported herein.

TABLE 4 DEPTH OF IMPREGNATION DETERMINATIONS

Impregnation			Comments
	Depth (in.)		
Core No.	Field[a]	Lab[b]	
1	3.25	3.0	Cores 1 through 4 taken along center line, coincided with jointure of heating units; expect lowest impregnation depths. Also, looking for gradient because of time lapse before impregnation
2	3	3.2	
3	3	3.5	
4	2.75	3.5	
5	3.25	[c]	Interior of a heating unit (typical conditions)
6	4.5	4.6	Overlap area of heater set-ups (expect deepest impregnation)
7	3.25	[c]	Interior of a heating unit (typical conditions)
11	2 to 2.75	2.3	Core taken 6 in. from parapet (expect shallow impregnation)
12	3.75	3.4	Interior of heating unit (typical conditions)

[a]Acid etched along one narrow vertical line immediately after coring.
[b]Average of at least four measurements on etched face of vertical slab cut from core.
[c]Compressive strength specimen—not sectioned.

REFERENCES

1. Bridge Program Cited. *Engineering News Record*, Vol. 214, No. 23, June 6, 1985, p. 6.
2. R. E. Weyers and P. D. Cady. Cathodic Protection of Concrete Bridge Decks. *American Concrete Journal, Proceedings*, Vol. 81, No. 6, Nov.–Dec. 1984, pp. 618–622.
3. P. D. Cady and R. E. Weyers. Deterioration Rates of Concrete Bridge Decks. *Journal of Transportation Engineering*, American Society of Civil Engineers, Vol. 110, No. 1, Jan. 1984, pp. 34–44.
4. Y. P. Virmani, K. C. Clear, and T. J. Thomas. *Time-to-Corrosion of Reinforcing Steel in Concrete Slabs. V, Calcium Nitrite Admixture or Epoxy Coated Reinforcing Bars as Corrosion Protection Systems*. Report No. FHWA/RD/83/012, Federal Highway Administration, U.S. Department of Transportation, Washington, D.C., Sept. 1983.
5. R. E. Weyers and P. D. Cady. Deterioration of Concrete Bridge Decks From Corrosion of Reinforcing Steel. *Concrete International Design & Construction*. American Concrete Institute, Vol. 9, No. 1, Jan. 1987, pp. 15–20.
6. M. T. Steinberg et al. *Concrete-Polymer Materials, First Topical Report*. Report No. BNL 50134 (T-509) and USBR General Report No. 41, Brookhaven National Laboratory, Upton, N.Y., and U.S. Bureau of Reclamation, Denver, Colo., 1968.
7. M. T. Steinberg et al. *Concrete-Polymer Materials, Second Topical Report*. Report No. BNL 50218 (T-560) and USBR REC-OCE-70-1. Brookhaven National Laboratory, Upton, N.Y., and U.S. Bureau of Reclamation, Denver, Colo., 1969.
8. J. T. Dikeou et al. *Concrete-Polymer Materials, Third Topical Report*. Report No. BNL (T-602) and USBR REC-ERC-71-6. Brookhaven National Laboratory, Upton, N.Y., and U.S. Bureau of Reclamation, Denver, Colo., 1971.
9. J. T. Dikeou et al. *Concrete-Polymer Materials, Fourth Topical Report*. Report No. BNL 50328 and USBR REC-ERC-72-10. Brookhaven National Laboratory, Upton, N.Y., and U.S. Bureau of Reclamation, Denver, Colo., 1972.
10. G. W. Depuy et al. *Concrete-Polymer Materials, Fifth Topical Report*. Report No. BNL 50390 and USBR REC-ERC-73-12. Brookhaven National Laboratory, Upton, N.Y., and U.S. Bureau of Reclamation, Denver, Colo., 1973.
11. L. E. Kukacka et al. *Concrete Polymer Materials for Highway Applications, Progress Report No. 3*. Report No BNL 50417 and FHWA-RD-74-17. Brookhaven National Laboratory, Upton, N.Y., and Federal Highway Administration, U.S. Department of Transportation, Washington, D.C., 1973.
12. E. Dahl-Jorgensen et al. Polymer-Impregnated Concrete: Laboratory Studies. *Transportation Engineering Journal*, ASCE, Vol. 101, No. TE1, Feb. 1975, pp. 29–45.
13. J. A. Manson et al. *NCHRP Report 190: Use of Polymers in Highway Concrete*. TRB, National Research Council, Washington, D.C., 1978.
14. P. D. Cady and R. E. Weyers. Field Performance of Deep Polymer Impregnation. *Transportation Engineering Journal*, ASCE, Vol. 113, No. 1, Jan. 1987, pp. 1–14.
15. R. E. Weyers and P. D. Cady. Development: Deep Grooving–A Method for Impregnating Concrete Bridge Decks. In *Transportation Research Record 962*, TRB, National Research Council, Washington, D.C., 1984, pp. 14–18.
16. R. E. Weyers and P. D. Cady. Application: Deep Grooving—A Method for Impregnating Concrete Bridge Decks. In *Transportation Research Record 962*, TRB, National Research Council, Washington, D.C., 1984, pp. 19–21.
17. P. D. Cady. *A Study of Policies for the Protection, Repair, Rehabilitation, and Replacement of Concrete Bridge Decks*. Report No. PTI 8103, Pennsylvania Transportation Institute, 1981.
18. J. A. Manson et al. *NCHRP Report 257: Long-Term Rehabilitation of Salt-Contaminated Bridge Decks*. TRB, National Research Council, Washington, D.C., 1983.
19. J. R. Craig, D. S. O'Commer, and J. J. Ahlskog. *Economics of Bridge Deck Protection Systems*. Paper No. 82. International Corrosion Forum, National Association of Corrosion Engineers, Houston, Tex., March 1982.

The authors are responsible for the accuracy of the data and the conclusions and opinions expressed in this paper.

Publication of this paper sponsored by Committee on Adhesives, Bonding Agents and Their Uses.

Bridge Management System Software for Local Governments

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A software system was developed for a microcomputer to conduct bridge management system studies for local agency bridge systems. The software system was developed so that weighting factors and level-of-service goals were kept in separate data base files. This way, criteria could be easily changed without modifying the basic program. By evaluating one local county bridge system, it was demonstrated that microcomputers provided a good computing base for managing local bridge systems. The results of the bridge management system analysis showed excellent correlation with the independently developed bridge replacement program in that county. When differences occurred, they were justified when other factors were considered. A procedure was proposed for implementing a bridge management system at the local level. This approach encourages input from all involved parties in setting policy and level-of-service goals. Particular emphasis is placed on the importance of accurate and consistent bridge parameter data.

Although the need for improving the infrastructure at the local level is well documented, the tools available to local officials for optimally using allocated resources in infrastructure rehabilitation are limited. Local agency bridges are among the most expensive infrastructure items. Most bridge management systems (BMSs) in use today were developed for relatively large state bridge systems. They were also developed to use on relatively large computer systems. Unfortunately, these types of computer systems are not usually available to personnel at the local level. Therefore, the objective of this paper is to present the results of a study to develop a software package for microcomputers to implement a BMS and to present the results of a study for one county BMS.

BACKGROUND

The purpose of any BMS is to provide the means to systematically rank all bridges in a given bridge system. In its simplest form, most BMSs use a ranking formula of the form:

$$\text{Ranking} = \sum_{1} K_i f_i\,(a, b, c,\ldots) \qquad (1)$$

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where

K_i = Weighting factors,
$f_i\,(a,b,c,\ldots)$ = Priority ranking formulas, and
a,b,c = Bridge parameters.

A good summary of several BMSs developed recently can be found in the Federal Highway Administration's report on BMSs (*1*). Typically, the priority ranking formulas evaluate three to six different bridge functions. For example, the system developed for the North Carolina Department of Transportation has four priority ranking formulas (*2*). They measure bridge load capacity, deck width, vertical (over and under) clearances, and estimated remaining life. Other BMSs have priority rating functions measuring parameters such as sufficiency rating, structural and deck condition, and so on. (*3–5*).

The objective of all priority ranking formulas is to develop a number for each bridge on the system. Although these priority ranking formulas have various forms, their sensitivity to various bridge parameters can be shown to vary over a significant range. For example, the sufficiency rating has been shown to be very insensitive to average daily traffic (ADT). Thus, two identical bridges with vastly different traffic patterns would end up with identical priorities if only the sufficiency rating were considered.

The priority ranking formulas are functions of bridge parameters. For a BMS to be implemented, all bridge parameters must be collected for all bridges in the system. The implications of this statement will be discussed later in this paper. However, for the ranking formulas to properly rank the bridges in the system, all bridge parameters must be accurate and consistent.

The last terms discussed in the ranking formula are the weighting factors. These factors provide a means to give relative values to the importance delegated to the various ranking formulas. For example, if bridge deck width is an important local consideration, the weighting factor for deck width should be increased. In general, for most systems, bridge capacity has a fairly high priority consideration. In most situations, a low load capacity is also a good indicator that deck width and remaining life are also low. However, there are exceptions. In most systems, the sum of all weighting factors is equal to 100.

In conclusion, most BMSs develop a priority ranking for each bridge based on weighting factors, priority ranking formulas, and certain bridge parameters. The development of these systems is a series of compromises. If every conceivable bridge parameter is used in the priority ranking formulas, then each bridge parameter must be collected for every bridge in the system. If the number of bridges is large, this could become a significant effort. Even for smaller bridge systems, this approach could become an unwise use of resources. Because the objective of any BMS is to set priorities and get a relative ranking of the system bridges, a more logical approach is to minimize the number of key bridge parameters collected.

North Carolina BMS

To develop a BMS for local agencies, several existing BMSs were evaluated. The one selected for implementation was developed by Johnston and Zia (*2*) for the North Carolina Department of Transportation. It is based on setting level-of-service goals for three different bridge parameters. These are load capacity, deck width, and vertical clearances for traffic over or under the bridge, or both. These levels-of-service were defined as a function of road classification, ADT, and number of traffic lanes. Bridges are ranked based on the number of deficiency points (DP) assigned to each bridge.

The DP are calculated based on the following formula:

$$DP = CP + WP + VP + LP \tag{2}$$

where CP, WP, VP, and LP are need functions for load capacity, deck width, vertical over/under clearance, and estimated remaining life, respectively. The ranking formula for CP is:

$$CP = WC * (CG - SV) * (0.6 * KA + 0.4 * KD)/10 \tag{3}$$

where

$KA = (ADT^{.3})/12$,
$KD = DL * ADTO/(20 * 4000)$,
CG = Capacity goal (tons),
SV = Single vehicle posting (tons),
ADTO = ADT of over route,
DL = Detour length (mi), and
WC = Capacity weighting factor.

The ranking formula for WP is:

$$WP = WW * (WG - CDW) * ADTO/(3 * 4000) \tag{4}$$

where

WG = Width goal (ft),
CDW = Present clear deck width (ft),
ADTO = ADT of over route, and
WW = Deck width weighting factor.

For vertical clearances of the bridge, the ranking formula is broken into two components to account for traffic over and under the bridge. It is:

$$VP = VPU + VPO \tag{5}$$

where

$VPO = WV * (UG - VCLU) * ADTU/(2 * 4000)$,
$VPU = WV * (OG - VCLO) * ADTO/(2 * 4000)$,
UG = Underclearance goal (ft),
VCLU = Present vertical underclearance (ft),
ADTU = ADT of under route,
OG = Overclearance goal (ft),
VCLO = Present vertical overclearance (ft),
ADTO = ADT of over route, and
WV = Vertical clearance weighting factor.

The last component considered is the estimated remaining life for the bridge. This parameter is obtained from the formula:

$$LP = WL * [1 - (RL - 3)/12] \tag{6}$$

where RL is estimated remaining life (yr), and WL is remaining life weighting factor.

TABLE 1 LEVEL-OF-SERVICE GOALS

Highway Function Classification	ADT	Load Capacity (Tons)	Lane Width (ft)	Shoulder Width (ft)	Under Clearance (ft)	Over Clearance (ft)
Major collector	≤800	25.0	9	1	14	14
	≤2000	25.0	9	2	14	14
	≤4000	25.0	10	2	14	14
	>4000	25.0	10	3	14	14
Minor collector	≤800	16.0	9	1	14	14
	≤2000	16.0	9	2	14	14
	≤4000	16.0	10	2	14	14
	>4000	16.0	10	3	14	14
Minor collector	≤800	16.0	9	1	14	14
	≤2000	16.0	9	2	14	14
	≤4000	16.0	10	2	14	14
	>4000	16.0	10	3	14	14

Note: Deck width goal = Number of lanes * lane width + 2 * shoulder width.

For all components of the DP formula, the value for each component shall not be less than zero nor greater than the corresponding weighting factor. After looking at the ranking formulas, the DP formula is a function of eight bridge parameters, three service goals, and four weighting factors. These bridge parameters are usually available because they represent basic data describing the bridge. These ranking formulas can be easily manipulated to give DP per unit component deficiency. These can then be plotted, if desired, as a function of the appropriate bridge parameter.

The weighting factors are presented in Table 1. The service goals are presented in the FHWA report on Bridge Management Systems (*1*) and in the report by Johnston and Zia (*2*).

Once the DPs are calculated for each bridge, the bridges can be ranked in numerical order. There are several approaches to further optimize the use of limited bridge resources. Although more complicated, the incremental cost/benefit ratio can be used to determine the optimal replacement and rehabilitation projects for a system. This approach has some advantages for determining which projects are involved and the degree of rehabilitation and replacement needed so that the maximum benefits are obtained for a given budget. The primary disadvantage is that cost data are required for a relatively large number of alternatives.

A simpler approach is to rank the bridges on the basis of a cost factor (CF) equal to:

$$CF = \text{Replacement costs (\$)/DPs} \qquad (7)$$

The ranking of bridges subject to replacement can be made on the basis of this CF. It would then be prudent to select bridge replacement projects with low CFs. As with any numerical scheme, the user must use judgment and experience when selecting actual projects for a planning period.

In conclusion, the North Carolina approach to bridge management has several advantages over using a single parameter such as the sufficiency rating. This approach assigns DPs nearly directly proportional to ADT. Detour length is also strongly considered in the most heavily weighted factor, load capacity. An additional advantage is that levels of service can be assigned for each highway functional classification of the bridge. The sufficiency rating is assigned based on one standard for all highway function classifications.

Application to a Local Bridge System

Many local agencies have microcomputers available to personnel. Because the computing power of these microcomputers is more than adequate for the analysis of most local bridge systems, the North Carolina BMS was programmed into the microcomputer using the dBASE III Plus (TM) data base management system. This data base system was chosen because of its widespread use in many agencies.

To demonstrate the applicability of the data base system, microcomputer, and the North Carolina approach to bridge management, a bridge system of a local county was selected for evaluation. This county, in Kansas, is located near a growing major metropolitan area. However, many of the county bridges are on rural roads.

The following description is provided to give an idea of the status of the county bridge system evaluated. There are 114 bridges in the system. Several are trusses. However, the majority are simple span bridges made of steel and concrete. Of the 114 bridges, 6 were closed and were included in the totals. The highway function classifications are local roads, minor collectors, and major collectors. The county also has minor and major arterials, and Interstate roads, but bridges on these systems are not a part of the county system. There are 81 bridges on the local system, 6 on the minor collector system and 27 on the major collector system.

There are 9 bridges (6 closed) with an operating rating of between 5 and 9 tons. Eight bridges (6 closed) have an estimated remaining life of less than 5 yr. Forty-four bridges have an estimated remaining life of between 5 and 9 yr. Another 44 bridges have an estimated remaining life greater than 20 yr.

The last variable to be discussed is the ADT count. The ADT range for local road bridges is 0–1,200. Forty-seven local bridges have an ADT of less than 99. Twenty-two have an ADT of between 100 and 199, and 5 bridges have an ADT of between 200 and 299. The remaining local bridges have an ADT of greater than 300. For the 6 bridges on the minor collector system, the range of ADT was 51–3,340. In the traffic ranges previously described, the distribution of ADT was 2, 1, 1, and 2, respectively.

The ADT range for the bridges on the major collector system was 276–4,782. One bridge had an ADT of less than 300, 4 were in the range of 300–499, and 9 were in the range 500–999. Twenty-seven bridges on the major collector system had an ADT of greater than 1,000.

Although very few bridges were replaced during the past 10 yr, the system has several relatively new bridges with good operating ratings and conditions. As with most local systems, there are 6 bridges that have been closed because of poor condition and load condition. In addition, there are several bridges that have load capacity restrictions and are narrow.

In general, this county bridge system is typical of most systems. Some bridges are in excellent condition and others are in desperate need of repair. Overall, some local agencies have bridge systems that are in worse need of replacement and other agencies have bridge systems in better repair.

Fortunately, the local agency had previously developed a complete data base for its bridge system. A significant effort was expended to accurately complete this data base. Because all data were not required to conduct the bridge management study, a new data base was developed using a data base manager system that only contained those bridge parameters required to conduct the analysis.

WEIGHTING FACTORS AND LEVEL-OF-SERVICE GOALS

To improve the flexibility of the system, separate data base files were created for the weighting factors and the level-of-service goals. This way each parameter could be modified without changing the basic system. A dBASE program was written to conduct all numerical calculations and to index or rank each bridge. Because replacement costs were not available, the ranking was done on the DPs. Although the weighting factors were varied later in the study, the baseline weighting factors used in the analysis are those given as follows:

Function	*Weight*
Single vehicle load capacity	WC = 70
Clear bridge deck width	WW = 12
Vertical roadway under/over clearance	WV = 12
Estimated remaining life	WL = 6

The next step was to develop the level-of-service goals. Because of the nature of the data base system, any number of highway function classifications can be defined. For this study, service goals for three highway function classifications were defined. The selected service goals are presented in Table 1. In general, the goals are similar to those outlined in the North Carolina study. Deck width goal varied with ADT so bridges with wider decks would be found on more heavily travelled roads. Because many bridges are on narrow, lightly travelled roads, single lane bridges were permitted. The establishment of these service goals is very flexible. Because they were stored in a separate data base file, they could be easily changed without modification of the program.

RESULTS

Although it is difficult to present the results of the analysis on the bridge system studied, some interesting observations could be made. The 114 bridges were analyzed using a 10 MHz AT clone microcomputer. To analyze the system completely took less than 2 min. This included calculation of all DPs and placing the bridge listings in descending order. Although most local agency bridge systems have less than 500 bridges, a microcomputer has more than sufficient computing power to handle the most sophisticated BMS.

For all bridges, the number of DPs for the entire system ranged from 0 to 72.7. Thirty-seven bridges had zero DPs. No bridge on the system had clearances less than the goals given in Table 1. Therefore, the maximum number of DPs was 88.

After the first analysis was complete, it was obvious that some bridges were not placed in the proper order. Upon review of the data, it became apparent that there were some errors in the data base. This illustrates the first observation. To use a BMS as a policy tool, it is imperative that a good, accurate data base of bridge parameters is available. If the bridge width of one bridge is missing, the ranking of that bridge will not be correct. Fortunately, the obvious errors are easy to spot. The subtle ones are much harder.

Because the weighting factor for load capacity was high, bridges with relatively high load capacity will obviously have small numbers of DPs. For bridges with relatively low load capacity values, bridges with high ADT had the higher number of DPs. The 10 bridges with the highest number of DPs were on the major collector system. The operating rating of these bridges varied between 5 and 16 tons. The ADT of the bridges varied between 496 and 4,782. The next 10 bridges were on the local or minor collector systems. The ADT of these bridges was generally lower.

Because the county had previously developed a comprehensive bridge replacement program, it was interesting to compare the results of the BMS and the independently developed replacement program. Except for specific instances, bridges with high numbers of DPs were scheduled for early replacement. Large discrepancies were observed in one or two instances, although there were good reasons for them in each case.

As discussed previously, all bridges in the county met the clearance goals for all highway function classifications. Therefore, the vertical clearance parameter did not provide useful information in the ranking process. For all bridges, no DPs were calculated for unsatisfactory vertical clearances.

Different weighting factors were considered. Variation of the vertical clearances' weighting factors was not considered for the reasons previously discussed. However, the load capacity weighting factor was reduced to 60 and the estimated remaining life weighting factor was increased to 16. After the analysis was complete, the results were compared. In general, the rankings were very similar with little change in relative rankings. However, two bridges changed their relative ranking approximately 10 to 15 positions.

IMPLEMENTATION FOR THE LOCAL AGENCY

What should be considered before a BMS is implemented at the local level? It would appear that the first step would be to get a commitment to the system from all persons responsible for selection of bridge replacement projects. This does not have to be a commitment to selection of bridge replacement projects based on the output of a "black box," but should be a commitment showing that the results from the BMS will be seriously considered as one important tool in the decision-making process. Because of the large amount of data required to implement a BMS, it is imperative that there be a commitment to the system.

The North Carolina system was used in this study. It was chosen because of its inherent flexibility and simplicity. Other BMSs could also be considered. However, the system selected should rank the bridges in a reasonable order with a minimum amount of data collection. Once the BMS has been selected, some interesting policymaking decisions can be made. It now becomes possible to set some long-term goals on the future configuration of the existing bridge system. For example, highway function classifications for the local agency road system can be defined. In some counties, a grid of high-capacity local roads at 3-mi intervals is being implemented. These roads will become major thoroughfares for the county and will have no posted bridges. In other cases, an existing system is working well and, with a stable environment, will not need to be changed.

Once the highway function classifications have been determined, it is time to set service goals for each classification. It appears that at the local level, any BMS should be flexible enough to accommodate local priorities and needs. When truck traffic that supports the local economy requires relatively high vertical clearances, it becomes desirable to pay particular attention to the vertical clearance goals. In other locations, posted bridges have a severe impact on the local economy. In these situations, load capacity goals should be given additional consideration. In western Kansas, clear deck width is a particular concern at the local level because of the machinery used in the production of wheat.

The last decision-making process is the adjustment of the weighting factors. This step is very important and could be a significant driver of bridge rankings. From the studies made for the county system studied, changing the load capacity and estimated remaining life weighting factors by 10 percent did not change the relative order of most bridges in the system. However, several individual bridges changed by approximately 10 ranking positions. The selection of the highway function classifications, level-of-service goals, and weighting factors will have a significant impact on the configuration of the bridge system in the future. Therefore, it is important to have a consensus about long-term objectives of future bridge systems. If all interested parties have contributed to the process of setting service goals, the entire organization could be working toward a common objective. As long as the objective remains the total bridge system, input from technical staff, politicians, and users is important in the development of BMS goals. Once the goals and policymaking decisions are made, it becomes time to collect the hard data about the entire bridge system in its current state. If the BMS is to be effective, it is imperative that accurate, consistent, and reliable data be available for each bridge. These BMS systems are inflexible with respect to missing or inaccurate bridge properties.

Fortunately, all of the information required to use the North Carolina system is available from the Structural Inventory and Appraisal (SI&A) forms currently required for all bridges (*6*). Because all bridges on the local system must currently be inspected every 2 yr, up-to-date bridge parameters should be available. However, it is suggested that if these data are used they should be carefully reviewed for consistency and accuracy.

After all bridge parameters are inserted into a data base, the analysis of the data and calculation of the ranking parameters would take place. This project demonstrated that microcomputers have sufficient computing power for use with local bridge systems.

After the bridges are ranked for DPs, or some cost factor, actual projects can be selected. Although the ranking of the bridges with the BMS is an important tool in the project selection process, it should not be used blindly. Other considerations such as funding sources, availability of plans, construction schedules, and so on, are important.

At the conclusion of the analysis required by the implementation of a BMS, a logical, justifiable bridge replacement program should result. This program will be developed based on existing bridge parameters that fairly compare one project with all other bridges in the system.

The setting of highway function classifications, level-of-service goals, and weighting factors should not be set once and never reevaluated. Conditions and needs do change and a periodic review of these parameters is appropriate. However, they should not be changed indiscriminately. To adjust the service goals so that the relative ranking of a particular project is improved or changed, for example, would defeat the purpose for implementing a BMS.

CONCLUSIONS

Through the analysis of a typical county bridge system, it was shown that the computing power of microcomputers is more than adequate for operating BMSs. The BMS developed for North Carolina was chosen for implementation in this project. The software was developed using dBASE III Plus data base management system. With appropriate programming, separate data bases containing weighting factors and level-of-services goals were developed. This way criteria could be changed without modifying the ranking program. This approach improves software flexibility and friendliness.

The bridge parameters from one local county system were thoroughly analyzed using several ranking criteria. With the baseline criteria, the bridge ranking was compared with the actual replacement program developed independently by the county. In general, the two approaches to the development of a bridge replacement program agreed closely. Where differences occurred, they could be explained by taking other factors into consideration. The time required to develop a bridge replacement program with the use of a BMS was significantly less than that required for the manual selection process.

Based on the results of this study, microcomputers provide a very good base for BMSs. Although improved productivity could be used when a BMS is implemented, collection of bridge parameter data could become a major effort. If the data on the SI&A forms are accurate and up-to-date, this effort would be minimized. The reliability of the results are dependent on the quality of the bridge parameter data. These data must be accurate and consistent if reliable results are to be obtained. Although this project evaluated bridges, it could be modified to include the culvert systems of local agencies. In most areas there are more culverts on the local system than there are bridges.

Therefore, the potential for additional productivity gains while setting replacement priorities would be greater for culvert systems than it would be for bridges.

ACKNOWLEDGMENT

The author wishes to express his sincere appreciation to Don Hovey and Rick Miller, Johnson County Public Works Department, Kansas, for their assistance in making the bridge parameter data available.

REFERENCES

1. *Bridge Management Systems*. Draft Report, FHWA-DP-71-01, Federal Highway Adminstration, U.S. Department of Transportation, Washington, D.C., March 1987.
2. David W. Johnston and Paul Zia. Level-of-Service System for Bridge Evaluation. In *Transportation Research Record 962*, TRB, National Research Council, Washington D.C., 1984, pp. 1–8.
3. *The Pennsylvania Bridge Management System*. Draft Final Report, Bureau of Bridge and Roadway Technology, Pennsylvania Department of Transportation, Harrisburg, Pa., Feb. 1987.
4. Wallace T. McKeel, Jr., and John E. Andrews. *Establishing a Priority of Funding for Deficient Bridges*. Virginia Department of Highways and Transportation, Richmond, Va., Feb. 1985.
5. *Interim Report on Bridge Maintenance, Rehabilitation, and Replacement Procedures*. Committee on Bridge Maintenance, Rehabilitation, and Replacement Procedures. Nebraska Department of Roads, Lincoln, March 1986.
6. *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges*. Federal Highway Administration, U.S. Department of Transportation, Washington, D.C., 1979.

Publication of this paper sponsored by Committee on Structures Maintenance.