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Preface

This publication contains papers presented at the 9th International Bridge Management Conference held at the Orlando Airport Marriott Hotel, April 28–30, 2003, in Orlando, Florida. The conference was sponsored by the Transportation Research Board in cooperation with FHWA. The objective of the conference was to provide a forum for the exchange of information about the state of the practice and state of the art in bridge management systems between practitioners and researchers in all levels of the public and private sectors. This publication contains papers on bridge management concepts, strategies and health indices, asset management, joints, coatings and concrete repair, life-cycle costs, load testing, utilizing the Internet and performance measures, deterioration and reliability, future directions and challenges in bridge management systems, management system implementation, safety and serviceability, scour modeling and experiences, expert systems and uncertainties, and concrete deterioration. The papers have not been subjected to the TRB peer review process.

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Supply and Demand System Approach to Developing Bridge Management Strategies
Comparison with Two Existing Approaches

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Optimal management strategies for a group of bridges that may incur damage from flooding during a 15-year period were determined using three approaches, and then the strategies were compared. The three approaches are the supply and demand system approach and two approaches used in existing bridge management systems: the supply bridge approach and the supply and demand bridge approach. The comparison investigates the approaches' abilities to determine optimal management strategies when multiple bridges are likely to be affected simultaneously. With probabilistic concepts, the likelihood of inadequate service resulting from the damage was determined. The comparison shows that when bridges may be adversely affected simultaneously, the use of the supply and demand system approach can result in increased savings as compared with both the supply bridge approach and the supply and demand bridge approach.

The approaches to developing bridge management strategies in existing bridge management systems (BMSs) have placed limitations on these systems' ability to find optimal management strategies. A source of these limitations is orientation toward performance of an individual bridge rather than performance of the transportation network as a whole. One result of the approaches' limitations is the inability to find optimal management strategies when multiple bridges may be adversely affected simultaneously, as a result of natural hazards such as floods and earthquakes.

A recently proposed supply and demand system (SDS) approach (*1*) can overcome this limitation. With the proposed SDS, optimal management strategy focuses on the ability of the network as a whole to provide an adequate level of service (LOS) and the expected additional costs (*EAC*) if an adequate LOS is not provided.

This paper briefly explains the SDS approach and compares it with two approaches that BMSs currently use: the supply bridge (SB) approach and the supply and demand bridge (SDB) approach. Optimal management strategies for a group of bridges that may incur damage from flooding during a 15-year period were determined using all three approaches, and the strategies were then compared. The comparison investigated the approaches' abilities to determine optimal strategies when multiple bridges were likely to be affected simultaneously. With probabilistic concepts, the likelihood of bridges' inadequate service resulting from the damage was determined.

OPTIMAL MANAGEMENT STRATEGIES

Optimal management strategies are defined in this paper as the order in which interventions are to be performed to minimize the *EAC* caused by inadequate performance over the next 15 years, for given amounts of resources. Interventions are to be performed immediately. Not considered were the different times at which interventions can be performed or further interventions due to the same hazard scenario for the investigated time period. This is because consideration of different intervention times adds a level of complexity to the calculations that is unnecessary to show the differences between the approaches. Loss of benefit, safety, and functionality is treated as cost. It is assumed that the agency costs of each intervention are the same, so that optimal strategies can be based solely on the possible reduction in the expected additional user costs. This simplifies the comparison of the approaches.

Optimal management strategies were found for single interventions (if only one intervention is to be performed) and multiple interventions (if multiple interventions are to be performed). A single-intervention optimal strategy determines the priorities for intervention if only one intervention is to be performed. For example, if the largest reduction in *EACs* is gained by reducing the probability of inadequate performance of Bridge 14, then it is Bridge 14 that should be repaired. However, if this action cannot be performed, the bridge whose repair will result in the second largest savings in *EAC* should be the point of intervention. The multiple-intervention optimal strategies are devised for situations in which more than one intervention is to be performed. For example, if only one intervention is performed, the largest *EAC* savings are achieved by reducing the probability of Bridge 14's inadequate performance, but if two bridges are to be repaired, the strategy determines, then the largest *EAC* savings are achieved by reducing the probability of inadequate performance of both Bridge 14 and Bridge 9.

APPROACHES TO DEVELOPMENT OF OPTIMAL MANAGEMENT STRATEGIES

The compared approaches to developing optimal management strategies were the SB approach, the SDB approach, and the SDS approach.

SB Approach

Optimal management strategies determined with the SB approach are based solely on the condition of the physical bridge. No consideration is given to user requirements (demand). This approach is taken in BMSs currently in use, such as Pontis (*2*), BRIDGIT (*3*), and KUBA-MS (*4*). In addition, only agency costs are considered, without consideration of expected societal costs attributed to lost safety or functionality. When agency cost is the sole decision criterion, the decision to perform an intervention is governed by the condition state of the bridge. In this paper the condition state of the bridge is defined as the probability of its inadequate service. The objective function is to minimize the collective probability of the structures' inadequate performance (Equation 1):

minimize

$$l = \sum_{i=1}^{n} P_i \tag{1}$$

where n is the number of bridges and P_i is the probability of inadequate performance of Bridge i as follows:

$$P_i = f_i\left(P_{oi}, res_i\right) \tag{1a}$$

where P_{oi} is the initial probability of inadequate performance of Bridge i, and res_i, is the resources allocated to Bridge i, subject to the constraint

$$\sum_{i}^{n} res_i \leq res_{tot} \tag{2}$$

where res_{tot} is the total resources allocated to the network, and subject to the nonnegativity condition

$$r_i \geq 0 \tag{3}$$

$$\overrightarrow{res} \in \overrightarrow{RES}$$

$$\vec{p} \in \vec{P}$$

SDB Approach

Optimal management strategies determined with the SDB approach are based on the ability of individual bridges to provide an adequate LOS and the *EAC* if an adequate LOS is not provided—bridge by bridge. The SDB approach does not explicitly take into consideration how the transportation network as a whole delivers an adequate LOS. The SDB approach is used in bridge management systems (*2*–*4*) when both agency and user costs are considered. The decision to perform an intervention is then governed by the condition state of the bridge (probability of inadequate service) and the expected reduction in functionality. Neglected is lost functionality resulting from simultaneously nonoperational bridges. In this investigation, the optimal strategies determined with the SDB approach were based on the effectiveness of interventions in reducing *EAC*. All monthly costs were compared by the equivalent worth method with a discount rate of 0—costs in each month of the investigated time period were considered equal. The objective function is to minimize *EAC* (Equation 4):

minimize

$$l = \sum_{i=1}^{n} EAC_i \tag{4}$$

where EAC_i is the *EAC* due to failure of Bridge i as follows:

$$EAC_i = f_i\left(P_{oi}, res_i, AC_i\right)$$

where AC_i is the additional user costs if Bridge i fails, subject to the constraint

$$\sum_{i}^{n} res_i \leq res_{tot} \tag{5}$$

and subject to the nonnegativity condition

$$res_i \geq 0 \tag{6}$$

$$\overrightarrow{eac} \in \overrightarrow{EAC}$$

SDS Approach

Optimal management strategies determined with the SDS approach are based on the ability of the network as a whole to provide an adequate LOS and the *EAC* if adequate LOS is not provided. Researchers have previously shown that network considerations can alter management strategies (*5–8*). The SDS approach explicitly takes into consideration how the transportation network as a whole delivers an adequate LOS. In this investigation, the optimal strategies determined with the SDS approach were based on the effectiveness of interventions in reducing *EAC*. All monthly costs were compared using the equivalent worth method with a discount rate of 0. The objective function in the SDS approach is to minimize *EAC* (Equation 7):

minimize

$$l = \sum_{i=1}^{n} EAC_i \tag{7}$$

where n is the total number of network condition states to be investigated, and

$$EAC_i = f_i\left(P_j^o, res_j, AC_i\right)$$

where

P_j^o = initial probability of inadequate performance of all j bridges in the network,

res_j = resources allocated to each j bridge in the network, and

AC_i = additional costs if a certain combination of bridges are nonoperational, a Network Condition State i,

subject to the constraint

$$\sum_{i=1}^{n} res_i \leq res_{tot} \tag{8}$$

and subject to the nonnegativity condition

$$res_i \geq 0 \tag{9}$$

STRATEGY FOR COMPARISON OF APPROACHES

To compare the SDS approach with the SB and the SDB approaches, optimal management strategies were determined for flood conditions for five bridges crossing a river system in the Morogoro region of Tanzania (Figure 1) (*1*). This multiple-bridge system offered the opportunity to compare the approaches' abilities to determine optimal strategies when multiple bridges may be adversely affected simultaneously.

The bridges' physical conditions were described with probabilistic concepts—a certain probability the bridge will perform inadquately over the next 15 years, ranging from 1 for certainty the bridge will perform inadequately to 0 for certainty the bridge will perform inadequately. The optimal management strategies were determined with the assumptions that the interventions were to be performed immediately and that the probability was neglible that the bridge would again perform inadequately for the duration of the investigated time period after the intervention. As only one bridge per link was investigated in the studied network, the bridges are referred to by their link number. For example, Bridge 14 is the bridge in Link 14. Table 1 shows the probabilities of the bridges' inadequate performance. The probabilities of inadequate performance of Bridges 6, 7, 8, and 9 were assumed to be 10^{-1} and of Bridge 14 were assumed to vary between 10^{0} and 10^{-3}. The failures were assumed to be perfectly correlated in time. These orders of magnitude of probability of inadequate performance were chosen to show the range of effects between the approaches from a maximum probability of inadequate performance (P = 10^{0}) to a minimum probability of inadequate performance ($P = 10^{-3}$). Smaller probabilities of inadequate performance are not shown, because the differences between approaches for this bridge network were negligible after $P = 10^{-3}$. Optimal management strategies were determined for single interventions and multiple interventions.

The *EAC* due to inadequate performance for the network condition states was estimated by simulating the traffic flow on both the fully operational and partially operational network on

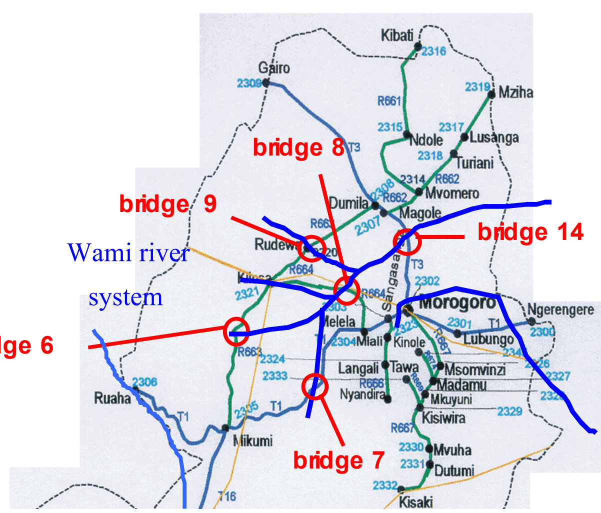

FIGURE 1 Location of the five investigated bridges (6, 7, 8, 9, and 14) in Tanzania.

TABLE 1 Probabilities of Inadequate Service for Determining Optimal Management Strategies for Each of Five Bridges

Bridges	Probability	Bridge	Probability
6, 7, 8, 9	10^{-1}	14	10^{0}
6, 7, 8, 9	10^{-1}	14	10^{-1}
6, 7, 8, 9	10^{-1}	14	10^{-2}
6, 7, 8, 9	10^{-1}	14	10^{-3}

the basis of established relationships between vehicle operating, travel time, and accident costs, and between traffic flow, road condition, and vehicle speed. Lost benefits if not all desired travel was possible were also determined (*9*).

EXAMPLE OPTIMAL MANAGEMENT STRATEGIES

SB Approach

The optimal management strategies using the SB approach are shown in Tables 2 and 3. There was no difference between the order of intervention in the single- and multiple-intervention strategies.

SDB Approach

The optimal management strategies using the SDB approach are shown in Tables 4 and 5. The order of intervention depends on the probability that Bridge 14 service will be inadequate, if the probabilities of inadequate service from Bridges 6, 7, 8 and 9 are equal ($P = 10^{-1}$), as expected. If no interventions are performed, the *EAC* in monetary units (mu) is as follows for the probabilities of inadequate performance of Bridge 14:

Probability	***EAC* (mu)**
10^{0}	32.1×10^{6}
10^{-1}	69.6×10^{3}
10^{-2}	414.0×10^{3}
10^{-3}	388.0×10^{3}

Changing the probability of inadequate service of Bridge 14 reduced the *EAC*, and the order of intervention in both the single- and multiple-intervention optimal strategies changed accordingly. There is no difference between the order of intervention in the single- and multiple-intervention optimal management strategies.

SDS Approach

The optimal management strategies using the SDS approach are shown in Tables 6–9. If no interventions are performed, the *EAC* in mu is as follows for the probabilities of inadequate performance of Bridge 14:

Probability	***EAC* (mu)**
10^{0}	5.31×10^{6}
10^{-1}	69.6×10^{3}
10^{-2}	505.0×10^{3}
10^{-3}	461.0×10^{3}

TABLE 2 Single and Multiple Optimal Management Strategies with SB Approach (Bridge 14 Probabilities 10^0 and 10^{-1})

	Bridge 14: Probability = 10^0		Bridge 14: Probability = 10^{-1}	
Bridge	**Ranking/Sequence**	**Probability**	**Ranking/Sequence**	**Probability**
6	2	10^{-1}	1	10^{-1}
7	2	10^{-1}	1	10^{-1}
8	2	10^{-1}	1	10^{-1}
9	2	10^{-1}	1	10^{-1}
14	1	10^0	1	10^{-1}

TABLE 3 Single and Multiple Optimal Strategies with SB Approach (Bridge 14 Probabilities 10^{-2} and 10^{-3})

	Bridge 14: Probability = 10^{-2}		Bridge 14: Probability = 10^{-3}	
Bridge	**Ranking/Sequence**	**Probability**	**Ranking/Sequence**	**Probability**
6	1	10^{-1}	6	1
7	1	10^{-1}	7	1
8	1	10^{-1}	8	1
9	1	10^{-1}	9	1
14	2	10^{-2}	14	5

TABLE 4 Single and Multiple Optimal Management Strategies with SDB Approach (Bridge 14 Probabilities 10^0 and 10^{-1})

	Bridge 14: Probability = 10^0		Bridge 14: Probability = 10^{-1}	
Bridge	**Ranking/Sequence**	**Reduction in *EAC* (mu)**	**Ranking/Sequence**	**Reduction in *EAC* (mu)**
6	5	10.0×10^3	5	10.0×10^3
7	2	323×10^3	1	323×10^3
8	4	11.0×10^3	4	11.0×10^3
9	3	42.0×10^3	3	42.0×10^3
14	1	2.82×10^6	2	282×10^3

TABLE 5 Single and Multiple Optimal Management Strategies with SDB Approach (Bridge 14 Probabilities 10^{-2} and 10^{-3})

	Bridge 14: Probability = 10^{-2}		Bridge 14: Probability = 10^{-3}	
Bridge	**Ranking/Sequence**	**Reduction in *EAC* (mu)**	**Ranking/Sequence**	**Reduction in *EAC* (mu)**
6	5	10.0×10^3	4	10.0×10^3
7	1	323×10^3	1	323×10^3
8	4	11.0×10^3	3	11.0×10^3
9	2	42.0×10^3	2	42.0×10^3
14	3	28.2×10^3	5	2.82×10^3

TABLE 6 Optimal Management Strategies with SDS Approach (Bridge 14 Probability at 10^0)

	Single Intervention		Multiple Intervention	
Bridge	**Ranking**	**Reduction in *EAC* (mu)**	**Sequence**	**Reduction in *EAC* (mu)**
6	5	53.0×10^3	5	10.0×10^3
7	3	686×10^3	2	387×10^3
8	4	174×10^3	4	13.5×10^3
9	2	1.51×10^6	3	45.3×10^3
14	1	4.85×10^6	1	4.85×10^6

TABLE 7 Optimal Management Strategies with SDS Approach (Bridge 14 Probability at 10^{-1})

	Single Intervention		Multiple Intervention	
Bridge	**Ranking**	**Reduction in *EAC* (mu)**	**Sequence**	**Reduction in *EAC* (mu)**
6	4	62.4×10^3	5	10.0×10^3
7	2	417×10^3	2	387×10^3
8	5	48.4×10^3	4	13.5×10^3
9	3	198×10^3	3	45.3×10^3
14	1	485×10^3	1	485×10^3

TABLE 8 Optimal Management Strategies with SDS Approach (Bridge 14 Probability at 10^{-2})

	Single Intervention		Multiple Intervention	
Bridge	**Ranking**	**Reduction in *EAC* (mu)**	**Sequence**	**Reduction in *EAC* (mu)**
6	3	63.3×10^3	5	10.0×10^3
7	1	390×10^3	1	390×10^3
8	5	35.8×10^3	4	13.5×10^3
9	2	66.2×10^3	2	60.8×10^3
14	4	48.5×10^3	3	30.0×10^3

TABLE 9 Optimal Management Strategies with SDS Approach (Bridge 14 Probability at 10^{-3})

	Single Intervention		Multiple Intervention	
Bridge	**Ranking**	**Reduction in *EAC* (mu)**	**Sequence**	**Reduction in *EAC* (mu)**
6	2	63.4×10^3	4	10.0×10^3
7	1	388×10^3	1	388×10^3
8	4	35.8×10^3	3	13.7×10^3
9	3	53.0×10^3	2	$46.9 \times 10_3$
14	5	4.85×10^3	5	2.8×10^3

The order of intervention depends on the probability that Bridge 14 service is inadequate, if the probabilities of inadequate service from Bridges 6, 7, 8 and 9 are equal ($P = 10^{-1}$), as expected. The SDS approach, however, shows that it is not necessarily most beneficial to repair the bridge with the highest probability of inadequate service. For example, when the probability of inadequate service of Bridge 14 is 10^{-2} (Table 8) it is still more beneficial to repair Bridge 14 as a single intervention than it is to repair Bridge 8 ($P = 10^{-1}$). As a multiple intervention, it is more beneficial to repair Bridge 14 than either Bridge 8 or Bridge 6 ($P = 10^{-1}$).

Comparison of Tables 7 and 8 shows that the reduction of the probability of inadequate service of one bridge may increase the possible reduction in *EAC* by decreasing the probability of inadequate service of another bridge. When the probability of inadequate service of Bridge 14 is decreased from 10^{-1} to 10^{-2}, the possible reduction in *EAC* for the repair of Bridge 6 actually increases (62.4×10^3 to 63.3×10^3 mu). This is because when the probability is decreased that all five bridges (Bridges 6, 7, 8, 9, and 14) are simultaneously nonoperational, the probability that only the four remaining bridges (Bridges 6, 7, 8, and 9, if Bridge 14 is repaired) are simultaneously nonoperational increases. Whether or not this increases the possible reduction of *EAC* because of the repair of a single bridge depends on the additional costs associated with each network condition state (e.g., the network condition state for Bridges 6, 7, 8, 9, and 14 as opposed to the network condition state of Bridges 6, 7, 8, and 9).

DISCUSSION AND COMPARISON

Optimal Management Strategies Determined with SDS Approach

Comparison of the optimal management strategies determined with the SDS approach shows:

- The reduction of *EAC* for each intervention does not depend on whether the intervention is performed as a single or one of multiple interventions, if the probability is negligible that the bridges will be simultaneously nonoperational.
- The reduction of *EAC* for each intervention does depend on whether the intervention is performed as a single or one of multiple interventions, if the probability is not negligible that the bridges will be simultaneously nonoperational. This difference in the reduction of *EAC* diminishes with decreased probabilities of inadequate service.
- It is not necessarily most beneficial to repair the bridge with the highest probability of giving inadequate service, as that bridge does not necessarily have the largest *EAC*.
- Reducing the probability of one bridge's inadequate service may increase the possible reduction in *EAC* by repairing of another bridge. This is because *EAC* is associated with network condition states, and the repair of a single bridge alters the probabilities of more than one network condition state.

SB Versus SDS Approach

The single- and multiple-intervention optimal management strategies determined with the SB and SDS approaches are different (see Tables 2 and 3, and Tables 6 and 9). For example, when Bridge 14 has a probability of inadequate service of 10^{-2} and Bridges 6, 7, 8, and 9 have probabilities of inadequate service of 10^{-1}, the multiple-intervention strategy with the SDS approach (Table 8) indicates that if three bridges are to be repaired, Bridge 14 should be the third. The SB approach (Table 3), however, indicates it should not be repaired. Comparison of the optimal management strategies shows

- If there is a larger variation in *EAC* than the variation between the probabilities of inadequate service, the SDS approach gives a clearer distinction between the bridges than the SB approach when there is, or is not, a negligible probability of simultaneous failures (when more than one bridge does not provide an adequate LOS at the same time). This is not necessarily a benefit of the SDS approach.
- Although no difference between the optimal management strategies determined with the SB and SDS approaches was observed when there is a negligible probability of simultaneous failures, there would be a difference in other circumstances. For example, if the SDS approach is taken, if Bridge A has high *EAC* associated with its inadquate performance (10^9) but a low probability of inadequate service (10^{-3}), then it will be selected over Bridge C, which has lower *EAC* associated with its inadequate performance (10^6) and a slightly higher probability of inadequate performance (10^{-2}). This is because *EAC* of Bridge A is 10^6 (10^9 x 10^{-3}), and *EAC* of Bridge C is 10^4 (10^6 x 10^{-2}), and the SDS approach suggests that the bridge to be repaired is the one that can most reduce *EAC*: ($10^6 > 10^4$). The SB approach, however, will suggest that the bridge with the highest probability of inadequate performance be repaired, regardless of the costs of inadequate performance. In this case, Bridge C would be repaired because $10^{-2} > 10^{-3}$.
- There is a difference in the optimal management strategies determined using the SB and SDS approaches when probability of simultaneous failures is not negligible.

SDS Versus SDB Approach

There are differences between the possible reductions in *EAC* determined using the SDS and SDB approaches for both the single- and multiple-intervention optimal management strategies (Tables 4 and 5 and Tables 6 through 9). For example, when the probability of inadequate service of Bridge 14 is 10^0 and the SDS approach is taken to determine the single-intervention optimal management strategies (Table 6), Bridge 9 (1.5 x 10^6 mu) is ranked second, and Bridge 7 (686 x 10^3 mu) is ranked third. If the SDB approach is taken (Table 4), Bridge 7 (323 x 10^3) is ranked second, and Bridge 9 (42 x 10^3 mu) is ranked third. In another example, when the probability of Bridge 14 is 10^{-1} and the SDS approach is taken to determine the single-intervention optimal management strategies (Table 7), Bridge 14 (485 x 10^3 mu) is ranked first, and Bridge 7 (417 x 10^3 mu) is ranked second. If, however, the SDB approach is taken (Table 4), Bridge 7 (323 x 10^3 mu) is ranked first, and Bridge 14 (282 x 10^3 mu) is ranked second.

Savings

To illustrate the financial significance of using the SDS approach to determine the optimal management two examples are shown. In both, it is assumed that *EAC* as estimated with the SDS approach is correct. In the first example, the optimal management strategies are determined when probabilities of inadequate service are 10^{-1} for Bridges 6, 7, 8, and 9 and 10^{-2} for Bridge 14. The savings are shown in Table 10. If two interventions were to be performed, using the SDS approach rather than the SDB approach would not result in any additional savings. However, savings would be between 16 x 10^3 mu and 396 x 10^3 mu, depending on which bridges were repaired, with the SDS approach rather than with the SB approach. With the SB approach, there is a still a choice between the bridges that are going to be repaired because the bridge conditions are the same.

In the second example, the optimal management strategies are determined for the case in which Bridges 6, 7, 8, 9, and 14 have probabilities of inadequate service of 10^{-1}. The savings are shown in Table 11. Only one intervention is to be performed. The additional savings by using the

TABLE 10 Savings with SDS Approach

Approaches	Bridges Repaired	Savings (mu)	Difference from SDS Approach (mu)
SDS	7, 9, 14	480×10^3	N/A
SB	6, 8, 9 or 7, 8, 9	464×10^3 or 84.3×10^3	16.0×10^3 or 396×10^3
SDB	7, 9, 14	480×10^3	0

TABLE 11 Savings with SDS Approach

Approaches	Bridges Repaired	Savings (mu)	Difference from SDS Approach (mu)
SDS	14	485×10^3	N/A
SB	6 or 7 or 8 or 9 or 14	62.4×10^3 (min) 485×10^3 (max)	423×10^3 (max) 0 (min)
SDB	7	417×10^3	68×10^3

SDS approach when compared with the SB approach are between 0 and 423×10^3 mu, depending on the bridge chosen for repair. The additional savings with the SDS approach when compared with the SDB approach are 68×10^3 mu.

Given the assumptions about the costs of intervention and the societal costs of inadequate performance, the savings shown in the examples are significant. The costs associated with network condition states with multiple failures were not estimated to be vastly different than those with single failures. In reality, multiple failures are usually caused by natural disasters, such as flooding or earthquakes, that result in sudden and unpredictable failures and disrupt traffic flow for relatively long periods of time. If the costs due to multiple failures are larger than those estimated, then the SDS approach would result in greater savings than the SB and SDB approaches.

CONCLUSIONS

- The SB and the SDB approaches regularly determine different optimal management strategies than those determined with the SDS approach.
- The SDB and SDS approaches generate the same optimal management strategies for bridges if there is a negligible probability of simultaneous bridge failures, and different optimal management strategies for bridges if the probability of simultaneous bridge failures is not negligible.
- To gain the maximum benefits, the bridge with the highest probability of inadequate performance is not always the bridge to be chosen for repair.
- The determination of optimal management strategies for bridges that may be adversely affected simultaneously requires taking the SDS approach.
- The use of the SDS approach can result in increased savings over the use of the SB and SDB approaches when bridges may be adversely affected simultaneously.

REFERENCES

1. Adey, B. A Supply and Demand System Approach to the Development of Bridge Management Strategies. Thesis 2519. Swiss Federal Institute of Technology, Lausanne, Switzerland, 2002.
2. Cambridge Systematics, *Pontis Release 3.1—Users Manual.* AASHTO, Washington, D.C., June 1996.
3. National Engineering Technology Cooperation. *Bridgit Bridge Management System: Technical Manual-Version 1.00*. NCHRP Project 12-28(2)A. TRB, National Research Council, Washington, D.C., December 1994.
4. Ludescher, H., and R. Hajdin. *KUBA-MS-Ticino Benützerhandbuch*. Swiss Federal Highway Authority, Eidgenössische Drucksachen und Materialzentrale, Berne, Switzerland, 1999.
5. Dalziell, E., and N. Nicholson. Risk and Impact of Natural Hazards on a Road Network. *Journal of Transportation Engineering*, Vol. 127, No. 2, 2001, pp.159–166.
6. Minami, M. The Highway Network Structure Design Assuring Connectivity. *Proc., 16th Congress of IABSE, Lucerne Switzerland* (CD-ROM), International Association for Bridge and Structural Engineering, Sept. 11–14, 2000.
7. Augusti, G., and M. Ciampoli. Optimal Distribution of Seismic Upgrading Interventions to Maximize the Expected Flow in a Road Network. *Reliability and Optimization of Structural Systems* (D. Frangopol, R. Corotis, and R. Rackwitz., eds.), Permagon, 1997, pp. 85–92.
8. Augusti, G., Borri, A., and M. Ciampoli, Optimal Allocation of Resources in Reduction of Seismic Risk of Highway Networks. *Engineering Structures*, Vol.16, No.7, 1994, pp. 485–497.
9. Watanatada, T., W. D. O. Paterson, A. Bhandarid, C. Harral, A. M. Dhareswar, and K. Tsunokawa. *The Highway Design and Maintenance Standard Model, Vols. 1 & 2*. Published for the World Bank by Johns Hopkins University Press, Baltimore, Md., 1987.

Life-Cycle Management of Concrete Infrastructures for Improved Sustainability

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The LIFECON Life-Cycle Management System (LMS) is a European model of a predictive and integrated LMS for concrete infrastructures. It is generic in nature and will be suitable for different categories of structures, from buildings, bridges, and nuclear power plants, to lighthouses. LIFECON LMS has two planning phases: long term and short term. The system is also divided into three levels of structural hierarchy: component and module, object, and network. The component- and module-level system addresses structural components such as beams and columns and their combinations in modules. The object-level system deals with complete structures or buildings. The network-level system treats networks of objects such as stocks of bridges or buildings. The LIFECON LMS is capable of considering the many requirements and aims involved in decision making. Besides a structure's observed condition and evaluated urgency of repair, many other requirements should be taken into account, such as life-cycle costs, user costs, minimum requirements of structural performance, structural risks, traffic and other operational requirements, aesthetics, environmental risks, and ecological pressures. Thus, the method of multiple-attribute planning is applied on all hierarchical levels of the system. The life-cycle analysis and optimization module involves the data applications for studying the economy of the life cycle and cost-effectiveness of optional maintenance, repair, and rehabilitation strategies. Alternative strategies are compared as life-cycle activity profiles over a defined time frame. The purpose of life-cycle analyses is to find the optimal activity profiles to reach the targets.

Civil engineering infrastructures represent about 70% of the national property in European societies. Operation, maintenance, repair, rehabilitation, and renewal of this infrastructure are consuming about 35% of all energy and represent 30% of all environmental burdens and wastes. The influence of business buildings on organizations' work productivity and on people's safety and health is important. The number of deteriorating concrete civil engineering structures and buildings is constantly growing with great impacts on resources, environment, human safety, and health. At the current time, maintenance, repair, and rehabilitation (MR&R) of these structures are reactive. The need for MR&R is usually realized when deterioration has reached a very advanced stage, which causes huge investments in repair measures or even the need for demolition. The European LIFECON project aims to address all these needs with a life-cycle management system (LMS).

LIFECON project belongs to the European Union Competitive and Sustainable Growth Programme as a Shared-Cost Research and Technology Development project of the Fifth Research and Technological Development Framework Programme. The project's duration is 3 years, from January 1, 2001, to December 31, 2003.

FRAMEWORK OF LIFECON LMS

General Characteristics

LIFECON LMS is a predictive and integrated LMS for concrete infrastructures. The system makes it possible to organize and implement in an optimized way all activities related to planning, constructing, maintaining, repairing, rehabilitating, and replacing structures, taking into account safety, serviceability, economy, ecology, and other aspects of life-cycle planning. LIFECON LMS is generic in nature and thus is suitable for different categories of structures, such as buildings, bridges, nuclear power plants, and lighthouses (*1*).

LIFECON LMS consists of three separate systems: the object-level system, the network-level system, and the network- and object-level system. This LMS structure gives the owner the possibility of choosing the system that will best suit the organization's needs and requirements. The three systems are described as follows:

- The object-level system is designed for companies and organizations that own only a limited number of concrete infrastructures. It is a practically oriented system that helps an organization plan and execute MR&R projects on the basis of data from inspection and condition assessments. It generates proposals for MR&R actions with optimized timing, composition of actions as a part of project planning, and annual project programs for infrastructure networks.
- The network-level system is designed for national road administrations and other organizations that are responsible for upkeep of a large network of concrete infrastructures. The network-level system is a tool for operative planning and decision making on the administrative level. It makes it possible for administration to evaluate necessary funding for MR&R activity and optional maintenance strategies.
- The network- and object-level system integrates the first two levels. Through a special interface system the optimal work programs produced by the object-level system can be compared and harmonized with the network-level optimum.

LIFECON LMS answers the typical needs and requirements of governmental organizations in the context of economic analysis and justification of maintenance decisions on infrastructures and supplies:

- Economic justification of decisions,
- Objective basis for decisions on engineering and economic grounds,
- Strategic guidelines for asset preservation,
- Determination of medium- and long-term objectives and appropriate maintenance strategies for achieving them,
- Optimized MR&R strategies based on engineering and economic grounds,
- Selection of justifiable maintenance decisions within budget constraints,
- Demonstration of value for money in infrastructure provision and maintenance,
- Integration of allocation of funds, and
- Evaluation of whole-life costing, including user costs.

Decision Making

LIFECON LMS is divided into three levels of structural hierarchy:

- Component and module level, which addresses structural components such as beams and columns and their combinations (i.e., modules);
- Object level, which deals with complete structures or buildings such as bridges and nuclear power plant units; and
- Network level, which treats networks of objects such as stocks of bridges or buildings.

Figure 1 describes the process of decision making at these three levels. The loop starts from the network level, at which the definition of strategic targets and general rules for maintenance policy are supported by life-cycle analyses and optimization.

MR&R actions are specified at the component level by using data related to specific components or modules and the recommended life-cycle action profiles (LCAPs). The timing of both repair actions and preventive maintenance actions is predicted by means of degradation models.

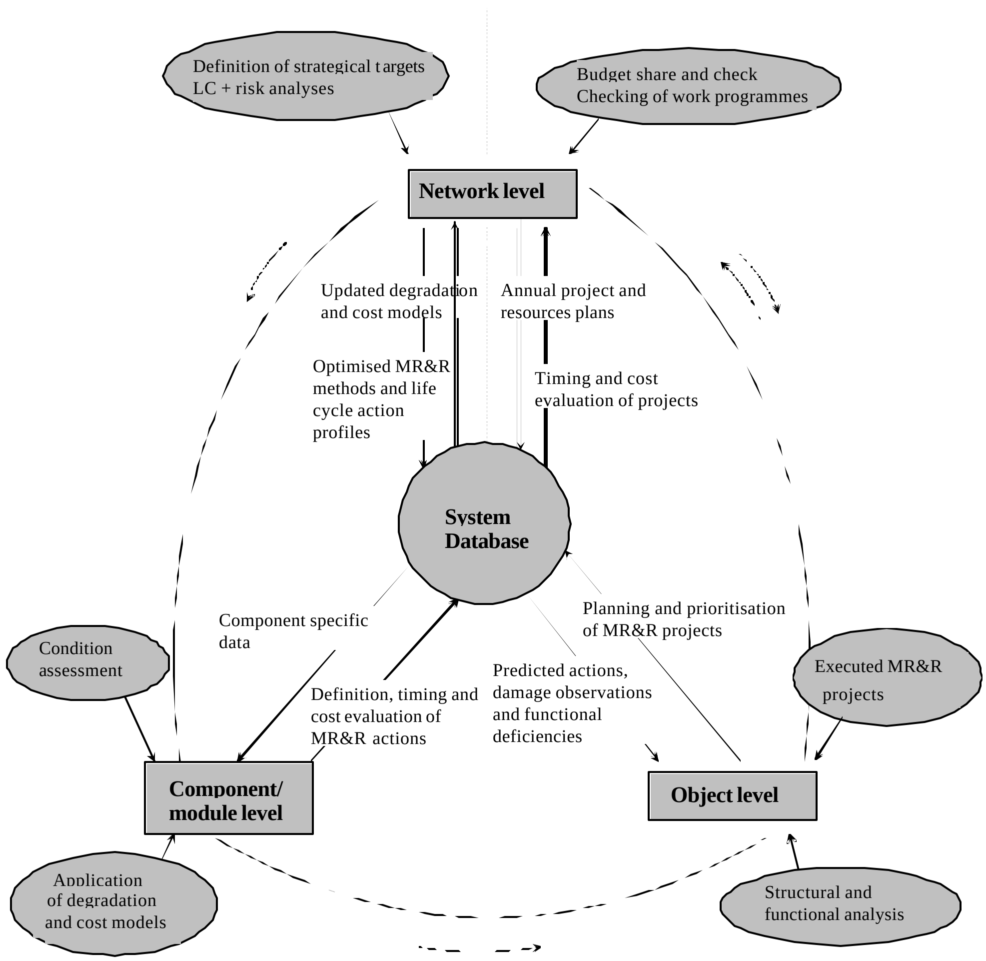

FIGURE 1 Three levels of decision making in LIFECON LMS.

Actions of the component level are gathered at the object level into projects, which are sorted and prioritized. Lastly the annual project and resources plans—the work programs—are prepared at the network level, where budget limits are also taken into account.

LIFECON LMS enables organizations to evaluate future needs for repair. This is possible through degradation models that assess structures' degradation. With the assessments from the degradation models and life-cycle analyses, organizations can identify the optimal MR&R actions and their optimal timing.

The planning involves two phases in the LIFECON LMS: preliminary and final. At the preliminary phase the long-term performance of structures is analyzed on the basis of data from degradation models and general inspection. In this way, a rough evaluation of the future MR&R needs and costs can be produced for up to 10 years. The potential objects for future MR&R projects are also identified through special inspections and structural analyses.

In the final phase, short-term analyses are performed on the basis of special inspection data. In this phase, the evaluation of MR&R needs and costs is more reliable than that obtained in the preliminary phase.

A feature emphasized in LIFECON LMS is the consideration of many requirements and aims in the process of making decisions on MR&R activity. In most systems, decisions are made only on the basis of the condition of the structures and the observed urgency of repair. In LIFECON LMS, other aspects—such as life-cycle costs, user costs, delay costs, minimum requirements of structural performance, structural risks, traffic and other operational requirements, aesthetics, environmental risks, and ecological pressures—can also be taken into account.

Definition of Strategic Targets

The infrastructure owner has responsibility for maintaining structures in good condition. A special maintenance policy is needed to ensure the safety level of the structures and also preserve the assets. When effective, the maintenance policy is the target-oriented practice of an organization for the upkeep of its building stock. This policy comprises the targets and rules that should be considered in all the organization's MR&R activities. The targets signify not only technical and functional needs but also needs such as effective use of funds, ecological considerations, and the structure's cultural value.

The purpose of an LMS is to ensure in practice that the strategic targets are taken into account in all decision making related to MR&R at the network, object, and component and module levels, within funding constraints. In some cases special tools, called multiple attribute decision aids, are required to balance conflicting requirements. In LIFECON LMS, needs can be prioritized by assigning weight factors for different attributes in decision making. An example of this weighting process is given in Table 1.

TABLE 1 Example of Target Definition

Target	Weight (%)
Life-cycle economy (agency costs)	20
User economy	20
Ecology	10
Aesthetics	10
Mitigation of risks	20
Preservation of cultural values	10

STRUCTURE OF LIFECON LMS

General Description

The main processes of LIFECON LMS are

- Administrational inputs,
- Analyses,
- Planning of MR&R actions,
- Planning of projects,
- Preparation of annual work plans, and
- Data collection and modeling.

Figure 2 presents a detailed flow diagram of the LIFECON LMS network and object—level. The boxed entities are system modules. The main relationships between modules are marked with arrows. The boxes plotted with a dotted line may or may not be included in the actual management system, as these activities may also be considered as separate sectors of MR&R activity. From the point of view of the system, however, the information produced by these modules is vital.

The combined network and object-level of LIFECON LMS has an advanced feature: a special interface between the object and network levels to ensure the overall optimality of work programs. It contains tools for checking, comparing, and harmonizing the MR&R work distribution generated by object-level planning process with the optimal work distribution generated by the network-level analyses.

Harmonizing the object-level work distributions with the network-level optimum is the link that comprises the network- and object-level system. The work programs produced by the object-level system are already in many ways optimized. However, through the link to the network-level system, the overall optimality can be checked. The handshake between the object- and network-level processes also checks the reliability of the network-level planning.

The object-level work program and the network-level optimum are harmonized by first recalculating the MR&R action distributions from the proposed object-level work program on the basis of the same modular breakdown used in the network-level short-term analysis. If a clear difference is observed between the proposed and the optimal distributions, the object-level work program is altered by changing the distribution of funding in budget categories or by changing the priority of objects in budget categories, or both. Thus, a combination of projects that better fit with the network-level optimum is searched. The internal composition of projects is not normally changed. The revised work—project and resources plan—is then returned to the object level.

Analyses

Life-Cycle and Risk Analysis

The life-cycle and risk analysis module involves the data applications for studying a structure's life-cycle economy and the cost-effectiveness of optional MR&R strategies. Alternative strategies, in the form of LCAPs, are compared over a defined time frame. LCAPs include both protective maintenance actions and heavy repair actions.

The purpose of life-cycle analyses and optimization is to find the most advantageous MR&R actions and LCAPs according to the given strategic targets. Risk analyses are performed to uncover possible risks associated with MR&R actions.

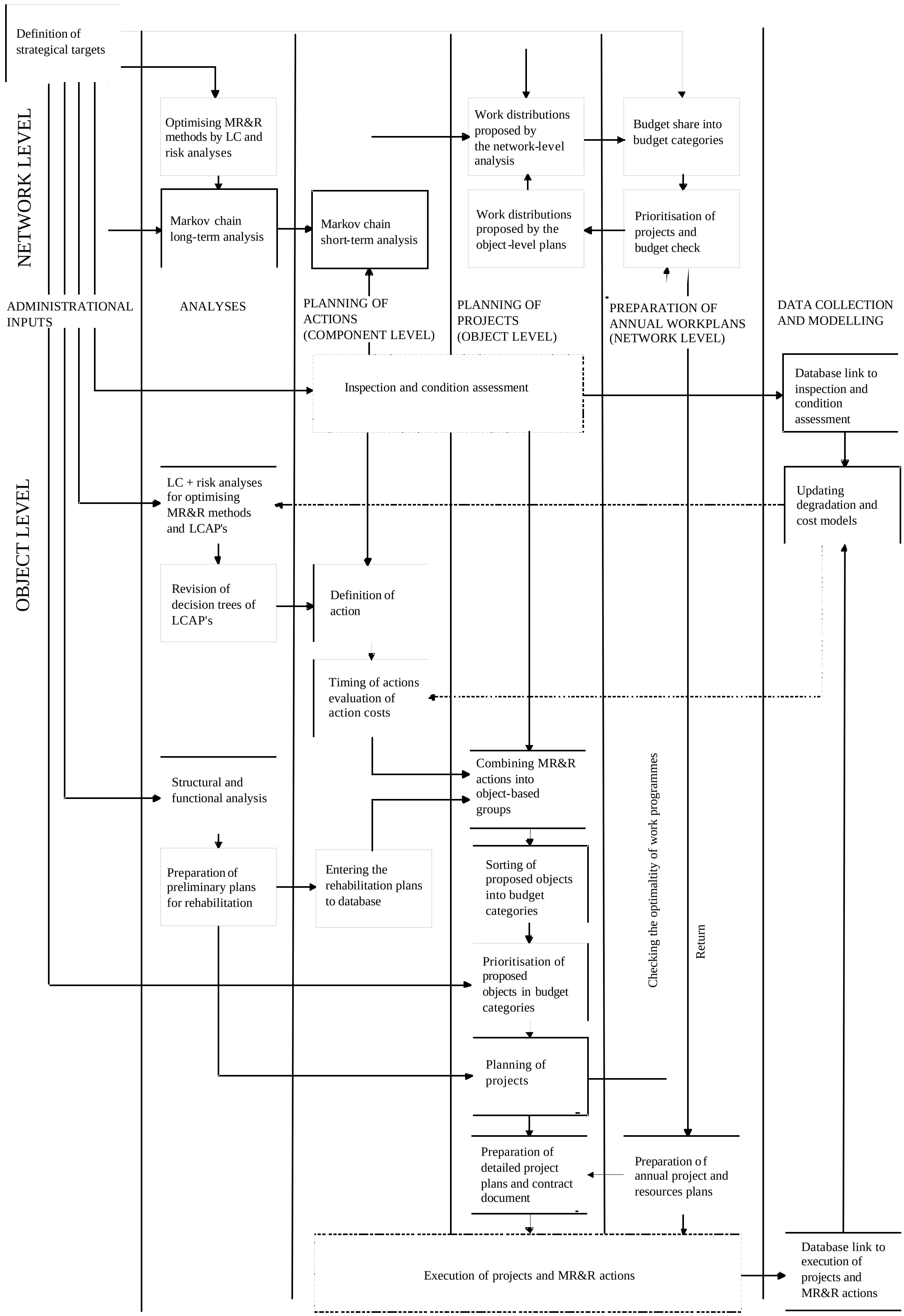

FIGURE 2 Detailed flow diagram of the network- and object-level LIFECON LMS.

A main aim of the LC analyses is to study the cost-effectiveness of coatings and other protective methods on structures. Protective maintenance methods reduce the degradation rate of structures but increase the maintenance costs between heavy repair actions.

Two alternative LCAPs are presented in Figure 3. The upper profile shows the degree of degradation over time when only heavy repair actions are performed in a given time frame. The lower profile shows the retarding effect of protective maintenance actions on the rate of degradation, with the result that only one heavy repair action is needed within the same time frame.

The life-cycle analysis is performed in such a way that the most cost-effective LCAPs in different environmental conditions can be identified. The analysis methods can also be modified for addressing ecological pressures if ecological data for optional repair methods and materials are available. Possible risks related to MR&R actions are also considered.

The methods for life-cycle analyses are essentially the same for the network level as for the object level. Life-cycle analyses can be combined with the network-level long- and short-term analyses so that MR&R methods are first optimized by life-cycle analysis for each condition state. The optimal condition states for triggering repair actions can then be solved by the long-term analysis.

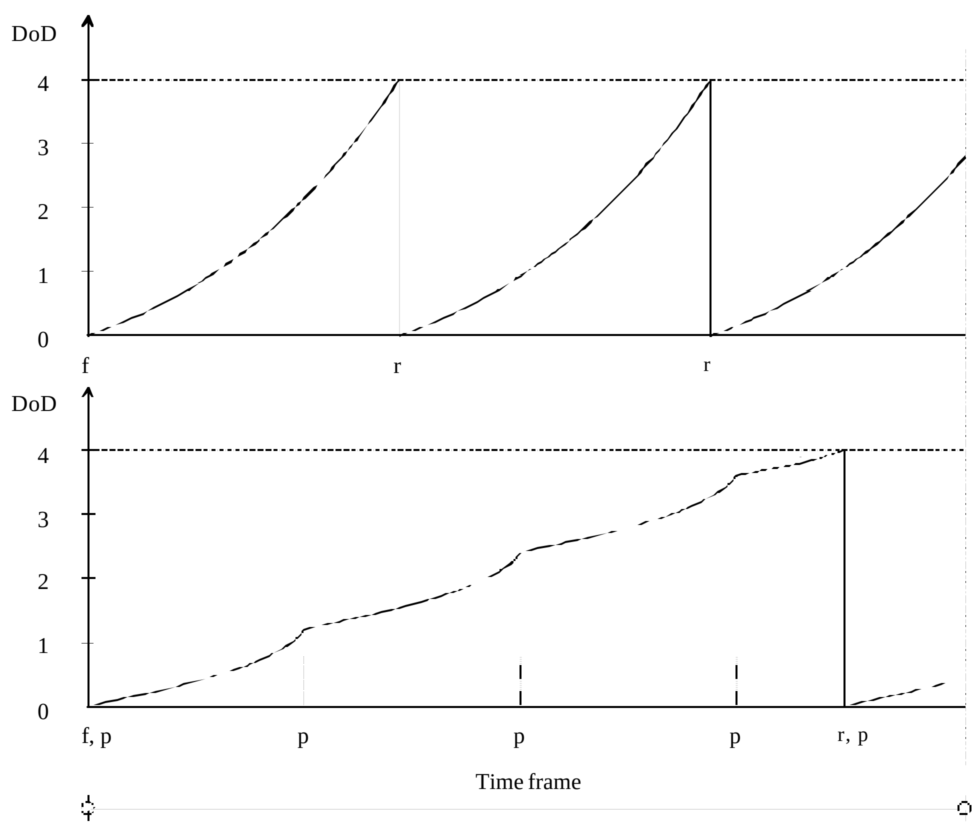

FIGURE 3 Comparison of two life-cycle activity profiles. (f = fabrication, r = repair action, p = protective maintenance action, and DoD = degree of damage.)

Decision Trees of LCAPs

The results of the life-cycle analyses for different environmental conditions with various material and structural parameters on the object level are presented in the form of a decision tree. The recommended optimal LCAP depends on parameters of materials, structure, environment, and condition. An LCAP defines

- Immediate actions to be performed once,
- Preventive maintenance actions to be performed periodically,
- Repair actions to be performed periodically,
- Conditions states at which the actions are performed, and
- Unit costs of protective maintenance and repair actions.

Decision trees are constructed on the basis of results of life-cycle analyses. Criteria at the branches of the tree are those found decisive in the analyses. The LCAP for recommendation can be obtained by going through the decision tree. The tree is checked by experts who evaluate the decisions it generates. Once the decision trees have been constructed, they are modified only when new or changed strategic targets are given by the administration.

Decision trees will specify the recommended MR&R actions for the future. As an example, Figure 4 shows a decision tree for the edge beams of a bridge.

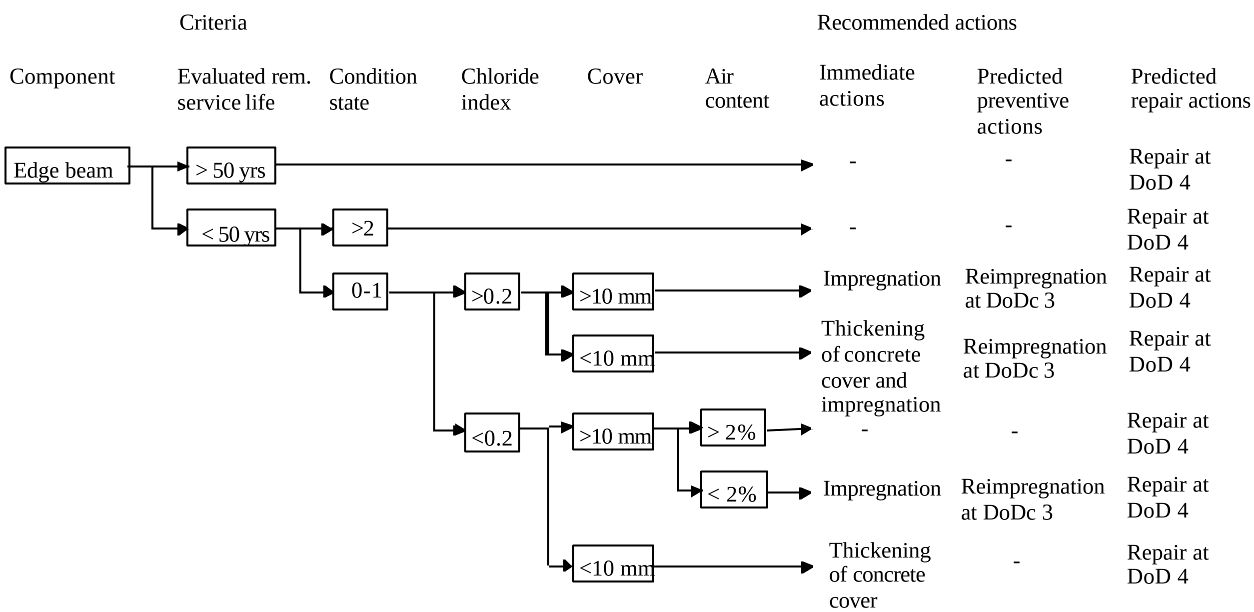

FIGURE 4 Example of a decision tree for edge beams of bridges. (DoDc = degree of damage of the preventive maintenance system.)

Structural and Other Risk Analyses

The aim of structural analysis is to reveal the deficiencies in the structural load-bearing capacity or functional performance. Hence, the system allows risk analyses for components, modules, and objects. The purpose of risk analyses is first to ensure the structural safety of deteriorated structures. Other types of risks, such as functional, economic, ecological, and health hazards, may also be analyzed.

The results of risk analyses are taken into account in project planning and in the sorting and ranking of repairable objects. A high risk indicates an urgent repair of the structure.

For structural analysis, a structure's load-bearing capacity can be studied, for example, with the following methods:

- Original design equation of structures and current loadings requirements,
- Three-dimensional numeric analysis of stresses (e.g., finite element modeling analysis), and
- In-situ loading tests.

In the preliminary planning phase, the load-bearing capacity can be evaluated with the help of the residual capacity factor, which expresses the load-bearing capacity of a damaged structure in relation to its original, undamaged load-bearing capacity.

The principle in this phase is to use automatic routines to reveal potential risks related to structures' bearing capacity. Normal design equations enable evaluation of the effect on load-bearing capacity of reduced cross sections that result from materials degradation. Examination may be extended by applying time-related models for material strengths and dimensions of structures. This enables prediction of the development in bearing capacity in the future.

For selected structures screened by the preliminary analysis, second-level analyses evaluate the performance of the whole structure. The performance of the structure and static load quantities can be examined by special methods that take into account the effects of the observed damage in the structure's stiffness, possible changes in the regulations of loads, and other factors. At this second level of accuracy, the calculation model can be verified by in-situ loading tests and measurements to obtain information on the structure's real performance and real loading.

Network-Level Long- and Short-Term Analyses

The aim of long-term optimizsation is to find the economically optimal long-term condition distribution of the building stock within safety and minimum service levels. The long-term optimum solution is a combination of the optimal-condition state distribution and the optimal MR&R action distribution.

The optimal condition state distribution corresponds to a certain optimal set of MR&R actions. These actions would, in the ideal case, be applied to the same extent annually. The optimal distribution of MR&R actions ensures the lowest MR&R costs in the long term.

The purpose of short-term optimization is to find an economically optimal way to reach the long-term optimum condition distribution during the next few years. Separate short-term solutions are posited for each coming year. Each short-term solution represents a step toward the long-term optimum.

The network level optimization offers the possibility of conducting what-if experiments for safety, minimum service levels, costs of repair measures, budget limits, and other variables.

The system also produces detailed information for future maintenance engineers on the deterioration mechanisms of specific elements and on the lifespan cost of different structures.

Budget Share Analysis

The administrative targets for MR&R activity are taken into account by distributing the total budget into budget categories, which are correlated with the targets. The budget categories are also consistent with the MR&R categories:

- Preventive maintenance,
- Repair,
- Restoration,
- Rehabilitation, and
- New construction.

The correlation between the targets and the budget categories is evaluated. For instance, maintaining safety is usually correlated with heavy repairs such as renewal, rehabilitation, and restoration. Economic and ecological factors probably correlate best with the categories of repair and preventive maintenance. The correlation factors are numbers between 0 and 1 expressing the relative importance of the budget category to promote a particular target. The sum of correlation factors must be 1.

The targets are weighted by the weight coefficients defined by the administration (see Table 1 for an example). The weight coefficients are also numbers between 0 and 1 expressing the relative importance of the targets compared with each other. The sum of the weight coefficients must be 1.

The priority index of a budget category, which shows the allocation from total budget to the budget category, is determined from Equation 1:

$$I_i = \Sigma T_j \times K_{ij} \tag{1}$$

where

I_i = priority index of ith budget category,
T_j = weight factor of jth target,
K_{ij} = correlation between ith budget category and the jth target,
i = index for budget category, and
j = index for target.

The budget in each budget category is then determined as follows:

$$B_i = I_i \times B_{tot} \tag{2}$$

where B_i is budget of ith budget category and B_{tot} is the total budget.

The Quality Function Deployment (QFD) method is recommended in the budget share. The analysis results in a budget allocation for each budget category. Table 2 gives an example of the distribution into budget categories of a budget of 5,000,000 € (US$5,125,850 December 2002).

TABLE 2 QFD for Budget Share

Attribute	Preventive Maintenance	Repair	Restoration	Rehabilitation	Demolition and Rebuilding	Weight
Total need of funding	0.3	0.2	0.1	0.2	0.2	0.1
Life-cycle economy (agency costs)	0.6	0.3	0.1	0	0	0.2
User economy	0	0	0	0.5	0.5	0.2
Ecology	0.3	0.3	0.3	0.1	0	0.1
Aesthetics	0.4	0	0	0.3	0.3	0.1
Mitigation of risks (human lives)	0	0	0	0.4	0.6	0.2
Preservation of cultural values	0.3	0.3	0.4	0	0	0.1
						Total
Share of Total	0.25	0.14	0.10	0.24	0.27	1.00
Budget, € (in 1000s)	1250	700	500	1200	1350	5000

Timing of Actions and Evaluation of Action Costs

Timing of actions and evaluation of action costs form a central part of the LIFECON LMS process. The aims of this module are

- Evaluating the present-day condition of the structures,
- Timing the MR&R actions,
- Evaluating optimal costs of MR&R actions, and
- Evaluating the delay costs of MR&R actions.

The inspections are performed at intervals of 3 to 8 years depending on condition. Therefore, the observed data at the moment of inspection may not be relevant at the present day. However, the present-day condition of the building stock is important for making correct decisions. Present-day conditions can be determined with updated degradation models.

The owner's strategy can be best promoted when repair actions are timed optimally. This timing can be determined using the calibrated degradation models and the benchmark condition states defined by the decision trees. These models can be determined as mathematical functions with the degree of damage (DoD) and the repair area as parameters (see Figure 5).

The costs of single actions are necessary to evaluate project costs and the annual need for recourses. The costs of delay are also needed to evaluate the urgency of projects. High costs of delay result in high priority. Both the costs of optimal timing and costs of delay are evaluated using cost models.

MATHEMATICAL METHODS IN LIFECON LMS

General

The LIFECON management system is created for organizations and agencies that have responsibility for maintaining concrete infrastructures. The system description is generic and includes different analyses, approaches, mathematical methods, and solutions and thus allows modification according to the special needs of organizations and agencies. The methods

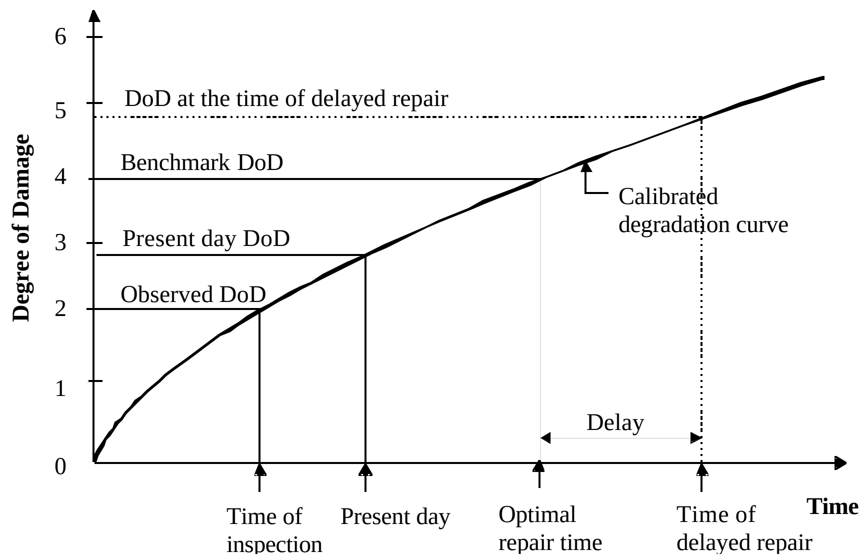

FIGURE 5 Prediction of DoD and timing of repair action.

presented here are based on the principles and common agreements of the partner organizations in the LIFECON project.

Markov Chain Method

The purpose of a LIFECON life-cycle cost analysis is to find the most feasible and economically most effective maintenance strategy to maintain the structures. It is a problem of optimization: The user seeks to find the most effective MR&R action profile for each structural part within defined period of time.

The Markov chain method makes it possible to depict a structure's performance over its whole life cycle, including the effects of degradation and also of MR&R actions, as a mathematical model in a probabilistic way. Thus it is an ideal basis for a complicated life-cycle analysis and also fulfils the requirements of a predictive and probabilistic LMS.

That is why the Markov chain method is used in LIFECON life-cycle analyses of the changes in the condition-state distribution of structures (*2*). The LCAP in a given time frame is reproduced in a probabilistic way. Through models based on Markovian transition probabilities, the LIFECON life-cycle analysis and optimization methods mathematically reproduce the effects of repair action on the structure's condition and degradation rate. In this way, the probability of the structure's being in any condition state at any moment in the treated time frame can be evaluated.

From Durability Models to Markov Chain–Based Life-Cycle Analysis

Different durability models with different types of degradation pose no difficulty in LIFECON LMS. The durability models can be evaluated in laboratories, by computer simulation, taking into account the local environmental effects or observations in situ. The most important types of degradation—such as from carbonation and chloride contamination of concrete, corrosion of reinforcement, frost scaling, internal frost damage, and alkali-aggregate-reaction—can be modeled. All these models are absolute and deterministic in nature.

The challenge of LIFECON LMS is to change the durability models into Markov chain–based transition matrices. The degradation models are automatically converted into degradation matrices. For the automatic conversion it is important to know how the transition probabilities between the condition states ($p_{i;\,i+1}$) depend on the degradation rate.

A great advantage of the Markov chain method as compared to analytic methods is the straightforward combination of sequential degradation processes. For example the models of depassivation phase and the active corrosion phase can be easily combined in the same transition probability matrix. In this way not only the average behavior but also the scatter of the two degradation processes can be combined without a problem.

Multi-Attribute Decision Making

Consideration of many objectives simultaneously calls for special mathematical methods to aid the decision making. Multiple-attribute planning is applied at all hierarchical levels of the LIFECON system (*3*).

The multiple-attribute planning of MR&R is more than a method. First, it is an attitude for planning. Figure 6 presents a list of attributes that could be considered in almost all engineering problems related to both new construction and maintenance of structures. These attributes describe the whole area of the LIFECON project and are grouped under the titles: human conditions, economy, culture, and ecology.

VALIDATION AND IMPLEMENTATION OF LIFECON LMS

Aside from the design and implementation of LIFECON LMS, the validation of the system is an ongoing process with two subsections: validation of the procedures themselves and the evaluation of the LMS against the original objectives.

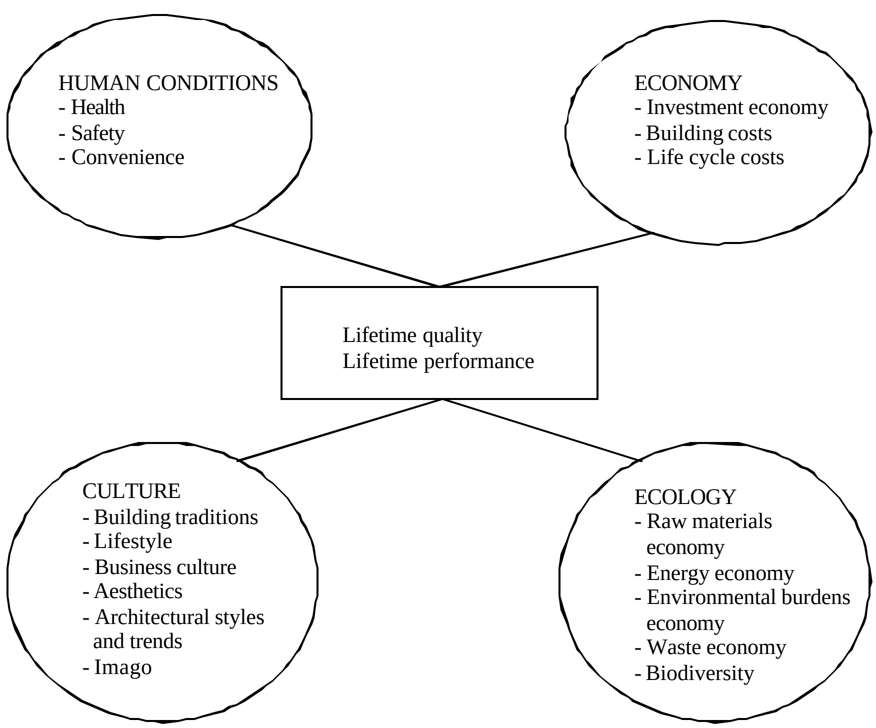

FIGURE 6 Attributes of LIFECON decision making.

For the validation process, nine structures were selected as case studies (*4*) to provide a range of common reinforced structural types and to cover the range of conditions to which a structure is subject during its lifetime and its internal conditions. The structures considered were limited to four bridges, two buildings, one wharf, one tunnel, and one lighthouse from five countries of north and central Europe.

The implementation of the LIFECON LMS prototype will start for testing in early 2003 among all project partners. After the project a commercial product will be released for the use by European organizations and other interested infrastructure owners.

ACKNOWLEDGMENTS

The authors thank the participants of the LIFECON project for their efforts and expertise given for the project.

REFERENCES

1. Vesikari, E., and M.-K. Söderqvist. LIFECON LMS Generic Technical Handbook for a Predictive Life Cycle Management System of Concrete Structures. LIFECON deliverable D1.1 draft 3. Sept. 2002.
2. Sarja, A., and E. Vesikari. Performance Systematics and Methodology for Lifetime Maintenance and Repair Planning of Structures. LIFECON Deliverable D2.1A-E draft 2. June 2002.
3. Lair, J., and A. Sarja. Multi-Attribute Decision Aid Methodologies. LIFECON Deliverable 2.2.
4. Law, D., and J. Cairns. Validation Plan of LIFECON Management System. LIFECON Deliverable 6.1 draft 2. Dec. 2001.

Pontis-Based Health Indexes for Bridge Priority Evaluation

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The Kansas Department of Transportation (KDOT) has been using priority formulas for the past 20 years to select projects for major rehabilitation, modification, or replacement of roadways and bridges. The department currently uses a bridge priority formula based on National Bridge Inventory (NBI) ratings for deck and structural conditions. Since 1994, KDOT has also conducted element-level inspections for use in the Pontis Bridge Management System. To avoid duplication of data collection and better utilize the Pontis system's detailed element-level data, KDOT evaluated whether to replace the NBI ratings with the Pontis inspection data in the bridge priority formula. Pontis also offers priority rankings of bridges on the basis of cost–benefit analysis. KDOT wanted to explore whether the Pontis system should replace the bridge priority formula as the method for prioritizing bridge improvements and selecting bridges for major rehabilitation or replacement. KDOT took several approaches for determining how best to integrate Pontis into the prioritization of bridge projects. The final approach selected for implementation uses health indexes that can be derived from Pontis element-level inspection analysis. The development of this approach and its impact on evaluating bridge priorities are discussed. The current bridge priority formula and the role of NBI ratings are described in the next section. Alternative approaches to integrating Pontis data or output into evaluation of bridge priorities are presented. Bridge priorities using alternative approaches are compared, and the factors in the choice of the approach for implementation are described. The final section contains a summary and conclusions.

CURRENT BRIDGE PRIORITY FORMULA

Background

Kansas Department of Transportation (KDOT) is charged with providing a safe and efficient transportation system for the citizens of Kansas. State transportation projects are funded through the Comprehensive Transportation Program (CTP) (see Figure 1 for the program's main components). While KDOT also has responsibilities for airport, freight, and transit services, its primary responsibility is to preserve, modernize, and expand the state highway system—comprising approximately 10,000 mi of roads and 5,000 bridges—as needed.

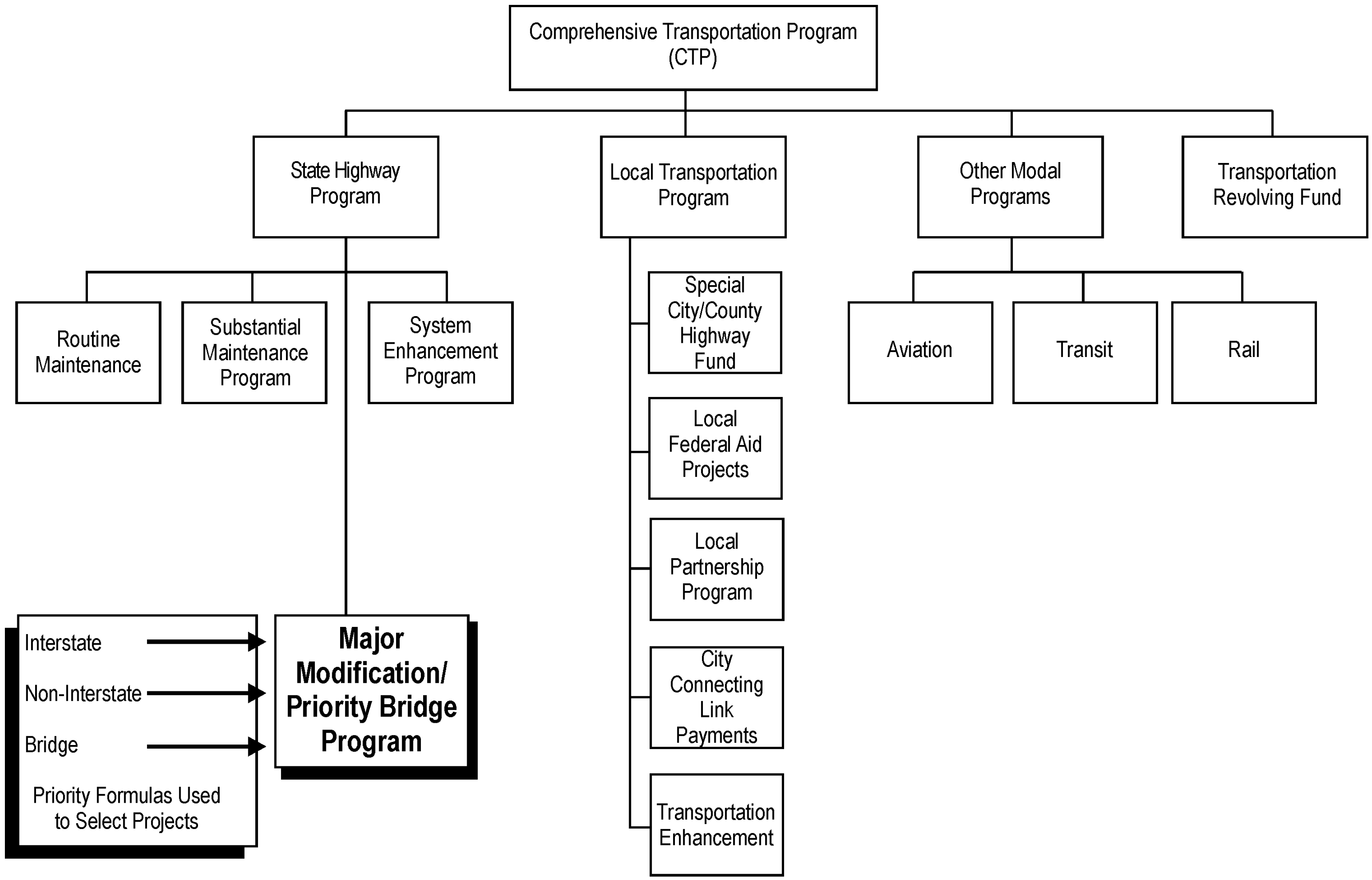


FIGURE 1 Main components of Kansas' CTP.

As shown in Figure 1, the State Highway Program consists of four parts:

- Routine maintenance. The objective of routine maintenance is to correct highway deficiencies on a temporary basis through minor maintenance until a more permanent action can be applied. These actions are generally performed by KDOT maintenance personnel.
- Substantial Maintenance Program. This program's objective is to protect the state's investment by preserving the as-built condition of roads and bridges for as long as possible. This category covers all work, from pavement patching and overlays to bridge painting and repair. These actions are performed by private contractors.
- System Enhancement Program. This program's objective is to substantially improve safety, relieve congestion, improve access, or enhance economic development. Examples of projects include new bypasses or four-lane improvements.
- Major Modification and Priority Bridge Program. This program's objective is to improve capacity and enhance safety. The projects include all kinds of actions, from completely replacing the pavement to adding lanes. The most deficient bridges are targeted for replacement or modernization under this program.

This paper pertains to project selection under the Priority Bridge Program. KDOT's 10-year budget for the Priority Bridge Program is approximately $300 million.

More than 20 years ago, the Kansas Legislature had the foresight to start the state of Kansas down the path of developing an objective, data-based method for selecting projects under the Major Modification and Priority Bridge Program. In 1979, the Legislature directed KDOT to develop a method of project selection that:

- Was clearly defined and used documented criteria,
- Was systematic and consistent,
- Was reproducible, and
- Used quantitative and verifiable factors in determining relative priorities.

Out of that directive, KDOT's priority formulas were developed. Originally, two formulas were developed: one for roadways and one for bridges. In the mid-1980s, the single roadway formula was split into separate formulas for Interstate and non-Interstate roadways. These three formulas are used to select the Interstate roadway, non-Interstate roadway, and priority bridge projects.

A need-based approach was taken to develop the three formulas. The fundamental principle of the need-based prioritization system is that the priority by which alternative roadway and bridge projects receive capital funding should be based on the overall need for improvement. Improvements are necessary because over time deficiencies develop that adversely affect a roadway segment's or bridge's operation. Deficiencies can develop for a variety of reasons. A bridge structure may become weaker after it has been subjected to traffic loading and environmental exposure. An old bridge built with past design standards may not have physical features (e.g., bridge width) consistent with modern design standards.

Different roadway or bridge deficiencies may not have the same degree of impact on the driving public. Structural deficiencies that can cause bridge failure are more critical than a rough deck surface that may affect drivers' comfort. Consequently, different deficiencies must be weighted differently in calculating the overall weighted need of each roadway segment or bridge.

Roadway segments and bridges are ranked in a descending order of the overall weighted need, with those ranked higher selected for funding ahead of those ranked lower. This need-based priority system (highest need first) is quite different from an optimization approach determined, for example, by a benefit–cost ratio.

Over the past 20 years, the three priority formulas have been fine tuned to some extent. In 1999, KDOT and its consultants conducted a comprehensive review of the formulas and found they were fundamentally sound, but could be enhanced by taking advantage of newly available technology and better data collection and reporting methods (*1*).

Details of the current bridge priority formula are presented below.

Overview of Current Bridge Priority Formula

To ensure that the priority formulas were based on what is important to the people of Kansas, the KDOT project team identified the reasons for improving a road or bridge, expressed as the objectives of a quality highway network. The bridge objectives were defined as

- Maximizing user safety,
- Maximizing preservation of investment, and
- Minimizing users' travel time and vehicle operating cost.

Each objective was then related to one or more attributes—physical features of a roadway or bridge that could be measured and corrected or improved. Figure 2 shows the objectives and attributes included in the bridge priority formula.

A need function was developed for each attribute that converted the raw data levels of the attribute into a need score: a relative measure of the need to improve, on a scale of 0 to 1. Figure 3 illustrates the need function for the attribute of National Bridge Inventory (NBI) deck condition rating. For example, the need score of a bridge with an NBI deck condition rating of 4 is 0.8 and of a bridge with an NBI deck condition rating of 7 is 0.2.

Certain adjustment factors may be applied to some attributes. An adjustment factor is not a deficiency and does not in itself create a need for improvement, but it may change the degree of concern related to a deficiency. For example, traffic volume may be an adjustment factor for an attribute such as bridge width. Traffic volume represents no deficiency, but the need to improve a bridge with substandard width may be assessed as less if the bridge's traffic volume is low.

The project team identified several potential improvements to the current bridge priority formula. A key recommendation was to explore approaches to revise, or completely replace, the current formula with Pontis data and analysis. The next section describes the different approaches taken for exploring this issue.

APPROACHES TO INCORPORATING PONTIS INTO BRIDGE PRIORITY EVALUATION

Motivation

The NBI records a bridge's structural condition primarily through three major components of a bridge: the deck, the superstructure, and the substructure. The condition of these components is assessed on a rating scale of 0 to 9. These ratings provide information on the condition's severity but do not identify or quantify the extent of the problem. The failure to quantify the extent of a

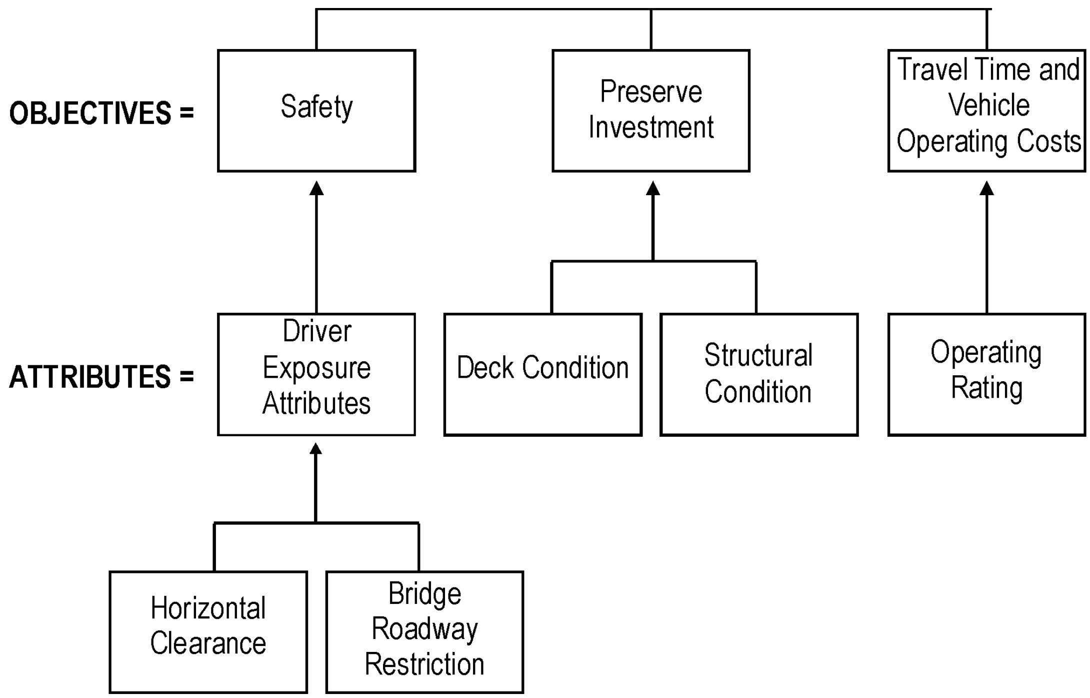


FIGURE 2 Bridge priority formula objectives and attributes.

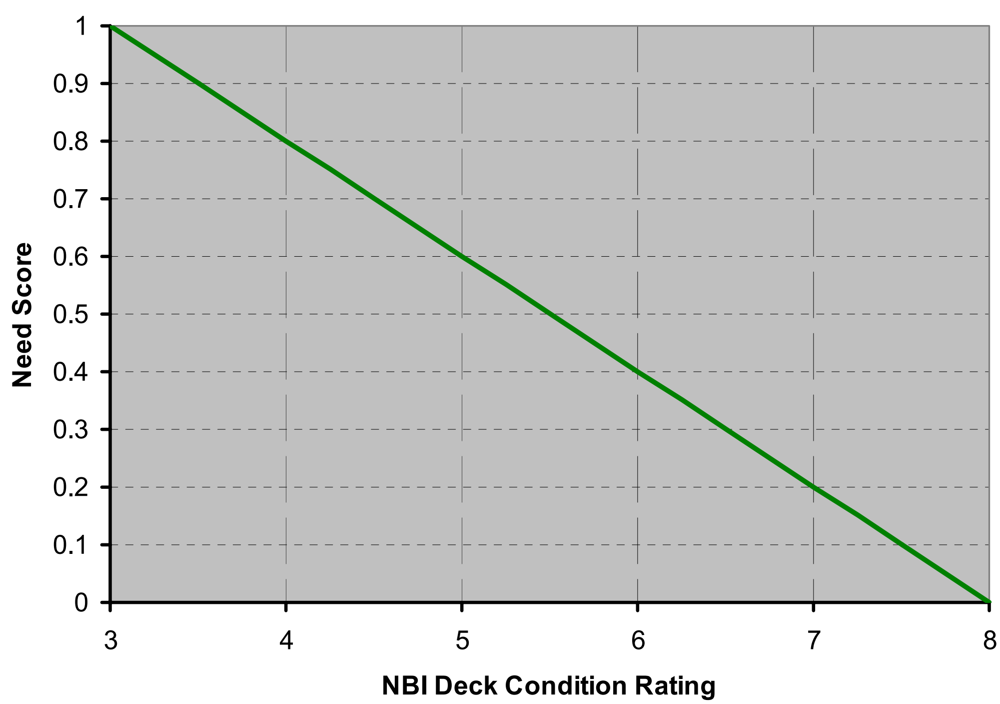


FIGURE 3 Need function of NBI deck condition rating.

deficiency minimizes the effectiveness of the NBI condition ratings in determining needs for bridge maintenance and rehabilitation. Relating the NBI ratings to needs requires a subjective interpretation by bridge inspection staff.

Pontis is a comprehensive bridge management system developed as a tool to assist in the challenging task of bridge management. Pontis uses element-level inspection techniques, as described in the AASHTO *Guide for Commonly Recognized (CoRe) Structural Elements* (*2*), which give a more detailed breakdown of bridge components. Instead of the NBI's single superstructure rating, the Pontis element-level inspection requires individual condition assessments for such components as girders, floor beams, pins, and hangers. Data from the element-level inspection define each element's condition-state in precise engineering terms for severity and extent of critical deficiencies. The Pontis data, thus, represents a more objective assessment of relative needs for improvement of different bridges.

Because of Pontis' superior conceptual framework, KDOT (and many other states) decided to use its approach for regular bridge inspection. If the Pontis inspection data could be incorporated in the bridge priority formula in place of the NBI ratings, this would avoid duplication in the collection of data and result in substantial cost savings to KDOT. Furthermore, the Pontis system evaluates priorities for bridge improvement and selection of bridges for major rehabilitation or replacement. KDOT, therefore, also evaluated Pontis to determine whether it should replace the NBI bridge priority formula.

Three alternative approaches were explored to incorporate the Pontis inspection data and the results of its analysis into the bridge priority formula:

1. Translating Pontis inspection ratings into NBI ratings,
2. Calculating health indexes from Pontis, and
3. Replacing the NBI bridge priority formula with Pontis.

KDOT tested each approach on bridges with NBI and Pontis ratings, and the resulting bridge priorities were compared to those obtained from the current priority formula. Test results for each approach and their implications for KDOT are presented below.

Translating Pontis Inspection Ratings into NBI Ratings

In this approach, the Pontis inspection ratings were translated into NBI ratings with a computer program developed for the FHWA by the University of Colorado. The translator program determined NBI condition ratings for deck, superstructure, substructure, and culverts using a table-driven procedure. To determine equivalent NBI ratings, the condition-state quantities of *CoRe* elements recorded in the Pontis inspection were compared to criteria for the NBI rating assignment. The translated NBI ratings were then used in the priority formula in place of the field NBI ratings.

Reliability was tested by correlating the field and translated NBI ratings. Figure 4 shows this correlation for structural and deck condition ratings. Note that for the priority formula, the structural condition rating is defined to be the lower of the super- and substructure condition ratings. As shown in Figure 4, the correlation between the two sets of ratings was very poor. The square of the correlation coefficient (r^2) is 0.117 for structural condition ratings—only 11.7% of the data variability in the translated NBI ratings was explained by the field NBI ratings. Many bridges that had a low-field NBI structural condition rating had a high translated NBI rating. Conversely, many bridges that had a high field NBI structural condition rating had a low translated NBI rating. Figure 4 shows a similar pattern for rating deck conditions, where r^2 between the field and translated NBI ratings was only 0.249. The apparent lack of correlation between the two sets of ratings was not surprising given the expected difficulty in converting the multidimensional Pontis ratings into one-dimensional NBI ratings.

On the basis of the results in Figure 4, the project team concluded that the NBI ratings translated from the Pontis inspection data would not provide reliable assessment of bridges with relatively high deficiencies and hence a greater need for improvement The approach of translating Pontis data into NBI ratings was considered ineffective and was not pursued further.

Calculating Health Indexes from Pontis

Overview

In this approach, health indexes for deck and structure were calculated using Pontis inspection data. These health indexes then replaced the NBI field ratings in the priority formula.

The concept of health index, as used in Pontis, is described in Shepard and Johnson (*3*). The health index is a single number assessment of a bridge's condition based on the bridge's element value, which is determined by element-level inspection. It can be calculated for any given group of bridge elements and is defined as the ratio of current element values (summed over all elements) to the initial element values (summed over all elements). The current element value takes into account the quantity in each of several possible condition states and the value of the quantity in each condition state. Similarly, the initial element value is calculated using the total element quantity and the value of the quantity in its initial condition state.

Using Pontis inspection data, the health index was calculated for the deck elements of 4,762 of the Kansas State Highway System's 4,911 bridges. Separate health indexes were also calculated for the bridges' super- and substructure using the appropriate elements in each group. As with the NBI ratings, the lesser of these two indexes for a bridge was taken as the structure's health index.

To gain an understanding of the characteristics of the statewide distribution of health indexes, histograms and summary statistics were prepared, as shown in Figure 5 for the deck and in Figure 6 for the structure. For purposes of comparison, the figures also show similar results for the NBI deck and structure condition ratings:

- The range of the deck health index is from 0 to 100, with an average of 84.
- The range of the structure health index is from 11 to 100, with an average of 91.
- The range of the NBI deck rating is from 3 to 8, with an average of 6.9.
- The range of the NBI structure rating is from 5 to 8, with an average of 6.9.

Because of the significant differences between the procedures used in NBI and Pontis inspections, a perfect match between the two sets of ratings was not expected. As stated earlier, NBI inspections survey the bridge components of deck, superstructure, and substructure, with discrete-number ratings on a scale of 0 to 9. Pontis inspections examine many bridge elements, with the health indexes rated on a continuous scale of 0 to 100.

Although for individual bridges significant differences may exist between NBI ratings and Pontis health indexes, it was expected that a group of bridges with low NBI ratings would, on average, have a low health index. Conversely, a group of bridges with high NBI ratings would, on average, have a high health index. To verify the expected correlations, graphs of NBI

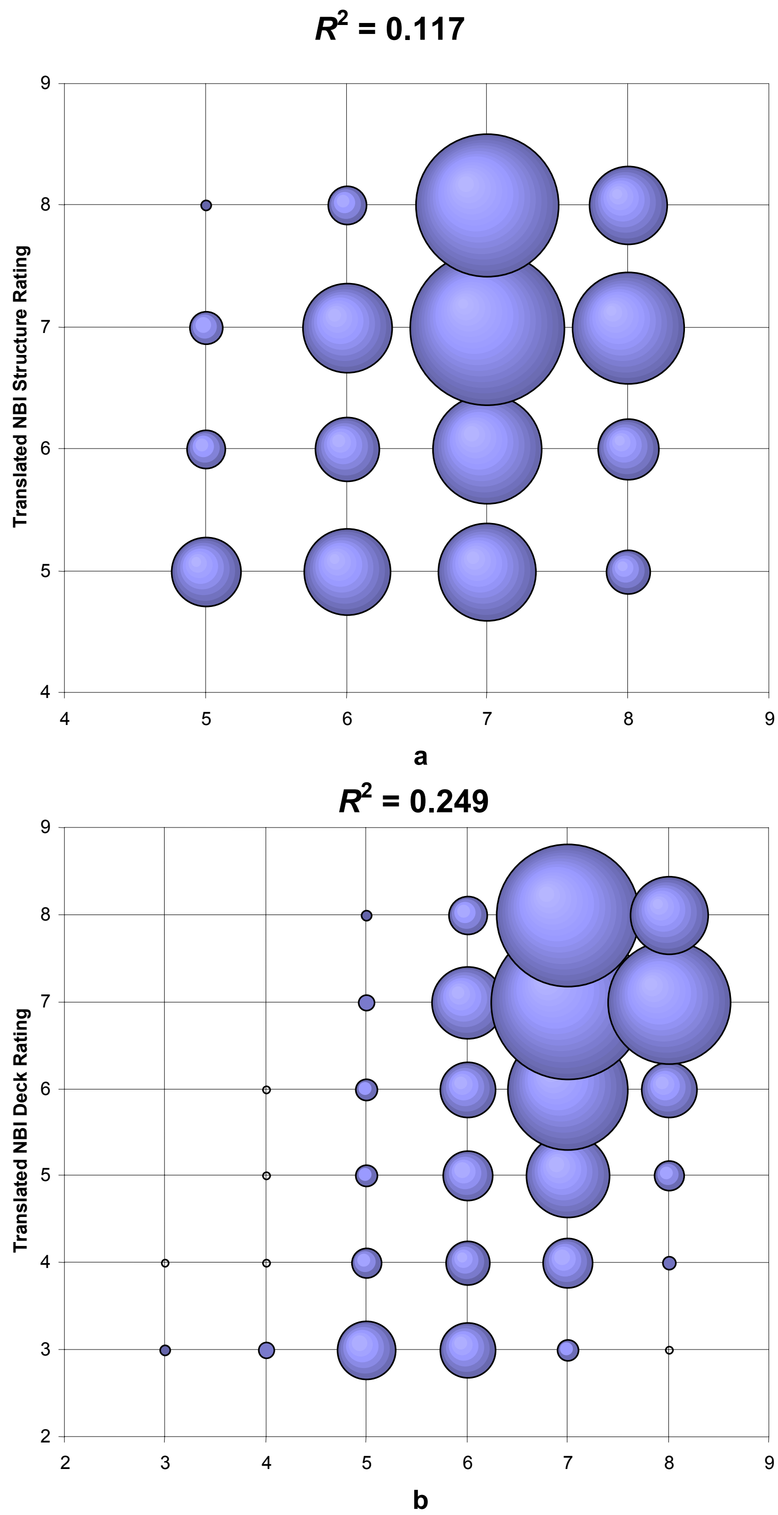
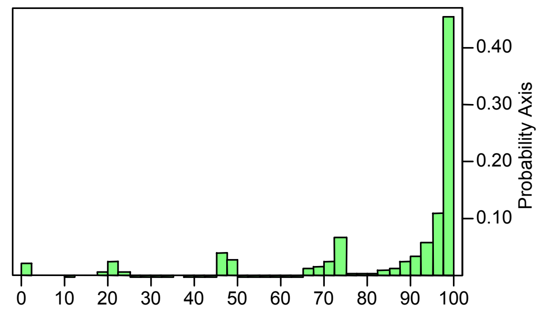


FIGURE 4 Graph of translated versus (a) field NBI structure ratings and (b) field NBI deck ratings.

Deck Health Index



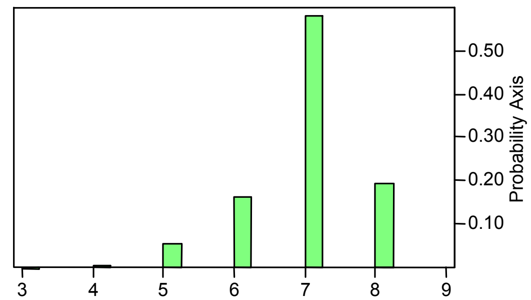
Quantiles

100.0%	maximum	99.700
99.5%		99.500
97.5%		99.500
90.0%		99.000
75.0%	quartile	98.600
50.0%	median	96.800
25.0%	quartile	72.900
10.0%		47.200
2.5%		19.800
0.5%		0.000
0.0%	minimum	0.000

Moments

Mean	83.783977
Std Dev	24.038775
Std Err Mean	0.3483515
upper 95% Mean	84.466907
lower 95% Mean	83.101047
N	4762

Deck NBI Rating



Quantiles

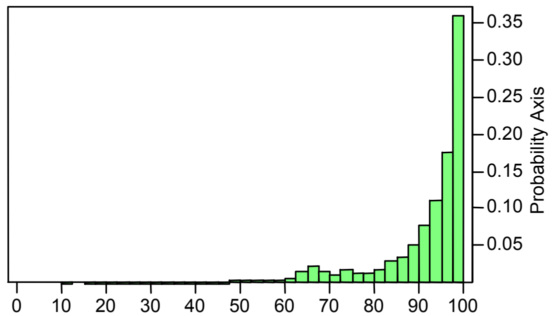
100.0%	maximum	8.0000
99.5%		8.0000
97.5%		8.0000
90.0%		8.0000
75.0%	quartile	7.0000
50.0%	median	7.0000
25.0%	quartile	7.0000
10.0%		6.0000
2.5%		5.0000
0.5%		4.0000
0.0%	minimum	3.0000

Moments

Mean	6.8926921
Std Dev	0.8136833
Std Err Mean	0.0117913
upper 95% Mean	6.9158085
lower 95% Mean	6.8695758
N	4762

FIGURE 5 Histograms and summary statistics of deck health index (left) and deck NBI rating (right).

Structure Health Index



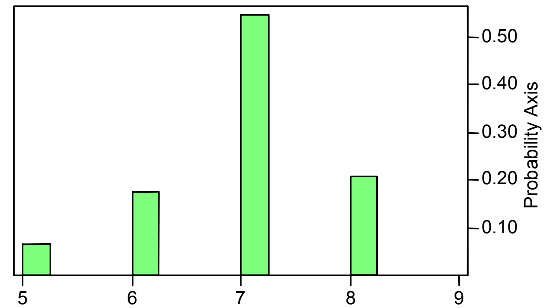
Quantiles

100.0%	maximum	99.700
99.5%		99.500
97.5%		99.500
90.0%		98.900
75.0%	quartile	98.500
50.0%	median	95.500
25.0%	quartile	89.000
10.0%		73.300
2.5%		61.200
0.5%		39.700
0.0%	minimum	10.800

Moments

Mean	90.924087
Std Dev	11.498006
Std Err Mean	0.1666203
upper 95% Mean	91.250739
lower 95% Mean	90.597434
N	4762

Structure NBI Rating



Quantiles

100.0%	maximum	8.0000
99.5%		8.0000
97.5%		8.0000
90.0%		8.0000
75.0%	quartile	7.0000
50.0%	median	7.0000
25.0%	quartile	7.0000
10.0%		6.0000
2.5%		5.0000
0.5%		5.0000
0.0%	minimum	5.0000

Moments

Mean	6.8971021
Std Dev	0.8021684
Std Err Mean	0.0116244
upper 95% Mean	6.9198913
lower 95% Mean	6.8743128
N	4762

FIGURE 6 Histogram and summary statistics of structure health index (left) and structure NBI rating (right).

ratings versus Pontis health indexes were prepared. The graphs for deck and structure conditions (Figure 7) show a fairly strong relationship between the two variables: lower NBI ratings were associated with lower health indexes and vice versa, and R^2 between the two variables was 0.66 for deck and 0.96 for structure.

These results confirm reasonable agreement between the two methods in identifying groups of bridges generally in good or poor condition. However, for individual bridges whose conditions vary across different elements, the Pontis health indexes may generate very different relative ratings compared with the NBI ratings. The health indexes are also evaluated on a continuous scale, while NBI ratings are evaluated on a discrete scale. Therefore, health indexes can better discriminate the rehabilitation needs among individual bridges.

For these reasons, the approach of creating health indexes was considered promising and was pursued further in evaluating bridge priorities. Bridge priorities produced by the Pontis-based health indexes and by the NBI ratings were then compared, and these results are presented next.

Bridge Priorities Using Health Indexes Versus NBI Ratings

The relative priorities for improvement for all of the state 4,911 bridges were evaluated with the current bridge priority formula using NBI ratings for deck and structure conditions and also with the corresponding Pontis-based health indexes. Given available funding for bridge rehabilitation projects, no more than 10% of bridges can be considered for a 10-year bridge rehabilitation program. The distribution of the top 10% of the bridges in the priority-ranked lists was analyzed.

Replacing the NBI ratings with Pontis health indexes significantly affected the bridge's priority ranking of bridges. Analysis showed that about 40% of the bridges that were in the top 10% with NBI ratings remained in the top 10% with the Pontis health indexes. However, about 60% of the bridges ranked in the top 10% were different for the two cases. About 30% of the bridges that were in the top 10% with NBI ratings dropped below 10% with Pontis health indexes. Conversely, about 30% of the bridges not in the top 10% with NBI ratings moved into the top 10% with Pontis health indexes.

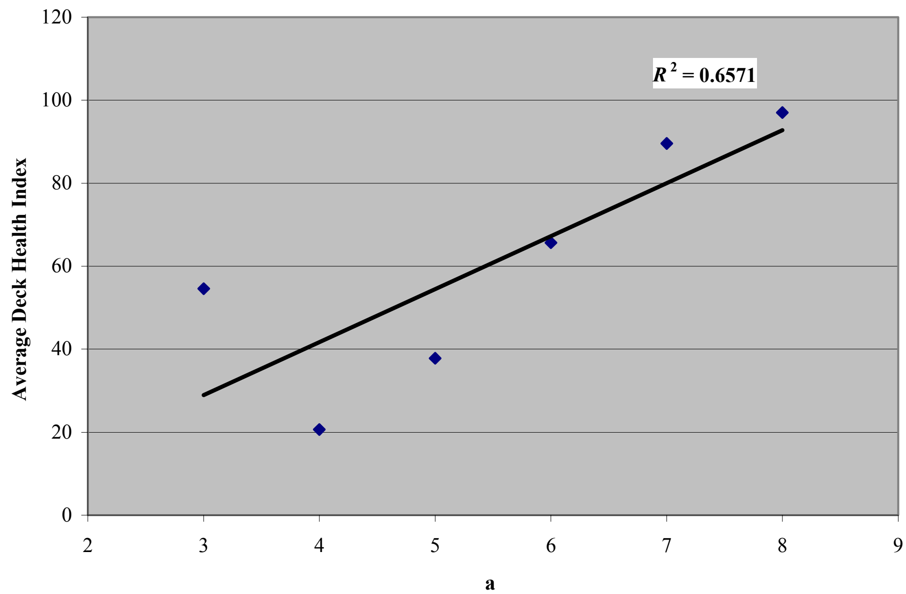
This discrepancy can be attributed to a number of causes. Pontis health indexes provides higher granularity in distinguishing the structural and deck conditions of bridges, because they are expressed on a nearly continuous scale. The NBI ratings are expressed on a discrete scale with a limited number of levels. Therefore, bridges with the same NBI rating may have different Pontis health indexes and therefore different needs for improvement, according to the Pontis health indexes.

Replacing the NBI Bridge Priority Formula with Pontis

Overview of Approach

In this approach, the priority ranks of bridges for rehabilitation were determined using the Pontis application and compared with the priority ranks determined using KDOT's current bridge priority formula.

The fundamental difference between the two formulas is that Pontis employs a cost–benefit analysis, whereas the bridge priority formula uses a need-based methodology. For a fixed amount of budget dollars, Pontis selects the bridges for improvement that return the highest amount of dollar benefits per unit cost. The dollar benefits are calculated on the basis of the expected reductions in bridge user accidents and delay related to detours. On the other hand, the bridge priority formula selects the bridges in the order of their need scores and deficiencies



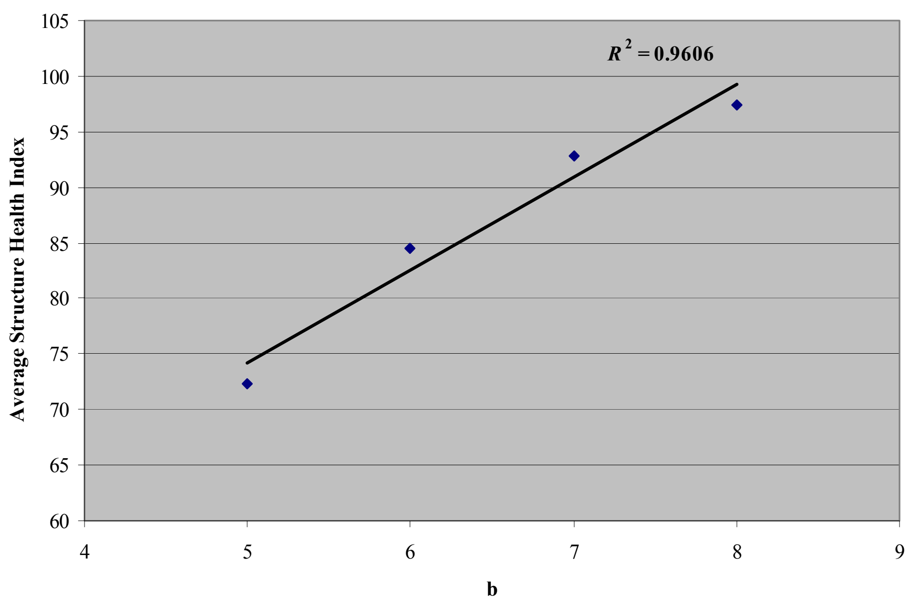


FIGURE 7 NBI ratings for (a) deck and (b) structure versus Pontis-based health indexes.

according to highest need first. As a result, the priority-ranked lists derived from these two methods were expected to differ. This section analyzes the differences in the lists of high-ranked bridges the two methods generated.

Method of Analysis

A priority-ranked list of bridge projects was developed using Pontis Version 3.4 and also using the current bridge priority formula. With Pontis, the default input parameters such as cost matrix settings, policy settings matrix, and improvement parameters were retained. These parameters address issues such as unit construction costs for different actions on given types of bridges, improvement in bridge condition following an action, and minimum serviceability life required for each action. KDOT bridge engineers used the default parameters for the base case Pontis runs to determine bridge improvement priorities. Only replacement or major rehabilitation actions were considered; substantial maintenance actions were excluded. These choices were consistent with the typical scopes selected for priority bridge projects. A budget of $286 million was set, equivalent to the budgeted dollars for KDOT's 10-year highway program for bridge replacement and rehabilitation projects in the current Priority Bridge Program. Other factors for the Pontis optimization run included a minimum cost threshold for a project of $50,000 and a minimum life of 10 years for the bridge rehabilitation action.

The Pontis-generated list consisted of projects for 361 bridges when all budget dollars were exhausted. The list was sorted in descending order of the benefit–cost ratio: projects with higher benefit–cost ratios were selected ahead of those with lower benefit–cost ratios. Both actual construction costs (agency costs) and user benefits and user costs were calculated in the benefit–cost ratio.

The ranked list of projects from Pontis was compared with the list generated by the current bridge priority formula. The results of the comparison are described below.

Results and Discussion

Of the 361 bridges Pontis selected for projects, 34 bridges were outside KDOT's jurisdiction (e.g., Kansas Turnpike Authority bridges) and were not considered by the bridge priority formula. Consequently, these bridges were excluded from the comparison, and the remaining 327 bridges were analyzed. The top 327 ranked bridges (approximately top 6.7% of bridges) ranked by the bridge priority formula were compared with the remaining 327 on the Pontis-generated list. Table 1 shows the cross-tabulation of bridge selection.

Of the top 6.7% of bridges, only 1.5% (74 bridges) appeared on both the Pontis and the bridge priority formula lists; 253 out of 327 bridges did not appear on both lists. This indicates that the two methods yield substantially different results. The top 327 ranked bridges were subdivided into two groups: those ranked in the top 1 to 163 were labeled as high priority in each list, and those ranked 164 to 327 were labeled as medium priority. Bridges ranked below 327 were defined as low priority. Figure 8 shows the crossed frequency plot between Pontis and the priority formula for the high, medium, and low definitions.

As shown in Figure 8, the match is a very low between the project selections of Pontis and the bridge priority formula. The primary reason for the differences is that Pontis uses the benefit–cost ratio as the ranking criterion, whereas the bridge priority formula uses the need score. Pontis, in general, selects projects with lower costs so that more work can be done within the fixed budget. The bridge priority formula does not consider user costs and benefits in ranking projects.

TABLE 1 Analysis of Priority Ranks with Pontis Versus Bridge Priority Formula

		PONTIS		
		In Top 327 Ranks	**Out of Top 327 Ranks**	**Total**
Priority Formula	**In Top 327 Ranks**	74	253	327
	Out of Top 327 Ranks	253	4331	4584
	Total	327	4584	4911

		PONTIS		
		In Top 327 Ranks	**Out of Top 327 Ranks**	**Total**
Priority Formula	**In Top 327 Ranks**	1.5%	5.2%	6.7%
	Out of Top 327 Ranks	5.2%	88.2%	93.3%
	Total	6.7%	93.3%	100.0%

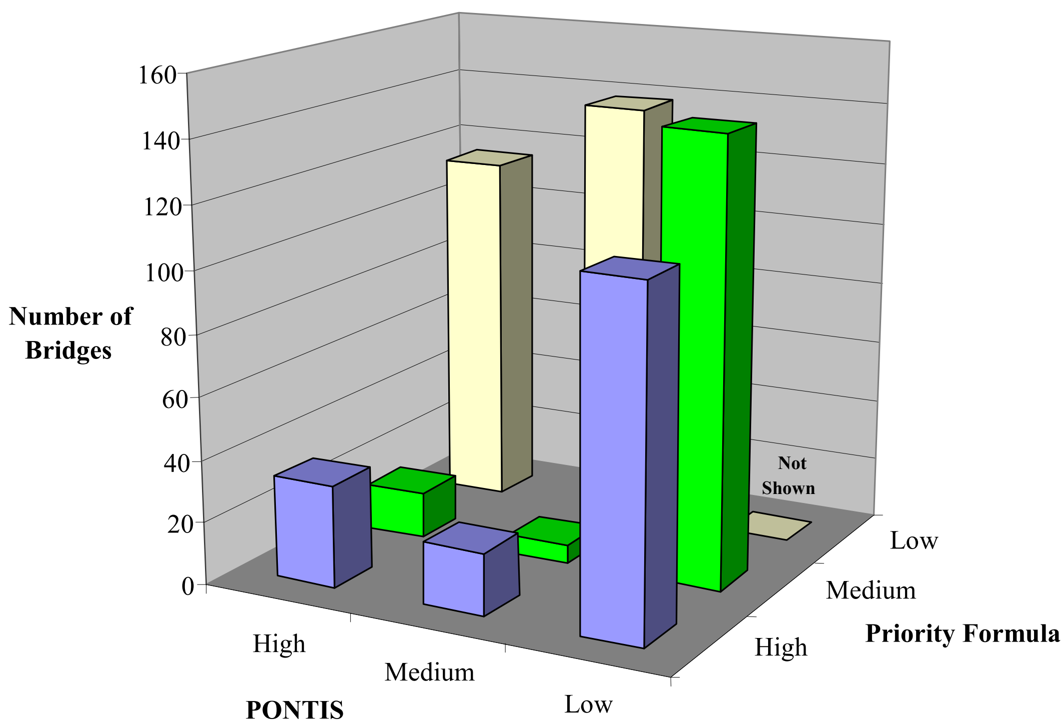


FIGURE 8 Bridge priority groups based on Pontis and bridge priority formula.

Further breakdown of the top 327 ranked bridges selected by Pontis as opposed to those selected by the bridge priority formula is shown in see Table 2 and revealed the following:

- Pontis ranked more bridges that have a higher strategic importance. The Kansas State Highway System is divided into five Route Classes—A, B, C, D, and E in descending order of importance—based on strategic importance and the amount of total and commercial traffic carried. Pontis selected more bridges in Route Classes A, B, and C, whereas the bridge priority formula selected more bridges in Route Classes D and E. This is likely because of Pontis' inclusion of user benefit as a project selection criterion, and higher traffic volumes in Route Classes A, B, and C would generate higher user benefits from bridge improvements. The bridge priority formula does include an adjustment factor for traffic, but its range is limited to 0.85 to 1.0. In contrast, the Pontis user benefits are proportional to traffic; user benefits double when traffic doubles. Therefore, traffic has a much greater effect on Pontis results than on bridge priority formula results.
- For the same reasons discussed above, Pontis selected significantly more bridges for eastern Kansas than for western Kansas and more bridges in urban areas or with higher volumes of average annual daily traffic.
- The analysis of deficiencies in the bridges selected indicated that Pontis tends to select bridges with primary concerns for geometric and deck condition. In contrast, the top 327 ranked bridges selected by the bridge priority formula were all driven by structural condition, which carries the most weight in the priority formula.
- The average structure length and surface area of bridges selected by Pontis appeared to be larger than those selected by the bridge priority formula. This may be because Pontis selected more urban high-traffic bridges, which are generally longer or wider, or both.
- Other parameters, including bridge type, number of lanes the structure carries, and roadway width, did not appear to significantly differ among the two sets of bridges selected.

Choosing Between Pontis and Bridge Priority Formula

Because of their fundamentally different strategies for project selection, Pontis and the bridge priority formula produced priority lists that differed substantially. Pontis employed a benefit-cost analysis in project selection, while the priority formula considered highest need. Pontis selects bridge projects in ascending order by benefit–cost ratio, whereas the bridge priority formula selects projects with the most severe bridge deficiencies. Thus, for example, a bridge in poor structural condition would be ranked high in the bridge priority formula list, but low in the Pontis list if its unit cost of improvement were relatively high. KDOT's philosophy over the years has been to select projects (roadway or bridges) based on need without regard to the unit cost of the improvement needed. KDOT places even greater emphasis on maintaining functional bridges, because unlike roads, when a bridge fails, motorists cannot travel on that route and a very lengthy detour may be needed to circumvent the feature that the dysfunctional bridge crossed.

The Pontis method appears to select a very different set of bridges than a list selected on the basis of need alone. Pontis selects a bridge with a lower need and lower cost ahead of a bridge with a higher need and higher cost, if the cost difference is greater than the benefit difference.

Project selection based on cost and benefit, although accepted by the user community, is

TABLE 2 Distribution of Top 327 Bridges: Pontis Versus Bridge Priority Formula

Factor	Level	Priority Formula Base Case	Pontis Project Priority List
Route Class	0	2%	0%
	1	9%	32%
	2	7%	15%
	3	21%	31%
	4	43%	14%
	5	18%	8%
	Max Shift		***29.4%***
Region	E	70%	84%
	W	30%	16%
	Max Shift		***14.1%***
Location	Rural	89%	59%
	Urban	11%	41%
	Max Shift		***30.0%***
Traffic (AADT)	(1) 0-249	6%	3%
	(2) 250-399	3%	1%
	(3) 400-874	23%	7%
	(4) 875-1749	27%	10%
	(5) 1750-3499	15%	20%
	(6) >=3500	25%	59%
	Max Shift		***33.6%***
Type of Primary Improvement (based on priority formula need scores)	(1) Geometric	0%	23%
	(2) Deck Condition	0%	3%
	(3) Structural Condition	100%	75%
	Max Shift		***25.4%***

Factor	Level	Priority Formula Base Case	Pontis Project Priority List
Structure Length (feet)	Mean	288	421
	Median	133	199
Area (Sq yard)	Mean	950	1772
	Median	408	674
Type of Bridge	Other	71%	67%
	Steel	29%	33%
	Max Shift		***4.0%***
Lanes Carried by Structure	1	2%	1%
	2	92%	83%
	3	2%	7%
	4	4%	8%
	6	0%	1%
	Max Shift		***8.9%***
Roadway Width (feet)	Mean	28.8	31.9
	Median	24.0	28.0
Accident Rate, 5-year (accidents per million vehicle miles)	Mean	2.16	2.17
	Median	1.90	1.58

more difficult to explain to the public. However, the project team did believe that incorporating Pontis' health indexes into the bridge priority formula allowed the department to take advantage of element inspection-level techniques, while maintaining KDOT's longstanding philosophy of fixing bridges with the most severe deficiencies first.

SUMMARY AND CONCLUSIONS

KDOT wanted to explore ways in which Pontis inspection data could replace NBI ratings in the bridge priority formula. KDOT's sought to avoid duplication of bridge inspections and collection and recording of data by dropping NBI inspections and taking advantage of the superior element-level Pontis inspection methodology. Three alternative approaches were evaluated for incorporating Pontis data into the bridge priority formula: convert Pontis data into equivalent NBI ratings for use in the bridge priority formula, use health indexes calculated with the Pontis inspection data, and completely replace the bridge priority formula with Pontis analysis.

Conversion of Pontis data into equivalent NBI ratings using FHWA's translator computer program produced inconsistent results and was dropped from further consideration.

Replacing the bridge priority formula with Pontis was considered unacceptable, because it was inconsistent with KDOT's longstanding philosophy of fixing bridges with the most severe deficiencies first—highest need first. Detailed examination confirmed that the rankings of bridges for projects by Pontis and by the bridge priority formula varied widely. The portfolio of bridges selected by the bridge priority formula was more consistent with the preferences of KDOT's management. For example, Pontis ranked a number of bridges with geometric or deck deficiencies ahead of bridges with structural deficiencies, apparently because of the lower costs involved in fixing geometric or deck problems relative to fixing structural problems. The bridge priority formula almost always selected bridges with structural deficiencies ahead of those with geometric or deck problems, a selection that better reflected KDOT's policies and philosophy.

Calculating health indexes proved to be the most effective way of incorporating Pontis data into the bridge priority formula. With the health indexes, the department can utilize the Pontis element-level inspection, which has strong advantages over the NBI inspection system. Use of health indexes was also validated by the reasonable agreement between the NBI ratings and the average Pontis health indexes for each group of bridges with the same NBI rating. KDOT is currently revising the bridge priority formula to replace the NBI ratings for deck and structural conditions with the corresponding Pontis-based health indexes.

REFERENCES

1. HNTB and URS Greiner Woodward Clyde. The Kansas Priority Formula Review: Technical Report. Kansas Department of Transportation, Topeka, 2000.
2. *Guide for Commonly Recognized (CoRe) Structural Elements.* AASHTO, Washington, D.C., 1997.
3. Shepard, R.W., and M. B. Johnson. California Bridge Health Index: A Diagnostic Tool to Maximize Longevity, Investment. *TR News*, No. 215, July–August 2001, pp. 6–11.

Management System Implementation

Pontis Bridge Management System
State of the Practice in Implementation and Development

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The Pontis bridge management system (BMS), a product of AASHTO, is now in its fourth major version, Release 4. Pontis continues to support the complete bridge management cycle, including bridge inspection and inventory data collection and analysis, recommending an optimal preservation policy, predicting needs and performance measures for bridges, and developing projects to include in an agency's capital plan. Transportation agencies licensing Pontis use the system in greatly varying ways and frequently have taken advantage of the system's flexibility to customize their implementation of Pontis to meet agency needs. A description is given of how U.S. transportation agencies are using the Pontis BMS, approaches taken to implementing the system, and agency-specific developments or customizations that may be of relevance to other agencies. Details are provided that include the level of use and degree of customization for agencies licensing Pontis with regard to supporting data collection and management, supporting the asset management process, and supporting other agency-specific needs.

The Pontis bridge management system (BMS), a product of AASHTO, is now in its fourth major version, Release 4. Pontis supports the complete bridge management cycle, including bridge inspection and inventory data collection and analysis, recommending an optimal preservation policy, predicting needs and performance measures for bridges, and developing projects to include in an agency's capital plan (*1*–*3*). Pontis is used extensively across the United States and has a significant presence internationally. As of October 2002, 46 agencies in the United States were licensing Pontis, including 39 state or territorial transportation departments and 7 other agencies, as shown in Figure 1. In addition, seven agencies outside the United States have licenses for the product.

Pontis is supported and maintained through AASHTO's joint software development technical service program. This program offers member agencies an opportunity to pool their resources and produce complex software solutions at a fraction of the cost of custom in-house development, while promoting a best-practices approach to design. By allowing agencies across the country to combine their resources, the joint development program provides enormous economies of scale, particularly in comparison to developing and maintaining individual custom

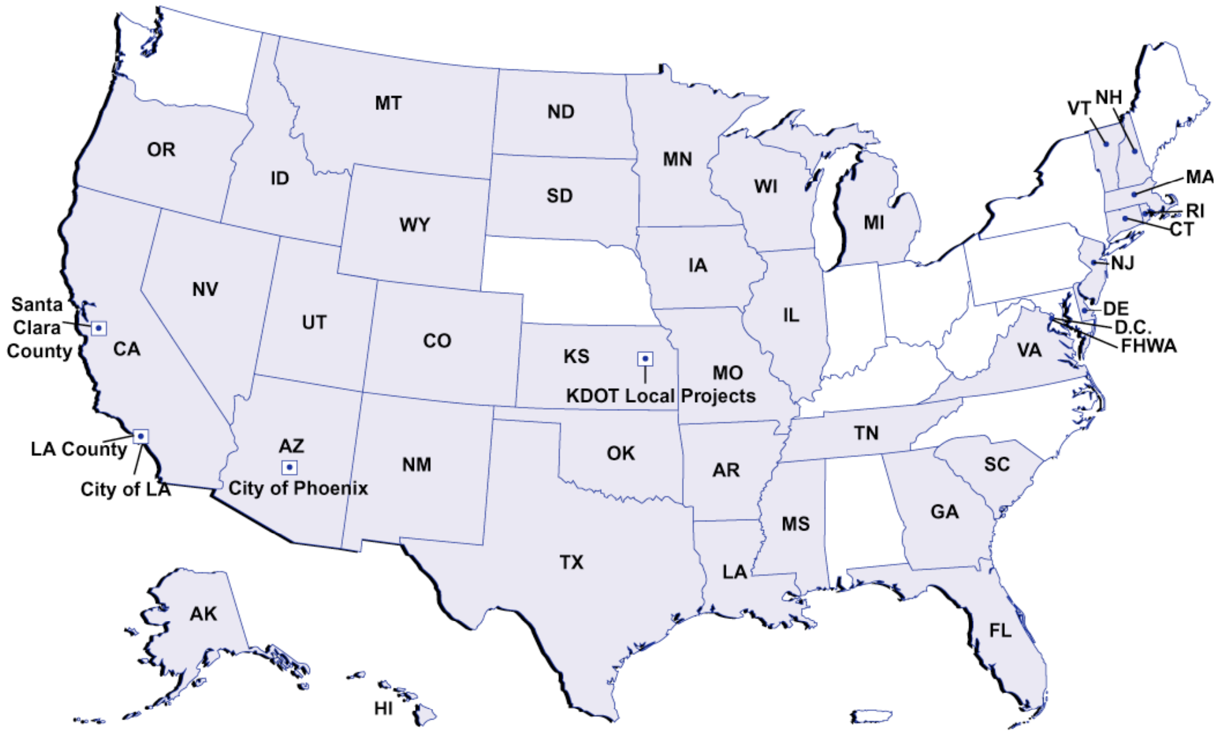

FIGURE 1 U.S. agencies licensing Pontis.
(KDOT = Kansas Department of Transportation.)

solutions. This results in significant cost savings, not only during initial software development, but also throughout the software product life cycle.

A critical factor in the success of Pontis has been its functionality for supporting a high level of agency customization. Agencies using the product may define their own bridge elements, deterioration models, and agency business process rules for use in the Pontis program simulation. They may develop their own screen layouts, reports, and data entry forms. Further, agencies may customize the Pontis database, take advantage of the open architecture of the product to add database tables, build their own applets or external applications for connecting to the database, and build interfaces between the Pontis database and other agency databases. Agencies are using all of these approaches to customizing Pontis to best meet their needs.

This paper summarizes the functionality available for customizing the Pontis BMS. Further, it details how U.S. transportation agencies are using Pontis, the approaches taken to implementing the system, and agency-specific developments or customizations that may be of relevance to other agencies. Details are provided including the level of use and degree of customization for agencies licensing Pontis with regard to supporting data collection and management, supporting the asset management process, and supporting other agency-specific needs.

FUNCTIONALITY FOR CUSTOMIZING PONTIS

Customizing Data and Product Behavior

Through the Pontis user interface, one may make a range of customizations to the data used by Pontis and to the product behavior. Customizations in this category include the following:

- Customizing data definitions and pick lists. In the Configuration module an agency may customize the data definitions used by the product, such as definitions of operating environments, bridge treatment action categories and types, and element types. Also, an agency may redefine the contents of the pick lists used in the application.
- Customizing element definitions. Central to Pontis is the representation of a structure as a set of structural elements. AASHTO has identified a set of commonly recognized (*CoRe*) elements (*4*). Agencies may supplement the *CoRe* elements with their own element definitions or replace them with a different set of element definitions, though this option of redefining *CoRe* elements is not acceptable to FHWA and is strongly discouraged by AASHTO. Action cost, action effectiveness, and deterioration models are defined for each combination of element and operating environment.
- Customizing product behavior. Using the application security and configurable options one may enable or disable selected fields or screens for all or selected users.
- Customizing the program simulation. Pontis uses inspection results and element models to predict future network-level and bridge-level conditions, and to make specific project recommendations through a program simulation. All significant parameters used in the program simulation are exposed for agency customization. In addition, beginning with Release 4 of Pontis, an agency may develop a comprehensive set of business process rules that allow Pontis to better simulate agency practices (*5*).
- Customizing data import and export. Pontis data interchange (PDI) files are used to exchange data between different Pontis databases, such as between a field inspector's laptop and a central database. Pontis users have an extensive set of options for customizing what database tables and fields are included in their agency's PDI files. Third party development efforts can also exchange data with Pontis through the PDI file.

Customizing the Database

The Pontis application connects to a relational database. The database may be a Pontis database, or an integrated BRIDGEWare database with data for Pontis, as well as for the other AASHTO BRIDGEWare products—Virtis (bridge load-rating program) and Opis (bridge design program). Agencies may implement the Pontis database either in Sybase Adaptive Server Anywhere (ASA) or Oracle. Agencies have full access to their Pontis database and may make customizations to the database. Additional tables and fields added to the database may be used in Pontis reports and may be exchanged by using PDI. Agencies may customize the security of the database outside of the Pontis application and add stored procedures and triggers as needed to support other needs.

Customizing Screens and Reports

Using the Sybase Infomaker application provided with Pontis, agencies licensing the software may develop their own custom screens and reports and include these in the product. Specifically, the bridge and project lists (desktop layouts) used to navigate in most of the product's modules may be customized using Infomaker. Also, agencies may define an arbitrary number of agency-specific reports; these are included in the Pontis Report Manager when the product is launched. Further, the basic screens provided with the product may be supplemented with agency-specific screens for collecting bridge, inspection, structure unit, roadway, and project data. Figure 2 shows an example of a custom screen used by the Kansas Department of Transportation (KDOT) for collecting bridge data.

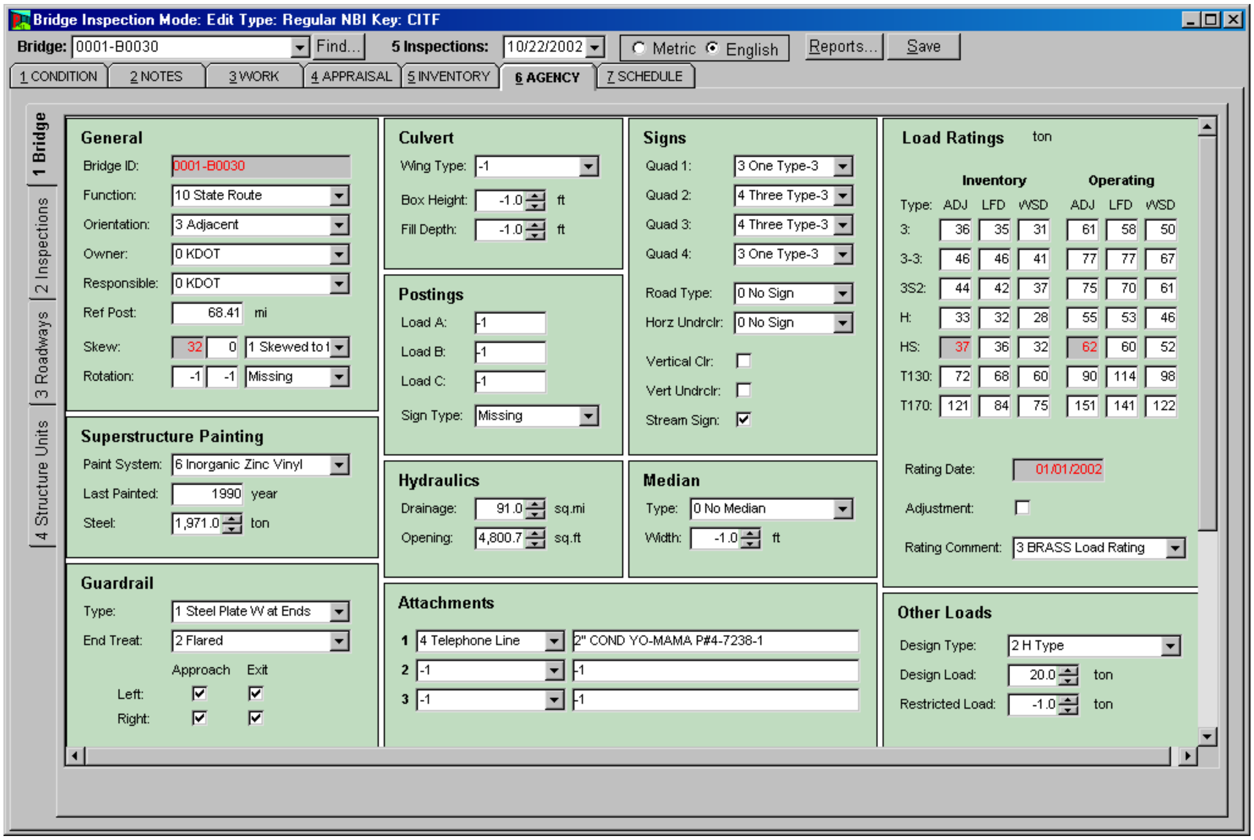

FIGURE 2 Example of a custom screen in Pontis. (Screen shot courtesy of KDOT.)

Developing Custom Applets and Applications

Given the open architecture of the Pontis application, it is possible to develop additional applications that interface with Pontis or that connect directly to the Pontis database to supplement the functionality offered by the system. Agencies with expertise in the Sybase Powerbuilder development environment may build Powerbuilder applets integrated with Pontis. These applets operate in conjunction with Pontis. They use the same database connection as the product and exchange information concerning what bridges are selected on the Pontis desktop. Agencies may use applets to provide additional data entry forms, reports, or other functionality.

External applications may be developed that connect to the Pontis database outside of the Pontis application. These applications may use application programming interface calls to access the modeling and data import and export functionality of the product (developed in C++) (*3*). Such applications may be used to collect additional data or provide functionality not available in Pontis.

ANALYSIS OF AGENCIES' USE OF PONTIS

Analysis Overview

Information on agencies' use of Pontis was compiled in September and October of 2002. This information was compiled primarily through telephone interviews with Pontis users. The interviews were supplemented with information gathered by authors at the annual Pontis User Training Meeting (the 2002 meeting was held in September in Phoenix, Arizona) and through information compiled from the Pontis Support Center.

Topics covered in the telephone interviews included the following:

- Current version of Pontis in use;
- Number of application and database users;
- Type of database in use (Sybase or Oracle, single user or multiuser);
- Information on functionality of the product used by the agency (e.g., data collection, program simulation, project planning);
- Information on customizations made to Pontis (e.g., custom screens, reports, forms, data definitions);
- Information on other applications used to connect to the Pontis database; and
- Feedback on the product.

Through the interviews and additional information, profiles were created detailing how each licensing agency uses the product, with emphasis on the extent to which the agency has customized the Pontis application or database, or both. Of the 46 agencies licensed to use Pontis, profiles were created for 34 agencies for which the authors could confirm that the agency has a production database. A total of 12 agencies was excluded in tabulating the statistics presented in this paper. Of the agencies excluded, one is a new licensee that had not yet received the product; two could not be contacted; and one (FHWA) uses the product for a mix of training, research, and data collection that are not directly comparable to the use of the product by other agencies. The other eight excluded agencies are state transportation departments that have not yet implemented a production database.

Statistics on Pontis Use

In each of the 34 agencies, Pontis is used on a regular basis, with the frequency of use depending on what functionality of the product is used by the agency. The number of users at each agency varies from one to several hundred. Each agency uses one of several options available for deploying the Pontis database. The product is shipped with a single-user version of the Sybase ASA product. Agencies may use the version of ASA provided with Pontis as their database management system, or they may upgrade to a multiuser version of Sybase or Oracle. This allows multiple users to connect to the database simultaneously and allows the agency to take advantage of other advanced functionality typically found in database management systems. As indicated in Figure 3, 50% of the agencies use Oracle as their primary database management system, often in conjunction with ASA used for field computers. Approximately 38% (13) use the single-user version of Sybase ASA shipped with Pontis, while the remaining 12% (4) use a multiuser version of Sybase.

Figure 4 summarizes what functionality is used by the different agencies. As shown in the figure, half of the agencies (17) use Pontis solely for inspection data collection and management through the Inspection module. These agencies do not, as of the time of this writing, use the advanced functionality of the system for performing program simulations or developing project plans. Approximately 12% (4) of the agencies use the advanced functionality of the system but do not use the Inspection module. Two of these agencies use systems integrated with the Pontis database for collecting inspection data, while two use external systems and have developed procedures for importing needed data into Pontis.

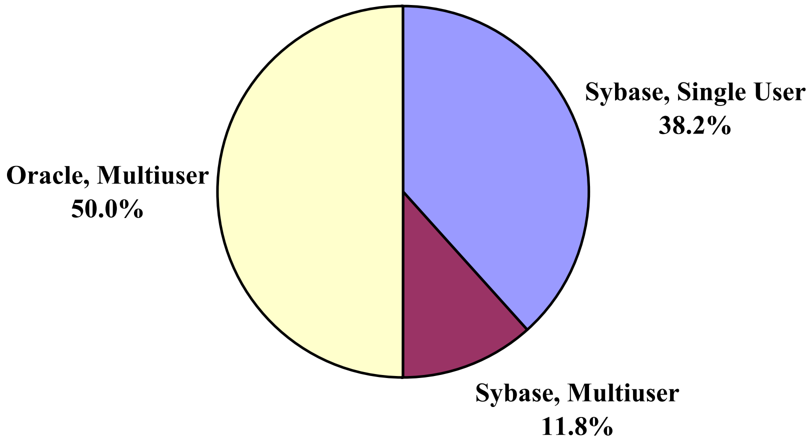

FIGURE 3 Approaches to database deployment used for Pontis.

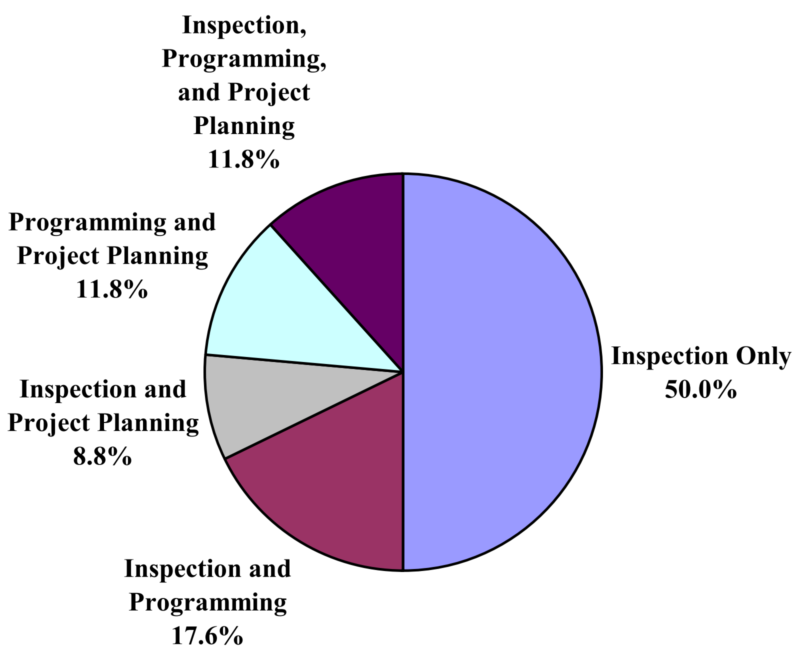

FIGURE 4 Pontis functionality used by licensing agencies.

Also shown in Figure 4, approximately 41% (14) use the program simulation functionality in Pontis (included in the Programming module) to analyze bridge needs at the network level. These agencies use the Programming module in conjunction with the Inspection module, the Project Planning module, or both. Approximately 32% (11) use the project planning functionality (included in the Project Planning module) in conjunction with the Inspection module, Programming module, or both. The level of use of the Project Planning module appears to have increased significantly since the last examination of agencies' use of this functionality performed prior to the release of Pontis Release 4 (*6*).

Level of Agency Customization

Most of the 34 agencies interviewed have made extensive customizations to the product. Nearly all have customized their element definitions or data definitions, or both, used in the product. Approximately 59% (20) have customized their database in some fashion. Over 58% (20) have developed data import and export procedures for exchanging data between the Pontis database and other databases.

Figure 5 summarizes the extent to which agencies have customized Pontis. The figure shows that most agencies using Pontis have customized the product. Approximately 46% (16) have performed moderate customizations. These include customizations that can be made through the product or through Infomaker, including developing forms, reports and desktop layouts, as well as customizing product behavior. Approximately 36% (12) have made major customizations, including development of applets or external applications to connect to the Pontis database. Over one-third of the agencies have used all of the basic approaches to customizing Pontis, including developing reports and desktop layouts, forms, and additional applets or applications. Approximately 18% (6) have made no customizations to the product except for customizing element definitions.

STATE OF THE PRACTICE IN PONTIS IMPLEMENTATION AND DEVELOPMENT

This section presents the state of the practice in current efforts to implement Pontis and develop additional functionality to complement the system by the agencies licensing the product. Several primary themes in Pontis implementation and development were observed through the interviews discussed in the previous section. These include an emphasis on customizing Pontis and

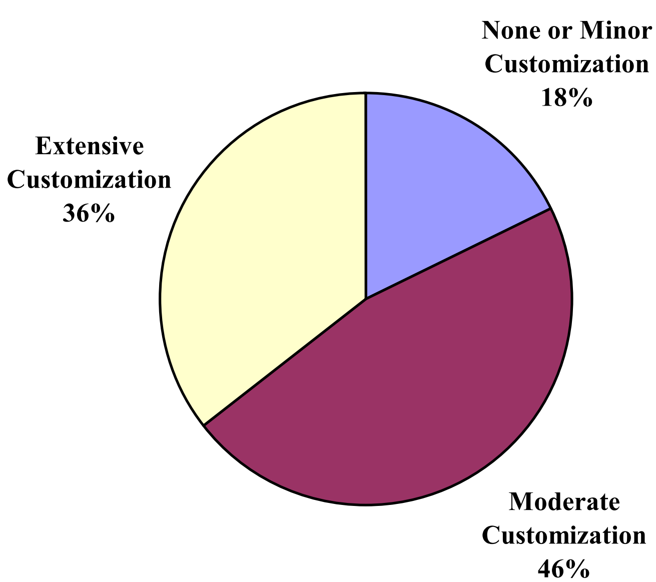

FIGURE 5 Extent of Pontis customization.

developing additional applications for supporting agency business processes, interest in developing thin-client applications, and a focus on using Pontis to implement asset management concepts. The discussion in this section is organized around these prevailing themes, with relevant examples of Pontis-related software implementation and development efforts currently being undertaken related to each.

Supporting Agency Business Processes

The primary reason that most agencies licensing Pontis customize the system, or develop additional applications to work with the Pontis database, is that they need to tailor the system to better match their agency's business processes. Given the diverse needs of the many agencies using Pontis, and the flexibility offered by the system for supporting customization, agencies have generally found that customizing Pontis provides an effective means to reconcile differences between the product's default settings and behavior and the needs of the agency.

The following are representative examples of efforts agencies are taking to customize Pontis to support their processes. The experience of the South Dakota Department of Transportation (DOT) provides an example of the customizations implemented in many agencies to support agency processes related to performing bridge inspections (*7*). South Dakota began supporting Pontis in 1993 and began customizing the system in 1996. Customizations performed by the agency include these:

- Adding six custom tables to the Pontis database to collect agency-specific data not included in the National Bridge Inventory (NBI) or the Pontis product.
- Developing four custom forms for use in the Pontis Inspection module for editing the agency-specific data items.
- Customizing database security.
- Customizing the procedures and files for using PDI to import and export data between the central database and field units. The customized procedures and files restrict the data that are exchanged based on agency processes and ensure that only the data from the latest inspection are sent from a field machine back to the central database.
- Developing custom desktop layouts.
- Developing procedures for updating the Pontis database with data from mainframe systems.

KDOT has made extensive customizations to its Oracle Pontis database to support the agency's processes. KDOT collects more than 100 agency-specific data items and has added these to the agency's Pontis database through customizations to the database and development of additional reports and data entry forms. Most of the agency-specific items are essentially NBI items collected at a more detailed level (e.g., at the structure unit level rather than the bridge level, or additional Kansas codes that map back to a single NBI code). An extensive set of database triggers has been established to calculate NBI fields automatically as a bridge inspector enters data in the agency's customized forms. In addition, the agency has developed custom reports and has developed a Powerbuilder applet for performing batch entry of inspection data, consistent with past agency practice. Further, the agency has developed an interface between the Pontis database and the agency's CANSYS-II system used to store information on bridges, pavements, and other assets.

Agencies such as the Illinois DOT have customized the behavior of the product to better support their practices for bridge programming and project planning. Illinois has defined its own set of elements as an alternative to the CoRe elements to better match its processes for inspections and recording costs and quantities. Further, the agency has developed an extensive set of program simulation rules to ensure that Pontis project recommendations are consistent with agency practice. To support these efforts, Illinois has made customizations to the Pontis desktop layouts and database to include additional data items relevant to bridge programming.

Developing Thin-Client Applications

Pontis has been developed as a client–server application. As such, it is typically necessary to install a client application (also called a thick client) on each machine running the system. The client can then access a database stored on a server or locally.

A number of agencies using Pontis have expressed an interest in implementing Pontis as a thin-client application. In this architecture, application logic is stored on a server and the footprint of the application on the client machine is minimal. This is an attractive approach to implementing management systems such as Pontis, for with this approach software deployment and maintenance costs are minimized. Also, training is typically simplified and security is enhanced, as there are fewer ways to access the Pontis database outside of the application. For the large number of agencies with geographically dispersed users, a thin-client approach is particularly advantageous.

Thus far, three agencies have developed approaches to using a thin-client architecture to implement Pontis or data collection applications tied to the Pontis database. The California Department of Transportation (Caltrans) and the Montana DOT have developed applications outside of Pontis that are used to update inspection and inventory data in the Pontis database. Both applications were developed in Oracle and allow users with access to a web browser to enter inspection data. At the same time, users in the central office have access to the Pontis application and can use Pontis to view the data being entered into the database. Both agencies use the Pontis database extensively but rely on the Pontis application primarily for use in supporting programming and project planning.

In the case of Caltrans, the agency's thin-client application contains additional screens for entering hydraulic and seismic data required for the agency. Caltrans distributes the Pontis application to local agencies with less stringent data requirements and uses PDI files submitted by local agencies to update the central database.

The Florida DOT (FDOT) has implemented thin-client support for Pontis using Citrix MetaFrames. With this approach, several hundred bridge inspectors, including state personnel and consultants, enter data into the agency's central Oracle database by using Pontis and additional Powerbuilder applets for collecting agency-specific data. Citrix software installed on each user's machine allows the user to run a copy of Pontis stored on a server without installing the application on the client side. FDOT is currently funding a series of enhancements to the Pontis product to improve bridge-level security and add multimedia support to the product. This functionality also will be supported using the MetaFrames approach and will enhance the functionality of the product for meeting the agency's needs.

At present, deploying Pontis using a thin-client approach requires an agency to invest significant additional time and effort. A number of other agencies have expressed an interest in making thin-client support a standard part of the Pontis product. AASHTO and its Pontis contractor Cambridge Systematics are currently engaged in developing the technical architecture for future versions of Pontis. As part of this effort, they are exploring the potential for supporting a thin-client approach in Pontis, particularly a thin-client version of the Inspection module.

Implementing Asset Management Concepts

As recognized by the Governmental Accounting Standards Board, Statement 34, asset management is an area of significant interest for U.S. state transportation departments, as well as for AASHTO and the FHWA. Asset management has been defined as "a strategic approach to managing transportation infrastructure" (*8*). Pontis provides exemplary support for implementing asset management concepts for bridges and other structures and includes functionality vital for implementing asset management at an agency, such as

- Determining the optimal maintenance policy for preserving a network of bridges;
- Allowing an agency to establish its funding needs and evaluate the impact of making trade-offs between different investments;
- Enabling detailed tracking of the inventory and condition of bridge assets, as well as the actions taken to preserve a bridge; and
- Supporting integration of bridge data with data for other assets.

In agencies such as the Michigan DOT and the Vermont Agency of Transportation, legislation has been passed requiring the agency to implement asset management concepts, highlighting the importance of implementing management systems. The Utah DOT has recently completed reconstruction of the Interstate 15 Corridor in Salt Lake City and has committed to using asset management concepts to maintain the corridor in the future. The agency is evaluating what changes are needed to its systems and business processes, including Pontis, to support this initiative.

Although Pontis provides excellent support for implementing asset management concepts even without extensive customization, agencies have nonetheless found it valuable to make customizations to the systems to support their asset management efforts. For example, agencies such as the Illinois DOT (described previously) and the Virginia DOT are pursuing implementation of the advanced programming functionality in Pontis, which incorporates asset management concepts. These agencies have carefully reviewed and customized the program simulation rules in the product to produce recommendations that best match agency practices.

At FDOT, Pontis has been customized with a number of additional element definitions. These new elements are used to describe other assets managed by structures personnel, including structure types such as tunnels and sign structures. By defining Pontis elements for these assets, FDOT can apply Pontis modeling functionality to assets besides bridges and manage these assets in an integrated fashion.

Data integration is an important component of asset management, for asset management emphasizes collecting quality data on all of an agency's key assets and bringing relevant data on those assets together to support integrated decision making and evaluation of trade-offs. Through its open database architecture, Pontis helps enable data integration efforts between Pontis and other applications—such as those recently performed in Michigan, Mississippi, and Kansas.

Recently, AASHTO completed an integration of the databases of its BRIDGEWare products—Pontis, Virtis, and Opis—into one integrated database. Additional work is being performed to explore adding support for extensible markup language to facilitate data exchange between BRIDGEWare databases and other agency databases. Additional opportunities for integrating data for additional assets and from other agency databases are being explored as part of the technical architecture work described previously.

CONCLUSIONS

The Pontis BMS provides a broad range of functionality for supporting bridge management. Of the agencies using the system, approximately half relies on the system primarily for supporting the bridge inspection process. The other half uses some combination of the system's functionality for inspection, bridge programming, and project planning.

Supporting agency customization is a critical component of the success of the Pontis BMS. Agencies licensing the software have access to extensive functionality for customizing the behavior of the Pontis product and the Pontis database. The product supports development of applets and external applications that can utilize the Pontis database. Agencies using the product have taken advantage of the customization options and rely on the functionality for customizing Pontis to allow them to tailor the product to meet agency needs. More than 80% of the agencies using the product have made customizations to the system, and more than one-third have made extensive customizations. Key themes in implementing and developing Pontis include customizing the system based on agency business processes, developing thin-client applications, and implementing asset management concepts.

Recognizing the diverse needs of the agencies licensing the product is an important step in considering how the product should evolve in the future and what functionality should be added. The study suggests that certain types of functionality would be valuable to many agencies, such as support for a thin-client architecture, and additional data integration and exchange functionality. However, future enhancements to the product should be made in a manner that preserves agencies' options for customizing the system to meet their unique needs.

ACKNOWLEDGMENTS

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REFERENCES

1. *Transportation Research Circular 423: PONTIS*. TRB, National Research Council, Washington, D.C., 1994.
2. Cambridge Systematics, Inc. *Pontis Release 4 User's Manual.* AASHTO, Washington, D.C., 2001.
3. Cambridge Systematics, Inc. *Pontis Release 4 Technical Manual.* AASHTO, Washington, D.C., 2001.
4. Subcommittee on Bridges and Structures of the Standing Committee of Highways. *AASHTO Guide for Commonly Recognized (CoRe) Structural Elements.* AASHTO, Washington, D.C., 1998.
5. Robert, W. E., A. R. Marshall, S. S. Lin, R. W. Shepard, and J. Aldayuz. Integration of Agency Rules with the Preservation Optimization Model in the Pontis Bridge Management System. In *Transportation Research Record: Journal of the Transportation Research Board, No. 1795*, TRB, National Research Council, Washington, D.C., 2002, pp. 74–81.

6. *Transportation Research Circular 498: Comparison of Pontis Bridge Project Recommendations to Programmed Work for Three U.S. Transportation Agencies*. TRB, National Research Council, Washington, D.C., 2000.
7. *Pontis States and Actions*, Vol. 8. AASHTO, Washington, D.C., 2002.
8. *NCHRP Web Document 41:* Phase *I Report. Task 2 of 3: Asset Management Framework*. Project SP20-24[11]). TRB, National Research Council, Washington, D.C., Feb. 2002.

MANAGEMENT SYSTEM IMPLEMENTATION

TISBO Infrastructure Maintenance Management System
Integrating Inspection Registration and Maintenance Management

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To practice output management, a more transparent and balanced determination process and a higher quality of justification for nationally desired maintenance budgets were required by the government of the Netherlands. Output management aims at allocating and justifying resources in relation to social demands or desired performance. With the aim of integrating functional, economical, and technical considerations into a standardized and transparent management process, the Dutch Ministry of Transport, Public Works, and Water Management (Rijkswaterstaat) is in the last stages of developing an Infrastructure Maintenance Management System, named TISBO. This computer program integrates inspection registration and maintenance management to produce rationalized and justified short- and long-term maintenance programs and required maintenance budgets on a local, regional, and national level in a controlled and transparent fashion.

The Dutch Ministry of Transport, Public Works, and Water Management (Rijkswaterstaat) maintains all infrastructure of national importance, which consists of roughly 6,500 structures, such as bridges, dams, and locks; 3,000 kilometers of road surfaces; and several hundred kilometers of rivers and canals; and more. Until recently, allocation of the budgets to maintain this infrastructure was largely determined by a combination of technical condition (restoring any imperfection as it surfaced) and organizational turnover capability. Budgets were set yearly, a practice that introduced disadvantages, including (*1*)

- A combination of over- and underachievement that resulted in a partially unutilized available budget,
- Insufficient consideration for or relationship with social benefits associated with maintenance,
- Subjective grounds for prioritization and allocation and reallocation of funds, and
- Insufficient insight into long-term financial maintenance needs.

Inefficiencies in fund allocation, combined with insufficient management means to respond, created a situation in which essential maintenance was being postponed for too long, which resulted in higher costs, and unnecessary or less effective maintenance was being implemented to realize turnover goals.

To increase the effectiveness of initial fund allocation and gain greater insight into long-term needs, a program management model was introduced (Figure 1).

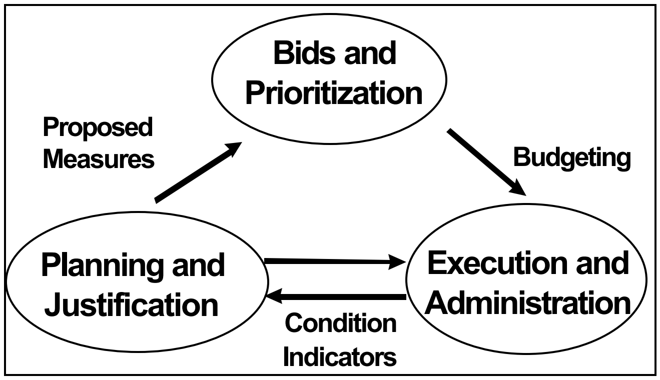

FIGURE 1 Program management model.

PROGRAM MANAGEMENT MODEL

The program management model contains three processes (*2*):

1. Bidding and prioritization,
2. Execution and registration, and
3. Planning and justification.

The maintenance measures are prioritized on the basis of the proposed measures relative to the desired quality of the infrastructure. Implementation of maintenance or inspection measures is based on this prioritization and the available budget. After maintenance or inspection is performed, the information on the condition of the structure is updated. These condition indicators are used for planning and justifying measures the next time round.

CURRENT SYSTEMS THAT SUPPORT THE PROGRAM MANAGEMENT MODEL

The program management model is already used to allocate maintenance budgets. At this time, a wide range of systems is used by maintenance managers to acquire the necessary information for producing an integral maintenance program. These systems, however, are not related. The data they house partially overlaps, although each system was developed for different purposes. This results in data inconsistencies, information holes, and inefficient data management. Four types of systems can be distinguished, all of which describe the relevant infrastructure at one or more levels.

Inspection Registration Systems: DISK for Structure and ROEVER for Riverbeds

These are the main inspection registration programs in use at this time. From inspection reports, the maintenance manager is advised on the maintenance to be done.

Maintenance Planning Systems: TISBO 1 and MaatPlan

Both systems are tools developed earlier for the manager in drawing up a maintenance management plan for up to 15 years. Standard maintenance measures (intervals and costs) for each type of element are the input for this planning. The disadvantage of these systems is that they do not run on a central database and are therefore difficult to maintain. The desired transparency is only partly achieved. No direct link to inspection results that show maintenance needs exists, except for a paper report acquired from the inspection registration system.

Maintenance Programming Systems: PROWEG and BOPPER
These programs are used for bids and prioritization. Maintenance measures for one or more structures are grouped on the basis of functional effect, practical feasibility, and urgency. This maintenance program is offered to the central office by a regional department and constitutes a bid. From this, maintenance measures are prioritized. Bids are composed of three levels:

- Rough estimate for a long-term plan 6 to 10 years ahead,
- Detailed estimate for 3 to 6 years ahead, and
- Prioritization for 1 to 2 years ahead at a point when budgets have been fixed for each regional department and only marginal reallocation is desired.

At this time, the link between the manager's maintenance plan and the bids is not always clear. The level of justification is often insufficient.

Project Management Systems: SAP
Once budgets are fixed, a maintenance project can be executed. SAP is the project management system that monitors operational progress and manages business administration processes. This system operates in the execution and registration stage.

Integrated Management Systems: IVON
This road management system integrates inspection registration and maintenance planning for road surfaces. This sophisticated tool provides a good justification for road surfaces. IVON is not suitable for structures. Similar to TISBO, it operates in the planning and justification stage.

NEW TISBO PROGRAM

Integrating Survey Data, Functional Demands, and Maintenance Prognoses
The new TISBO (*2*) is intended to replace and integrate several of the systems named earlier—DISK, ROEVER, TISBO 1, and MaatPlan. It will communicate directly with the PROWEG and BOPPER systems. This will synchronize the bidding and prioritization process and improve transparency. Because of its character, IVON, the road management system, will not be integrated with TISBO, and the project management system, SAP, also does not yet communicate with TISBO. TISBO is a Web-based application that runs on a central server. A fair amount of shared knowledge is put into the central database and is available for each user. Figure 2 presents an overview of TISBO modules.

Infrastructure System and Functional Demands
Output management aims to adjust required maintenance to fit a defined performance. The goal is to maintain a functioning infrastructure against minimal costs and with minimal interference (*3*). To achieve this, it is required to break down existing infrastructure into manageable units and to define the functional demands per unit, as shown in Figure 2 in the boxes labeled "infrastructure" and "policy."

Functional demand finds its origin in policy and bills passed through the parliament. Politically relevant functional themes have been defined for managing performance and accountability to society on a political level (e.g., environment, transport, recreation, water quality, land protection, traffic safety, and mobility). Functional goals and demands are defined

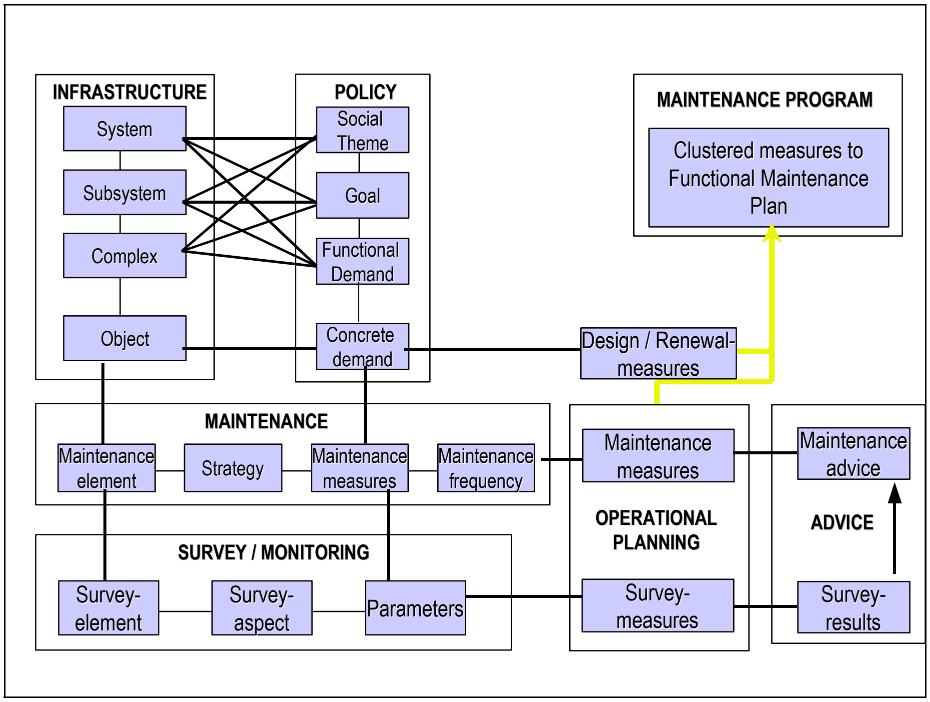

FIGURE 2 Contents figure of the new TISBO program.

for the main infrastructure types (e.g., open waters, rivers, canals, and main traffic routes). This infrastructure is managed and maintained by regional departments.

Infrastructure System
Each main infrastructure type within the boundaries of a regional department is called a system. For each system, nationally defined goals and demands per theme are set or declared relevant and translated into regional goals and demands. This also permits regional departments to communicate their interpretation of policy to the national level.

Infrastructure Subsystem
Different systems within a regional department are divided into subsystems for further delineating goals and demands for parts of infrastructure to realize the overall goal or demand. These units can be managed on a different level within the organization.

The existing infrastructure has been described as a collection of facilities that realize defined goals and demands and are considered functioning and thus are functional. Further breakdown requires focus on types of facilities that contribute in a specific way or types of facilities that have a specific impact on a functioning infrastructure. For example, structures, water, road surfaces, water banks, and waterbeds are distinguished.

Infrastructure Complex or Group of Objects
Earlier defined goals and demands will be selectively relevant for different types of facilities. Each facility type's technical character requires further translation of policy, which is also desirable. Groups of facilities that have similar goals, demands, and technical characteristics are formed as practical units for managing, monitoring, and maintenance. Such a unit is called a complex.

Until now, descriptions of goals and demands are still abstract and difficult to measure. Monitoring can only be achieved on a larger scale with a birds-eye-view and a higher degree of subjectivity.

Infrastructure Objects

Each type of facility is broken down into objects. As with complexes, objects are distinguished by their contribution to or impact on goals and demands and their technical characteristics. For example, distinctions are made between locks, tunnels, static bridges, viaducts, buildings, dams, and opening bridges. These objects are all tangible and recognizable in the field. As such, functional goals and demands on this level of description can be translated into concrete demands that can be measured by monitoring performance and that are similar to a set of design demands or design specifications. Monitoring these specifications can lead to measures to build additional facilities (new object or object modification) or to readjust goals and demands (e.g., adjusting realization time or quality).

These concrete demands are also tied to the specific technical demands put to each element that makes up the object. This is the most detailed level of description, which is used for management, accountability, and bid negotiations.

All of the above assumes that insight is achieved into the long- and short-term costs associated with defined functional goals and demands on different levels of detail infrastructure description (management tool).

On the level of elements, a further specification of technical demands enables us to determine the necessity and impact of maintenance and also generate an objectively determined sense of urgency as justification for aggregated costs on the level of infrastructure objects and higher levels of infrastructure description.

Decomposition

Each object, such as a bridge, lock, or tunnel, is decomposed into maintenance and survey elements, as shown in Figure 2 in the boxes labeled “maintenance” and “survey/monitoring.”

Maintenance elements are groups of one or more survey elements used for maintenance planning purposes. Survey elements are usually one level deeper in detail. Survey results are registered at the level of these elements. Reference decompositions, which describe the relation between maintenance and survey elements, are used for all common object types.

Each element is related to a standard element type. Standard element types are available in a shared national database. Standard element types are general element descriptions with a predetermined set of available information or required information to further specify the subject element. The following information is accumulated:

- Element characteristics. These characteristics differentiate the possible forms of the element type. For example, the element type bridge deck has an element characteristic form that contains a value or “orthotrope.”
- Discipline. Maintenance and inspection are usually organized by technical discipline, such as steel, concrete, electric, and mechanical.
- Quantities. A set of relevant quantities is needed for maintenance management and should be determined (m^2, mm, etc.).
- Survey instructions. Standard descriptions for element aspects are checked (good or bad) or measured (value) during inspection.

- Repair instructions. Instructions are required for standard maintenance programs
- Maintenance measures. Standard maintenance measures are defined with a default unit price per quantity and at a default interval.

This set of standard ingredients is available for each element in an object. The central database is kept up to date for each element type on a national level. Each local maintenance manager can add local data to the standard set, which the maintenance manager must keep up to date.

Maintenance Strategy

Maintenance strategies are described in a general sense as "reference documents" for most element types. These strategies are translated into standard maintenance measures per element type in the central database. For simple objects such as most highway bridges, the maintenance manager has only one function: to sustain traffic accessibility within the boundary condition of traffic safety. In most cases, the manager can adopt the offered standard maintenance strategies for planning purposes. For multifunctional structures such as locks—which are relevant to flood protection, transport, ecology, and recreation—maintenance strategies are determined to fit the purpose. Failure analyses are performed, and critical elements are determined per function. From these analyses, optimal inspection and maintenance strategies are determined. Maintenance measures for the most critical elements have a defined intervention level based on a measured parameter, as shown in Figure 2.

Survey Strategy

Survey measures are planned on the object level either incidentally or periodically. The objective of the survey is to update the maintenance plan with the data on the actual condition of the bridge. Survey planning is geared to maintenance planning, as shown in Figure 2 in the box labeled "operational planning." Survey measures have a unit price and an interval and consist of the elements to be inspected. Sometimes survey instructions, which may or may not be replenished with inspection parameters, are specified.

Operational Planning and Maintenance Program

Analogous to the maintenance programming systems, operational planning is roughly divided into three stages:

- The long-term plan. This plan is a rough estimate of the maintenance to be done 6 to 15 years ahead. Most figures in this plan are not supported by survey results.
- The mid-term plan. This plan is a detailed estimate of the maintenance that needs to be performed 3 to 6 years ahead to sustain the desired functions. The bidding and justification take place during this period. Maintenance measures are clustered to economical and financial optimal groups of work that are combined in a proposed maintenance program. In case of insufficient budget, measures are prioritized, and new bids are issued.
- The short-term plan. This consists of maintenance projects in preparation. During this period a contract survey is performed to estimate the exact costs of the planned maintenance. At least 1 year ahead of execution, the final budget is allocated.

Figure 3 illustrates how the planning grows in accuracy from the long-term plan through maintenance execution (*4*).

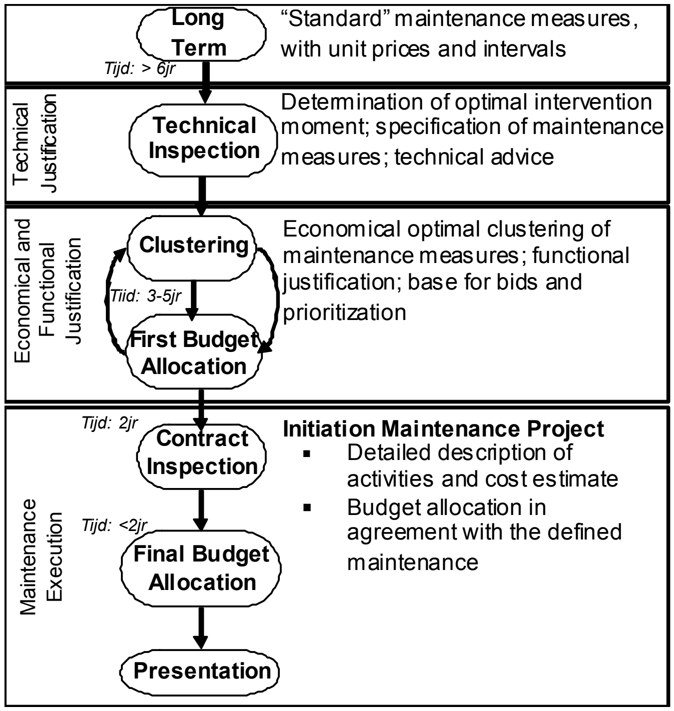

FIGURE 3 Accuracy of the planning in time.

Survey Results

Survey registration can be based on a survey instruction or visual defect. The result of survey instructions can either be a "good" or "bad" indication or a measured value. Visual defects are described in a standardized manner. The central database provides a set of standard damage descriptions. The inspector may specify the defect in a free comment field. Both for survey instructions and defects, inspectors are asked for their opinions on the seriousness of the function at stake. The inspector can recommend any of the following:

- Improve the element, if repair or replacement will not help;
- Replace the element;
- Repair the element;
- Conduct a detailed study, if the cause of the defect is not clear;
- Monitor a defect in time, in which case a measurement instruction should be defined for future inspections; or
- Leave a default unrepaired, if the defect has no consequences for the object's functions.

The inspector groups the inspection results into maintenance actions. For instance, corroding reinforcement, concrete cracking, and a measured high chloride content result in a maintenance action: concrete repair.

Maintenance Recommendation

The maintenance recommendation provides the technical justification for the maintenance plan and consists of the following steps (as shown in Figure 4):

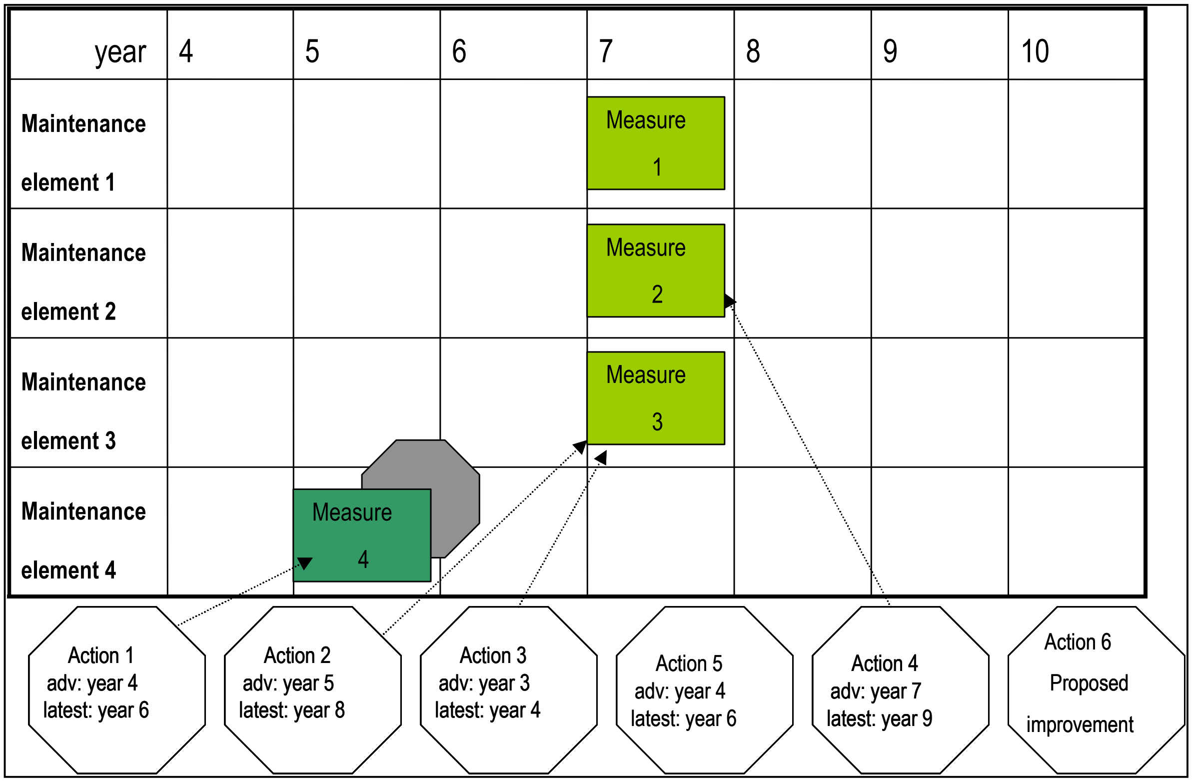

FIGURE 4 Justification of maintenance measures.

- The maintenance actions (group of indicators, such as defects and measurements) are supplied with a recommendation of the optimal and latest moment of action.
- The maintenance actions are related to the maintenance or inspection measures in the maintenance plan.
- New maintenance and inspection measures are planned on the basis of those actions that do not comply with the planned measures.

If a maintenance measure is planned earlier or later than the boundary conditions stated in the maintenance actions, then this measure is marked. The technical recommendation does not automatically lead to adjustment of the maintenance plan, because technical aspects are not the only considerations that determine the plan. As illustrated in Figure 4, Maintenance Measures 1, 2, and 3 are planned in Year 7. Action 3, however, shows that Maintenance Measure 3 should be performed in Year 3 or 4, whereas Maintenance Measure 1 is not justified by any maintenance action. Given this information and the manager's functional requirements, the manager should recluster the activities.

A technical advisement is not needed when intervention parameters are defined for maintenance measures beforehand. Measurements of critical parameters can be a technical justification for maintenance without defining a maintenance action (clearly, because the relation between the measurement instruction and the maintenance measure has already been made).

CONCLUSIONS

TISBO is an infrastructure maintenance management system that will help achieve the political goal of output management for the maintenance budget. TISBO's great advantage over most other maintenance management programs is that maintenance management and inspection registration are integrated. TISBO is proactive instead of reactive; maintenance measures are based on a combination of strategy and inspection results, rather than solely on inspection results. Where most bridge management systems are primary inspection registration systems, which translate inspection results into condition parameters, TISBO is primarily a management program. Money is not earned by postponing activities as long as possible, but by doing maintenance effectively and efficiently. Maintenance is done to sustain a desired functional quality level in the infrastructure, and the justification process is transparent. Prioritization is performed on the basis of the functional aspects rather than the locally available budget.

The TISBO program uses a central database. This makes it possible to supply and improve maintenance knowledge at the national level. It offers great opportunities for national analyses of the database and for learning from structure behavior in practice.

REFERENCES

1. Bakker, J. D., and E. Plooij. Relating Survey Results to Management Data for Maintenance Budgets. *Structural Faults and Repair 2001* (CD-ROM). London, July 2001.
2. Klatter, H. E., J. M. van Noortwijk, and N. Vrisou van Eck. Bridge Management in the Netherlands: Prioritisation Based on Network Performance. *IABMAS 2002* (CD-ROM). International Association for Bridge Management and Safety, Barcelona, Spain, July 2002.
3. Klatter, H. E., and J. M. Van Noortwijk. Maintenance of Structures Based on Functional and Economical Requirements. *Proc., Workshop on Optimal Maintenance of Structures*. Delft, Netherlands, October 2000.
4. Bakker, J. D., N. Vrisou van Eck, and E. Plooij. New Infrastructure Maintenance Management System to Support Output Management in the Netherlands. *IABMAS 2002* (CD-ROM). International Association for Bridge Management and Safety, Barcelona, Spain, July 2002.

MANAGEMENT SYSTEM IMPLEMENTATION

Florida Project-Level Models for Pontis

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In 2002 the Florida Department of Transportation (FDOT) developed a project-level bridge management decision support model to complement the existing network-level functionality of AASHTO's Pontis. This work was done as part of a series of research projects FDOT has undertaken in recent years to adapt Pontis to its own management processes and to build useful functionality on top of Pontis to fill unmet needs. In previous efforts, FDOT had developed a new user cost model, with emphasis on accident risk; a new model of maintenance, repair, and rehabilitation costs; and a new set of transition probabilities for the Pontis element-level deterioration model. All of these models were designed to fit within the existing network-level analytical framework of Pontis. In the current project, the Pontis analysis has been significantly extended to better serve the needs of project-level decision making. A graphical decision support tool was developed to help the engineer explore the economic trade-offs of scoping and timing inherent in project-level decisions. Developed in Microsoft Excel and Visual Basic, the tool uses Pontis network-level results for policy guidance and long-term economic consequences of actions, and draws on the Pontis inventory and inspection process supplemented with FDOT enhancements. A new analytical process was developed to generate candidate alternatives that are tailored to FDOT policies and that explore the range of practical scope and timing alternatives. Using the graphical presentation, engineers can quickly grasp the economic structure of each individual bridge and incorporate this perspective into their decision making.

Since 1998, Florida Department of Transportation (FDOT) has been implementing the Pontis bridge management system provided by AASHTO. More than 16,000 public bridges and other structures located statewide have been inventoried and inspected, including all bridges over 20-ft long, and numerous sign structures, high-mast light poles, and retaining walls. A centralized Pontis database with all of this information is accessible to, and primarily maintained by, the eight district offices. A custom application built into Pontis for work order management was deployed statewide in 2000. FDOT has recently begun a project to develop database security and document management enhancements to Pontis.

Frontline FDOT decisions regarding the maintenance, repair, rehabilitation, improvement, and replacement of more than 8,000 state-maintained bridges and other structures are made by district structures and facilities engineers (DSFEs). On a few major routes, some of this decision-making responsibility has been delegated to consultants through asset management contracts. The DSFEs and their staff also provide technical assistance and inspection services to local governments for more than 8,000 additional structures. As in most transportation agencies, decision-making authority is shared: DSFEs initiate work plans, but these plans are negotiated in an annual process with the Headquarters Maintenance Office, from a policy perspective, and the Work Programs Office, from a funding perspective.

The department has engaged in a series of research projects to adapt Pontis to its own needs and to provide the necessary planning input data for the system's decision support models. These efforts have resulted in several products:

- A new user cost model with economic parameters derived from earlier research within Florida. A significant part of this model is a new accident risk model based on bridge roadway width, approach alignment, traffic volume, number of lanes, length, and functional class. This was developed using Pontis bridge data and a Florida database of crash statistics (*1*).
- Unit costs of all maintenance, repair, and rehabilitation actions defined in Pontis for Florida's structural elements. Florida uses most of the AASHTO commonly recognized (CoRe) elements (*2*) as well as a set of non-CoRe elements for movable bridge components, sign structures, light poles, decks, joints, and drainage systems. These costs were derived from historical project data in three existing information systems, supplemented by expert judgment (*3*).
- Transition probability models to predict element deterioration for all Florida elements. These were developed using an expert elicitation process (*3*).

In the current research several additional products have been developed:

- Failure costs—both the minimum cost needed to satisfy the requirements of the Pontis network optimization model and a maximum cost designed to represent the full economic impact of allowing an element to fail.
- Truck height histogram—describing the fraction of the traffic stream composed of trucks above any given height. This is used in estimating the detour costs associated with bridges having impaired vertical clearance. The truck height histogram was derived from new measurements of actual traffic by using laser equipment.
- Truck weight histogram—describing the fraction of the traffic stream composed of trucks above any given weight. This is used to estimate the user costs of bridges with low operating ratings. The model was derived from weigh-in-motion data collected in Florida.
- User cost model for movable bridge openings—contributing to the justification for replacement of movable bridges, which are quite numerous in Florida.
- A project-level decision support tool—incorporating Pontis network-level results along with all the products of the earlier research, to give DSFEs a clear picture of the economic health of a bridge and the economic implications of scoping and timing decisions for structure maintenance, repairs, rehabilitation, improvement, and replacement.

The latter product is the focus of this paper. The decision support tool is designed to be compatible with, and take advantage of, the existing Pontis network-level models (*4*) but is intended to be used as a part of project-level decision making. This means adapting the Pontis economic definitions and life-cycle cost model so that they are most useful in the context of individual structures, adding a few additional submodels to address certain project-level concerns, and building a display tool that is informative for scoping and timing decisions.

OBJECTIVES AND PHILOSOPHY OF PROJECT-LEVEL ANALYSIS

Pontis began in 1989 as a purely network-level model, and it continues in release 4.1 to take primarily a network-level view of bridge management decision support. Florida, like most states, has decision-making processes at both the network level and project level. The requirements of project-level decision support can readily be explored by contrasting it with the network level:

- Network level focuses on the uniform processing of groups of bridges. Project level focuses on one bridge at a time.
- Network-level inputs concentrate on uniform rules for scoping and cost estimation. Project-level engineers are expected to make these decisions individually.
- Network-level analysis uses techniques, like simulation, suitable for automating decisions over large groups of bridges. Project level uses techniques that provide quick feedback on a larger number of bridge-specific decision variables.
- The primary modes of presentation at the network level are lists of bridges and network-wide summaries. The primary modes at the project level are lists of elements and needs on one given bridge and predictions of future conditions and performance of that bridge and its elements.
- Network-level optimization in Pontis most conveniently divides the inventory by element type. Project-level analysis divides it by bridge.
- Network level can use only data that can be cost-effectively collected systemwide. Project level can use data that may be collected for only a few bridges.
- Network-level costing has, as its most important objective, using methods that produce network-wide budgetary requirements that are sufficient and realistic, even if project-level estimates are imprecise. Project level is more concerned with precision and realism of each bridge individually and not as concerned that a methodology can be automated across the whole inventory.
- At the network-level, every bridge contributes probabilistically, in at least a small way, to the expected value of funding requirements during the programming horizon. At the project level, only a few bridges are realistically considered for implementation.

Obviously the network level and project level are very different. They are complementary, because both perspectives can be used together in an agency's bridge management process. They also are linked: The network level contributes predictive models (e.g., deterioration, life-cycle costs) needed by the project level for evaluating possible outcomes of decisions; the project level produces a set of candidate projects, with costs and benefits, that can readily be used in a network-level priority-setting and budgeting analysis.

Significantly, the project-level perspective allows more data to be collected cost-effectively because such data are needed only on a small number of bridges. Some of the most difficult issues in bridge management today may benefit from this orientation. For example, a project-level deck analysis can incorporate material testing data; cost models can include indirect costs and work zone user costs; and vulnerability analysis can focus on those structures for which vulnerability is an issue. The project-level analysis tool can be a powerful test bed for new research in these areas.

For more immediate use with currently available knowledge, the main benefit of a project-level model is the ability for project-level decision makers, primarily the DSFEs and others with whom they must cooperate, to interact with Pontis data at a level with which they are comfortable and can take maximum advantage of the resources available in Pontis. Much of the usage of the tool will be during an annual process of program negotiation known as "gaming," which occurs during the fall of each year for the program period beginning the following year.

LIFE-CYCLE COSTING FRAMEWORK

An important goal of the research is to develop project-level models that can work in concert with the existing network-level models of Pontis, relying on much of the same data and assumptions. This is a significant challenge: Pontis models are mainly probabilistic, focused on the inventory as a whole and not as much on individual bridges. The economic snapshot given by the project-level models needs to provide deterministic project scopes and costs, and it needs to evaluate candidates in a manner consistent with traditional life-cycle cost and net present value analysis.

Figure 1 presents a schematic example of a project-level model framework designed to satisfy these goals. The scale in the lower portion of the diagram shows the relationship in time between the point of decision ("today"), the most recent inspection, the time frame in which candidate projects are to be developed (start and end of program period), and the long-term outcome of the decision. On the vertical scale, filled areas indicate cost streams during and after the program period. The line chart overlaid on this depicts the typical pattern of bridge condition, expressed as a health index (*5*). The overall pattern of costs over time is called a life-cycle activity profile (*6*).

The programming process within FDOT considers specific projects as far as 9 years into the future, though only bridge replacement projects are important to the process that far away. Maintenance projects are programmed only 3 years into the future. It is assumed as a matter of policy that, once a project has taken place on a bridge, that bridge is not revisited until the 10th year following the project. A long-term cost model, described below, provides a general estimate

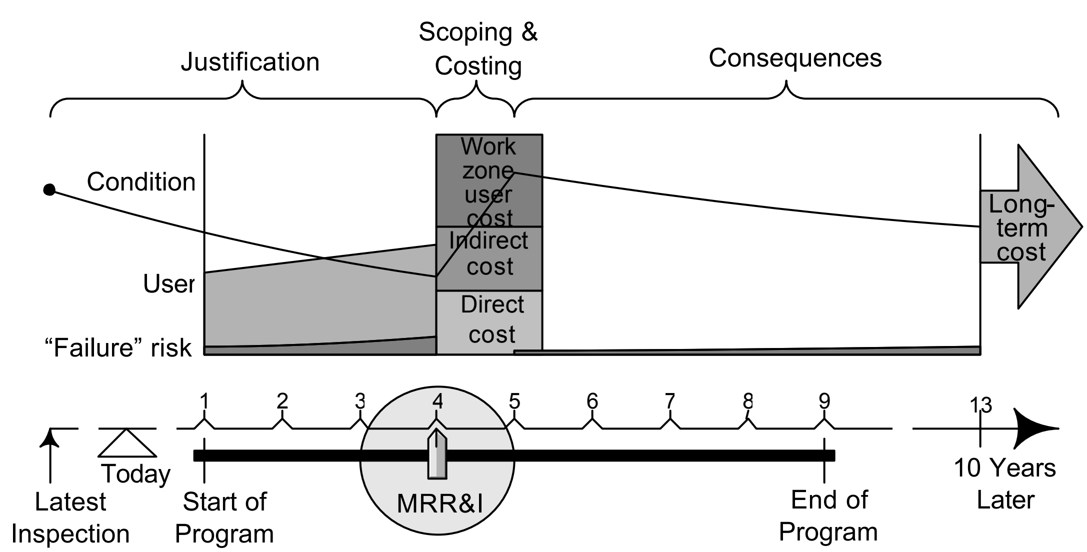

FIGURE 1 Project-level life-cycle costs.
(MRR&I = maintenance, repair, rehabilitation, and improvement.)

of life-cycle costs for the time beyond this required interval. As a matter of convention, inspections and condition forecasts are assumed to occur at the end of a year, while projects (including their costs and improved conditions) are assumed to occur at the beginning of a year.

The engineer is asked to make decisions about the scope and timing of candidate projects. Although the decision support tool supplies a great deal of useful information about bridge economics, the engineer also must rely on significant inputs from other sources:

- Status of the ongoing project development work flow affecting the readiness of individual projects;
- Information about funding availability for various types of work, from the Work Programs Office;
- Policy guidance from the Maintenance Office; and
- Information about interrelationships with other projects, including those of other districts, local governments, and asset management contractors, as well as nonbridge projects.

During the 2 to 4 months of the gaming process, the dynamic nature of these inputs and decisions will cause numerous adjustments in the scope and timing of work candidates. The project-level models need to be sensitive to these adjustments so as to inform decision makers of their implications. Much of this feedback is given in the form of conditions, deficiencies, and life-cycle costs. **Figure 1** shows the life-cycle cost components that are modeled, organized into three phases:

- Justification phase, which predicts deterioration from the latest inspection up to the year in which a candidate is being considered. If functional deficiencies (e.g., narrow bridge roadway, limited load capacity, impaired vertical clearance, and movable bridge openings) are present, there may be a user cost representing the adverse effect on the public. If conditions are very deteriorated, there may be a risk of loss of functionality, necessitating emergency repairs. This is called "failure," the same concept used in the Pontis network-level models (*4*).
- Scoping and costing phase, in which predicted needs at the investigated point in time are converted to a definition of a realistic candidate project. This candidate has direct costs, indirect costs (primarily maintenance of traffic and mobilization), and work zone user costs. It has an immediate effect on condition. The engineer can adjust the action selection and quantity at the element level.
- Consequence phase, predicting the long-term outcome resulting from the considered project. No further work is done for 10 years after completion of the project, during which time the bridge deteriorates. A risk of loss of functionality may occur, especially if the candidate project did not address all of the needs present on the bridge. A user cost may occur if functional deficiencies were not remedied. Beyond this 10-year waiting period, the Pontis network optimization provides a probabilistic estimate of subsequent life-cycle costs, sensitive to the ending condition of each element.

All of these costs are discounted to present value. If the timing of a candidate is delayed, user costs and failure risk costs may increase. Needs may increase, forcing an increase in the scope and cost of work. Offsetting these effects, the initial cost and long-term cost are discounted by a greater amount, since they are farther away in time. If a candidate is downscoped, then not all of its needs will be met. Even though the initial costs are lower, this may be offset by higher failure risk, possible user costs, and higher long-term costs.

All candidates are evaluated in comparison to a default "Do Nothing" candidate. This is the same as postponing work to beyond the end of the program period. The life-cycle cost of Do Nothing includes an elevated failure risk and long-term cost because of uninterrupted deterioration. A user cost may also be present. A work candidate is considered beneficial if it reduces user cost, failure risk, and long-term cost by an amount greater than its initial cost, all on a discounted basis. Economic benefit is calculated as the difference between the life-cycle cost of doing nothing, and the life-cycle cost of the candidate under consideration. Any benefit greater than zero is desired.

The project-level analysis automatically generates separate preservation and replacement candidates for each year of the program period so as to give the engineer a starting perspective on scope and timing. Typically, the engineer then may adjust the scope of the candidates to more realistically describe the choices available on the bridge. The tool responds by providing new evaluation results from the life-cycle cost model.

Justification

Justification is defined here as the portion of life-cycle costs that accumulate when needed work is not conducted on a structure. It consists of cost components that could be avoided if all deficiencies were to be relieved at the beginning of the program period. By convention, avoidable life-cycle costs are not recognized prior to the start of the first year of the program period (called the "base year"). If implementation of a candidate is delayed for any reason, its justification normally increases.

The FDOT project-level analysis tool currently estimates the following components of the justification phase of life-cycle costs:

- Failure risk. Pontis uses failure cost as a penalty for allowing portions of an element to remain in the worst-defined condition state without remedy. At the project level, the natural interpretation of this concept is the possibility of needing emergency repairs to maintain an acceptable level of service on the bridge.
- User cost of functional deficiencies. This includes the cost of excess accident risk and truck detours, according to the FDOT Pontis user cost model (**Figure 2**).
- User cost of movable bridge openings. This includes the delay to all road users caused by frequent opening of movable bridges to allow passage of ships.

To calculate these quantities as well as those needed for subsequent phases, the model simulates the deterioration of each bridge element from the most recent inspection to the year in which a candidate project is contemplated, using the same Markovian transition probabilities used in Pontis. When a portion of the element reaches the worst-defined condition state at least 1 year before the contemplated work, it is assigned a failure risk penalty. By convention, the failure penalty is recognized at the end of a year based on conditions forecast at the end of the preceding year.

WIDENING of roadway on the structure

INPUT DATA (all lengths in meters)

On_under:	On	DeckWidth:	10.300	FuncClass:	17	ApprAlign:	9
Length:	117.300	RoadWidth:	8.500	ADT (now):	6,314	DkRating:	7
Lanes:	2	ApprWidth:	7.200	Growth%	0.732		

This is a long bridge (>60 m.)
LOS width: ReqWidth = Lanes*LOSLaneWidth+2*LOSShldWidth = 2 * 3.4 + 2 * 0.9 = 8.600 m.
Design width: NewWidth = Lanes*DesLaneWidth+2*DesShldWidth = 2 * 3.7 + 2 * 2.4 = 12.200 m.
Level of service and design standards based on the roadway's functional class
***** RoadWidth<ReqWidth and RoadWidth<NewWidth so roadway needs widening.**

ACCIDENT COST PARAMETERS based on the FDOT accident risk model

Name	Coefficient	Description
Coef1	-377.3701	Constant (based on urban arterial functional class)
Coef2	0.7323	Coefficient for Lanes * Length
Coef3	0.3409	Coef for ADT * Lanes / RoadWidth (based on ApprAlign and DkRating)
AccCost	94,291	User cost per accident
Weight	1.0000	User cost weight

Accident risk = (Coef1 + Coef2*Lanes*Length + Coef3*ADT*Lanes/RoadWidth) / 1000
Excess cost = (Unimproved minus improved risk) * AccCost * Weight (but not less than zero)
Since ADT varies by year due to traffic growth, so does accident cost.

EXCESS ACCIDENT COST BY YEAR

Year	ADT	OldRisk	NewRisk	ExcessCost
2003	6,360	0.305	0.150	14,589
2004	6,407	0.308	0.152	14,696
2005	6,454	0.312	0.155	14,804
2006	6,501	0.316	0.158	14,912
2007	6,549	0.320	0.160	15,021
2008	6,597	0.324	0.163	15,131
2009	6,645	0.327	0.166	15,242
2010	6,694	0.331	0.169	15,354
2011	6,743	0.335	0.171	15,466

Long-term potential user cost (perpetuity with no growth) is $293,854 discounted to $175,941.
This is not yet capped at replacement cost.
Widening will add 645.150 m^2 of deck area and cost $412,896 ($640/m^2).

FIGURE 2 Example of detailed computation log.

Certain bridge elements—namely, expansion joint seals and drainage systems—exist primarily to slow the deterioration of other elements. The secondary effect of one element on another cannot be modeled effectively in the Pontis network optimization but is significant and should be addressed at the project level. During the simulated deterioration, this effect is modeled by changing the environment classification of protected elements (most superstructure and substructure elements) according to the predicted condition of protector elements on the same bridge. It is important that the transition probabilities used in Pontis for deterioration prediction describe the “median” behavior of bridges in the inventory, so the rules for deterioration adjustments are balanced to roughly equalize the number of upward and downward shifts in environment.

The justification phase of life-cycle costing was defined in a general way, as described here, for several reasons: It is a distinct part of the software code to implement the models; it has a clear delineation on a time scale, so the effect on project timing is easy to see; and it has a clear economic interpretation (avoidable costs) that can be expanded in the future. Potential future expansion of the concept could include the effect on agency and user costs of unmitigated vulnerability to natural or man-made hazards, and the economic harm of lost opportunities for preventive actions (e.g., waterproofing or cathodic protection systems) on bridge decks.

Scoping and Costing

Evaluation of the economic implications of project decisions—rather than automation of those decisions—is the main purpose of the project-level analysis tool. Therefore, automatic scoping of projects is not in itself a desired feature of the system. Nevertheless, it is very convenient for the tool to be able to create reasonable first-cut candidates, consistent with the Pontis network optimization, while respecting certain constraints on realism of candidate definitions. These initial candidates are not optimized, but they do provide a reasonable measure of need, project urgency, and economic merit. It is expected that the engineer will revise the scope of the project based on his or her own knowledge of the bridge.

Three candidate types are always generated automatically:

- Do Nothing—no action in any year of the planning period;
- Auto MRR&I—do a reasonable set of actions in response to all MRR&I needs on the bridge, in 1 year of the period (a separate life-cycle activity profile is generated for each of 9 possible implementation years); and
- Auto Replace—replace the bridge in 1 year (again a separate definition for each of the 9 years).

In addition, the engineer may specify up to three additional candidate scopes and analyze each one in any implementation year.

The process for generating the Auto MRR&I candidates is very similar to what is done in the Pontis program simulation, using the Pontis network optimization results to identify preservation actions on each element, and using level-of-service standards to identify functional improvements. A few refinements are imposed on this process to generate realistic project candidates:

- Minimum and maximum thresholds for feasible scale of actions;
- A rule requiring replacement of railings, joints, and drainage systems any time a deck is replaced;
- A threshold criterion for replacing the entire paint system on a bridge, rather than spot painting or overcoating; and
- Allowing an action to be larger or smaller than the quantity in states for which the action has been defined.

The latter point is especially important, because it gives the engineer a great deal of flexibility to scope a candidate project in any way he or she believes necessary, even if the action quantities differ from what Pontis would recommend. An output prediction model in the system divides up each scope item and assigns the parts to Pontis actions in a manner as reasonable as possible to determine the cost and effectiveness of the work.

A cost model estimates the direct and indirect costs of each candidate. The preservation costs developed in the earlier FDOT agency cost study were divided into direct and indirect components, based on rules of thumb that depend on the type of element. Currently, both direct and indirect costs are proportional to the quantity of work, but in the future the framework is designed to allow indirect costs to be constant or to vary in a nonlinear way with quantity. This is a high-priority topic for future research that may require collection of new data on traffic control and mobilization activities. Addition of a work zone user cost model is also a high priority.

Consequences

In the idealized life-cycle cost analysis, costs of a candidate project are assumed to occur on the first day of its implementation year, followed by 10 years of inactivity when the bridge is allowed to deteriorate. User costs may occur during this period if any functional needs are left uncorrected. The analytical framework allows for the possibility of further work within the programming period, but FDOT does not currently need this capability in its planning process.

At the end of the 10-year hiatus, it is not necessary to plan specific projects, but it remains necessary to estimate residual life-cycle costs. Such an estimate must be sensitive to the ending condition of each element: If work is done during the program period, ending conditions will be relatively good, and further needs will be relatively small, compared with the do-nothing case. Pontis provides in its network optimization an estimate of long-term costs, sensitive to the starting condition state and choice of action. To use this as an estimate of long-term residual costs, only a few refinements are necessary:

- Model 10 years of do-nothing deterioration from each condition state, and then calculate long-term costs from the resulting conditions by using Pontis optimal actions.
- Whenever a portion of an element reaches the worst-defined condition state, impose a failure risk penalty 1 year later in the same way as is done in the justification phase (described above).
- Long-term costs are assumed to occur exactly 10 years after the candidate project, since Pontis already accounts for all discounting beyond that. Failure risk costs are assumed to occur on the last day of each year, consistent with the way they are used in the justification phase.

As a matter of convention, the failure risk penalty is imposed during the 10-year period when no action is allowed, but it is not imposed afterward. This is because element failure is not allowed to occur in a Pontis optimal policy. If any functional needs are uncorrected, they are either assumed to continue indefinitely beyond the 10-year period, or a bridge replacement cost is incurred, whichever gives the lowest life-cycle cost. In the former case, traffic growth is assumed to stop. A topic for further research is to analyze the network-level effects of assumptions such as these, to determine whether they have much effect on the results, and to decide whether different assumptions are warranted.

DECISION SUPPORT TOOL

The framework described here was implemented as an Excel 2000 spreadsheet model, with most of the analysis written in Visual Basic for Applications. This proved to be a very effective tool in terms of system development cost as well as execution speed. Built-in features of Excel were used for most of the data management and user interface needs, so the majority of new software code was devoted to the analysis. Excel's programming model is quite stable, its worksheets easy to modify, and the analytical code nonproprietary. Together, these factors make the software an attractive test bed for further development of bridge management models in a research setting.

For production use, the system is very fast and is sufficiently secure for its intended use as a means of displaying analysis results. (It does not write data to the Pontis database but stores all of its results in Excel worksheets.) A full screening analysis of all 16,000 Florida structures takes just over a minute (on a 1.2 GHz Pentium III computer), with database access requiring an additional 1 to 4 min depending on the server configuration. In normal usage in which only one bridge at a time is accessed, the total time to access, analyze, and display the results is well under a second. This speed is extremely important to the usefulness of the system, as it encourages engineers to experiment with many alternative project definitions until they are satisfied with the results. This gives it the potential to become a true programmatic design tool.

All of the primary functionality of the tool is presented on one screen, as indicated schematically in Figure 3. This Excel worksheet is called the digital "dashboard" because, like the dashboard of a truck, it presents an organized set of gauges and controls for the model. Examples of some of the contents of the dashboard worksheet are the following:

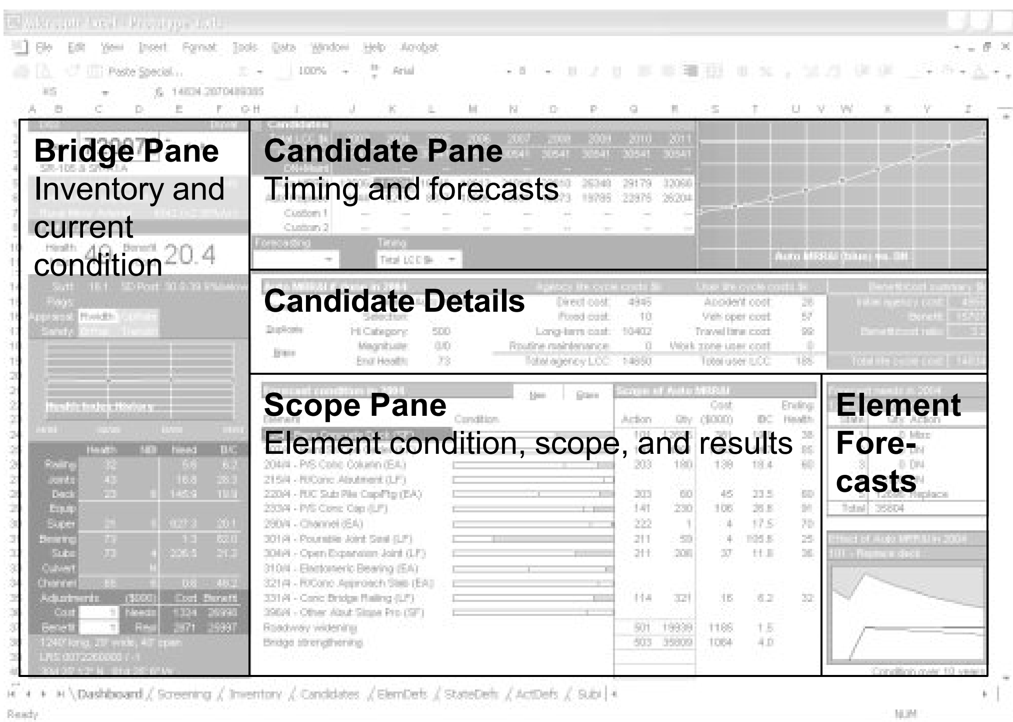

FIGURE 3 Main sections of the dashboard.

- Flags indicating functional deficiencies and vulnerabilities, including Smart Flags (*2*) noted in the bridge inspection process (**Figure 4**).
- A life-cycle cost analysis sensitive to project timing (**Figure 5**, explained below). This analysis can show total life-cycle costs, as in the example, or agency and user costs separately. It can also compare the timing implications of initial cost, action type, benefits, and benefit–cost ratio.
- The prediction of element conditions and anticipated preservation needs in any year in which a candidate project is being considered (**Figure 6**). The bar graph is interactive, changing to show condition each year as the cursor is moved, helping the engineer to visualize the relative deterioration rates of the various elements on a bridge.
- A forecast of future condition trends of an individual element if a candidate is implemented in a given year (**Figure 7**). The example shows an immediate improvement in condition in the first year if deteriorated joint seals are replaced, followed by normal deterioration.
- A forecast of the future condition trend of the bridge, if a candidate is implemented in a given year. This can be expressed as a health index, preservation needs, benefit–cost ratio, user cost, or excess accident risk.

The analysis of project timing exemplified in **Figure 5** is an important new capability. In this example, life-cycle cost is plotted against implementation year for each of three candidates. The Do Nothing candidate always has a flat line because no work is implemented in any program year.

In this example, bridge replacement has a higher life-cycle cost than Do Nothing, regardless of the implementation year. Therefore, the bridge is not an attractive candidate for replacement. The Auto MRR&I line shows the effect of deterioration. Life-cycle costs increase the more the work is delayed. In the first year, Auto MRR&I has a lower life-cycle cost than Do Nothing, indicating an attractive preventive maintenance opportunity that will be lost if the work is delayed. By 2011, replacement becomes more attractive than preservation, though still higher than the Do Nothing case.

Figure 8 shows another example. Here, both do-something candidates have lower life-cycle costs than Do Nothing. In the first 2 years, there is a preventive maintenance opportunity that becomes infeasible by year 2005. Beyond that point, replacement of the bridge is the only economical alternative.

Engineers use the dashboard to determine at a glance the economic health of a structure, and they use it as a design tool for candidate projects to enter the programming process. When the engineer modifies a candidate by changing element action selections, quantities, or various cost factors, the dashboard responds as a spreadsheet is expected to do, by immediately updating its predictive results.

In addition to the dashboard, the system has a screening worksheet containing a list of bridges in a selected subset of the inventory. This list may be sorted by various screening criteria, such as the sufficiency rating, National Bridge Inventory condition ratings, health index, type of work, cost, benefit, benefit–cost ratio, and economic urgency of actions. Economic urgency in this system is defined as the difference in life-cycle costs between the first and second years of the Auto MRR&I candidate generated by the models. It shows how much is lost by delaying action.

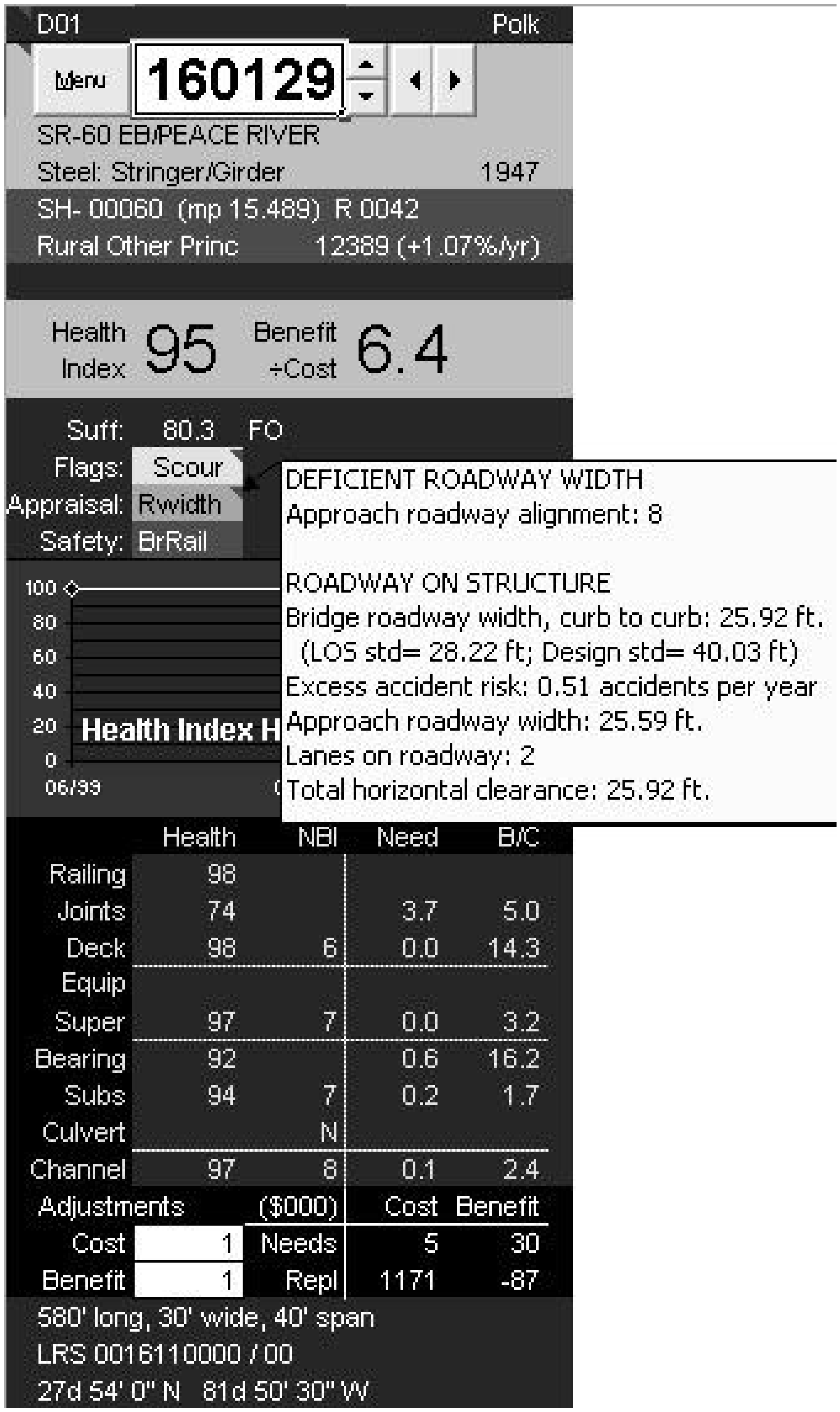

FIGURE 4 Bridge pane showing appraisal and smart flags.

Auto MRR&I if done in 2007		Agency life cycle costs $k	
New name	Auto MRR&I	Direct cost:	15
Dir. cost adjust (%)	0.00	Indirect cost:	6
MOT (%)	17.27	Near-term risk:	127
Mobilization (%)	12.58	Long-term cost:	131
Design+CEI (%)	10.00	Agency LCC:	275

User life cycle costs $k		Adj. benefit/cost summary $k	
Accident cost:	0	Initial agency cost:	20
Truck delay cost:	0	Agency benefit:	-39
Work zone user cost:	0	User benefit:	0.0
Long-term cost:	0	Benefit/cost ratio:	-1.9
User LCC:	0	Total life cycle cost:	275

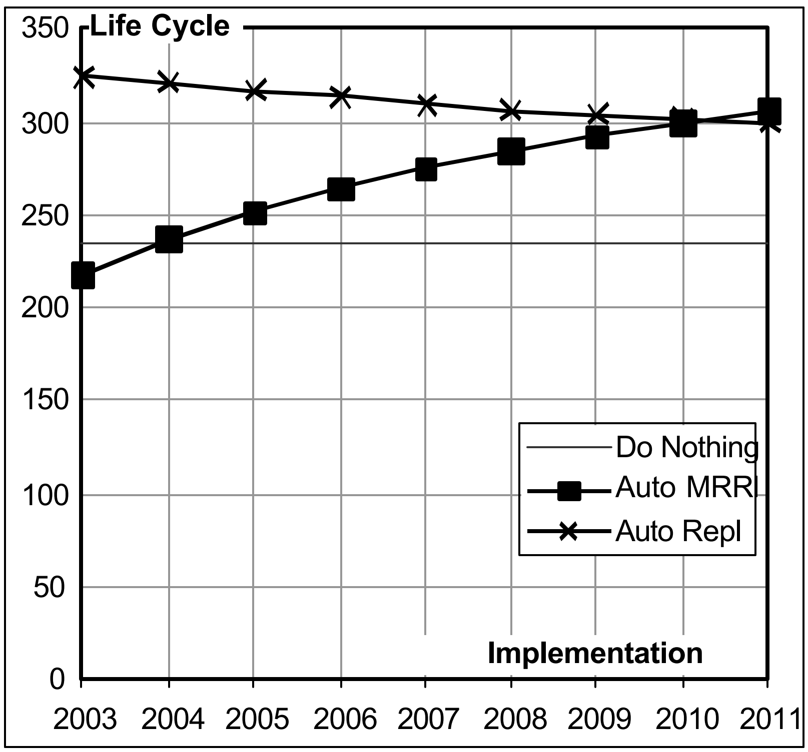

FIGURE 5 Selected portions of the candidate pane, using life-cycle cost to evaluate candidate timing.

Forecast condition Jan 1, 2011		Scope of Auto MRR&I		
Element	Condition	Action	Qty	Cost ($000)
39/3 - Unp Conc Slab/AC Ovl (SF)				
204/3 - P/S Conc Column (EA)				
215/3 - R/Conc Abutment (LF)		Repair	11.64	2
234/3 - R/Conc Cap (LF)				
290/3 - Channel (EA)				
301/3 - Pourable Joint Seal (LF)		Replace	137.80	10
321/3 - R/Conc Approach Slab (EA)		Rehab	1.24	0
330/3 - Metal Rail Uncoated (LF)				
333/3 - Other Bridge Railing (LF)				
396/3 - Other Abut Slope Pro (SF)		Rehab	2456	8
475/3 - R/Conc Walls (LF)		Maint	16.41	1

FIGURE 6 Predicted condition and candidate actions.

Forecast needs Jan 1, 2003		
301/3 - Pourable Joint Seal		
State	Qty	Action
1	0.00	DN
2	111.69	Cln&Reseal
3	26.11	Patch&Rsl
4		
5		
Total	137.80 (LF)	

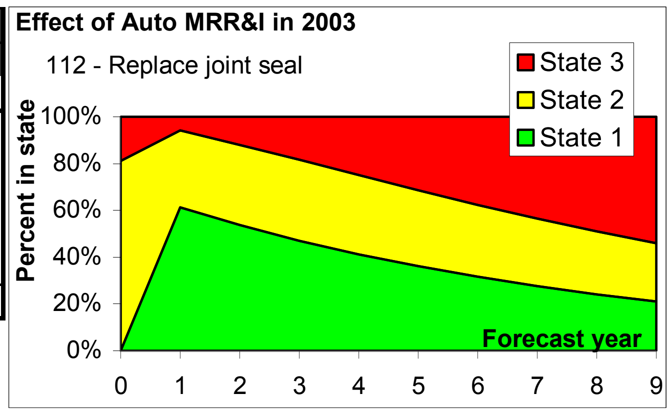

FIGURE 7 Element forecast pane.

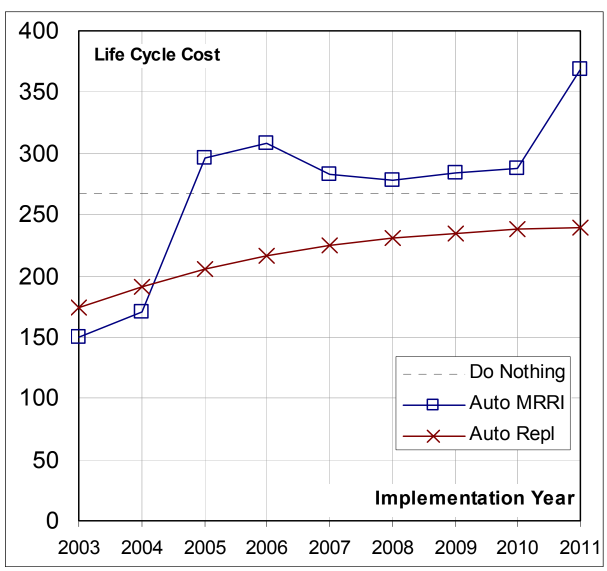

FIGURE 8 Example of candidate selection.

A third worksheet provides a detailed rationale for the results reported on the dashboard, describing the effects of each part of the project-level model. For example, **Figure 2** shows the analysis conducted for a bridge roadway width deficiency. The three main worksheets, supporting functionality, and help features are all accessed by means of a custom toolbar.

CONCLUSIONS

The Pontis analytical framework has always had the potential to support project-level decision making, but up to now it has not been easy to find this information or use it effectively. Project-level decisions are often influenced by noneconomic considerations, such as project readiness and interrelationships with other activities. Timing of the work, in particular, is an important decision variable.

By adopting a project-level perspective on the Pontis analysis, it is possible to define a new analytical framework that is sensible and valid when evaluating potential work on an individual bridge, while remaining compatible with the Pontis network-level models. When this information is presented in a suitable way, engineers who are planning a multiyear bridge work program can use the analysis to gain a quick, intuitive view of the economic health of a bridge and the urgency of completing work on it.

REFERENCES

1. Thompson, P. D., and F. T. Najafi. *Florida DOT Pontis User Cost Study*. Technical Report. Florida Department of Transportation, Tallahassee, 1999.
2. *AASHTO Guide for Commonly Recognized (CoRe) Structural Elements*. AASHTO, Washington, D.C., 1997.
3. Sobanjo, J. O., and P. D. Thompson. *Development of Agency Maintenance, Repair, and Rehabilitation (MR&R) Cost Data for Florida's Bridge Management System*. Technical Report. Florida Department of Transportation, Tallahassee, 2001.
4. *Pontis Technical Manual*. Release 4.0. AASHTO, Washington, D.C., 2001.
5. Shepard, R. W., and M. B. Johnson. California Bridge Health Index. In *8th Conference on Bridge Management: International Bridge Management Conference*. Preprints, Vol. II. TRB, National Research Council, Washington, D.C., 1999.
6. National Engineering Technology Corporation. *BRIDGIT Technical Manual*. Unpublished report. NCHRP Project 12-28(2), TRB, National Research Council, Washington, D.C., 1996.

MANAGEMENT SYSTEM IMPLEMENTATION

Integration of AASHTO's BRIDGEWare Products

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The AASHTO BRIDGEWare Task Force has launched an effort to link its bridge-related software products—Pontis for bridge management, Virtis for load rating, and Opis for bridge design—into an integrated suite of tools to be known as BRIDGEWare. The three software systems share many features, including a common client-server architecture and several individuals who have served on all three projects over the years. The task forces overseeing the products were recently merged. In 2002 development was completed of an integrated database and new versions of all three products to operate on this database. The BRIDGEWare Strategic Plan anticipates that, for the foreseeable future, the three products will remain separate for project management and licensing purposes but will gradually develop useful linkages through adoption of common standards and tools. For example, all products will share a common eXtensible markup language schema for data interchange and Internet-enabled functionality. Initially this standard will be put to work in an integrated report writer that can extract and merge data from any of the three systems. Shared capabilities for security and document management are also in the works. In the long term, the potential for exploiting this linkage is exciting and includes incorporation of life-cycle costs from bridge management into bridge design; greatly enhanced cost estimation for bridge construction and rehabilitation; incorporation of seismic, fatigue, and scour analysis in bridge management; and integration of freight movement operational policies, including permit management, with bridge design and management.

AASHTO develops and supports three software products (*1*) to address the needs of three distinct phases of the life cycle of transportation bridges:

- Pontis for inspection, management, and work planning for in-service bridges;
- Virtis for load rating of existing bridges; and
- Opis for design of new bridges.

Collectively, this suite of products has become known as BRIDGEWare. The products are intended to fully support

- Bridge engineering and bridge business processes of preliminary and final design;
- Routine and special inspections;
- Rating for posting and permitting;
- Analysis of deterioration and its effects;
- Planning of maintenance, repair, rehabilitation, improvement, and replacement activity at the network and project levels;
- Setting priorities and budgeting;
- Design of rehabilitation and improvement projects;
- Development of improved planning models;
- Satisfying federal reporting requirements and public information needs; and
- Ad hoc analysis and querying to satisfy management information needs.

Pontis is a comprehensive bridge management system that supports the entire bridge management cycle. It stores bridge inventory and inspection data; formulates policies for networkwide preservation and improvement for evaluating the needs of each bridge in a network; and makes recommendations of projects for an agency's capital plan that will derive maximum benefit from limited funds. A benefit of Pontis is its ability to translate engineering concerns of deterioration and costs into economic and policy performance measures for management in decision making.

AASHTO began developing Pontis in 1994, continuing development begun in 1989 by the California Department of Transportation and FHWA of earlier Pontis versions in the public domain. The AASHTO project converted the system to Microsoft Windows and adopted an enterprise relational database for managing bridge data. Subsequent work, funded from product license revenue, has enhanced the bridge inspection process and increased functionality in database management, report and program generation, and project planning. In 2002 Pontis had 48 licensees, of which 38 were in state transportation agencies (DOTs) and the District of Columbia.

Virtis and Opis were developed in tandem beginning in 1997 to replace earlier AASHTO software packages, BARS for bridge load rating and BDS for design.

Used for bridge superstructure load rating, Virtis features graphical tools that speed preparation of data and application of results. With Wyoming DOT's BRASS as its analytical engine for load factor rating, Virtis provides an integrated database where rating inputs and outputs, including potentially a three-dimensional (3-D) bridge description, can readily be stored, reviewed, and reused. These bridge data then can be used by line-girder, 2-D, or 3-D analysis packages, for permit and routing systems, and other third-party-produced applications. In 2002 Virtis had 71 licensees, of which 31 were in state DOTs.

Opis is currently a bridge superstructure design-review software product that uses the *AASHTO LRFD Bridge Design Specifications* (*2*). Opis employs the same database and graphical user interface as Virtis and shares much of its source code. BRASS-LRFD provides the system's structural analysis and specification checking engine. The Opis superstructure product in 2002 had 33 licensees, of which 22 were in state DOTs.

Sixteen states have funded the Opis substructure project, a 5-year effort that began in 2001. This project is developing a load and resistance factor design (LRFD) substructure system with a custom-developed analysis engine to handle the complex loading calculations of LRFD specifications for piers, walls, and foundations. The project will also develop design support tools for reporting, design process management, and comparison of design alternatives.

A major goal of AASHTO's strategic plan for the three products is to take maximum advantage of their synergy. All three systems have captured significant amounts of extremely valuable data that, if made sufficiently accessible, have potentially many uses. High-level objectives shared by the BRIDGEWare products include

- Developing the capability to access related data from all the products as needed to support existing and new applications, with an adequate and uniform level of reliability, coverage, security, and convenience;
- Saving development costs by sharing new software modules among all the products to the extent the new features add value to all of them;
- Saving licensee costs by streamlining testing, data management, and deployment of product updates; and
- Launching new applications of compelling value to transportation agencies, building on the existing products' combined data and functionality.

The idea of integrating BRIDGEWare products is similar to the integration of Microsoft Office Suite: each product maintains its own identity for development and licensing purposes, but portions of each system are gradually standardized and shared. The systems' interoperability and concurrent testing save money and create value for licensees.

A first step in product integration was the merger in 2001 of the Pontis and Virtis/Opis Task Forces to create the BRIDGEWare Task Force under a single chair. Although subcommittees and technical advisory groups (TAGs) continue to operate separately for each product, the full BRIDGEWare Task Force meets frequently and maintains an integrated strategic plan.

DATABASE INTEGRATION

In August 2002, the first tangible output of the integrated task force and strategic plan was the simultaneous release of Pontis 4.1, Virtis 4.2, and Opis 4.2. Although the three products can still be licensed and installed separately, they now share the ability to operate with an integrated database. This immediately gives the licensees of the combined suite a number of new capabilities:

- Running up-to-date releases of Pontis, Virtis, and Opis all together from one integrated physical database with appropriate referential integrity enabled, using approved versions of Oracle or Sybase Adaptive Server Anywhere (Each product may also support other databases according to the needs of its user community. For example, Virtis and Opis have an academic version using Microsoft Access.);

- Using InfoMaker, Excel, or any other open database connectivity–based report writer or analytical tool to develop reports combining any Pontis, Virtis, or Opis data items via a single data source connection;
- Implementing an integrated set of database security rights for an integrated user list for the combined database (separate from the security features built into Pontis, Virtis, and Opis);
- Combining or converting earlier versions of Pontis, Virtis, and Opis databases into the new integrated schema with cross-references among products;
- Sharing the same set of code tables (the Pontis Parameters table) in all BRIDGEWare programs for Pontis data items;
- Developing confidence in the database's stability with a change process that controls modifications to mature parts of the schema;
- Continuing to add agency-specific data items through Pontis user tables, which will now be as accessible as all other BRIDGEWare tables.

In addition, AASHTO or third parties will be able to develop new applications that are registered in the BRIDGEWare cross-reference table. This enables developers to create their own database tables to serve needs not covered by the existing products. Such bridges can be shared among any or all BRIDGEWare products in the same physical database.

A major issue encountered in BRIDGEWare database integration is the fundamental difference among the three products in the definition of "bridge." Pontis, Virtis, and Opis data tables describing a bridge as a whole differ as follows:

- The Pontis bridge table contains a large number of data items intended for bridge management. Most agencies have constructed reports and organizational processes that assume that all Pontis bridges are subject to bridge management processes, such as inspection, National Bridge Inventory (NBI) reporting, and maintenance and rehabilitation project planning.
- The Virtis bridge table contains data collected from bridges for load rating. Usually, the Virtis table lists a much smaller number of bridges than Pontis, though in most cases agencies have a long-range intention to rate a higher fraction of their bridges than is currently in electronic form. Virtis may contain bridges that are not required in Pontis, such as pedestrian bridges, railroad bridges, closed bridges, and bridges with spans less than 20 ft. Also, the inclusion of local bridges differs with states' bridge management and load rating practices.
- The Opis bridge table contains mostly bridges that have not yet been built. It is envisioned that structural data are entered as part of the design process, and after the bridge is built, these data are retained for subsequent load rating in Virtis and bridge management in Pontis. Depending on each agency's policies, a bridge that is recently completed but not yet open to service may need to be rated even though it is not yet ready to be inspected.

Another issue is that each agency may license any combination of Pontis, Virtis, and Opis. Thus, agencies with multiple products want easy migration of data among them; while agencies licensing only one product want to be sure that each product can stand alone. An agency that uses one product alone may at a later time add another product and want the ability to use the existing bridge data in the new product without reentering it.

In addition, each system has been designed and developed to serve a different set of requirements. Aside from identification information, the list of bridge data items stored in Virtis and Opis is quite different from the list in Pontis.

Because of these bridge and product life-cycle considerations, a bridge may be used in any combination of Opis, Virtis, and Pontis at different times. If the products were to share the same bridge table, the licensees would have to make major changes in the way they now use the three separate products, including changes to hundreds of Pontis custom reports. Similar concerns would arise if products were added to the BRIDGEWare suite later. The conclusion, then, was to minimize the impact on the separate products by constructing an "adapter"—a new linkage table—to connect the databases.

Figure 1 describes this linkage mechanism. Any structural asset (bridge, sign structure, tunnel, wall, etc.) used in any BRIDGEWare product is listed in the abw_asset_xref table. If the structure is used in Pontis, its identification and classification data appear in the Pontis bridge table; otherwise, this information appears in the abw_overflow table so it is available to all the other products. If the structure is used in Virtis or Opis, structural data appear in the abw_bridge table. The abw_asset_xref table contains foreign keys that identify specific records in each product where the structure is used.

When separate Pontis, Virtis, and Opis databases are first merged, a utility program creates and populates the linkage table by matching bridges from the separate databases. Each product has features that make it easy to adopt a bridge from one of the products, without data reentry. A bridge may also be deleted from any product without affecting any of the other products. The adapter mechanism minimizes the interdependencies among the three products, so each can continue on its own development life cycle. However, whenever a database is shared, changes in one product could potentially effect unintended consequences in others. In addition, it is desired to encourage third-party developers and licensees to create add-on modules for BRIDGEWare, and such developers need assurance that frequent database changes will not add to their maintenance costs. For this reason, a database change process, overseen by a Database TAG, was established. Each table in the integrated database is classified according to product interdependency, as follows:

- Green. Parts of the BRIDGEWare database under development are understood to be in flux, so end users and third parties are discouraged from relying on them. AASHTOWare

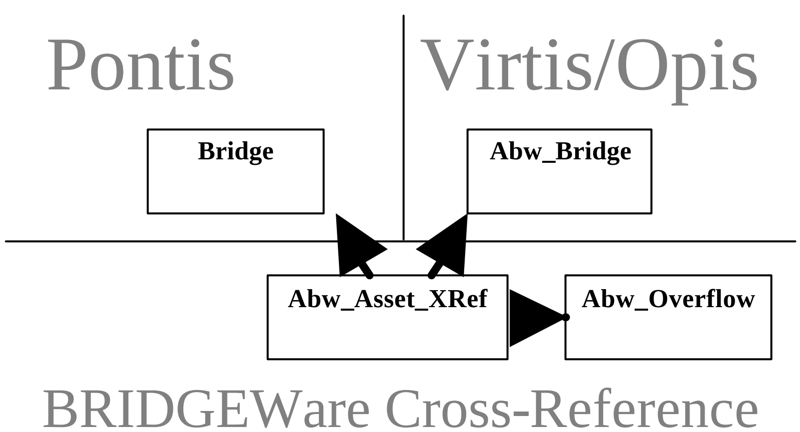

FIGURE 1 Database linkage mechanisms.

contractors are free to modify such data as often as necessary.

- Yellow. Certain parts of the BRIDGEWare database considered mature will be restricted to infrequent changes. End users and third parties are encouraged to create reports or applications from these data. Generally only one BRIDGEWare system is affected by changes to these tables, so release dates do not need to be coordinated among the products if only these tables are affected.
- Red. The tables abw_asset_xref, abw_overflow, bridge, roadway, paramtrs, and abw_bridge, are even more strongly restricted. Changes are very infrequent. Multiple BRIDGEWare systems are affected, so release dates must be coordinated among the products whenever these tables are modified.

The Database TAG determines whether a proposed modification to the yellow or red parts of the database justifies the disruption it would cause. This deliberation is based on factors such as the degree of usage of the items in question, the contractor costs to make the changes, the number of licensees wanting the change, and any others TAG wants to consider. Another important TAG mission is determining which parts of the database are to be designated yellow or red.

INSTALLATION AND PLATFORM SUPPORT

An important advance in the 2002 release is the ability to install and run any or all of the three BRIDGEWare products on the same computer through accessing the same integrated database or separate databases. Previously some of the licensees had trouble with this because of driver incompatibilities and dynamic link library conflicts.

To ensure this level of interoperability, it was necessary to catalog the many combinations of licensees' database managers, drivers, and operating systems and decide which ones to test. **Table 1** summarizes these platform choices. The official list is published in the AASHTOWare catalog (*1*) and is subject to change each year.

The BRIDGEWare products are true enterprise solutions, and as such they must coexist with many other products in each user organization's information technology environment. Each agency tends to customize its installation to best fit its own business processes and existing platforms. As a result, guidance is needed to help the organization perform this customization successfully. To meet this need, the 2002 releases featured the first integrated Startup Guide, which brings together in one place all the possible installation choices. The BRIDGEWare Startup Guide covers

- Planning the installation,
- Creating backups of bridge data,
- Uninstalling BRIDGEWare products,
- Ensuring correct versions of server hardware and software,
- Ensuring correct versions of client hardware and software,
- Installing BRIDGEWare products,
- Creating new databases,
- Migrating data to the latest version,
- Merging data into an integrated database, and
- Adding new users to a database.

TABLE 1 Client Hardware and Software Requirements

Hardware Requirements			
	Pontis	**Virtis/Opis**	**Integrated**
Machine	Intel Pentium III 300 MHz	Intel Pentium III 300 MHz	Intel Pentium III 300 MHz
Memory	64 MB	128 MB	128 MB
Video	800x600, 256 colors	1024x768, 256 colors	1024x768, 256 colors
Mouse	Microsoft® or compatible	Microsoft® or compatible	Microsoft® or compatible
Hard Disk	100 MB + (varies based on bridge inventory in single-user installations)	200 MB + (varies based on bridge inventory in single-user installations)	300 MB + (varies based on bridge inventory in single-user installations)
Disk Drives	16X CD ROM	16X CD ROM	16X CD ROM
Software Requirements			
	Pontis	**Virtis/Opis**	**Integrated**
Operating Systems	Win 95, 98, NT, 2000	Win NT SP4, Win 2000	Win NT SP4, Win 2000
MS Internet Explorer	4.5	5.0	5.0
Sybase Versions (i)	6.0.1, 7.0.3, 8.0.0	6.0.4, 7.0.3, 8.0.0	6.0.4, 7.0.3, 8.0.0
Oracle Versions (ii)	8.1.6. 8.1.7	8.1.6. 8.1.7	8.1.6. 8.1.7
MSDE versions	N/A	1.0	N/A
MS SQL Server Versions	N/A	7.0 (untested)	N/A
Access Versions (iii)	N/A	2000 (Academic Version)	N/A

As part of beta testing, volunteer agencies conducted many versions of the full installation and migration process on many different hardware and software platforms. The Startup Guide documents the combined experience of these testers over several months of work.

RATING PROCESS SUPPORT

Included in the 2002 releases is the ability to perform a rating in Virtis and copy the results into Pontis. Rating information in Pontis is used by agencies for NBI reporting, developing functional improvement needs, and managing the bridge load posting process. The following Pontis data items may be populated:

- Rating date,
- Operating rating and type (NBI Items 63 and 64),
- Inventory rating and type (NBI Items 65 and 66),
- Alternate inventory and operating ratings, and

- Inventory and operating ratings for three specific trucks (for load posting).

Within Virtis, the engineer can configure this data transfer to determine which rating trucks are used for each Pontis item. Then each time a rating is completed, it takes only a few clicks to transfer the results to Pontis. Figure 2 shows the Virtis screens for this purpose. Although simple, this capability begins to demonstrate the utility of a live database linkage among the products.

NEAR-TERM OBJECTIVES

As the basic functionality of the BRIDGEWare products becomes widely adopted and understood, end users have begun to demand more functionality to better serve their business needs and to take advantage of all the data they have collected. The task force and contractors are eager to satisfy this demand, but it is important to make the development efforts as economical as possible to fit within the existing licensing structure. In certain important cases, all three products can benefit from the same functionality, so it makes sense to develop each feature only once and share it across the whole product suite.

Report Writing

Up to now, Pontis and Virtis/Opis have different approaches to preparation of custom reports. When Pontis was first converted to the PowerBuilder architecture, the system's relatively small number of database tables was well suited for use with InfoMaker, a PowerBuilder derivative that allows end users to directly manipulate database tables to create standardized or ad hoc reports. This customization capability has become extremely important to end users.

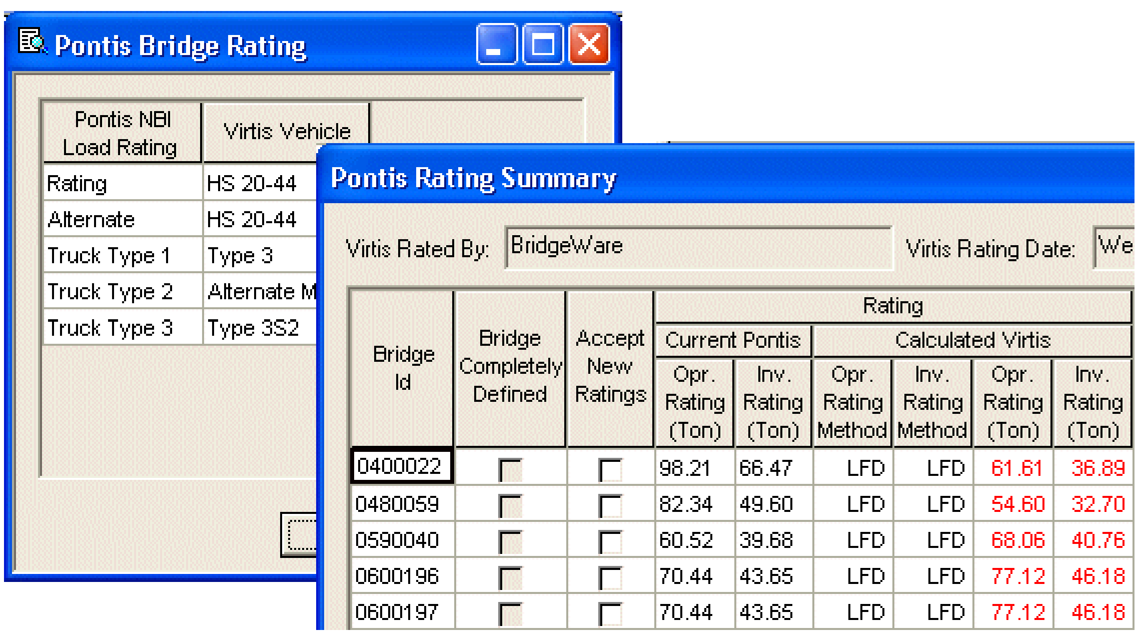

FIGURE 2 Transferring Virtis ratings to Pontis.

For Virtis and Opis, on the other hand, custom reports were not as central to the system architecture, and the database schema was always too large and complex for a generic database report writer. The solution is a custom report writer that is based on a simplified view of the database, modeled after the Bridge Workspace. The software creates standardized ASCII files in eXtensible Markup Language (XML) that can be formatted for display in Internet Explorer.

Now that it is possible to combine Pontis and Virtis data in one database query, a user-friendly report writing option is required that can use the combined data in standardized or ad hoc reports. A near-term priority is to add many of the Pontis tables to the Virtis and Opis report writer, so data from both systems can be combined in a way convenient for both. This is included in the 2002–2003 Virtis work plan.

Database Security

With funding from the Florida DOT, the Pontis project is currently adding new security capabilities to its database, primarily to allow access rights to be associated with groups of bridges. Virtis and Opis already have this capability. Once the specific needs of the funding agency are satisfied, it is envisioned that the projects will take any additional steps necessary to merge the two security schemes into one. For example, it could be possible to define a common set of bridge folders, or groups, and attach access rights to each group that apply to any or all three products.

In the longer term, an integrated security scheme would open the door to other integrated features on the products' front end. For example, a user could log into an integrated BRIDGEWare database just once and view a list of bridges that provides access to features of all three products. Agencies in which one person has both bridge management and rating responsibilities would appreciate this feature.

Document Management

Also with funding from Florida, the Pontis project is adding a document management capability. This feature is generic and able to accommodate nearly any type of data file that can be associated with a bridge or inspection. A logical extension is to make this capability available to Virtis and Opis. Engineering drawings, inspection photos, environmental reports, and many other items could be fruitfully shared among the three systems.

Import and Export

AASHTO has an initiative under way across all its software products to adopt XML as a data transfer mechanism to improve the synergy among the various systems. A high priority is developing an XML schema, which is basically a dictionary of key words used in XML files to identify the individual pieces of data within them.

Any two information systems, even if developed totally in isolation from each other, can use an XML schema to interpret a data file passed between them, if each has the generic ability to parse XML files. The schema defines what data items each system can provide in its export feature. Each system importing an XML file can use the schema to find data items it can use and ignore the rest.

As a part of its report writer, Virtis already has an XML export feature that will be extended to parts of Pontis in a planned 2003 release. A possible additional task is extending XML support to Virtis rating results and incorporating the full functionality of Pontis data interchange files. What remains is reconciling this feature with the combined AASHTOWare schema as it evolves and developing an import capability for it.

Once this functionality is available, it can be used for many purposes:

- Sharing data among AASHTOWare systems and with other transportation agency systems;
- Exchanging data with inspection, rating, and design consultants;
- Archiving old data; and
- Communicating with remote third-party systems, such as data collection devices and monitoring systems.

Web-Based Access

In recent years a number of architectural decisions have been made, especially in the Virtis and Opis project, to enable future use of the Internet for many purposes, such as software deployment and data queries, data reporting, and data capture. For example, the component-based architecture of Virtis and Opis is organized to allow remote communications among components and deployment of software updates to users in small chunks. The report writer uses XML files, a format frequently used for sharing data across the Internet, and renders its formatted reports in a standard web browser.

Pontis is currently investigating its own architectural changes that may enable it to exploit the Internet for several purposes, especially bridge inspection. If Pontis does move more closely to an Internet-enabled architecture, this would likely result in functionality of value also to Virtis and Opis users.

LONG-TERM POTENTIAL

Leaders in the fields of bridge engineering and management have always appreciated the need to continually evolve and improve, to better understand the bridge life cycle, and to make better decisions at every stage of it. The demands of fiscal austerity and growing traffic increasingly require designing to minimize life-cycle costs; planning maintenance interventions more strategically, with fewer but more effective actions; and managing network load capacity and traffic to best fit the changing needs of heavier trucks.

The integrated BRIDGEWare database and product suite are a tremendously valuable resources, unlike anything before, to serve the advancement of the field. Among the initiatives that can be undertaken in future years to build on this platform are

- Research into the relationship between structural characteristics and life-cycle deterioration and performance of bridge elements;
- Use of an agency's own life-cycle cost experience in bridge design decision making;
- Development of reliable cost models for bridge management and design;
- Research on the long-term performance of preservation actions;
- Ability to streamline the bridge inspection process to combine the needs of bridge management and load rating;
- Integration of seismic and scour analysis and general vulnerability assessment with bridge management;
- Integrated management of network load capacity;

- Improved integration of freight movement operational policies with bridge design, management, and rating; and
- Integration with existing or emerging nonbridge software systems such as Trns•Port, DARWin, HWYCON, TSIMS, and future asset management applications.

As transportation agencies continue to make these systems an integral part of their essential bridge activities, the volume and quality of structural data will continue to increase. This will bring all these goals closer to fruition.

REFERENCES

1. ASHTOWare Transportation Software Solutions. www.aashtoware.org. AASHTO, Washington, D.C.
2. *AASHTO LRFD Bridge Design Specifications*. AASHTO, Washington, D.C., 2002.

Bridge Management System for City of Moscow

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The Moscow Bridge Management System (MOST) provides oversight for approximately 850 structures, including bridges, overpasses, tunnels, embankments, and pedestrian crossings. MOST contains the software necessary for inspecting structures taking into account their deterioration, for servicing structures, and for planning the optimal strategy for maintenance and repair within budgetary constraints. To describe a structure, MOST employs a specially developed expandable catalog of standard structural elements (SSEs), to which correspond a deterioration model, a set of several condition states, and a set of standard repair procedures. Unlike other bridge management systems (BMSs), MOST develops a bridge's inspection plan and establishes bidirectional correspondence between bridge drawings and the list of SSEs in the database. This is extremely helpful in making adequate repair assignments. Maintenance, repair, and rehabilitation activities and their associated budget are planned and optimized from data obtained during standard inspections. Plans can be drafted for the entire network of bridges, for several selected structures, or even for a particular group of SSEs. Calculations are performed in an interactive mode, which makes it possible for the user to change coefficients and criteria in relation to specific conditions. Through implementation of this BMS, municipal services of the Moscow government have gained the ability to objectively evaluate the condition of city structures and optimize planning for long- and short-term repair activities.

In 2001 and 2002 under the aegis of the Moscow Project Implementation Unit Foundation of the World Bank, the Moscow Bridge Management System (MOST) was developed and turned over for experimental utilization. The development was carried out by Promos of Russia and AGA, Inc., of the United States with participation from Cambridge Systematics, Inc., and Ove Arup & Partners, Ltd.

MOST services the following structures of city transport and infrastructure:

- Bridges across the Moscow River, the Yauza River, the Moscow Canal, and other water barriers;
- Overpasses both in the city and along the Moscow Ring Road;
- Trestles, including multilevel trestles;
- Transportation tunnels;
- Above and underground pedestrian crossings; and
- Embankments and retaining walls.

The total number of objects within MOST is more than 850. MOST is an analytical system designed to help engineers in

- Obtaining objective data from uniform criteria on the technical condition of bridges and other structures,
- Predicting changes in the condition of the structures over time,
- Setting priorities for maintenance, repair, and rehabilitation (MRR) activities on the basis of prognostication and normative documents currently valid for the territory of the Russian Federation,
- Calculating financial expenditures necessary for the structures' maintenance while taking into account budgetary limitations, and
- Calculating load-bearing capacity of structures through use of recent quantitative design methods and accounting for defects.

MANAGEMENT SYSTEM METHODOLOGY

The fundamental principle underlying MOST's methodology is conceptualization of the management system's object as a totality of standard structural elements (SSEs), each of which has a standard indicator of its condition that can be determined in the inspection of the structure.

MOST permits the description of a bridge or other structure in the form of a multilevel branching structure of SSEs and makes it possible to evaluate the elements' mutual interaction. The branching structure of elements corresponds to the object's structure and is linked to its working blueprints.

The methodological foundation for MOST is a selection of catalogs and instructions. The expandable catalog of standard elements currently consists of 209 positions. This large selection of elements can be attributed to two factors. Because MOST is an engineering system that makes it possible to designate repair measures for a precise location, a greater level of detail is required in the initial data in comparison with standard management systems. For structures in a megalopolis like Moscow, it is necessary to consider the wide range of nonbridge factors, such as architectural and advertising forms and hydrotechnical and emergency equipment, among others. Among the catalogs and instructions in MOST are the following:

- A catalog of condition states, which makes it possible to definitively determine during inspection the condition of each SSE on the basis of a five-point scale;

- A catalog of repair actions, which determines the nomenclature of repair and utilization measures to bring the SSE into better condition;
- A catalog of prices of repair actions, which determines the cost of repair actions and which takes into account
  — Cost of construction materials,
  — Cost of mechanisms necessary for carrying out the given repair action,
  — Cost of labor,
  — Coefficients for the structure's complexity, the difficulties in carrying out the job, and a series of other factors, and
  — Coefficients for inflation and refinancing (in developing this catalog, plans for the repair and reconstruction of Moscow bridges over the last 5 years were analyzed; the catalog's software permits comparisons of predicted and actual costs of completing jobs over a given time span);
- A catalog of deterioration models of SSEs, which is based on analysis of the behavior of bridge structure elements over time according to data from both Russian and foreign scholars (te catalog's software makes it possible to adjust parameters of deterioration models over time on the basis of MOST data on the structure's behavior); and
- Instructions on MOST structure performance derived from standard inspections.

MOST SOFTWARE SUPPORT

The MOST architecture consists of the following basic elements:

- A wb-oriented relational database;
- A subsystem for protection of information and authorized access;
- A user interface equipped with an advanced context-dependent help system and a smart system for monitoring data entry and error correction;
- A search subsystem that permits a search of MOST objects by a series of selected criteria designated by the user in a dynamic mode (with the total number of search criteria exceeding 200, the number of possible combinations is almost unlimited);
- A dynamically controlled report generator;
- Software that realizes the system's analytical and modeling capabilities in prognosticating technical condition and budget optimization;
- An external analytical and number-crunching complex for assessing stress–strain conditions and solving problems related to passing overlimit loads;
- A subsystem of real-time data entry for uninterrupted monitoring;
- A subsystem for connecting MOST with ACAD 2002; and
- Supplementary functions and utilities.

Oracle is used for database management.

MOST Database

MOST is an economic-engineering system meant for economic tasks but also for tasks connected with planning MRR activity and choosing optimal transit routes for oversized loads. These functions are the reason the database stores such a large volume of information. The database is conditionally divided into interconnected categories:

- Cartographical fundamentals of the city of Moscow. The database contains a map of the city, and with the help of a special navigator program, the user can search for the object by moving around the city map.
- Structure technical data. The technical manual for the structure consists of a series of descriptive forms in which is preserved all technical information on the structure as a whole and on its components, such as supports, spans, roadways, architectural details, communication, and engineering equipment. Photographs of the structure (in unlimited quantity) and any other kind of dot-based depiction are integral parts of the technical manual. The software serving the graphic component of the technical information makes it possible to carry out all necessary transformations, such as expansion, shrinking, displacement, and rotation.
- Structure archival data. The technical archive of Moscow's bridges has a history of more than 100 years, and naturally all documentation is preserved in paper. In the process of developing an archival complex for MOST, the vectorization was taken from the most important schematic diagrams for all the city's bridges. A list of the diagrams held in the technical archive was also entered. ACAD 2002 was the basis for graphic vector analysis.
- Mathematical structural analysis. It is well known that the most labor-intensive and important task in performing calculations is the construction of mathematical finite element models that reflect with sufficient accuracy the structure under analysis. The accuracy of the model can only be confirmed by the correlation of the values derived from calculation with analogous parameters derived from inspection of the structure. Only when a mathematical model's accuracy is confirmed can it be used for planning tasks. Within the framework of MOST's development, finite element models were constructed and tested for a number of structures. In constructing the models, actual defects revealed during inspections were taken into account. This information will be used in the future for planning and for calculating the structure's actual load-bearing capacity.
- Results of inspections. The MOST database contains the results of all inspections of the structures. The results are in a spreadsheet that indicates the actual condition state of each SSE as ascertained by inspection and the kind of repair action the inspector has designated for the element. As a supplement to the spreadsheet information, each SSE is shown in an inspectional schematic drawing in ACAD format of each structure. With the aid of a special utility, MOST can link the schematic drawing in the inspection plan with the spreadsheet that reports the inspection results. Thus, in the analysis of inspection results the SSE can be precisely located, and the interconnection of the SSEs and their influences on one another can be determined.

MOST Economic Programs Module

The programs of this module prognosticate the conditions of the MOST object over time from the inspection data and also assign deterioration criteria for the object's structural elements. From the received prognosis, the necessary financial expenditures are calculated with budget limitations taken into account. All programs of the module function for both single structures and groups of structures, as requested by the user. The calculations are carried out for a user-designated time interval. Economic indicators are calculated in base or current prices. It is also possible to account for inflation.

The basic components of the module include

- Calculation of an index of health,
- Prognosis of technical condition, and
- Budget calculations and optimization.

MOST Calculation and Planning Module

Spatial finite element calculations are performed using the module STRAP, which was developed by the Israeli company ATIR. Schematic diagram and construction planning tasks are fulfilled using the ACAD 2002 system, which was developed by AutoDesk.

STANDARD BRIDGE INSPECTIONS IN MOSCOW

Through summer and fall of 2002, standard inspections for all of Moscow 450 bridge structures were conducted with the methods developed by MOST's designers. Fifty inspectors took part in the work. MOST operators then entered the results of the field observations into the database. In this process, the number of simultaneously functioning terminals at the station reached 20. No significant problems in this work during the peak load were recorded. In the process of this work, individual elements of the user interface subsystem were enhanced in a way designed to increase the efficiency and quality of the operators' work.

MOST SYSTEM OF CONTINUOUS MONITORING

In developing the system's design, the decision was made to install a continuous monitoring system for at least one object. This system assigned real-time measuring and recording of such parameters of bridges' utilization as deformation, loads, and temperature. This pilot project was implemented with the aid of specialists from Triada Holding and Mogormash, both of Russia, and SmarTec of Switzerland. The monitoring system equipment was assigned for installment on the Bolshoi Moskvoretskii Bridge over the Moscow River and on the Kutuzov interchange on the third transport ring.

MOST CONTROL CENTER

The Moscow municipal government has created a special dispatch center for utilization of MOST with Promos, the system's Russian developer. The center is provided with all necessary equipment for the system's effective functioning, including telecommunication facilities that support the continuous monitoring system.

The center is designed to accommodate six MOST operators along with the analytical and calculation groups, each of which has three people.

CONCLUSIONS

As a result of the MOST implementation, the official services of the Moscow government have attained the ability to objectively evaluate

- The actual condition of the city's artificial structures,
- Changes in the condition of the structures over time, and
- Necessary financial expenditures for maintenance of bridge and other transport structural stock.

The use of MOST also enables

- Calculations of the actual stress–strain status of structures given their defects,
- Calculations on the passage of overnorm and oversize loads,
- Significant enhancement of work efficiency with archival materials, and
- Real-time monitoring of the functioning of the most important transportation infrastructure objects in the city.

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Synchronizing an Agency's Bridge Information Between Multiple Vendor Systems

Kansas Department of Transportation Approach

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The Kansas Department of Transportation (KDOT) has implemented AASHTO's Pontis Bridge Management System in its Bridge Management Section, where the AASHTO product serves a number of important roles in managing the state's structures. At the same time, KDOT has implemented Exor Corporation's Highways product, which provides a Structures Manager module for bridge information, where data created and managed within the Pontis system must also be stored and changed. KDOT business practices mandate that new structures be set up initially within the Exor Highways software and be transmitted to the Bridge Management Section for incorporation in the Pontis database. In attempting to maintain data consistency and high levels of availability across disparate software systems, KDOT faces challenges very similar to those of other transportation agencies. To facilitate the transmission process and eliminate significant staff intervention wherever possible, a suite of automated data synchronization brokerage routines has been created by KDOT that periodically synchronizes all changes for shared data items between the two software systems. A combination of Oracle database procedures effects this synchronization nightly and generates automatic reports with database log tables and e-mail notification to track data exchange activities. The organization of the synchronization system is described within the context of KDOT business rules, the data organization of the separate software packages, and user requirements. The results of a period of active service for the brokerage software were reviewed to identify benefits realized or anticipated, difficulties encountered, and potential improvements to the system that KDOT may undertake in the future, including application of extensible markup language technology to enhance and strengthen the brokerage mechanism.

Kansas Department of Transportation (KDOT), as other large transportation agencies, has a wealth of transportation-related data it collects and uses. The information includes system utilization statistics, program information, performance indicators, financial data, and inventory. KDOT has selected the Exor Highways product to fulfill the inventory and mapping roles, among others, and includes all types of KDOT data in the highways system in different fashions.

The state's bridge data constitute a key component of the newly implemented KDOT CANSYS-II system, which is based directly on the Exor Highways product. The Exor Highways system is usually referred to as CANSYS-II in KDOT. In the recent past, highway information was maintained in dedicated mainframe systems. These have been replaced with implementation of the CANSYS-II system, the official primary repository for KDOT agencywide bridge information and the source accessible to most KDOT users.

Even before CANSYS-II was put in place, KDOT had actively used the Pontis Bridge Management System (BMS) for several years. This system, used by more than 35 state departments of transportation and other agencies, has been described fully elsewhere (*1–3*). In the course of their normal system assessments, KDOT bridge inspectors perform full Pontis inspections on state structures and collect condition and inventory information that is critical for safety, maintenance planning, program decision making, and other purposes. This Pontis data must be published for use by a variety of KDOT decision makers.

The Pontis data provide a more thorough assessment of the condition of state bridges. In KDOT, the question arose as to whether the Exor Highways product should provide the user interface for entering this data, or the data should be entered via Pontis with a means then provided to synchronize the data with CANSYS-II. The decision was to customize Pontis for the entry of data but publish it to KDOT as an agency with the CANSYS-II system. This change involved a paradigm switch. With Pontis as the primary data entry and management system for bridge data, the Bridge Management Section would effectively own the bridge data, and the data would be provided to KDOT Planning via an automated synchronization process with CANSYS-II. A direct advantage of this paradigm switch was that more people in the Bridge Management Section could enter the bridge data sooner and would not need training and user privileges for CANSYS-II. The reliance on Pontis also offers the opportunity for the entire Bridge Management Section to view the data through Pontis. Finally, the state is revising its programming process to incorporate Pontis information directly within the prioritization formula, which further amplifies the need to share bridge information effectively in KDOT.

OVERVIEW OF KDOT SYNCHRONIZATION PROCESS

To accomplish the synchronization of the same information stored redundantly in two disparate systems, KDOT staff and consultants have designed and developed a batch exchange mechanism, using Oracle tools, to transfer data back and forth. In designing the synchronization system, several key challenges were considered:

- Pontis uses a conventional table and row database organization, whereas CANSYS-II uses a linked-list database organization. The databases are organized very differently: CANSYS-II is highly normalized compared with Pontis. Every row in CANSYS-II is an attribute value, whereas the Pontis database stores attributes more conventionally as columns within a single table row.
- Pontis defines logical entities that are at some level of variance from the CANSYS-II equivalents, and vice versa. For example, a Pontis roadway entry to store road attributes for the road on or under a structure is not the exact same logical entity as a CANSYS-II feature crossing item, although the two share identical attributes.
- Pontis stores its data in differing data types, and CANSYS-II uses character data almost exclusively for attribute storage, a factor that necessitates some type conversions.

- CANSYS-II uses synthetic nonintelligent row identifiers, and Pontis uses a multipart key in many parallel contexts (e.g., bridge and inspection identification to indicate a unique inspection session).
- Pontis requires that a core set of records be in place for every structure to operate properly, while CANSYS-II can operate with a much sparser information matrix that is populated later. Pontis ensures this coherence in its business logic, and the synchronization routines emulate this in code.
- Codes, terminology, and data definitions initially differed for apparently identical data items between the systems.

In short, a number of the common challenges data warehousing experts face in data integration in an agency were experienced at KDOT in microcosm.

The solution was to design a data-driven rule-based system that arbitrates the exchange between CANSYS-II and Pontis. Any given data item (and context) that must be exchanged, other than primary key information, is registered with the rules, which then effect the exchange by driving the application code logic.

The rules system incorporates the following core assumptions:

- An exchange rule must be formulated for each data item to be exchanged.
- Only essentially identical pieces of information are exchanged. No data requiring unpacking or other transformations to facilitate differing representations in different systems are exchanged.
- Some information is not exchanged because of policy or need.
- One system or the other has been designated as the primary authority for a piece of information, and the rules document that precedence. This reflects agency staff responsibilities.
- Rules are not necessarily in effect for all types of exchanges. Some exchanges only occur for updates of existing records, for example, and not for creating new records.
- Duplicative exchanges are resolved using the precedence.
- Date and time are used to determine stale or out-of-date exchanges during preprocessing.
- An autonomous broker process is used to apply changes relying on the rules.
- A standard convention for communicating a change is used (e.g., a table structure), and the rule used accompanies the change.
- Each piece of information has a specific set of lookup keys or identifiers that are necessary to correctly communicate the change to the target system.
- Rules drive the underlying code behavior.

SYSTEM DESIGN SUMMARY

The system is designed to perform the data exchange following the flowchart shown in Figure 1. Two streams of changes must be propagated: changes in Pontis and changes in CANSYS-II. Changes are column- or row-level data modifications and additions or deletions. In sum, these changes are collected in the background during normal data entry operations, preprocessed as necessary, and applied during a comprehensive batch process on a scheduled basis.

The following key characteristics are important:

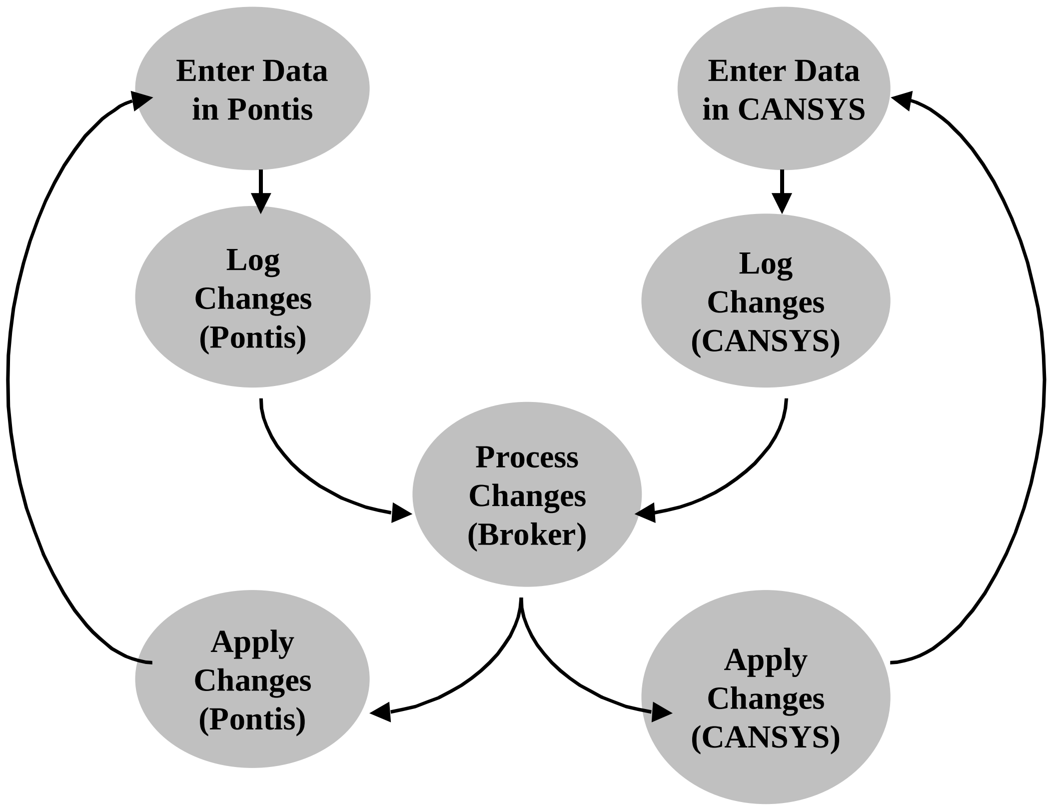

FIGURE 1 High-level view of synchronization process.

- Change collection is automatic and invisible to users.
- The exchange process is asynchronous.
- All work is tracked or logged.
- The application process can be restarted, to some degree, in the event of a failure of one or the other databases involved.

Further, the process takes full advantage of built-in system services for coordination and messaging functions.

APPLICATION ORGANIZATION

The application is divided into a set of user triggers and a processing engine. Triggers are Oracle executable constructs that in this system trap and publish changes to specific tables and fields in the Pontis database and CANSYS-II. The changes are stored for later exchange. These triggers fire without user intervention on the basis of update or insert events in the database such as data modifications. The Oracle database management system, as with most types of relational databases, recognizes that a change is occurring to a column that has a synchronization trigger and executes that trigger in the course of the transaction.

The processing engine, called the broker, consists of three main parts: a preprocessor, an updater, and a message manager. The preprocessor performs several data grooming and conditioning activities, including rejecting invalid changes, type conversions, and checking for complete information. The preprocessor moves data in two directions: change made in the CANSYS II application are transferred to queue tables in the Pontis database to be applied in there, and similarly, changes made in the Pontis application are moved to queue tables in the CANSYS II database to be applied there. These changes have already been arbitrated for change concurrency and staleness to prevent looping propagation of redundant or out-of-date changes. The transfer process does require that both systems be online and in communication. The pending changes are then applied by the updater to the respective databases by dedicated processes that perform the updates to the target systems.

While these engines are collectively complex, they are vastly simplified by compartmentalizing the preprocessing activities from the database-updating activities. These procedures are written entirely in Oracle PL/SQL as packages and can be scheduled for execution using standard database administration tools. Typically, the synchronization batch operations are run at night by the agency.

DATABASE ORGANIZATION

Synchronization is configured and driven by the database control tables shown in Figure 2. There are two main sets of tables: the change logs and the rules tables. Although full elaboration of the roles of the various tables is beyond the scope of this document, in sum, the change log tables hold all pending changes, and their contents are ephemeral, depending on update activity in KDOT, whereas the rules tables store persistent data that changes infrequently under security controls. Change entries are related to the rules that apply to them. The rest of the synchronization database consists of purposeful instances or copies of the ephemeral tables, and the system messaging and parameter tables. All tables are owned by a dedicated robot user identifier to prevent unauthorized access to the exchange information.

A typical rule entry consists of the following data, which is static after initial specification and not changeable by end users:

EXCHANGE_RULE_ID = 2370
TRANSFER_KEY_MAP_ID = 1
TABLE_NAME = INSPEVNT
COLUMN_NAME = UNDERCLR
SIT_ID = RTNG
SAT_ID = RD40
PRECEDENCE = P
CREATEDATETIME = 2001-11-08 09:46:07
CREATEUSERID = ROBOT
MODTIME = 2001-11-08 09:46:07
CHANGEUSERID = ROBOT
REMARKS = Initialized from old rule RD40

This rules entry is interpreted as follows: “To exchange the Pontis underclearance measurement (COLUMN_NAME = UNDERCLR) performed during an inspection (stored in TABLE_NAME = INSPEVNT) will be applied to CANSYS-II item type RTNG (SIT_ID = RTNG) and roadway underclearance attribute RD40 (SAT_ID = RD40). When performing this

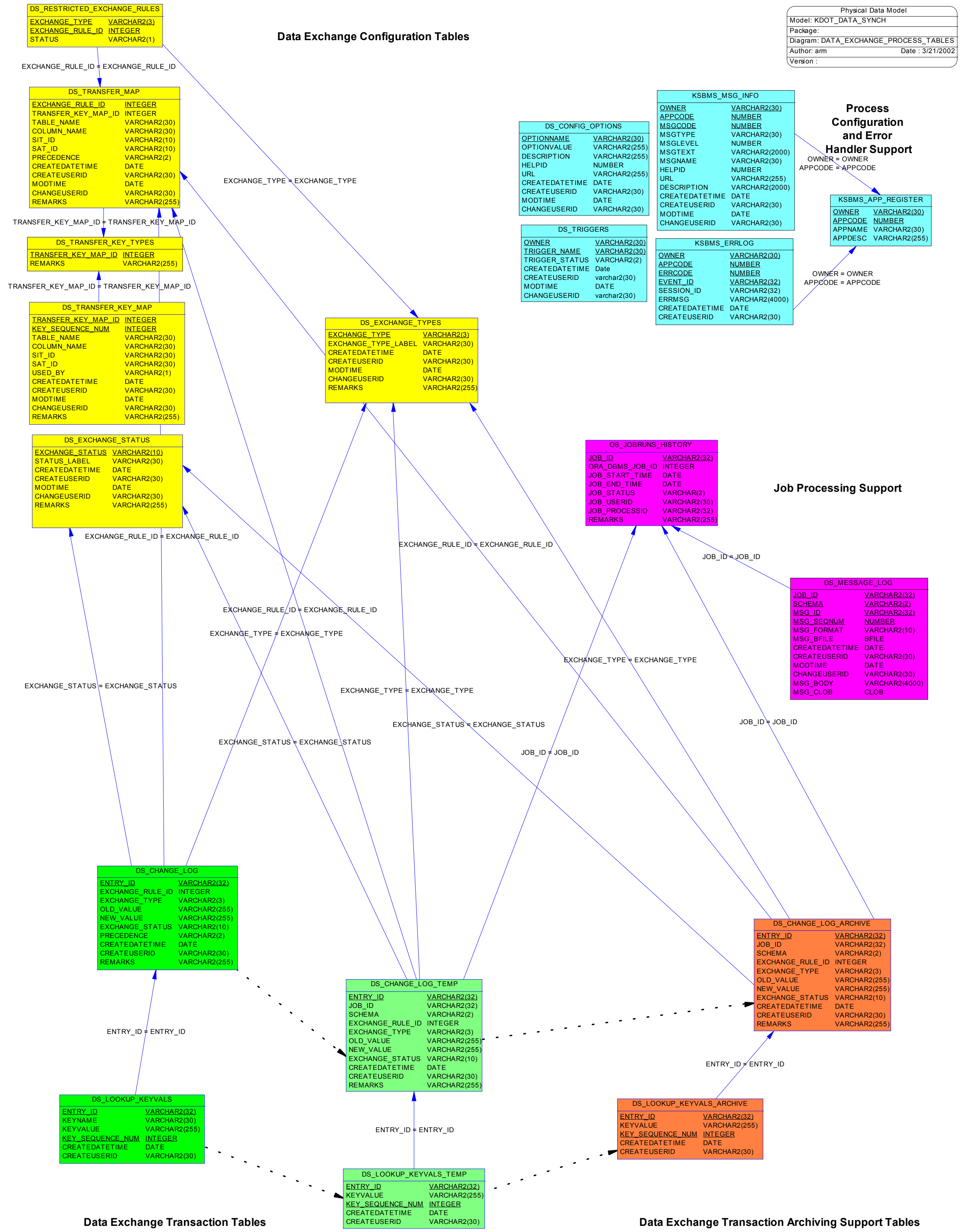

FIGURE 2 KDOT data synchronization table organization.

exchange, you need only a bridge ID to coordinate (TRANSFER_KEY_MAP_ID = 1) between the two systems. If a conflict arises, for example, because the same data item was changed for the same bridge in both systems, then the Pontis change will be kept because Pontis is the agreed owner of this data item (precedence = P).”

A typical change entry is as follows:

ENTRY_ID = AE2A551B98A2F908E03098C0188831B7
SEQUENCE_NUM = 4984
EXCHANGE_RULE_ID = 2370
EXCHANGE_TYPE = UPD
OLD_VALUE = <NULL>
NEW_VALUE = N
EXCHANGE_STATUS = UPD
PRECEDENCE = P
CREATEDATETIME = 2002-10-29 13:51:14
CREATEUSERID = PONTIS
REMARKS = From f_pass_update_trigger_params() via aur_ds_INSPEVNT_UNDERCLR

This change log entry is interpreted by the synchronization system as follows: “A time stamped and sequenced change has been made to a piece of data that is coordinated using rule 2370 (EXCHANGE_RULE_ID = 2370). The change is an update (EXCHANGE_TYPE = UPD). The old value was null, and the new value is an N. Apply this change in order by this change entry’s sequence number (SEQUENCE_NUM = 4984) when processing all pending changes.”

It can be seen that these two entries are coordinated through the rule identification. The solution for matching entries between CANSYS-II and Pontis is to determine the pair rule, as above, then use the key match subrule (TRANSFER_KEY_MAP_ID) to find the target record, and apply the change.

LESSONS LEARNED

The synchronization system described here has been developed and was in final testing in October 2002. It is being considered as a starting point for implementation of systems in other agencies faced with similar challenges.

From this experience, KDOT and the consultants have determined that creating a system of this sort requires several key considerations:

- Follow the standard of “Keep it simple.” Although it is more or less self-evident that a simpler system makes for simpler exchange, this standard is very important in a system like this so it is possible to track changes and verify correctness. A more monolithic exchange process, without granular management of the data exchanges, would be difficult to troubleshoot and extend to support new exchanges. As a particular design consideration, the rules should not be overloaded to behave differently in different situations. A new unambiguous rule should be should be made instead.
- Follow an enterprise data model. Inconsistent data definitions within an agency for apparently equivalent attributes are a bugbear for a system of this sort. The rules table in this system assumes that it is linking identical attributes between the systems.

- Incorporate logging and auditing. Logging and auditing are critical for increasing user confidence in the exchange process and have already generated the expected benefit of providing an early warning capability.
- Use server tools and services wherever possible. Development of the synchronization system was streamlined by incorporating built-in vendor services, particularly for job scheduling and messaging, and for the more conventional trigger and procedural language support. Many features of the system are built into the database.
- Use data models and prototypes in design**.** Formal data modeling was a key success factor. This allowed KDOT staff to comprehend the system more fully and facilitated their recommendations for improvements. Prototypes have the evident benefit of communicating the design in visual terms.
- Consider user information technology capabilities in the design. The end users of this system operate it largely without information technology support. For this reason, the procedures, messages, and other visible features must not be arcane but be straightforward for end users to understand and execute.
- Ensure good communication in the project team. With several parties involved and multiple technology vendors, it is important to assign roles and responsibilities for the system elements and establish a concrete process for resolving issues in the system.
- Design against stable databases**.** The synchronization system does depend heavily on the individual product database designs. If the databases change during implementation, this will have implications for the synchronization system. Further, if the database is static, then problems with synchronization can be more readily attributed to design issues in the system, not the database.
- Allow for special circumstances and workarounds. Although the bulk of the code in the system operates generically on the basis of the rule tables, nevertheless a small set of exceptional data transfers requires in-code workarounds. These are minimized in the system but must be anticipated as a normal part of an exchange system operating between two very differently organized databases.
- Anticipate a cyclical development process. This type of system is inherently evolutionary, and provision must be made for adaptations and adjustment of the design to address unforeseen issues that may arise, particularly in light of the previous considerations.

POTENTIAL SYSTEM ROLE FOR XML

The synchronization system relies on database tables exchange data. As shown in Figure 2, these tables are hierarchically organized with logic implicit in the rules used, naming, exchange type flags, and other database elements. Using the tables as raw data, the application code interprets the meaning of the fields. The extensible markup language (XML) standard for data exchange offers the ability for databases and applications to utilize robustly formatted data files, which are internally validated and incorporate their own processing logic for routine data operations. XML is a new standard or exchange of structured, complex data between applications. Unlike earlier standards such as hypertext markup language—a permissive file format that can be ill formed and is unreliable for data exchange—the XML specification mandates robust, concrete validation of file format and contents. XML files are then, by definition, well formed and reliable. This XML characteristic ensures confidence in the contents and organization of the exchange file for a system such as the KDOT synchronization.

Because of the formal syntax, XML files are also self-documenting. Finally, the formats can be extended to address future needs.

Practically speaking, the strength of XML technology is that the file is self-validating against a formal schema definition. This means that the agency no longer must rely on application code to logically and expensively ensure that a file is intact and usable for data exchange. Importantly, with XML, the file can be generated from any system to apply to any system with no particular language dependency. This permits essentially any system to emit or incorporate XML data. Finally, an XML file-based exchange is not dependent on interdatabase communication links or network sensitivities that may interrupt an exchange, because the file can be processed asynchronously on demand.

Incorporation of XML into the synchronization process would manifest itself in the KDOT system as follows: an XML file system would replace the change log, configuration, and message history, and the XML files would incorporate the exchange logic. This would move the application's exchange logic out of compiled code into configurable XML files, and this would offer the potential for easier end user data review, process maintenance and customization, and reduced database interdependencies and connectivity requirements.

In the future, the synchronization system also can be extended to use XML files to obtain data from and transfer information to systems other than CANSYS-II and Pontis. Building the next generation of the synchronization system on an XML base rather than conventional fixed-format SQL tables would offer KDOT, or other agencies considering building a similar interface, the option to enhance and extend the data exchange readily to accommodate future agency needs for data exchange.

CONCLUSION

This paper has presented an overview of a technical solution to the common problem of exchange of bridge inventory and management data between disparate database systems in a typical U.S. state department of transportation. Functional requirements and data exchange issues facing developers have been identified. The implemented system has shown the feasibility of a rule-based exchange mechanism for automatically transferring information to and from a popular BMS such as Pontis, that is organized in a conventional database model, to an entirely different type of database design. The system bridges the database divide successfully by using a combination of a metadatabased rule system and application logic. This first generation implementation offers a viable approach to data integration for other agencies that are considering use of Pontis and Exor Highways. There are several areas for potential extension of the system, particularly for the exchange of programmatic and financial information within the agency. The potential for using XML in a next generation of this synchronization system was introduced, and several potential advantages of XML over the current approach were identified.

ACKNOWLEDGMENTS

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REFERENCES

1. Thompson, P. D., and R.W. Shepard. *Transportation Research Circular 423: Pontis*. TRB, National Research Council, Washington, D.C., 1994.
2. Marshall, A., W. Robert, K. G. Anderson, R. L. Floyd, and F. Corso, Jr. *Transportation Research Circular 498: Comparison of Pontis Bridge Project Recommendations to Programmed Work for Three U.S. Transportation Agencies*. TRB, National Research Council, Washington, D.C., 2000.
3. Cambridge Systematics, Inc. *Pontis Release 4 Technical Manual*. AASHTO, Washington, D.C., 2001.

Implementation of Ontario Bridge Management System

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Development of the Ontario Bridge Management System (OBMS) began in 1998 and proceeded into the first steps of implementation in 2000. Implementation began with the bridge inspection and data management features, which are used heavily by inspectors in ministry regional offices as well as by consultants. Analytical features of the software, to be used by headquarters and regional offices, were completed in 2002, and their implementation is now under way. OBMS was designed with equal priority given to project-level and network-level functionality, but the implementation process has tended to favor the project level so far. Innovative features for navigating the data about each bridge, including photos, documents, and commentary, were completed first and have been well received in conjunction with a new element-level bridge inspection manual. The inventory is fully populated (except for smaller culverts) with a first round of inspections. The system's project-level decision support features have very flexible scoping rules, Markovian deterioration models, and a costing procedure based on the ministry's highway cost estimation system. Engineers can modify the scope and cost of a project, and predicted performance measures will be updated automatically. The OBMS network level is a graphical trade-off analysis in which the manager can experiment with funding levels and performance targets and view immediate answers to such questions as: how much does it cost to achieve a given standard of performance, or how much performance can be purchased for a given investment?

The Ontario Ministry of Transport (MTO) has under its jurisdiction approximately 2,600 bridges, 300 culverts of more than 6 m in span, and 1,400 culverts of 3 to 6 m. The management of the ministry's bridges and culverts is carried out through five regional structural engineering offices throughout the province and a central office known as the Bridge Office. Responsible for the inspection of all bridges and culverts, the regional offices also handle the determination, planning, and design of work on MTO structures. The Bridge Office is responsible for developing policies related to bridge inspection, rehabilitation, design, and construction and maintains and supports any systems related to these activities in the regions. The Program Management Branch within the MTO head office is responsible for planning and funding the ministry's capital construction program.

MTO began design and development of its OBMS in 1998 and began Ontario Bridge Management System (OBMS) deployment in 2000, replacing an earlier mainframe-based system. Development was performed by Stantec Consulting, Ltd., with Paul D. Thompson as design subcontractor. A set of project-level and network-level decision support models was subsequently completed in 2002.

INVENTORY AND INSPECTION

Since a bridge management system (BMS) is only as good as the data provided to it, the initial priority was placed on improving the computerized inventory and instituting an element-level bridge inspection process. A major effort by the ministry was revision of its bridge inspection manual.

Inspection Manual

MTO carries out a detailed visual inspection of its bridges once every 2 years and of culverts once every 4 years. The inspections have been done according to the *Ontario Structure Inspection Manual* (*1*) since 1985. In its first part, the manual describes the various components of a bridge and details the types and severities of material and performance defects for these components. The second part of the manual groups the components into convenient elements for inspection and describes the inspection procedure. Table 1 lists the components or elements to be inspected. The inspector has the option of subdividing these components further to identify particular areas where defects may be more prominent. For example, the ends of girders at expansion joints can be identified as a subelement to the main girder element.

A major revision to the manual was officially released in October 2000 following the initial release of the data management portion of OBMS in July 2000. One change dealt with the basic inspection philosophy. Previously, MTO rated the material condition of each bridge component on a scale from 1 to 6, with 6 indicating an element is in new condition. The numbers on the scale represented a combination of severity of defects and their extent. The 2000 manual modified the inspection philosophy by recording severity and extent separately and requiring the inspector to record the quantity of defects in each of four condition states for each bridge component: Excellent, Good, Fair, and Poor.

Each condition state combines defects on the basis of their severities. For example, Good condition state refers to an element (or part of an element) where the first sign of “Light” (minor) defects is visible. Material-specific condition state tables are used to describe the severity of defects and assign this to the appropriate condition state. For example, concrete components might have “No Observed Defects” (Excellent), “Light Scaling” (Good), “Medium Scaling” (Fair), or “Severe to Very Severe Scaling” (Poor). The recording of quantities was necessary for OBMS to estimate types of repairs and their costs.

Another revision to the manual dealt with the recording of performance deficiencies for each element. Before the October 2000 revisions, performance of each element was rated on a 1 to 6 scale, with 6 indicating an element is performing as required. The 2000 manual instead requires the inspector to identify the type of performance deficiency rather than to rate the performance. This was done in recognition that it is difficult in some cases to rate performance, such as load-carrying capacity, from a visual inspection without further analysis. The main purpose in recording the performance deficiency is to flag that some follow-up action may be required, such as strength evaluation. The recording of type of deficiency also allows OBMS in the future to estimate repairs to address the performance deficiency. Performance deficiencies include “Excessive Deformations,” “Seized Bearings,” or “Jammed Expansion Joints.”

The inspector is required to record a recommendation for capital work, including timing.

TABLE 1 Ontario Structural Elements

Group	Element Name and Units	Group	Element Name and Units
Abutments	Abutment walls (m^2) Ballast walls (m^2) Bearings (each) Wingwalls (m^2)	Embankments and Streams	Embankments (all) Slope protection (all) Streams and waterways (all)
Approaches	Approach slabs (m^2) Curb/gutters (m) Drainage system (all Sidewalk (m^2) Wearing surface (m^2)	Foundations	Foundation (below ground, no units)
Barriers	Barrier and parapet walls (m^2) Hand railings (m) Posts (m^2) Railing systems (m)	Joints	Armouring/retaining devices (m) Concrete end dams (m^2) Seals/sealants (each)
Beams/Main Longitudinal Elements	Diaphragms (each) Floor beams (m^2) Girders (m^2) Inside boxes (m^2) Stringers (each)	Piers	Bearings (each) Caps (m^2) Shafts/columns/pile bents (m^2)
Bracing	Bracing (each)	Retaining walls	Barrier Systems on walls (m^2) Walls (m^2)
Coatings	Railing coatings (m^2) Structural steel coatings (m^2)	Sidewalks/curbs	Curbs (m^2) Sidewalks and medians (m^2)
Culverts	Barrels (m^2) Inlet components (m^2) Outlet components (m^2)	Signs	Signs (each)
Decks	Deck top (m^2) Drainage system (all) Soffit—inside boxes (m^2) Soffit—thick slab (m^2) Soffit—thin slab (m^2) Wearing surface (m^2)	Trusses/Arches	Bottom chords (m^2) Connections (each) Top chords (m^2) Verticals/diagonals (m^2)

Although the system generates this automatically on the basis of element-level needs, this recommendation may prove useful when reviewing any work recommendations generated by the system. The inspector is also required to record any maintenance-related work as selected from a standard list of maintenance operations typically carried out by MTO staff or maintenance service providers.

System Capabilities

OBMS contains all inventory and inspection data for all bridges on highways under MTO jurisdiction. Three ways are provided to navigate among the data:

- Table View lists all structures in the database in a table form along with data about each structure. Filters can be built to select a subset of structures. The data in Table View can be exported to MS Excel for building reports or charts, and the inspector can also print blank inspection reports directly.
- Map View (Figure 1) presents a set of bridges through a geographical information system interface. Built within OBMS using map objects, the Map View can be used to locate and select a bridge site or to construct a thematic map by building a filter or by specifying ranges for the values of a particular field.
- Detail View (Figure 2) presents a drop-down list of sites and a tree showing the screens for viewing the data about any selected bridge. Detail View is launched from Table View or Map View when a site is selected. In Detail View the user can see all inventory, inspection, and project-level data associated with a bridge.

There are three bridge-level inventory screens within Detail View: identification, description, and appraisal. The identification screen (Figure 2) describes the location of each bridge and presents a photograph of the bridge, the type of bridge, and other general information. Most fields can be annotated with a Post-it Note to provide clarification or additional information. The description screen as shown in Figure 3 contains span data, services on and under the structure, utilities, and some overall dimensions. Appraisal evaluates the structure in terms of seismic, scour and flood, fatigue, load capacity, barriers, and geometrics data.

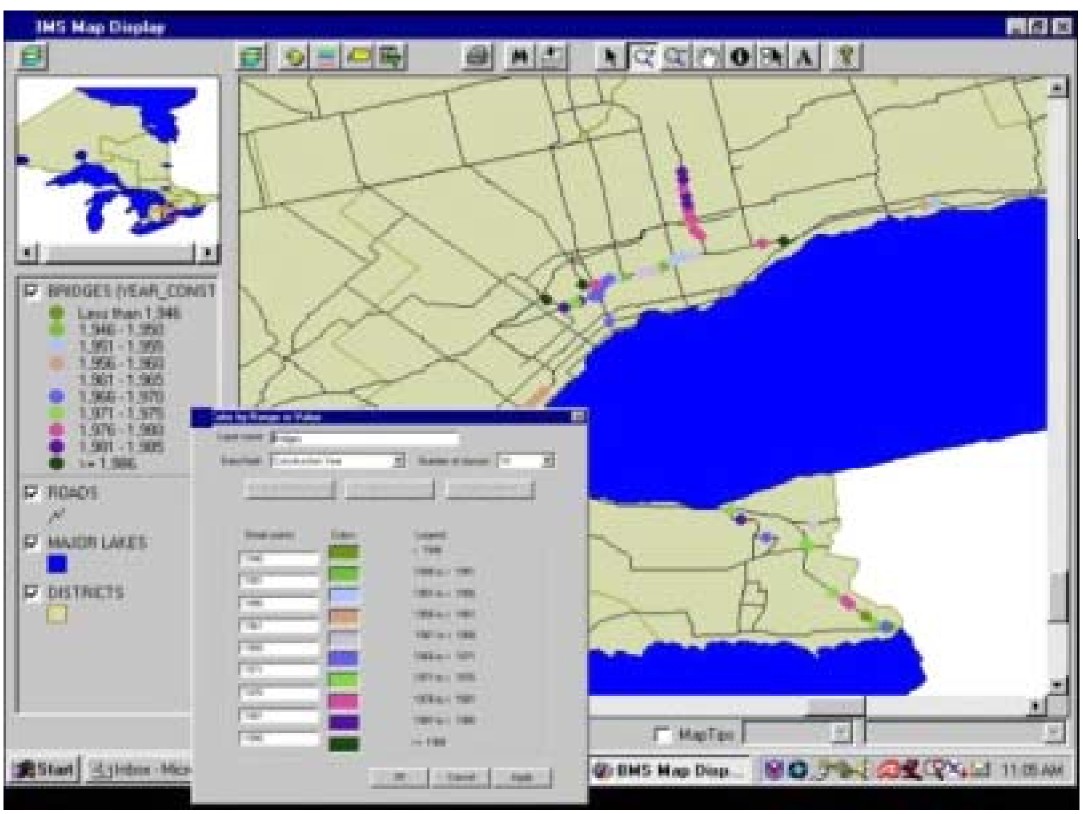

FIGURE 1 Map View.

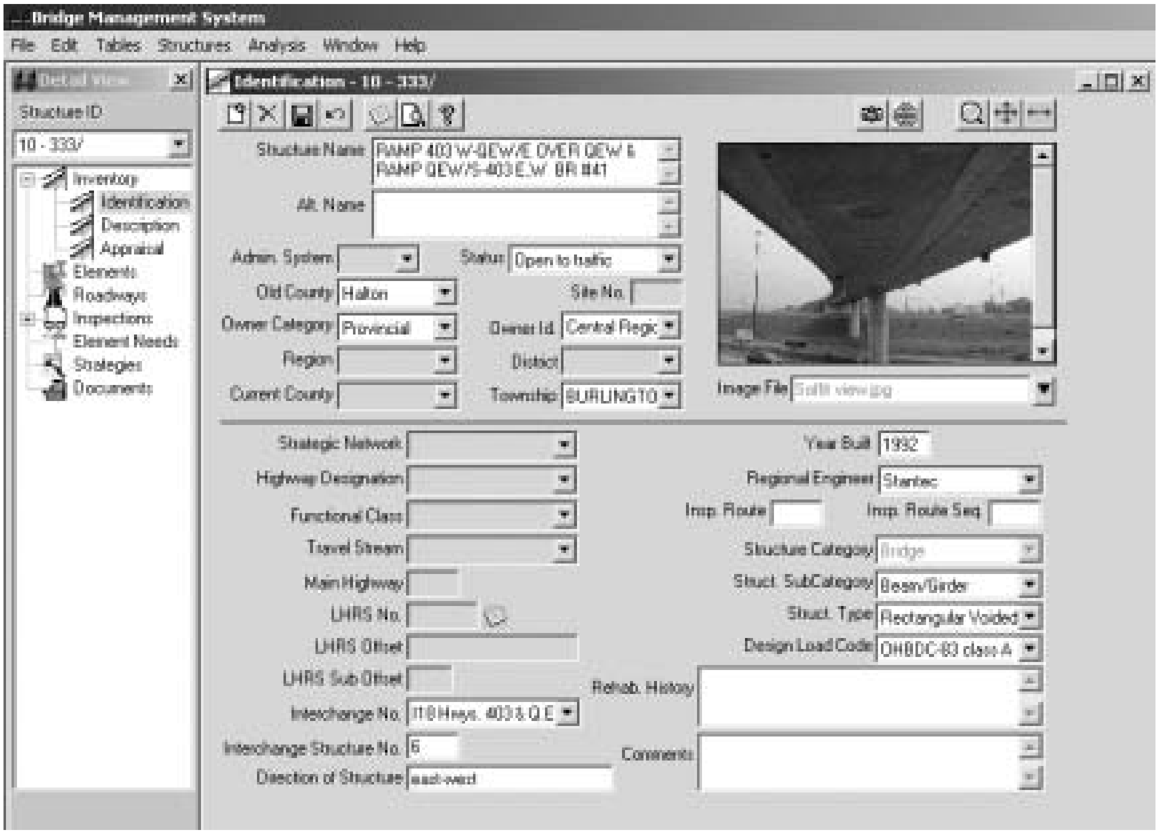

FIGURE 2 Inventory identification screen.

For each bridge, a list of elements is recorded, as described above and classified according to Table 1. Once a bridge's elements have been defined, the inspection data can be entered in the inspection screen, as shown in Figure 4. The inspector can also view inspection photographs and print inspection reports from this screen. Electronic documents such as reports, plans, and photographs can be linked to the bridge site and viewed in the document screen (Figure 5).

In addition to data about the bridge, the system also stores data about the roadways associated with the bridge, including type of roadway, speed, lane widths, traffic volumes, and percentage of truck volume. These data are to be updated annually from another MTO system for future use in a model of functional needs and user costs.

Implementation Experience

OBMS was first released to the regional offices in July 2000 to initiate the update and collection of inventory and inspection data. It was installed on individual user workstations running Windows 95. Each region was given a database with data loaded from the ministry's previous bridge inventory system. In addition, a few consultant assignments were issued for collection of the data. Most of the inventory and element data were gathered from plans available in digitized format. Once the element data were entered into the system, a paper inspection form was generated from OBMS and used by inspectors in the field. In some cases, data were loaded onto a notebook computer and updated in the field.

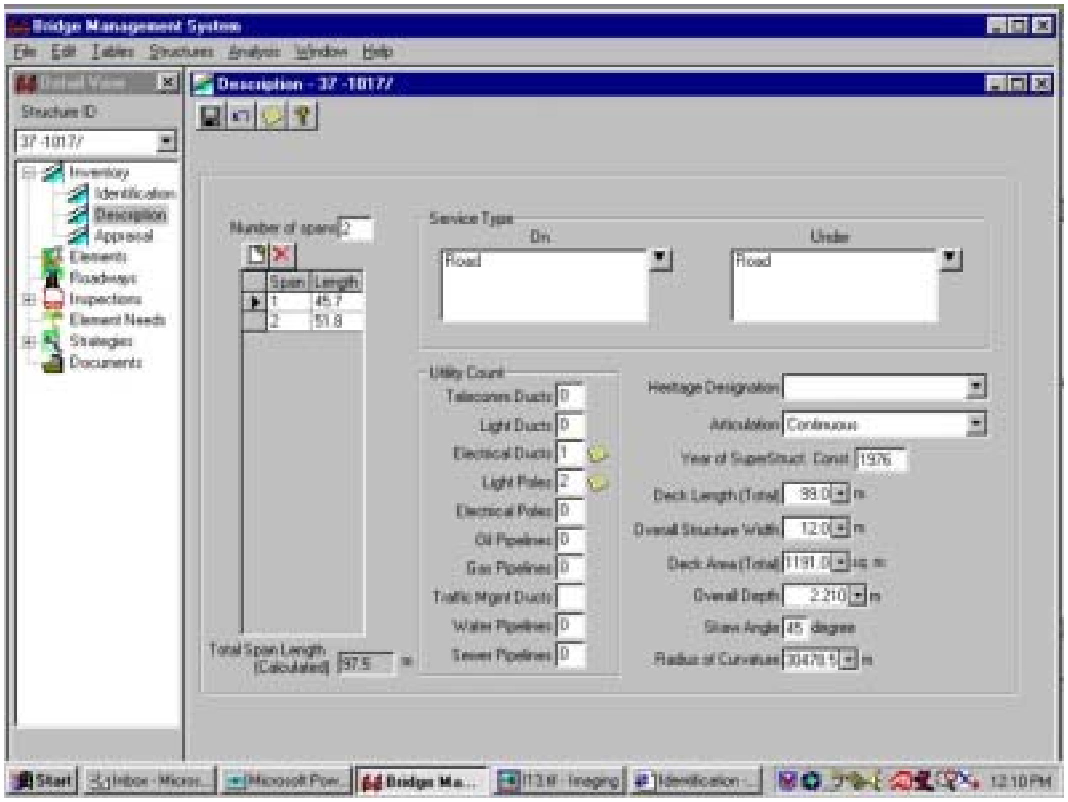

FIGURE 3 Bridge description screen.

During the summer of 2002 all data collected up to that point were gathered from the regions and consolidated into one central database, on a central Oracle 8 database server in the head office. This server also contains data for other ministry infrastructure applications, such as the Pavement Management System, the Integrated Highway Information System, and the Highway Costing System.

OBMS was also modified to link to the ministry’s application security system. This security system is used by the ministry to manage users and their roles for all infrastructure management applications and to control access to the Oracle database. In 2002 the ministry converted all workstations to Windows 2000 and released a Windows 2000 version of OBMS to approximately 80 users in the regions and head office.

This version allowed all users to link to the central Oracle database and to check data out from the server to a local workstation for updating. Once updated on the local computer, the data are checked back to the main server. Any inspection checked in to the server is marked as an official inspection and can no longer be modified. Users can access the main database to view individual bridges or to query the data. Since July 2000, numerous photographs were collected and stored on workstations in the regions. Work is under way to consolidate all sthese photographs into a central document management and archiving system, to which OBMS will be linked.

An important factor in the acceptance of the new BMS was strong participation in the design process from regional representatives and head office personnel through workshops conducted during the project’s first 6 months. Regional engineers participated from the first day of the project in brainstorming and conceptualization to develop the system’s vision and

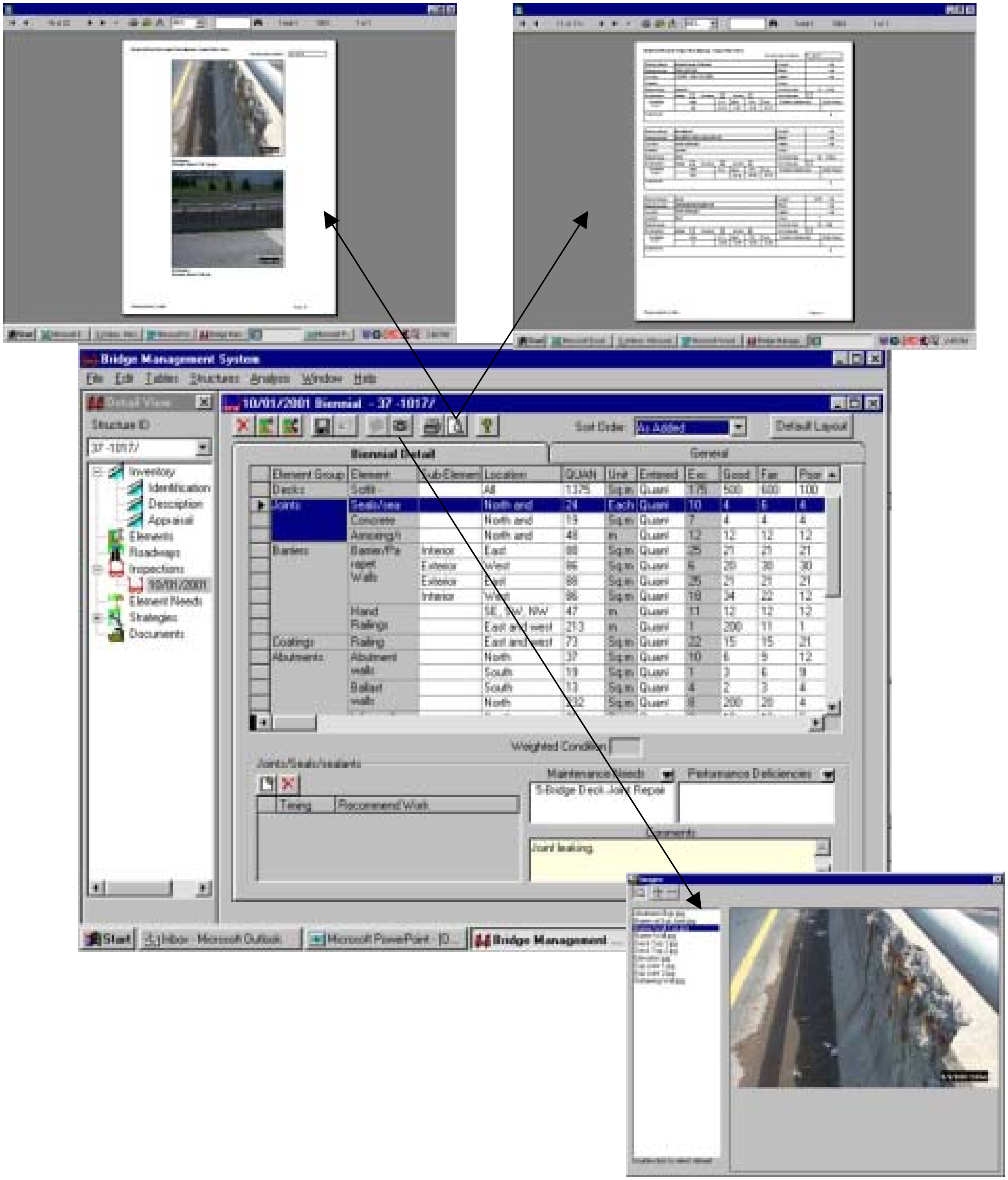

FIGURE 4 Element inspection screen.

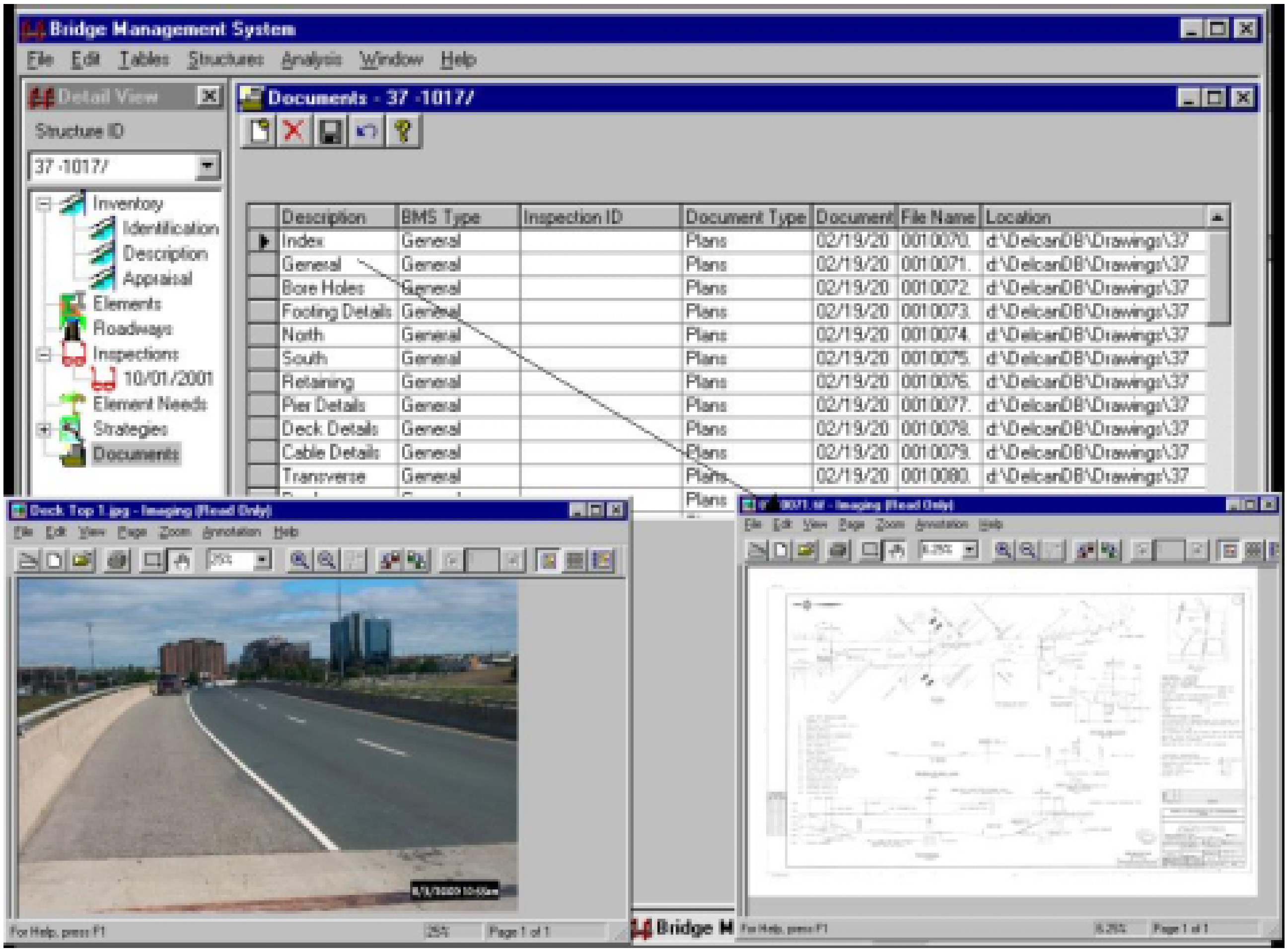

FIGURE 5 Document access.

continued their participation through the design stages to final review of a completed design specification. The system as developed adhered closely to this specification.

DECISION SUPPORT

OBMS contains bridge management decision support models designed to meet the needs of several potential user groups within the ministry. The decision-making processes served by the system include

- Monitoring—inspection and inventory updating performed by regional staff and consultants;
- Needs identification—analysis of bridge and element data to decide what work is needed on each bridge, performed in the regional offices;
- Policy development—analysis of the economic implications of planning models and policies, performed in the headquarters Bridge Office;
- Priority setting—deciding on the order and timing of projects, performed in the regional offices and the headquarters Program Management Branch; and

- Budgeting and funding allocation—trade-off analysis of funding versus performance, performed in the regional offices and the headquarters Program Management Branch, as part of an annual negotiation with the Provincial Treasury Board.

These decision-making processes occur in parallel, so each has an ongoing effect on the others. It is necessary, therefore, for OBMS to record the current status of all these processes and report information that is always consistent with them. For the analytical framework, this led to a requirement that the project level and network level always be consistent with each other.

Figure 6 depicts the main features of the decision support model. What is traditionally called project level (*2*) is divided into two levels for OBMS. These share responsibility for a life-cycle cost analysis, with condition modeling taking place at the element level and cost estimation at the project level. As the arrows indicate, these parts of the system operate in a bottom-up fashion, similar to Bridgit (*3*).

Network-level analysis in OBMS uses projects generated at the lower levels to create programs. Unlike Bridgit, this analysis is noniterative, operating in a top-down fashion in a way more similar to that of Pontis (*4*). However, unlike Pontis, the OBMS network level is not intended to optimize policies but instead optimizes the selection of projects from among a set of alternatives on each bridge, subject to a budget constraint. This is because MTO has long had a separate process to develop its structure rehabilitation policies, as embodied in the Ontario *Structure Rehabilitation Manual* (*5*).

The three levels of analysis in Figure 6 are connected by sets of alternatives for consideration by the next level higher. The levels of analysis influence each other through these alternatives, in the following ways:

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

Each level of analysis in this framework provides a set of evaluated alternatives to feed the broader-scale tool above it. Network-level analysis provides a context for the project level, because the network-level budget constraint affects which Project Alternatives can be programmed. For example, in a very constrained funding scenario many bridges will be selected for the do-nothing alternative.

Similarly, the project-level analysis provides a context for the element level, influencing the cost-effectiveness of individual element treatments. For example, if a particular treatment on a given element has substantially larger traffic control costs than the other elements on a bridge, it reduces the overall cost-effectiveness of the project and therefore the project might not be selected.

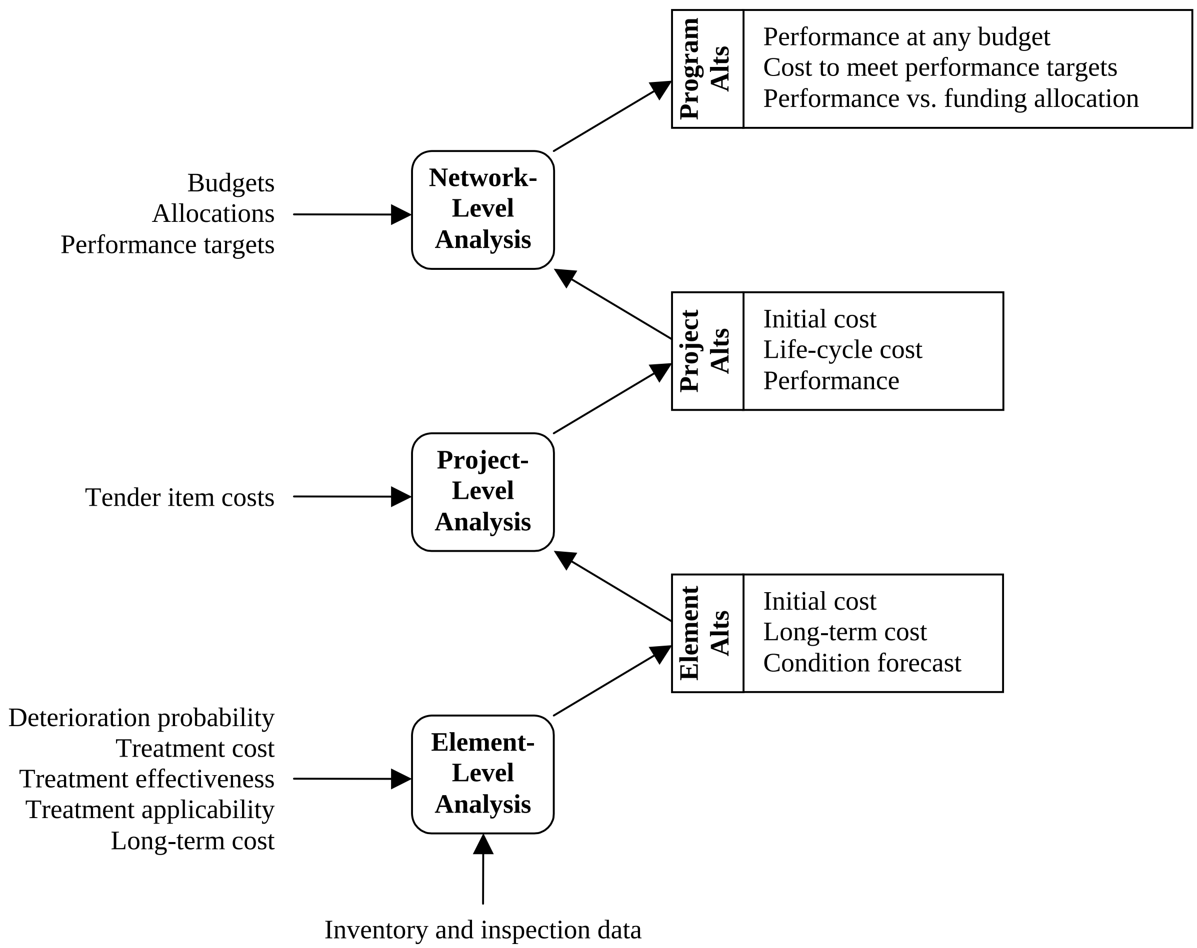

FIGURE 6 Inputs and outputs of primary models.

Life-Cycle Costing Framework

OBMS performs its life-cycle cost analysis over a program horizon of 10 years, with long-term effects extending to 60 years, and focuses on programmed rehabilitation and replacement of bridges. Smaller bridge maintenance activities in Ontario are not programmed but are performed according to routine maintenance standards or by contractors according to contract terms. Therefore, at most one intervention is allowed during the 10-year program. The analytical framework is designed to include smaller, more frequent activities if this is ever needed in the future.

Because of uncertainties in project readiness and inter-relationships with other projects (including nonbridge projects), the implementation timing of any particular bridge project cannot be planned with certainty, especially for lower-priority long-range work. OBMS therefore divides the future into three periods with successively less detail:

- Years 1 to 5. A total budget for the entire period is developed in the annual budgeting process. Projects are identified for this budget in the network-level analysis, to minimize total life-cycle costs. Within the 5-year selected project list, priorities are set according to the effect on life-cycle costs, and a target implementation year is then determined for each bridge within the 5-year period. It is understood that the timing of individual projects may change for a host of practical reasons unrelated to life-cycle costs.

- Years 6 to 10. A total budget for the entire period is developed as in the first period. To minimize total life-cycle costs, projects are selected for this period on the basis of needs that were not met in the first period. Priorities may be set from among the selected projects, but an exact implementation year is not needed.
- Beyond Year 10. A network-level life-cycle cost analysis for each type of element is conducted for a subsequent 50-year period, to estimate the long-term preservation costs for any element given its forecast condition at the end of the first 10 years. To estimate residual life-cycle costs, the results of this analysis are applied to the element-level results for each bridge. The MTO programming process does not require that specific projects be defined beyond year 10, so it is sufficient to have a consistent estimate of remaining life-cycle costs.

Every bridge has multiple project alternatives, each with its own life-cycle cost. In OBMS, life-cycle costs currently have two components:

- Initial costs, estimated at the project level and recognized at the beginning of either the first or second period; and
- Long-term costs, estimated at the element level and then summed over all elements on the bridge and recognized at the end of the second period.

The framework is designed to eventually include user costs of functional deficiencies, but this is not currently implemented due to work on a related system that will provide necessary roadway data.

Element-Level Analysis

The element-level analysis of OBMS produces a life-cycle cost profile of a single element from a deterioration model, a set of feasible treatments, and a long-term cost model. The result is a set of Element Alternatives that is used further in the project level.

Current and forecast conditions are expressed as the percentage of the element in each of four condition states: Excellent, Good, Fair, and Poor. These states are defined precisely in the *Ontario Structure Inspection Manual* (*1*) for each element. The deterioration model uses Markovian transition probabilities, modified by knowledge models (rules of thumb), to estimate the percentage of the element that will deteriorate in each period.

Knowledge models are a unique feature of OBMS and are used in several places in the analytical software. They are expressed as Excel worksheet formulas and rely on Microsoft Excel behind the scenes to provide model parameters and to execute the formulas. Using Excel for this purpose saved MTO on development costs, provided a powerful means of maintaining and enhancing the models, and proved to be fast and seamless.

For deterioration, the knowledge model is a formula for calculating an additive increase or decrease in the median number of years to deteriorate from one state to the next. This is mathematically combined with the transition probability matrix to calculate the fraction in each condition state in the following period. Each element may designate another element on the same bridge, called a protector element, which may influence its deterioration rate. In this way, for example, joint elements can affect the deterioration of caps.

The OBMS treatment model provides a list of agency actions that are relevant to each element. Each treatment is applicable to one or more condition states. A treatment applicability model calculates the quantity of the treatment on the basis of the quantity of the element in each condition state. A knowledge model determines whether the treatment is feasible for the given element on the given bridge. For each feasible treatment, a life-cycle activity profile is generated. Following a treatment, the condition of the element is modeled to change according to the treatment effectiveness model, followed by normal deterioration. Do-nothing is always a feasible treatment and always generates one Element Alternative.

Although project costing occurs at the project level and not the element level, the element model does provide approximate benchmark costs for a first-order cost estimate for use in the first iteration of project-level optimization. This number is a part of the calculation of element-level life-cycle costs.

When the life-cycle cost model looks further into the future, past the end of the program horizon, it is no longer necessary to program specific treatments, and conditions and treatments in general become more uncertain. The long-term cost model is a proxy for the expected value of life-cycle cost of all treatments that will be needed after the end of the program horizon. This value is sensitive to the condition state probabilities at the end of the horizon: worse condition states will have larger and more immediate needs. In the project-level analysis, this serves as a penalty for leaving an element in deteriorated condition. Long-term costs are expressed on a unit basis without any information about any specific bridge. Therefore, they are calculated in advance to save time. Each long-term unit cost is the result of a 50-year simulation of deterioration and actions, with all simulated costs discounted to 10 years before the start of the simulation.

Figure 7 shows the OBMS screen for navigating among elements and Element Alternatives for project planning.

Project-Level Analysis

OBMS generates and evaluates a set of Project Alternatives for both implementation periods for each bridge. Each Project Alternative is defined as a combination of Element Alternatives generated from the element-level analysis (Figure 8). The alternatives evaluated by the models are

- A do-nothing alternative. In this option, no treatment is applied to any element in any period. Life-cycle costs are based on element-level long-term costs, which are based on the predicted conditions of the elements at the end of the program horizon for the do-nothing Element Alternative.
- Two replacement alternatives. With one alternative in each period, the cost of this work is estimated from a unit cost per square meter of deck area. Life-cycle costs include the initial cost plus element-level long-term costs. The latter are estimated for each element on the basis of its replacement Element Alternative.
- Two preservation alternatives. With one alternative in each period, these options are produced by an optimization process. This process finds the combinations of Element Alternatives that minimize life-cycle costs. Life-cycle costing takes the project-level cost model and long-term costs from the selected Element Alternatives.

Each Project Alternative is analyzed as a long-term strategy, with work programmed in one period and no work in the other period, followed by a continuing stream of long-term costs. Each evaluated strategy has an implementation year, a list of Element Alternatives, initial cost, life-cycle cost, life-cycle benefit, and performance measures.

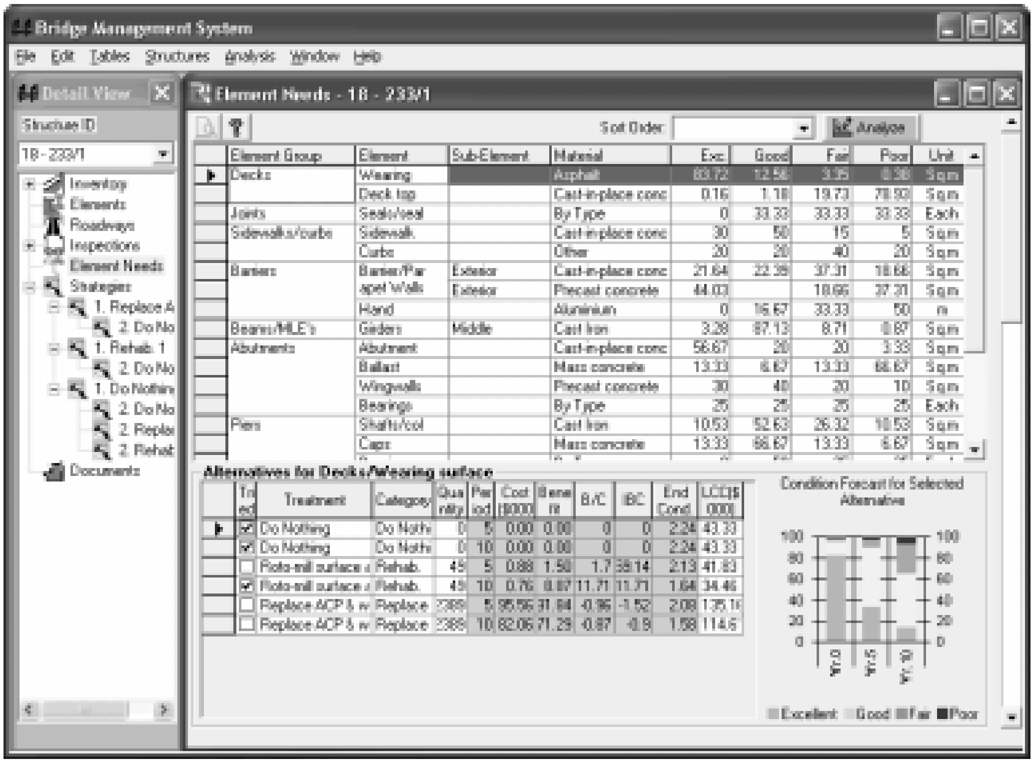

FIGURE 7 Element-level analysis.

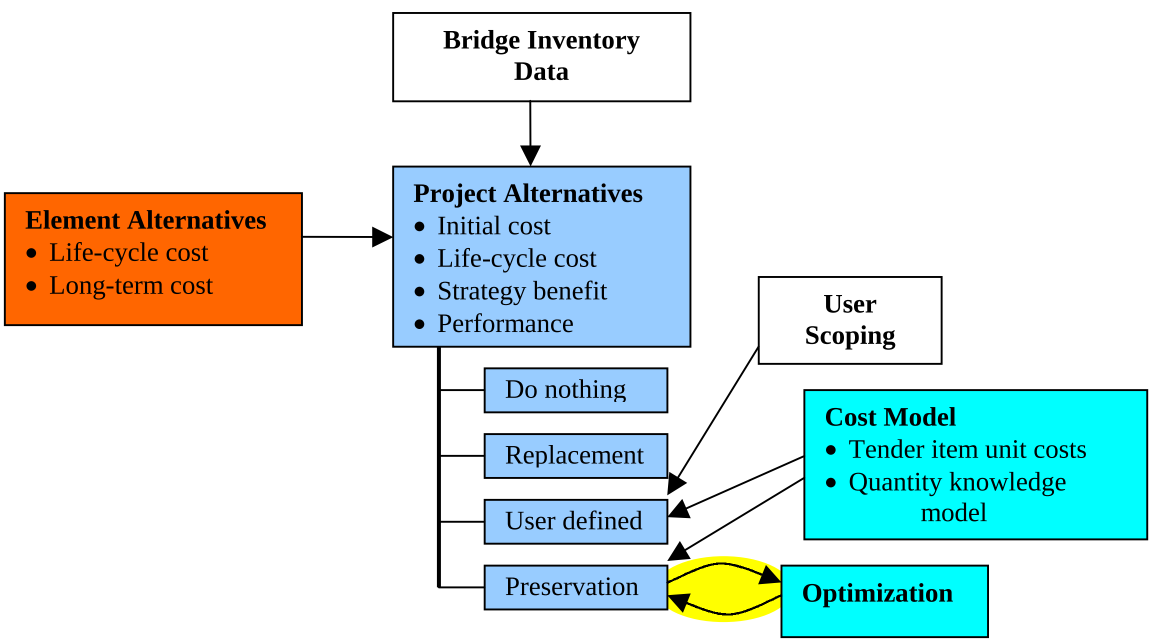

FIGURE 8 Project-level analysis.

OBMS estimates the initial cost of a Project Alternative by estimating the quantities and costs of tender items needed to construct the project. Tender items describe work activities or materials in greater detail than in treatments, which fit the level of detail at which unit costs are normally developed. The MTO estimating office routinely gathers cost information from in-house activities and bid tabulations to maintain a database of unit costs used throughout the agency for project estimation, planning, and bid checking.

Tender item quantities and costs are calculated in OBMS using knowledge models. These can be a function of treatment quantity, or any attributes of the bridge, treatment, or element. They can even be indirect costs unrelated to the quantity of the treatment.

Network-Level Analysis

The network level of OBMS operates on the entire inventory or a subset of the inventory. It identifies a set of Project Alternatives that maximizes the economic benefit of the bridge program by minimizing life-cycle costs, subject to budget constraints. It also reports on predicted future performance measures as they are affected by the amount of funding available.

Each bridge has a list of Project Alternatives generated from the project-level analysis. For each Project Alternative, the following quantities are needed:

- Cost—the initial agency cost of the project, defined as the amount of money to be deducted from the budget during the period of the project;
- Benefit—the savings in life-cycle cost, relative to the do-nothing alternative with consideration in general only of projects with positive benefit; and
- Performance measures.

On any given bridge, only one of the alternatives can be chosen. The network-level optimization finds the set of Project Alternatives that satisfies the budget constraint while maximizing benefits. Inputs to the analysis are constraints and targets that the model attempts to satisfy, including:

- Budget constraints—the total funding available in each planning period,
- Allocation targets—the percentage of the budget to be allocated to each subdivision of the inventory and usually each region, and
- Performance targets—desired levels of a selected performance measure.

The network-level analysis uses an incremental benefit–cost procedure to optimize the selection of projects. Because OBMS is architected as a system of cooperating objects, it has very fast access to the data it needs to conduct this analysis. As a result, the budgeting module (Figure 9) is highly interactive: slide the budget level up or down, and performance measures adjust themselves accordingly. The quickness of this trade-off analysis makes it easy to answer some very important budgeting questions, such as:

- How much improvement in performance can be purchased at any given budget level?
- How much will it cost to maintain performance at its current level?
- What is the true cost of favoring one part of the inventory over another in funding allocation?

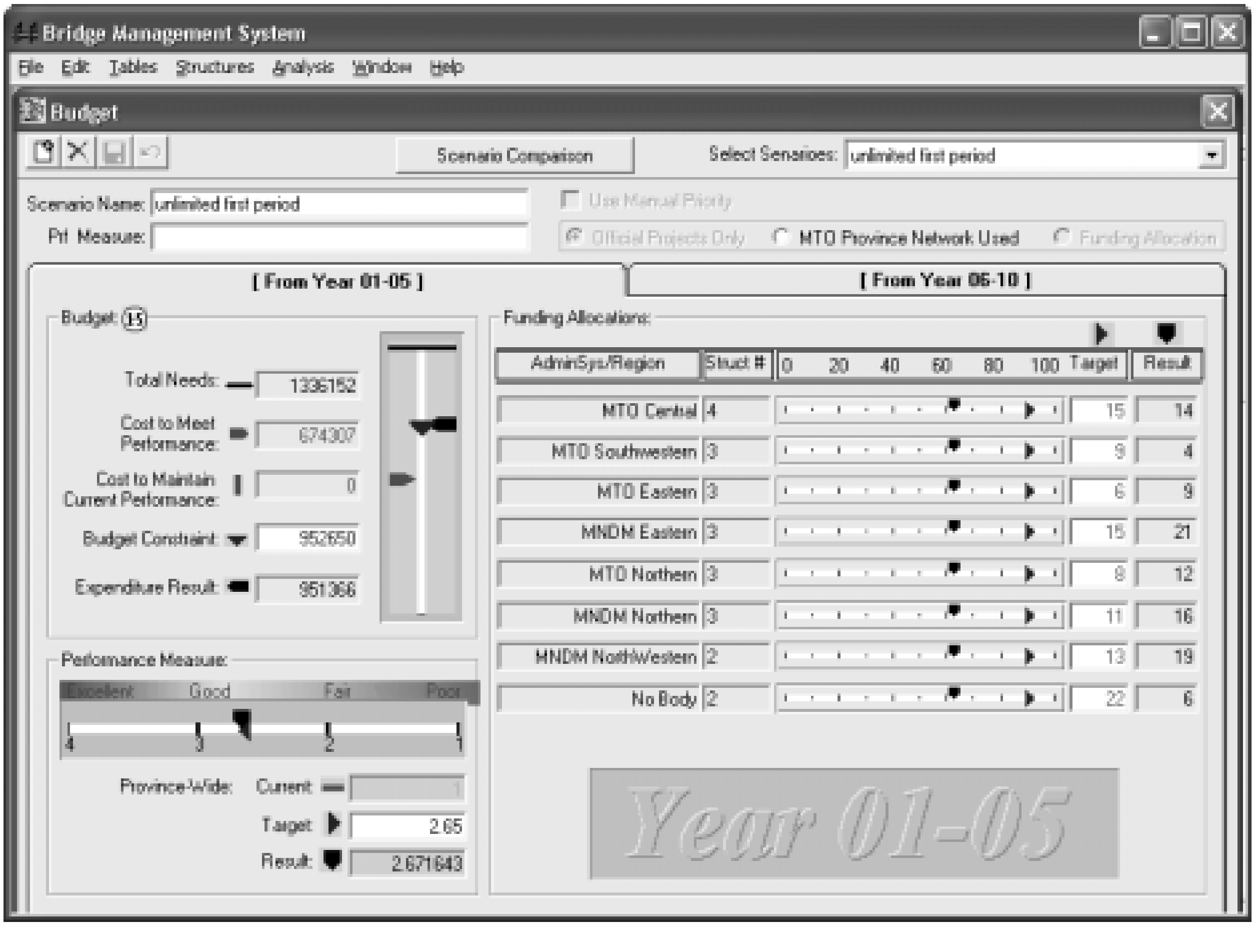

FIGURE 9 Network-level analysis.

- If funding is tightly constrained today, what effect will this have on future costs?

Whenever the budget is changed, this changes the selection of Project Alternatives on each bridge. A priority-setting worksheet shows the selected alternatives in each period. If decision makers disagree with the priorities thus prepared, they can use the mouse to drag projects to what they feel are appropriate locations in the list. This has the effect of adjusting project benefits. The magnitude of the adjustment can be viewed at both the project and network levels so that the effect of all such adjustments is apparent.

FUTURE DIRECTIONS

It is expected that the ministry will begin to make use of the system's project-level and network-level analysis features in routine decision making in the spring of 2003.

The ministry is working on a performance measure for its bridges from the condition data in OBMS. This measure is referred to as the Bridge Health Index, and it is based on the California Bridge Health Index (*6*). It is expected that this new measure will be implemented in 2003. Most bridge decks in Ontario are asphalt-covered, making it difficult to properly assess the condition of the deck top from visual inspection. To supplement the visual inspection data it is expected that data from bridge deck condition surveys will be incorporated into OBMS. These surveys measure corrosion potentials in deck reinforcing and identify areas of high corrosion potential.

Functional needs and user costs are currently not considered in OBMS, though they have always been a part of the analytical design concept. The ministry is undertaking an asset management study to provide direction on the incorporation of user costs in the analysis. Finally, further work is planned on appraisal indexes to represent functionality and vulnerability issues in the management of Ontario's bridges.

REFERENCES

1. Ontario Ministry of Transportation. *Ontario Structure Inspection Manual*. Ronen House, Toronto, Canada, 2000.
2. *AASHTO Guidelines for Bridge Management Systems*. AASHTO, Washington, D.C., 1992.
3. National Engineering Technology Corporation. *BRIDGIT Technical Manual*. NCHRP Project 12-28(2). TRB, National Research Council, Washington, D.C., 1996.
4. *Pontis Technical Manual*. Release 4.0. AASHTO, Washington, D.C., 2001.
5. Ontario Ministry of Transportation. *Structure Rehabilitation Manual*. Ronen House, Toronto, Canada, 2002.
6. Shepard, R.W., and M. B. Johnson. California Bridge Health Index. In *International Bridge Management Conference*, Vol. 2, TRB, National Research Council, Washington, D.C., 1999.

Customization of the Pontis Bridge Management System for a Transit Agency

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The Massachusetts Bay Transportation Authority (MBTA), the nation's sixth largest mass transit system and the oldest continuously operating light rail system in the country, has recently customized a version of Pontis—the most widely used program for bridge management for use with transit structures. This implementation addresses the specific needs of MBTA for managing the maintenance and reconstruction of highway, rail, and pedestrian bridges. Key customization activities include extension of the MBTA database to include information specific to rail bridges; new structure lists to facilitate views of the bridge network by line and bridge type; development of an inspection applet that allows MBTA to continue use of its existing inspection forms—the applet supports several different inspection types for highway, rail, and pedestrian bridges, including routine, fracture critical, and underwater inspections; and development of an applet to store information about utilities on each bridge. The MBTA experience provides an example of how Pontis customization can provide a quick and cost-effective bridge management solution for transit properties.

The Massachusetts Bay Transportation Authority (MBTA) is the nation's sixth largest mass transit system and the oldest continuously operating light rail system in the country. Currently, MBTA owns and maintains 506 bridges. In 1985 MBTA initiated a focused program on bridge management to better manage this valuable capital resource. A computerized bridge management system (BMS) was developed in dBase IV under DOS and provided information on basic administrative data, inspection dates, and complete structural information, such as description, dimensions, condition appraisal, and load-rating information, among others.

In 1998, as the last DOS-based machine was replaced at the authority, MBTA evaluated the available, off-the-shelf Windows-based BMSs. Several existing bridge management programs were evaluated for use. The most widely used program for bridge management was Pontis, a product available from AASHTO. The only drawback to using Pontis was the program's main focus on highway bridges. The inclusion of unique rail bridge characteristics required customization of the existing bridge management application.

A customized version of Pontis was implemented that addresses the specific needs of MBTA for managing the maintenance or reconstruction of highway, rail, and pedestrian bridges. Key customization activities included

- Extension of the MBTA database to include information specific to rail bridges;
- New structure lists to facilitate views of the bridge network by line and bridge type;
- Development of an inspection applet that allowed MBTA to continue use of its existing inspection forms—the applet supports several different inspection types for highway, rail, and pedestrian bridges, including routine, fracture critical, and underwater inspections; and
- Development of an applet to store information about utilities on each bridge.

The MBTA experience provides an example of how Pontis customization can provide a quick and cost-effective bridge management solution for transit properties.

CONTEXT FOR BMS DEVELOPMENT

MBTA

MBTA, established in 1895, was one of the first combined regional transportation planning and operating agencies in the United States. Over the past century, MBTA has responded to an ever-increasing demand for transportation services from the more than 80 communities it serves in eastern Massachusetts. Currently, MBTA is the nation's sixth largest mass transit system and the oldest continuously operating light rail system in the country. It serves a population of more than 3.5 million people within an area of 1,038 mi^2 and has an average daily ridership of approximately 680,000 passengers per day. To provide these services, MBTA maintains 160 bus routes, 4 rapid transit lines, 5 streetcar routes, 4 trackless trolley lines, and 11 commuter rail lines. Its roster of equipment currently consists of 1,108 buses, 220 light rail vehicles, 12 cars, 50 trackless trolleys, 52 commuter rail locomotives, and 305 commuter rail coaches. The rail lines that provide these services are carried at grade, in tunnels, and over elevated structures.

Bridge Infrastructure

Currently, MBTA owns and maintains 506 bridges, which comprise 277 railroad bridges, 58 transit bridges, 83 highway bridges, 77 pedestrian/utility/signal bridges, and 11 freight-only bridges. The average age of MBTA bridges is more than 70 years.

Initial BMS

In 1985 MBTA initiated a focused program on bridge management to better manage this valuable capital resource. The authority let contracts to perform the inventory inspection and load-rating analysis of all MBTA-owned bridges. A computerized BMS was also developed to create a database of relevant bridge information. The system, initially developed in dBase IV, running under DOS on a PC, provided information on basic administrative data, inspection dates, and complete structural information, such as description, dimensions, condition appraisal, and load rating information, among others.

This program was in use from 1985 to 1998 and proved very durable. However, it was quite limited in compatibility with Microsoft Windows. As the last DOS-based machine was replaced in 1998, it was decided to develop a new Windows-based BMS and import the valuable existing database. A team led by Parsons Brinckerhoff, Inc., with BMS software development services provided by Cambridge Systematics, was engaged in 1999 to conduct a comprehensive program of bridge inspections and to implement a new BMS.

NEW BMS DEVELOPMENT PROCESS

Overview

Work on the new BMS began in April 2000 and included requirements analysis, evaluation of off-the-shelf packages, system customization, and data-loading activities. Within 1 month, an operating prototype with MBTA's old data was available for the BMS team to review. The initial version of the customized system was installed in January 2001. Over the past 2 years, a series of additional enhancements (e.g., new forms and reports) have been made, and MBTA staff has undertaken a significant effort to fill in data gaps.

Requirements Analysis

As a first step, the requirements for a new BMS were defined. The primary MBTA bridge management need was to collect data and to establish a systematic procedure to record inventory and inspection data, while a secondary need was to develop an agency bridge program. In addition, MBTA wished to retain much of the data from its existing BMS and provide equivalent or better query and reporting capabilities. The old system was examined to determine which data items were actively used and which could be dropped in the new system.

Evaluation of Off-the-Shelf BMSs

The initial phase of this work included a survey of currently available systems that would meet MBTA's requirements. There were several on the market at the time, including IBIS, BRIGIT, and Pontis. No packages were identified that were oriented toward railroad bridges.

After reviews of each BMS, it was determined that Pontis offered the best option for MBTA for the following reasons:

- Pontis was sponsored by AASHTO, a national transportation agency.
- Over 49 departments of transportation (DOTs) and other agencies were involved in its development.
- A large installed base and users group already existed.
- Software was available to MBTA at no cost through the state highway department.
- Pontis offers capabilities to both record inventory and inspection history and develop a bridge program that includes project generation, planning, and tracking.
- Local enterprise (Cambridge Systematics, Inc.) was capable of providing additional development, continuous support, and on-demand training.

Customizing Pontis for MBTA

The Pontis application was developed with the primary intent of meeting the bridge management requirements of highway agencies. To be of value to MBTA, Pontis had to be customized to accommodate not only highway bridges but also transit and railroad bridges.

Workshops were held to review the current features of Pontis and to explore what aspects of the system could be customized. Pontis allows integration of new data items and tables. It is packaged with the Sybase InfoMaker product that allows development and integration of custom reports and forms. In addition, mini-applications called “applets” may be developed and integrated that provide specialized functionality. Some aspects of Pontis, for example, the standard tools for searching and subsetting the structure list, as well as standard tab cards of information, may not be modified without source code changes.

The BMS team developed a list of new features and where they should be available in the program. They developed a list of desired data elements that were not part of the standard Pontis database and a design for where these items would be viewed and edited.

One of the key decisions was whether to implement element-level inspections supported by Pontis, which provide the basis for the system’s analytical capabilities including optimization and simulation. MBTA decided to defer this type of data collection, since its primary BMS needs were for bridge data management and reporting. The agency did not feel that the effort necessary to define elements and estimate their quantities for each structure was yet warranted. However, the door was left open for adding capabilities at a later date.

The customization effort involved the following major components:

- Database customization,
- Screen and report customization,
- New MBTA inspection applet development, and
- New MBTA utilities applet development.

Each of these components is briefly discussed in the following section.

KEY FEATURES OF THE MBTA PONTIS SYSTEM

Database Customization

The standard Pontis data structures for bridge inventory information, based on the National Bridge Inventory (NBI), were able to accommodate most of the data that MBTA wanted to store. In some instances, standard Pontis fields were used for a slightly different purpose than intended —e.g., the administrative area field typically used to identify maintenance areas (below the highway district level) in a state DOT was used to identify transit lines and commuter rail branches. In addition, the standard Pontis (and NBI) fields for inventory and operating ratings were used to store separate ratings for stress and fatigue for transit and commuter rail bridges. A number of the pick lists in the system were customized to accommodate the administrative environment of MBTA and the engineering characteristics of railroad and transit bridges.

Pontis provides standard tables for agency-specific information related to bridges and inspections. Bridge inventory items added for MBTA included a bridge type category (highway, transit, railroad, pedestrian, utility), clearances left and right of track, number of tracks under the structure (for inspections) included the name of the inspection team leader, the name of the quality assurance engineer, and the inspection equipment required.

MBTA wished to computerize its entire suite of existing inspection forms, which include routine and underwater inspections based on a 0 to 9 rating for the various bridge elements (e.g., columns, pedestals, footing, etc.), as well as fracture critical and damage inspections, which provide details on specific members. A series of new tables were added to the database to accommodate these inspection forms—which varied by bridge type. The basic design was to have the standard Pontis inspection event table store a record of every inspection conducted. However, detailed inspection information associated with each inspection is stored in a table specialized for that type of inspection.

Screen and Report Customization

A large number of standard bridge lists were developed to allow MBTA staff to quickly select the view of the inventory they wished to work with—for example, railroad bridges on the West Route Commuter Rail branch, highway bridges on the Green Line, or fracture critical bridges on all lines. These views were also implemented as reports.

Two custom forms for the agency-specific Pontis bridge and inspection information were added to the inspection module tab cards using standard Pontis features. The MBTA-specific inspection tab card was used to indicate the type of detailed MBTA inspection to be added to the system. Figures 1 and 2 show the agency bridge and inspection tab card screens.

Specialized reports were added to the system to allow MBTA to print any of its standard inspection forms, to provide summaries of bridge condition, and to view inspection scheduling information.

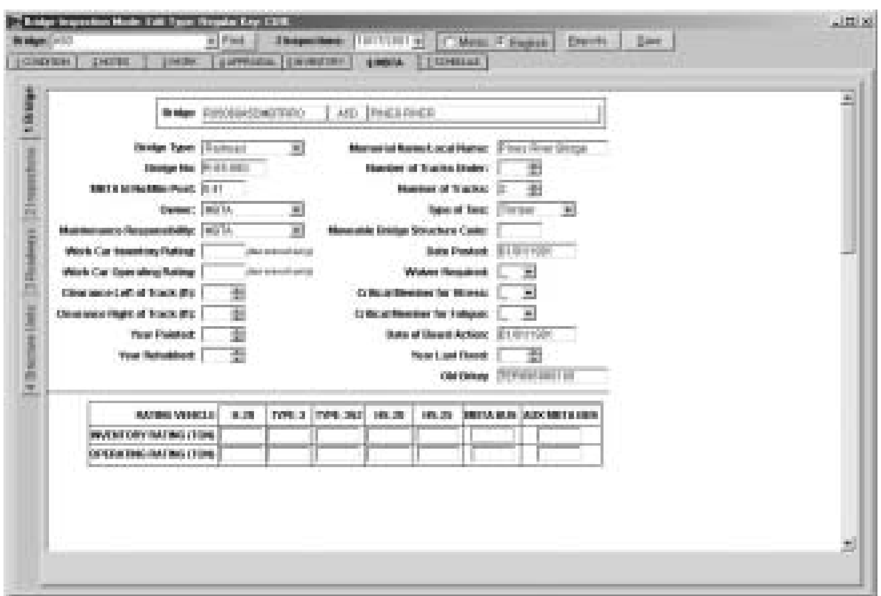

FIGURE 1 Agency bridge tab card—sample screen.

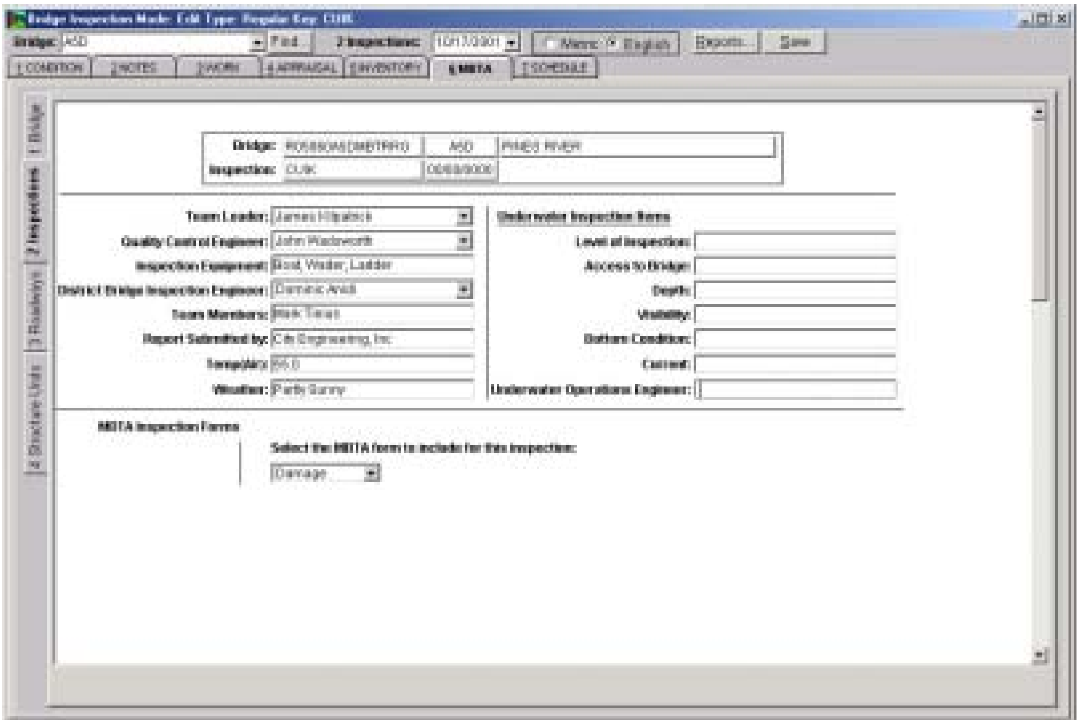

FIGURE 2 Agency inspection tab card—sample screen.

MBTA Inspection Applet

An inspection applet was developed to support data entry for MBTA's suite of inspection forms. To add an MBTA inspection, the user selects the bridge from the desktop structure list, navigates to the agency inspection tab card, and indicates the type of inspection to be performed. Then, the user selects the applet from the Pontis desktop and views past inspections or completes a new one. Database triggers are used behind the scenes to keep the new inspection data coordinated with the standard Pontis inspection tables. A sample screen from the inspection applet is provided in Figure 3.

MBTA Utilities Applet

A second applet was developed to track utilities on each bridge. To view or enter information about what types of utilities are on a specific bridge, the user selects the bridge from the desktop structure list, and then clicks on the utilities button on the desktop. Figure 4 shows a sample screen.

Mapping of Bridges

In a parallel effort, a GIS mapping task was completed that located all MBTA bridges on a large colored map. Included were MBTA's service lines, stations, and local street network. These maps proved valuable to MBTA police forces during a recent security alert.

FUTURE INITIATIVES

An important initiative is to store photographs taken by MBTA inspection teams with the bridge inspections to provide a visual record of damages. The new multimedia enhancement in Pontis 4.2 provides this capability.

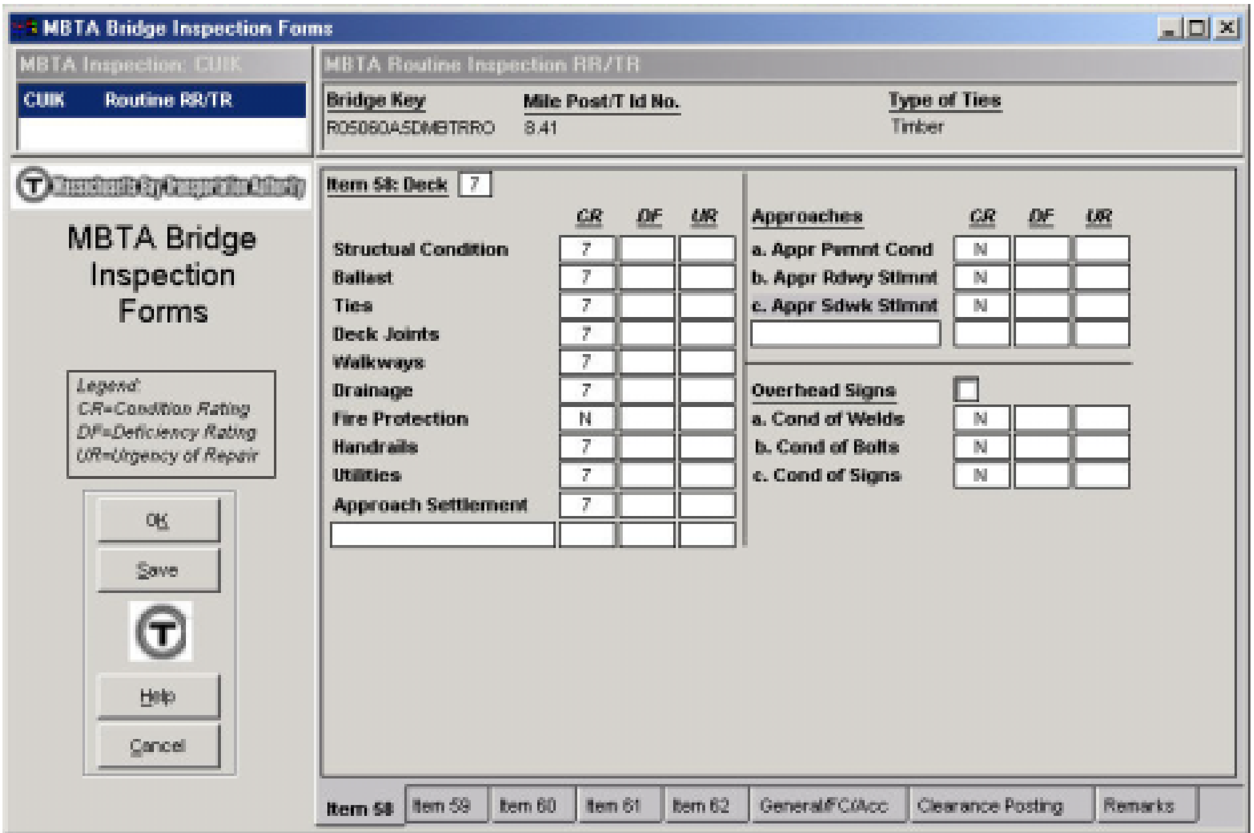

FIGURE 3 Inspection applet—sample screen.

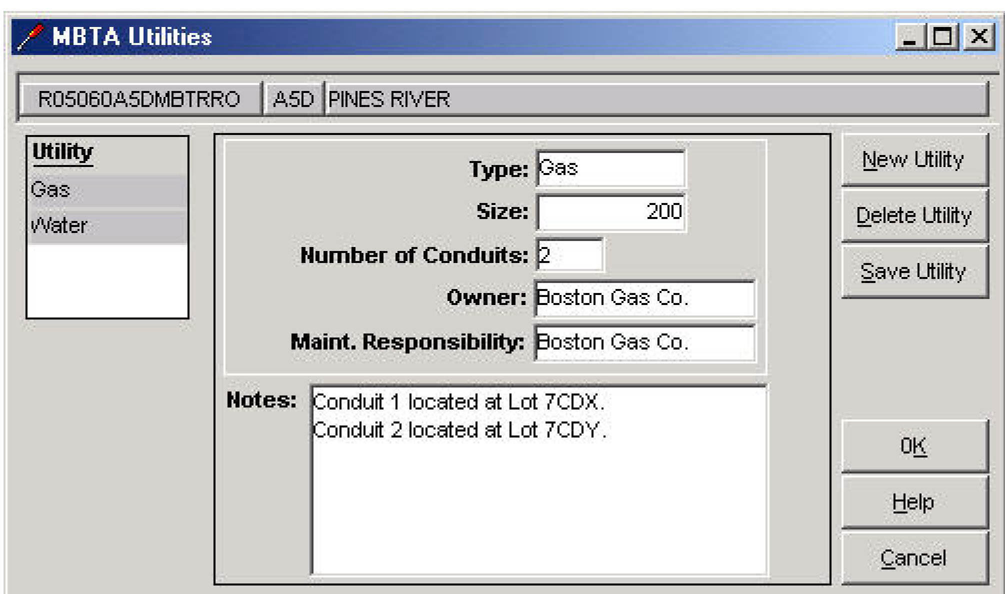

FIGURE 4 Utilities applet—sample screen.

In addition, MBTA is investigating the use of palm handhelds or pocket PCs to allow inspectors to gather data in the field for direct upload into Pontis. Looking further to the future, even though MBTA decided not to implement element inspections at the present time, the capability is in place to do this to take advantage of the modeling features of the Pontis program.

CONCLUSION

MBTA has been very pleased with the BMS development process. The Pontis database is now installed on MBTA's network, and the program is running on four workstations. The software allows for multiple concurrent users and preserves data integrity. Currently, MBTA is entering data from periodic field inspections, printing out basic reports, and preparing advance bridge inspection schedules.

The Pontis customization process completed by MBTA was the first undertaken by a transit property. However, the value and potential of MBTA's Pontis customization extends to other rail transit properties.

Bridge Management Aspects of Asset Management

Asset Needs from the Bottom Up

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During the last decade increasing effort has been applied in asset management. Many ideas and reports were created during this period by or for top management. The goal of asset management is to create tools for the use of the final decision maker, so that each project will be evaluated in an easy and comparable manner. During a long period, various management systems have used different methods to rank major repair projects, and many administrations are using poor tools and methods for spending their funds. The gap between the ministry, the top administration of infrastructure, bridge management, and the final decision maker is too big, and understanding among the parties is limited. The projects evaluated here are those for which a reconstruction is expected to take place within the next 10 to 15 years. Long-term budget needs are evaluated through good and proper inventory combined with studies of lifetime. Ranking may be complicated, and it is not always clear which parameters should be included. Parameters are evaluated here, and parallels are drawn to the net present value method. The need for funds has to be well documented, and consequences have to be explained clearly. Methods must be easy to understand at all levels of administration, and it must be easy to see and explain the included parameters in the evaluation and planning of maintenance. A preliminary introduction is given to net present value, an extended and well-considered ranking method from an economical point of view, to discuss why assets may be evaluated from the bottom up.

For several years asset management has become increasingly popular, especially within the circles of top management. However, it is very important to create a common understanding of assets on all levels of administration from government and ministry to the final decision makers.

Top management must understand all the factors that will lead to a decision on a lower level and must create understandable guidelines and take relevant steps to secure a uniform platform for asset management. The final decision makers also have an important role to play. It is not only important for them to understand guidelines and instructions given from above, but it is also important to send signals up in the system, to correct mistakes, and to improve the situation.

There is very often limited contact and understanding between the two parties. Those in top management do not spend sufficient time on lower-level activities, while those at the lower level then shake their heads and continue to do what they are accustomed to do.

The different levels of government administration are typically as follows:

Government–politicians
↑↓
Ministry
↑↓
Asset manager (high ranked)
↑↓
Local asset management staff
↑↓
Local decision maker
↑↓
Consulting companies - contractors

At the lower level many activities have an important influence on the use of assets. Important economical influences besides those mentioned later are these:

- The right choice of consultant,
- The right use of the contractor,
- A good description of the project, and
- Good tender documents.

A mistake in one of these activities may easily influence the net present costs by 20%. On the other hand, on a project with a value of US$10 million (all reinforced concrete), an initial savings of 20% will be equivalent to the costs for replacement of the structure after approximately 55 years if you have a discount rate of 3% (Figure 1). Figure 1 shows in principle the deterioration of a reinforced-concrete structure compared with the time development of initial savings of 10% and 20%, respectively, at discount rates of 3% and 7%. If the discount rate is 7% and the initial saving is 10%, replacement can take place after approximately 33 years.

For Figure 1 the deterioration is assumed to be bilinear. In a case in which U.S. $2 million is saved during the bidding and construction phase, the savings (if properly invested) will result in an amount equivalent to the cost of a new structure after 55 years. If the savings are US$1 million, this point is reached after 75 years. Both examples consider a discount rate of 3%. It is important for the top manager to consider the discount rate properly, and it might be beneficial to study present investment much more than estimating the lifetime of the structure to 80 or 100 years based on various lifetime models. If the discount rate is considered at 7%, the change of structure could take place for 22-33 years with a net present saving of US$1-2 million.

On the other hand, Figure 1 also shows that even a small saving today is a good investment, since the return of these investments (savings) will be similar to the reinvestment on the structure a long time before its service life is ended.

To send out tender documents in the right period of the year can easily influence the price by 10%. Further on, the final decision maker may wish a steady activity flow. Experienced contractors and experienced personnel may become unavailable if you have long periods without activity, and this may lead to accepting bad workmanship or high prices. It might be necessary to split major projects into minor ones, so that a certain amount of competition can take place.

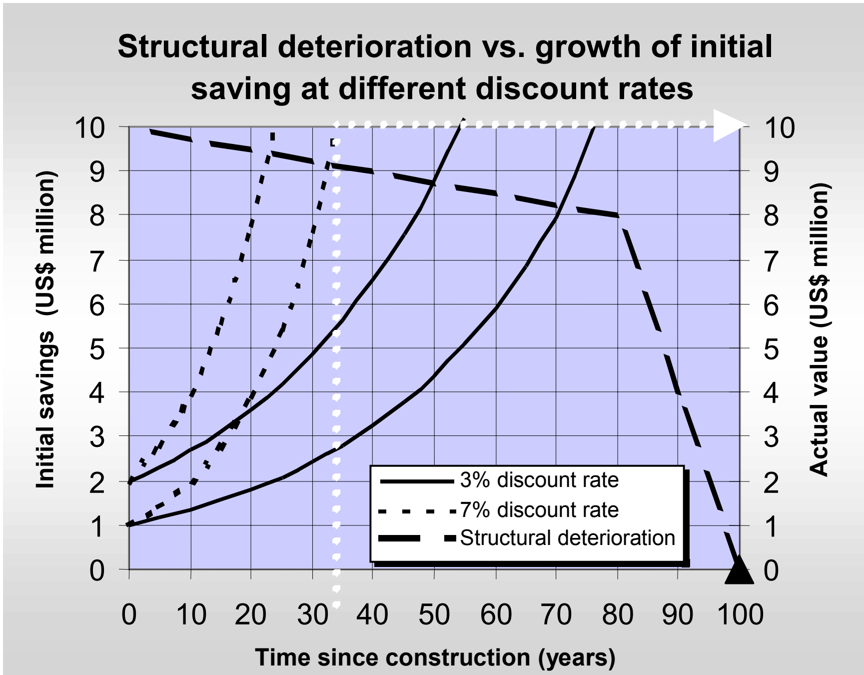

FIGURE 1 Deterioration versus initial savings for a reinforced-concrete structure with a service life of 100 years.

Figure 2 shows the development in costs as a function of the discount rate and time. It is very simple to calculate these changes; however, it often comes as a surprise to decision makers to see how much the discount rate affects the net present value of the costs.

Administrative routines and inspection routines, which lead to repair projects, are major projects in themselves, with regard to both manpower and economy. These include

- Superficial inspection,
- Principal inspection, and
- Special inspection

It is important to revise the routines now and then. The results must be reliable. Clear interfaces and well-defined description of activities must be clear for all involved parties.

In some countries the responsibility of the superficial inspectors has been increased, as their judgment leads directly to execution of work in the field.

Principal inspection is related to economy—not only damages and condition marks, but also an estimation of the next inspection year (the maximum may be set to 6 years). The increased responsibility makes the inspector more concerned about the results, and the allowable time span between inspections makes the inspection economy better. Another factor is the psychological one: if nothing happens to the structure (no deterioration takes place) from one

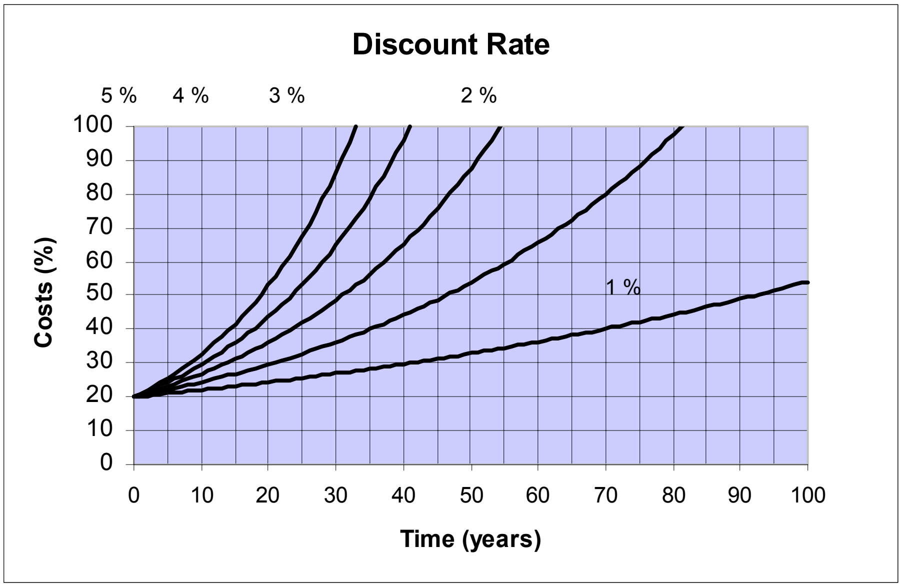

FIGURE 2 A simple calculation that shows the discount rate and time influence on the development of costs.

inspection to the next, the inspector could become "blind" in general. Formerly, ranking of bridges for repair was only based on condition marks, but now many authorities look on additional aspects related to the project.

Special inspection provides for both in-depth investigation of the structure and evaluation of repair strategies, which include construction costs, road user costs, safety and risk, impact on environment, etc. These strategies are one of the cornerstones connecting bridge management to asset management. Many of the aspects mentioned below are included in these analyses. See the strategy form in Table 1.

DETERIORATION OF ELEMENTS, DIFFERENT FACTORS

One of the major tasks of asset management is to make estimates of long- and short-term needs and to be able to create repair costs analyses in connection with strategies based on life-cycle costs. These needs are very much influenced by the estimation of service life of the various bridge elements. To support evaluation of strategies, a list of various elements related to service life and replacement costs should be created. An example for Danish conditions and environment is given in Table 2.

It is very important to continue research in estimation and prediction of service lifetime and deterioration periods—first of all, to update the above-mentioned considerations but also to find designs, materials, and systems with longer service life without increasing initial costs.

To be able to support asset management with reliable estimates, it is necessary to run a price book in the bridge management system. This price book will be a valuable tool to estimate replacement and repair costs. However, the local knowledge of deterioration factors is very important. The lifetime may be much shorter or longer depending on local conditions. The lifetime is influenced by the following:

TABLE 1 Strategy Form

Bridge 1: Repair of Pavement and Waterproofing						**Discount Rate (%): 3**			
	Direct Costs					Indirect Costs			
Investment Year	Road Repair	Bridge Repair	Superficial Road Maintenance	Superficial Bridge Maintenance	Traffic Management	User Costs	Safety	Environment	Other Items
0		3.000			300	12.000			
5				50					
10				50					
15		1.000			100	3.300			
20				100					
25									
30		1.000			100	3.300			
35									
40		3.000			300	12.000			
45				50					
50				50					
55		1.000			100	3.300			
60				100					
65				100					
70		1.000			100	3.300			
75				200					
80		3.000			300	12.000			
85				50					
90				50					
95		1.000			100	3.300			
100				100					
Costs	0	14.000	0	1.200	1.400	52.500	0	0	0
						Total Costs			69.100
NPV	0	5.639	0	345	564	21.549	0	0	0
						Total Net Present Value			28.097

Bridge 2: Repair of Pavement and Waterproofing						**Discount Rate (%): 3**			
	Direct Costs					Indirect Costs			
Investment Year	Road Repair	Bridge Repair	Superficial Road Maintenance	Superficial Bridge Maintenance	Traffic Management	User Costs	Safety	Environment	Other Items
0		1.000			100	3.300			
5				200					
10		3.000			300	12.00			
15				50					
20				50					
25		1.000			100	3.300			
30				100					
35				100					
40		1.000			100	3.300			
45				200					
50		3.000			300	12.000			
55				50					
60				50					
65		1.000			100	3.300			
70				100					
75				100					
80		1.000			100	3.300			
85				200					
90		3.000			300	12.000			
95				50					
100				50					
Costs	0	14.000	0	1.3000	1.400	52.500	0	0	0
						Total Costs			69.200
NPV	0	5.151	0	426	515	19.187	0	0	0
						Total Net Present Value			25.278

(*continued on next page*)

TABLE 1 (*continued*) Strategy Form

Road: Repair of Pavement						**Discount Rate (%): 3**			
	Direct Costs					Indirect Costs			
Investment Year	Road Repair	Bridge Repair	Superficial Road Maintenance	Superficial Bridge Maintenance	Traffic Management	User Costs	Safety	Environment	Other Items
0			500						
5	50.000				600	12.00			
10			200						
15			200						
20	3.000				300	3.300			
25			400						
30			400						
35	3.000				300	3.300			
40			500						
45	50.000				600	12.00			
50			200						
55			200						
60	3.000				300	3.300			
65			400						
70			400						
75	3.000				300	3.300			
80			500						
85	50.000				600	12.00			
90			200						
95			200						
100	3.000				300	3.300			
Costs	165.000	0	4.3000	0	3.300	52.500	0	0	0
						Total Costs			225.100
NPV	64.000	0	1.553	0	1.097	18.589	0	0	0
						Total Net Present Value			85.364

Road and Bridges: Repair of Pavement (and Waterproofing)						**Discount Rate (%): 3**			
	Direct Costs					Indirect Costs			
Investment Year	Road Repair	Bridge Repair	Superficial Road Maintenance	Superficial Bridge Maintenance	Traffic Management	User Costs	Safety	Environment	Other Items
0			500	400	600				
5	50.00	6.500				12.000			
10			200	100					
15			200	100					
20	3.000	2.000			300	3.300			
25			400	200					
30			400	200					
35	3.000	2.000			300	3.300			
40			500	400					
45	50.000	6.000			600	12.00			
50			200	100					
55			200	100					
60	3.000	2.000			300	3.300			
65			400	200					
70			400	200					
75	3.000	2.000			300	3.300			
80			500	400					
85	50.000	6.000			600	12.00			
90			200	100					
95			200	100					
100	3.000	2.000			300	3.300			
Costs	165.000	28.500	4.300	2.600	3.300	52.5	0	0	0
						Total Costs			256.200
NPV	64.125	10.160	1.553	987	1.097	18.589	0	0	0
						Total Net Present Value			96.510

TABLE 2 Estimated General and Local Service Life of Different Bridge Components

Main Elements	Subelements	Type of Product	Estimated General Lifetime, Years	Estimated Local Lifetime, Years
Superstructure	Slab	Reinforced concrete	50	40
		Posttensioned concrete.	50	40
		Steel1	35	45
		Steel2	40	45
		Wood	30	25
Pavement	Wearing course	Type1	15	12
		Type2	20	17
		Thin layer	12	10
	Base course		40	40
	Bearing course	Latex concrete	50	40
Edge beam	Cast in situ	Reinforced concrete	50	40
		Posttensioned concrete	50	40
		Stainless steel	80	70
	Pre-cast elements	Black steel	35	30
Waterproofing membrane	Membranes, bitumen		40	35
	Polyurethane		35	30

- Traffic,
- Climate,
- Design and geometry of elements,
- Contractor experience and execution, and
- Supervision during construction.

The figures in Table 2 could be considered as national data, and a corresponding local service life could be created by the final decision maker based on his or her own experience. A corresponding table with national and local unit replacement costs could be created in a similar way. Data in the table have to be updated now and then, for they are important factors in asset management. Variations may also appear from one bridge to the next. Consequently, lifetime estimates may in some cases be governed by local conditions.

The top-level authorities often require estimates for reliable long-term funding; consequently, they have to give the lower administration levels the means (money and manpower) to collect inventory data and ensure that they are continuously updated. A detailed inventory (year of construction, element, design, material and quantity) combined with the lifetime estimates are the main ingredients in an accurate forecast of coming needs.

To estimate the total bridge assets, a broad but precise definition is needed to determine what is a bridge. Many authorities use minimum length of span or bridge length as the criterion for defining a structure as a bridge. However, minor box and pipe culverts should be included in the funding estimates, since normally there are 5 to 10 times more culverts than bridges and they represent a significant part of assets in the infrastructure.

BUDGETS

Top management could improve the use of money for rehabilitation projects by these simple means:

- Require well-documented investment plans for at least 3 years.
- Use rolling budgets known for at least 3 years. (The management must be able to argue with the ministry or government to make this arrangement possible.)
- Allow for a certain "+" or "–" margin in the yearly budget.

These conditions can have an important influence on repair projects. Very often, the final decision maker works under a strong time pressure to implement repair projects, which can often lead to bad and expensive results.

Planning with other authorities and road users is needed. Negotiation with supply companies, water, gas, electricity, telephone, cable TV, etc., is needed.

Evaluation of various traffic-handling possibilities during the repair period is important, and road users and neighbors should be informed.

If time is too limited, the work can be affected adversely, and the contract sum may increase by as much as 20%. This has a dramatic effect on the economy for the structure's lifetime (Figure 1).

In many administrations too many efforts are made to spend the allocated budget by the end of the financial year, which very often leads to misuse of budget funds. It is very strange, as the same political leaders easily accept major contracts, with private companies, covering work through many years. Public administrators should have the same possibilities.

DISCOUNT RATE

To estimate net present values, it is important to use the same discount rate and reference year for all the projects or strategies to be compared.

The actual discount rate might be defined as the actual interest rate of the society minus the actual inflation. However, normally the discount rate is set to an arbitrary value by the top-level decision makers (board of directors or political authorities).

If the rate used is too high, and there are no other changing factors besides deterioration, it will not be beneficial to repair anything before the point of no return for the structure on which significant lower load-carrying capacity or other dangerous situations start to occur. This actually means that the strategy turns from repair and maintenance to replacement.

The discount rate has to be reconsidered now and then, and top-level management has to inform the final decision maker.

TRAFFIC COSTS, ROAD USER COSTS

Various Methods to Handle Traffic During Repair Periods

Traffic handling in repair periods can be an essential part of a project, and alternatives may be considered together with all parties involved (road users, other road administrators, police, cable owners, etc.).

Traffic regulations may be very expensive, and in many cases it can be an advantage to change the project because of traffic handling, i.e., widening the bridge, splitting the project into minor parts, or constructing an intermediate bridge—all very costly efforts.

The final decision maker has to get some indications from the top level to be able to select among the various possibilities or to get a clear signal that it is not necessary to take certain factors into account.

Road User Costs Including Detours for High, Heavy, and Wide Vehicles

The price for delays (price per minute) for cars and trucks is evaluated for each of the various possibilities for passing the project.

Further, the maximum accepted waiting period for a single vehicle must be stated. During planning of the projects it is possible to calculate the expected tailbacks due to the repair work and the expected time to get through the tailback (to pass the work area).

The price for detours (accepted price per kilometer) for cars and trucks (also heavy, high, and wide vehicles) and the maximum accepted length of detours must be stated.

In many administrations there are no clear guidelines on these points. When a traffic jam occurs, the basis for dispute is present between asset managers at the top and lower level and between the managers and the press.

It is also very much needed to have approved estimates for future development in the amount of traffic. This parameter is important for considering all strategies for life-cycle analyses.

Areas where traffic intensity is near the capacity of the road, and there is a high yearly increase in traffic volume, can create a severe problem to handle. In this situation, it is very important to have some guidelines; maybe you will have to “repair” (update) bridges, even when no signs of damage are seen, because of the expected low remaining lifetime caused by, e.g., capacity problems.

RECONSTRUCTION PLANS FOR ROADS AND BRIDGES AND COMBINATION OF BRIDGE AND ROAD MAINTENANCE

It is crucial that the final decision maker gets information on planning issues for the future (for instance, the following 20 years). A huge amount of assets are used up when invested in a wrong manner.

Traffic may be handled on new roads, and reconstruction of existing roads (widening, higher clearance, higher load-carrying capacity) may influence existing structures so dramatically that recent repair investments are lost. In some states and countries up to 80% of the bridge repair and reconstruction costs are used on “new” roads, which means that contact with planning authorities is needed.

Road user costs and traffic management might be very expensive, and too often closing of roads and lanes is inconvenient and psychologically unacceptable to the users and the public. Consequently, it is desirable to combine road and bridge repair when possible.

The above aspects must be considered when Table 1 is filled in. As can be seen from the tables, there might be tremendous savings by carrying out proper planning of projects along the road and combining different bridge and road maintenance works.

SAFETY AND RISK FACTORS

Risks both with regard to workers and road users may be considered during the period of repair, but normally these expenses are neglected. However, depending on the repair contract, the contractor can take responsibility for all accidents in the period of the repair work.

In some countries, insurance companies will not insure the contractor’s risks. Consequently, the asset managers must consider if the public should take the risk, as the public authority often is responsible for design and approval of the traffic regulation around the project.

One safety factor is related to design according to old standards, and safety is also affected by deterioration. One example could be old railings, not strong enough according to today's design rules, or a bridge could be too narrow. Very few top asset managers have clear rules for replacement or widening. Another factor could be limited friction of the wearing course; clear rules are often missing here too.

To prevent bridge collapse, an unacceptable occurrence, all bridges have to be inspected often and detailed, and this may be far from feasible in the long run. Here, a probabilistic approach may be used to point out the risk considering a whole bridge stock. A minor problem compared to collapse, but still important, is the situation in which elements or pieces of elements fall down on a highway. This may lead to an overreaction requiring that all the involved elements shall be secured immediately—even though many other severe problems then have to be neglected. Risk factors of this kind are very difficult to handle in an objective way; however, it is an important topic to evaluate—how to handle assets in a proper manner.

CALCULATION FORM FOR SIMPLE AND UNDERSTANDABLE NET PRESENT VALUE ESTIMATES ON DIFFERENT ALTERNATIVES

In Table 1 a simple example of a strategy form is shown. As mentioned before, the form must include all parameters that the top asset management wishes to include in prioritization. The simple example in Table 1 includes the above-mentioned factors that influence the total costs, both direct costs and indirect costs. The idea is also to inspire the proper planning of all projects along a road because a relatively large amount of money could be saved in this way.

For the various strategies, the total net present values for each strategy have to be compared and the most feasible one chosen. One strategy may include more bridges and also road repair. This strategy may be compared with the sum of net present values for each individual project. For each project it is possible to calculate an interest rate of investments and compare this with the postponed solutions.

All strategies should have a postponed strategy (for instance, 10 years). The postponed strategies should include the additional costs due to the postponement so it will be possible to see how much the costs for a postponement will be. The postponed solution is not shown in the tables.

FUTURE NEEDS FOR OPTIMAL ASSET NEEDS MANAGEMENT

Asset management is needed, and more and more efforts have been applied in this discipline in recent years. Used in the right way, asset management may help to optimize the use of allocated funds.

Asset management can be used to compare investments between very different activities (roads, bridges, education, hospitals, social welfare, etc.). It is important, however, that the principles for implementing asset management be understood on all levels, and that personnel at all levels have some understanding of the needs of other levels. Today, there is a lack of understanding among the involved parties. A dialogue may improve this by using common guidelines and simple tools.

Life cycles of various elements are important, but as explained in this paper, the final decision makers need better decisions and guidelines from politicians and top management to improve their work. There are many evident and “easy” decisions that may improve bridge management significantly.

It is necessary to implement a well-understood guideline for the contents of Table 1. A dialogue and better understanding are needed!

Condition-Based Bridge Asset Valuation

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Understanding the relative value of infrastructure assets such as roadways, bridges, and other transportation facilities is important when making decisions regarding the allocation of funding across different infrastructure types. The criterion used to allocate available funds is generally a function of identified needs weighed against a required service level or political influences. Oftentimes the relative value of assets is completely ignored in the fund allocation process. By understanding the relative value of the various infrastructure assets, better decisions can be made in the split of preservation funds. In addition to the benefits in the preservation fund allocation process, new requirements coming from the Governmental Accounting Standards Board (GASB) Statement 34 have required bridge owners to provide a value for all major infrastructure assets constructed after June 30, 1980. To comply with the requirements of GASB 34, the California Department of Transportation has developed a methodology for asset valuation that is based on the various materials, design types, and current condition of the bridges in the state. The procedure developed to assess the value of the state-owned bridges in California is described.

Understanding the relative value of infrastructure assets such as roadways, bridges, and other transportation facilities is important when making decisions regarding the allocation of funding across different infrastructure types. These infrastructure assets can vary substantially in usage, construction material, design type, service level, and condition. Though much work has been done in the area of infrastructure asset management, objective economic analysis among different asset types is still greatly lacking. At the root of the problem is the variability in the measure of benefits across assets. In the absence of management system tools that can objectively compare the benefits of actions on various infrastructure asset types, other methods of distributing funds for preservation must be explored.

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In addition to the benefits in the preservation fund allocation process, new requirements coming from the Governmental Accounting Standards Board (GASB) Statement 34 have required bridge owners to provide a value for all major infrastructure assets constructed after June 30, 1980. To comply with the requirements of GASB 34, the California Department of Transportation (Caltrans) has developed a methodology for asset valuation that is based on the various materials, design types, and current condition of the bridges in the state. This document outlines the procedure developed to assess the value of the state-owned bridges in California.

GASB 34 REQUIREMENTS

GASB Statement 34 requires infrastructure owners to retroactively determine the value of all assets acquired, renovated, or improved after June 30, 1980. Statement 34 further clarifies this requirement by stating that ". . .if determining the actual cost of general infrastructure assets is not practical because of inadequate records, then report the estimated historical costs of those assets acquired, significantly reconstructed or significantly improved in fiscal year ending June 30, 1980." Statement 34 has forced many agencies to begin looking at infrastructure valuation for the first time. In California, the value of the highway vehicle transportation system had been estimated at $300 billion for several years. This figure was repeated in numerous documents, but it lacked supporting documentation. No breakdown of value was available for the different classes of infrastructure assets. More than 12,000 bridges in the state were assumed to be included in this $300 billion estimate, but the value of the bridges as a group was not known. In addition to the highway pavement and bridges, Caltrans also began looking at other assets such as park and ride facilities, vista points, roadside rest areas, landscape areas, highway pump facilities, and other ancillary infrastructure items.

The historic records for the infrastructure assets were readily available for some assets, such as bridges, but were nonexistent for other assets. The records for almost all of the infrastructure assets were not in an electronic format, which would have facilitated the valuation process. For example, the plans for highway projects and bridges were available in a paper form that would require the department to manually review these documents to determine the cost of the construction or rehabilitation project. The year of construction and reconstruction for bridges was readily known as these items are required as part of the National Bridge Inventory, but the costs of the construction projects were not electronically available. Plans were available for the roadway projects, but the project cost, project limits, and year of placement were not available in an electronic format. The ancillary infrastructure items had even less information available than the roadways and bridges.

Though there was a lack of electronically available construction costs information, Caltrans does have a complete set of condition assessments for the infrastructure assets in the state. With an understanding of the records that were readily available, Caltrans began to look at methods to determine the value of the infrastructure assets in the state.

POSSIBLE VALUATION APPROACHES

To satisfy the requirements of GASB 34, the value of infrastructure put into place or significantly modified after June 30, 1980, had to be reported. Bridge managers in Caltrans considered three possible methods for determining the asset value: depreciated historic cost, deflated replacement cost, and written-down replacement cost. Although the following paragraphs illustrate the various approaches as they apply to bridges, the basic principles apply to all infrastructure assets.

The actual historic cost method was quickly ruled out because of the labor required to review paper documentation on construction, improvement, and maintenance projects for the past 23 years. This approach also required the determination of the life span of all bridges. Bridges that outlive their assumed life would have zero value but would still be providing service to the agency. It is believed by Caltrans that bridge life can vary substantially based on the preservation level, environment, and operation practices of each bridge. For these reasons, the actual historic cost method of valuation was not chosen.

The deflated replacement cost method would work fairly well for Caltrans; however, Caltrans believes that this approach does not adequately capture the value because it does not account for the level of preservation that the bridge has experienced over its life.

For example, consider two bridges that were constructed in the same year and that had experienced similar deterioration over time but had dramatically different preservation actions performed on them. At the time of the asset valuation, both bridges would have the exact same asset valuation even though the bridge receiving adequate preservation would be in substantially better condition. Because Caltrans has an active infrastructure preservation program, the agency believes that this method would result in a valuation that is substantially below the current worth of California's bridge infrastructure.

The final method of valuation that was explored is a written-down replacement cost method of valuation. This method involves calculating the current replacement cost and reducing (writing down) the value based on the current condition of the asset. This approach has two distinct advantages for most transportation agencies: (1) the actual historic construction cost and year of construction are not required; and (2) the level of preservation performed on a structure will be recognized in the valuation process. This method is superior to other alternatives because it is the only method that takes the current condition of the asset into consideration. This is a critical point, because the preservation of our assets is at the core of our infrastructure management program. For agencies that are opting to use the modified approach for GASB 34 reporting, this valuation method is preferred, since it is consistent with the preservation focus of the modified reporting approach.

In Figure 1, all three approaches for asset valuation are presented for a year 2000 asset valuation of a bridge constructed in 1960 for $1,500,000 with an assumed 60-year life. Between construction in 1960 and the year 2000, a moderate level of preservation activity has been carried out on the structure. The year 2000 replacement cost was determined to be $2,450,000, and the condition was measured at 71% of new condition. The lower line on the graph represents the depreciated historical cost approach. This approach is simply a linear depreciation over an assumed life of 60 years. The depreciated replacement cost method shows a linear deflation of the replacement cost back to the construction year of 1960 that is followed by a linear depreciation over the assumed life of the structure. The vertical line shown in year 2000 represents the written-down method. Figure 1 illustrates that even a moderately preserved bridge can have an asset value that is significantly above the depreciated value approaches for the same year.

During the asset valuation process, it was recognized that the majority of the bridges in the state were constructed long before the 1980 cutoff date identified in GASB 34. To present the true investment in bridge infrastructure, the value of all bridges regardless of construction or improvement year was included in the statewide analysis.

CALTRANS APPROACH TO BRIDGE VALUATION

The bridge infrastructure valuation was determined using a written-down replacement cost approach. Using this approach, the current replacement value of each bridge was determined on the basis of average statewide unit replacement costs from contract bid documents. Table 1 presents the material and labor cost calculations by design type for concrete structures in the state.

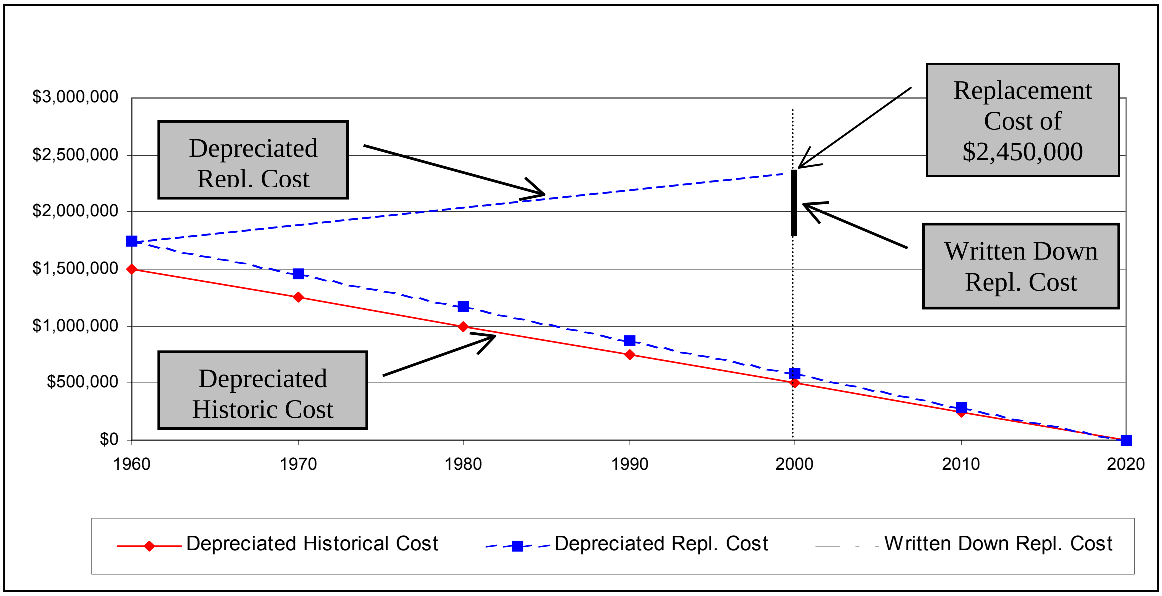

FIGURE 1 Asset valuation approaches. (Repl. = Replacement.)

TABLE 1 Sample Cost Calculations

Concrete	Deck Area (sq. m)	$/sq. m	Material and Labor Cost ($)
00 Other	23,517	1,500	35,275,500
01 Slab	927,392	1,500	1,406,088,000
02 Stinger/Girder	54,273	1,500	81,409,500
03 Girder and Floor Beam	271	1,500	406,500
04 Tee Beam	1,414,763	1,500	2,122,144,500
05 Box Beam/Girder/Multiple	6,359,307	1,500	9,538,960,500
06 Box Beam/Girder/Single	58,846	1,500	88,269,000
07 Frame	1,634	1,500	2,451,000
11 Arch-Deck	136,190	4,000	544,760,000
18 Tunnel	7,280	9,000	65,520,500
19 Culvert	473,854	1,000	473,854,000
20 Mixed Types	4,435	1,500	6,652,500
22 Channel Beam	8,166	1,500	12,249,000
Subtotal of Concrete	**9,479,928**		**143,78,040,000**

The material and labor costs for the replacement were then added to project delivery costs (design, contract administration, construction inspection, etc.) to determine the overall current replacement cost of each structure. The replacement cost for each structure was then written down based on the current condition of the bridge as measured by the bridge health index (BHI) of that structure.

The BHI is a 0 to100 numerical rating that utilizes element inspection data collected during a bridge inspection to determine the remaining asset value of a bridge or network of bridges. The BHI operates on the premise that each element on a bridge has an initial asset value when the element is in new condition. Over time an element may deteriorate to a lower condition state resulting in a reduction in the asset value of the element. When maintenance or rehabilitation actions are performed, the condition of the element will likely improve and the corresponding asset value of the element will be increased. At any point in time the current element condition state distribution can be ascertained by field inspection or predicted for future years using a deterioration model. Once the condition distribution is known, the current element value can be determined for all elements on the bridge. The BHI is the ratio of the current element value to the initial element value of all elements on the bridge. The availability of the BHI greatly facilitated the asset valuation process.

The example in Table 2 illustrates the approach used to determine the current condition-based assets value of the bridges in California. The valuation methodology shown in Table 2 was applied to each of the bridges in the Caltrans inventory and then summed to determine the current value of the bridge assets for the state as a whole.

SUMMARY

By using a condition-based written-down replacement cost valuation methodology, Caltrans was able to determine the asset value of 12,656 state-owned bridges with relative ease. The value of state-owned bridges was determined to be close to $50 billion in current value. The resulting valuation recognizes the current condition of each structure and does not require assumptions to be made regarding the life span of any bridge. A similar valuation approach is being developed to assess the value of roadway assets in the state.

The publicizing of the asset value information for bridges has caused managers to ponder the value of other infrastructure items. Highway pavement valuation has begun with a great deal of anticipation. Upon completion of the valuation process for the remaining infrastructure assets, Caltrans will have an objective measurement of value across asset classes. The availability of this new information is already beginning to provide comparisons between assets classes.
New measurement tools, such as the annual preservation expenditures-to-value ratios, can now be determined for various assets classes. By comparing the amount of preservation expenditures against the value of an asset, Caltrans has been able to develop baseline information that can be used to compare expenditures across asset classes. The value information has the potential to influence future allocations of preservation funds across competing asset classes.

TABLE 2 Written-Down Replacement Cost Example

Bridge A is in poor condition with a BHI of 72. The 27-year-old concrete girder bridge has a deck area of 100 m^2.

Bridge B is in good condition with a BHI of 92. The 48-year-old steel girder bridge has a deck area of 100 m^2.

Sample Calculations

Item	**Age**	**Deck Area**	**BHI**	**Unit Replacement Cost ($/sq. m)**	**Replacement Cost**	**Current Value (Replace $*BHI)**
Bridge A	27	100 m^2	72%	$1800	$180,000	$129,600
Bridge B	48	100 m^2	92%	$3000	$300,000	$276,000
Totals					$480,000	$405,600

Statewide Implementation of a Bridge Management System as Part of Integrated Asset Management in Kentucky

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The Kentucky Transportation Cabinet (KYTC) has implemented an integrated highway asset management system on a statewide basis. The commercial off-the-shelf software suite used in the implementation has primary systems for bridge management, pavement management, equipment and fleet management, and maintenance management. As part of this implementation, the TRDI bridge management system, Visual/BMS, was specifically configured, through its flexible framework, for the state of Kentucky and will thus be referred to as the KYBMS. This primary asset management subsystem assists KYTC decision makers in the process of managing a population of bridges within the context of an entire roadway network. The software is a Microsoft Windows-compatible program designed as a client–server application to ideally function within a coordinated suite of transportation asset management systems. KYBMS provides extensive capabilities in the key elements of an effective BMS. These key elements include inventory, environment, and traffic data; history of construction, maintenance, and improvements; condition inspection data; database management system; data analysis capability; bridge improvement program; bridge maintenance program; and report generation. A comprehensive and easily accessible database is the foundation for the system. On the basis of these data, the analysis results can be clearly understood by many groups both inside and outside of the agency. A flexible framework permits an agency to configure the entire set of analysis and simulation models. Thus, the system may be configured and calibrated to match the agency's engineering philosophy and operating procedures. An integrated maintenance module interfaces with the MMS and provides bridge-specific maintenance recommendations and history. The KYBMS allows a wide range of predefined and user-defined reporting and graphics capabilities.

Kentucky Transportation Cabinet (KYTC) has recently implemented a new asset management system to manage the life cycle of all assets (bridges, pavements, right of way, drainage, traffic appurtenances, etc.) on the state highway network. KYTC wanted one of the components to be a modern BMS that would

- Be based upon current inspection practices,

- Identify bridge needs and costs,
- Prioritize these needs based on funding sources,
- Be a tool for prioritizing Federal Highway Bridge Replacement and Rehabilitation Program bridges, and
- Provide for multiyear and multifinding analyses.

An important component of the system is TRDI's Visual/BMS. This powerful, state-of-the-art system was specifically configured, through its flexible framework, for the state of Kentucky and is therefore referred to as the KYBMS. The system was implemented by TRDI under subcontract to American Management Systems. The project included implementation of TRDI's full suite of transportation asset management systems, which also includes subsystems for maintenance management (Visual/MMS), pavement management (Visual/PMS), equipment and fleet management (Visual/EMS), and live interfaces to the statewide financial management package and other key systems. This paper describes the KYBMS in terms of an overview of its implementation within the asset management framework. It then describes the functionality of this new generation of highly informative, highly flexible analytical models packaged in an easy-to-use graphical environment and takes the practice of bridge management to a new plateau in technology.

The KYBMS implementation uses both U.S. federally mandated data required by the National Bridge Inspection Standards (NBIS) and a number of additional fields for Kentucky-specific inventory and bridge element condition data. These additional fields are flexible and can be adapted to additional or changing data needs in the future or to accommodate custom fields for other state departments of transportation (DOTs) that may be interested in using the flexible framework of the KYBMS software. The Kentucky implementation contains more than 160 data elements per bridge. The additional Kentucky-specific data allow for expanded analysis capabilities to determine maintenance, repair, rehabilitation, and replacement needs of the population of bridges; to estimate deterioration; and to use such estimated models in the system's predictive analyses.

The KYBMS software provides a strong degree of user-controlled configuration. Analytical parameters, such as bridge improvement criteria, maintenance and rehabilitation treatments, treatment costs, decision processes, and bridge condition formulas, are all controlled by the user and thus are implemented in a Kentucky-specific configuration. All of the U.S.-required federal SI&A (structural inventory and appraisal) data are included in the system, including all of the sufficiency rating calculations required to produce the annual federal reporting requirements. In addition to the predefined sufficiency ratings, any form of user-defined bridge condition or serviceability indexes can be set up within the system and used as decision criteria in the comprehensive decision analysis and support components of the software.

PURPOSE OF THE SYSTEM

The purpose of the implemented system is to provide KYTC with a comprehensive, unified approach to managing its bridge population in concert with all other transportation-related assets. It is intended to serve all of the bridge management requirements including all federal requirements, desired bridge analyses, recommendations for bridge program maintenance and improvements, and extensive reporting capabilities. The system is designed for integration with the MMS and other asset management components to allow for single entry of data and immediate sharing across all such related modules. In this regard, the KYBMS addresses bridge assets within the framework of an overall asset management system to a more complete extent than any other BMS implementation currently known to the authors.

The system is linked to network referencing information, traffic data, maintenance history and plans, and other useful asset management information to allow for comprehensive bridge management functions. On the basis of the current condition information for the population of bridges and estimated budgets for future years, the system recommends expected improvement needs and anticipated future conditions for the bridge population. It also stores and maintains a bridge repair history and uses it to schedule preventive maintenance and repair work as well as to provide background information for the bridge manager on each individual bridge in the population. In effect, the system provides for complete bridge management functionality with a great deal of power, control, and flexibility within the framework of an integrated asset management system for KYTC.

EXTENDED BENEFITS OF KYBMS

On the basis of the clear benefits that the system provides to KYTC, other agencies can also benefit from the implementation of the KYBMS. The flexible framework of the software allows it to be easily configured and customized, without additional programming, for almost any state DOT or other similar agency that owns and manages a population of bridges. The system already has a built-in set of implementation guidelines in that it is already configured for a set of data variables, functions, and setup data to accommodate rapid implementation for almost any typical bridge management agency. The software is designed to accept a client's own values for system variables, which can be easily added to the software and database—both as data and as analytical functions, which are then automatically available for use throughout the software. Therefore, any such agency can benefit from the new, advanced analytical and reporting capabilities of the system and be able to start with a default set of operating parameters. The bridge experts within a new agency can easily review the transparent models provided within the system and make adjustments based on their expert knowledge of their own bridge population and its performance. Over time, these parameters will continually be updated, based on actual data, to improve system model calibrations and predictive abilities.

SYSTEM DATA REQUIREMENTS

The KYBMS contains a wealth of information about individual bridges and the bridge population as a whole in the state of Kentucky. It also contains additional setup information that allows the interrelated framework of analytical models to function. This setup information is a source of the power and flexibility of the system that allows it to be calibrated to any agency's characteristics and to be continually improved with use. In addition, information related to maintenance needs and desired schedules allows for establishing maintenance requirements for each bridge, whose implementation is then tracked in the state's new MMS.

For every bridge in the state, a complete set of inventory and condition data is accommodated, including all of the federally specified NBIS and any other type of information that the agency wishes to store. For example, KYTC has added more than 50 variables to describe additional attributes and provide additional information about each of its bridges. Such information is then available to be combined with other variables or to be used independently as a deterioration or decision variable or in a formula to compute the combined decision variables for use within the analytical models.

For the system to determine expected improvement needs and future conditions for the bridge population, it requires a certain amount of setup information. This includes formulation of bridge condition and performance indicators, deterioration rates and models of these indicators, priority criteria to improve bridges, maintenance and rehabilitation treatments and the unit costs, condition improvement amounts due to each treatment, the formulation of combined inventory and condition variables and performance indicators into a decision process, and various other rules and criteria that drive the analytical models of the system. This allows the system to be immediately used to make reasonable bridge-related decisions. The initial set of this information has been developed interactively with several expert bridge engineers, including some from KYTC and some familiar with bridges in the state of Texas. The accessibility of this information to the user allows continual improvement of the predictive abilities and calibrates the system to the actual performance and environmental conditions experienced in the bridge agency.

The system also has the ability to help develop a bridge preventive maintenance program. For this function to work efficiently, the system needs, for each bridge and maintenance type, the latest date when that type of maintenance was last performed on each bridge. In addition, a desired or anticipated maintenance cycle frequency for the specific activity is specified. With this information, the system recommends a preventive maintenance program whose implementation is then tracked in the MMS.

REPORTING FEATURES OF KYBMS

One very powerful component of the system is the integrated flexible reporting tool. This tool allows users to create tabular or graphic reports about any information contained in the database. Reports can be developed and saved by a user or made available to the entire set of users across the network. In addition, reports may be developed for a specific snapshot of data or be set to refresh every time the report is run with the current set of data at that time. Reports developed for a specific geographical area, bridge type, functional class, or other specific criteria can be generalized so that the same report can be run for any other value of the specified criteria.

APPLICATION AND DATABASE ENVIRONMENT

The KYBMS is a Windows-based client–server application. The application component resides on the user's Windows-based PC connected to the statewide network. The database is a centralized Oracle database, which drives all of the modules of the KYTC asset management system. The centralized database stores all of the bridge data discussed in previous sections. In addition, it contains all data for the MMS, PMS, EMS, and other inventory modules contained in the system. The unified database allows for the integration of the systems as discussed above and provides for the single point entry and immediate sharing of data necessary for a truly integrated system. Users from around the state can access the central database on the state network and gain access to the data, analyses, and reports of the KYBMS

DETAILED DESCRIPTION OF SYSTEM

This section provides a detailed description of the capabilities of the KYBMS. Four areas are addressed—information, analysis, maintenance, and reporting.

KYBMS Information

The information portion of the system contains structure, inventory, and appraisal information for each bridge. KYBMS is designed to handle a large number of bridges; consequently, the first informational feature is its data selection capability. KYBMS allows Kentucky to look at its bridges by county, by district, by type of bridge design, or by type of material. The KYBMS is designed so that different agencies can have their own sets of data selection criteria.

Structure, inventory, and appraisal data that indicate the current condition and status of each bridge constitute the major portion of the informational section of KYBMS. For Kentucky this includes all of the 116 U.S. federally required data fields, as well as more than 50 Kentucky-specific variables. Those variables are identified and categorized in Table 1. This information is spread across eight tabbed pages in a window as shown in Figure 1 (three tabbed pages shown). In this window, navigation from bridge to bridge is accomplished in the leftmost subwindow by highlighting the appropriate bridge; then the bridge-specific information on the right is displayed and can be edited. Data quality is assured by the use of drop-down data-selection controls, which limit value selection to legal values. The KYBMS is designed so that every agency can use its own set of informational data.

Data are also retained historically. In Kentucky's case, these are mostly condition data, including more detailed condition rating sets as applied to several bridge elements. Figure 2 shows Kentucky's detailed condition rating data. On average, each USA federally required condition rating has 7 to 10 Kentucky-specific condition element ratings associated with it. For this window, data navigation is even more important because, not only is there data for every bridge, but also each bridge can have many years of data. The top part of Figure 2 shows all the data collected on one bridge for 1 year's inspection. The normal "page up" and "page down" navigational tools are available for moving from year to year and bridge to bridge; however, when there are 10,000 bridges, each with 10 surveys, there needs to be an alternative, more efficient navigational tool. The bottom part of Figure 2 is that tool. In the KYBMS the full-page form view (top) and the table view (bottom) are coordinated so that, when you move from bridge to bridge in one view, you're also moving in the other view. In the table view, you can sort by any column or set of columns you want by simply clicking on the column heading.

Analysis begins with the calculation of current improvement needs for every bridge in the population accompanied by the current condition calculation of every agency-determined condition formula or index. Kentucky uses more than 50 condition variables of whose current values 20% are calculated and 80% come directly from data. The KYBMS allows calculation of current improvement needs and condition for all bridges very quickly. For example, the recalculation of 13,000 bridges takes less than 1 min.

KYBMS Analysis

The Bridge Master File is a good way to see current needs and condition, and it contains the data upon which all future years' analytical simulations are based. From the user's perspective, all such analyses are conducted in the scenario analysis window, whose first tab is demonstrated in the top part of Figure 3. The scenario analysis window in KYBMS contains as many simulated projections of the future as a user wishes. One particular scenario analysis is defined by the following:

TABLE 1 KYBMS Data

Inventory Route–Item 5	Super Type	Classification
Record Type (5A)	Sub Type	Functional Class of Inv. Route #1 (26)
Route Signing Prefix (5B)	Asphalt Thickness In	Hwy. System of the Inv.y Route (104)
Designated Level of Service (5C)	Deck Type (I) (107)	Hwy Sys 2
Route Number (5D)	Age and Service	Parallel Structure Designation (101)
Directional Suffice (5E)	Year Built (27)	Parall Str 2
SI&A Data	Year Reconstructed (106)	Direction of Traffic (102)
Features Intersected	Type of Service on Bridge (42A)	Traf Dir 2
Facility Carried	Type of Service Under (42B)	Toll (20)
Structure Number (8)	Lanes on the Structure (28A)	Maintenance Responsibility (21)
Location Description No. 1	Lanes under the Structure (28B)	State/County (22)
Highway Network (12)	Bypass Length #1 (19)	Historical Significance (37)
Lrs Route	Average Daily Traffic #1 (29)	Federal Lands (105):
Lrs Subroute	Average Daily Truck Traffic (109)	Temporary Structure Designation (103)
Border Bridge	Year of Average Daily Traffic #1 (30)	Temp Str 2
Border Bridge–Neighboring State Code (98A)	Truck Highway Code	Condition
Border Bridge Struct No P (99F)	Geometric Data	Deck Condition (I) (58)
Border Bridge Struct No 1 (99b)	Length of Maximum Span (48)	Superstructure Condition (I) (59)
Border Bridge Struct No 2 (99d)	Structure Length (49)	Superstructure Condition (I) (60)
Border Bridge Struct No Route (99c)	Left Curb or Sidewalk Width (50A)	Channel Condition (I) (61)
Border Bridge Struct No County (99a)	Right Curb or Sidewalk Width (50B)	Culvert Condition (I) (62)
Border Bridge Struct No Bridge (99e)	Bridge Roadway Width (51)	State Underwater
Other Data	Deck Width (52)	Load Rating and Posting
Facility Name	Approach Roadway Width (32)	Design Load (31)
Rd Sys No	Bridge Median (33)	Structure Posted (41)
Bridge County	Skew Angle (34)	Method Used to Determine Operating Rating (63)
Road Name	Structure Flared (35)	Inventory Rating Method (65)
School Route	Minimum Vertical Clearance #1 (10)	Bridge Posting (I) (70)
Bridge Bridge	Total Horizontal Clearance #1 (47)	Sufficiency Rating
Earthquake	Minimum Vertical Clearance Over Bridge Roadway (53)	Operating Rating (64)
Structure Type and Material	Minimum Vertical Underclearance (54B)	Inventory Rating (66)
Kind of Material and/or Design (43A)	Minimum Lateral Underclearance on Right (55B)	Type Load Trucks 1
Type of Design and/or Construction (43b)	Min. Lateral Underclearance on Left (56)	Type Load Trucks 2
Number of Spans in Main Unit (45)	Vrt Clr Eb Nb Ft	Type Load Trucks 3
Kind of Approach – Material/Design (44A)	Vrt Clr Eb Nb In	Type Load Trucks 4
Type of Approach – Design and/or Construction (44B)	Vrt Clr Wb Sb Ft	Appraisal
Number of Approach Spans (46)	Vrt Clr Wb Sb In	Structural Evaluation (67)
Bridge Description No 1	Navigation Data	Deck Geom. Apprais. (68)
Bridge Description No 2	Navigation Control (38)	Underclearances, Vert. & Horiz. (69)
Deck Type	Navigation Vertical Clearance (39)	Waterway Adequacy (I) (71)
Wearing Surface (I) (108A)	Navigation Horizontal Clearance (40)	Appr. Roadway Alignment (I) (72)
Wear Srfc Mmbrn (I) (108B)	Pier or Abutment Protection (111)	Priority Factors Lcp
Deck Protection (I) (108C)	Min Vrt Nav Clr Lift Brd	Priority Factors Wp

(continued on next page)

TABLE 1 (*continued*) **KYBMS Data**

Priority Factors Vp	Bypass Length #2 (19)	County Num
Priority Factors Lp	Bypass Length #3 (19)	Crit Feat Insp Details Months
Priority Factors Dp	Bypass Length #4 (19)	Crit Feat Insp Special (Y/N)
Priority Factors Pf	Total Horizontal Clearance #2 (47)	Crit Feat Insp Special Months
Bridge Railings (I) (36A)	Total Horizontal Clearance #3 (47)	Crit Feat Insp Underwater (Y/N)
Transitions (I) (36B)	Total Horizontal Clearance #4 (47)	Crit Feat Insp Undrwtr Months
Approach Guardrail (I) (36C)	Length 1	Critical Facility Ind
Approach Guardrail Ends (I) (36D)	Speed 1	Des Load Imp
Scour Critical Bridges (I) (113)	Length 2	District Item 2
Analysis Location	Length 2	Hist Proj No
Date Postedmm	Speed 2	Inv Rating Type
Improvements	Milepoint #2 (11)	Inv Route
Type of Work Proposed (75A)	Milepoint #3 (11)	Inv Route Des Lev Serv 2
Type of Work Done (75B)	Milepoint #4 (11)	Inv Route Des Lev Serv 3
Length of Structure Improvement (76)	Functional Class of Inv. Route #2 (26)	Inv Route Des Lev Serv 4
Bridge Improvement Cost (94)	Functional Class of Inv. Route #3 (26)	Inv Route Dir Suf 2
Roadway Improvement Cost (95)	Functional Class of Inv. Route #4 (26)	Inv Route Dir Suf 3
Total Project Cost (96)	Average Daily Traffic #2 (29)	Inv Route Dir Suf 4
Year of Improvement Cost Est. (97)	Year of Average Daily Traffic #2 (30)	Inv Route Rec Type 2
Average Daily Traffic (115)	Average Daily Traffic #3 (29)	Inv Route Rec Type 3
Future Average Daily Traffic (114)	Year of Average Daily Traffic #3 (30)	Inv Route Rec Type 4
Qcrit	Average Daily Traffic #4 (29)	Inv Route Route Num 2
Total Cost Impr	Year of Average Daily Traffic #4 (30)	Inv Route Route Num 3
Pe Cost Imp	Miscellaneous	Inv Route Route Num 4
Dem Cost Imp	No Lanes Imp	Inv Route Route Pref 2
Sub Cost Imp	Oper Rating Type	Inv Route Route Pref 3
Super Cost Imp	Paint Area	Inv Route Route Pref 4
Draw Number 1	Paint Color	Latitude Degrees
Draw Number 2	Paint Condition (I) (59A)	Latitude Min
Draw Number 3	Paint Date Latest	Latitude Sec
Draw Number 4	Paint Lead	Longitude Degrees
Inspections	Paint Type	Longitude Min
Critical Feature Insp Dates B	Paint Weight	Longitude Sec
Critical Feature Insp Dates A	Pier Protect Req	Min Lat Under Clr Ref
Designated Inspection Freq. (I) (91)	Rd Width Imp	Min Vert Under Clr Ref
Inspection Date (I) (90)	Remlife	Parallel Bridge
Critical Feature Insp Dates C	Repl Cost Block	ADT Count Imp
Indepth Insp Type	Scour Watch	ADT Imp
Indepth Cond Insp Type	Station Number	Approval Code67
Indepth Cond Inspmm	Str Deficient Obs Code	Approval Code68
Insp Note	Strahnet Highway	Approval Code69
Maint Hist 1 Type	Type Srv Imp	Approval Code70
Maint Hist 1	Upn County	Approval Code71
Maint Hist 2 Type	Upn Mile Point	Approval Code72
Maint Hist 2	Upn Prefix	Coal Haul
KYTC Specific Data	Upn Route	Coal Haul Check
Location Description No. 2	Yr Imp	Contract Award Date
Minimum Vertical Clearance #2 (10)	Yr Need Imp	
Minimum Vertical Clearance #3 (10)	Yr Rehab	
Minimum Vertical Clearance #4 (10)	Date Update:	

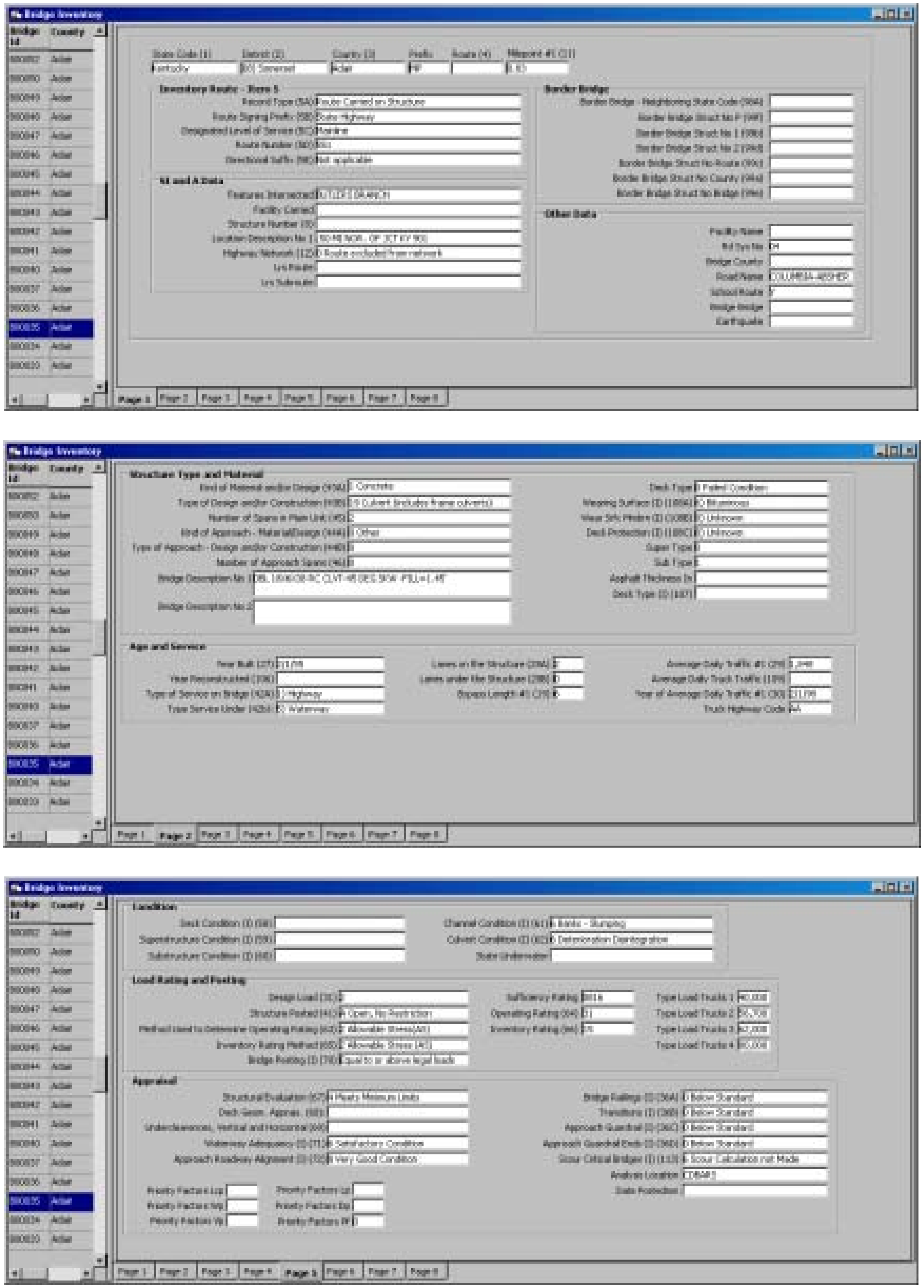


FIGURE 1 Three examples of eight total pages of bridge inventory and appraisal data.

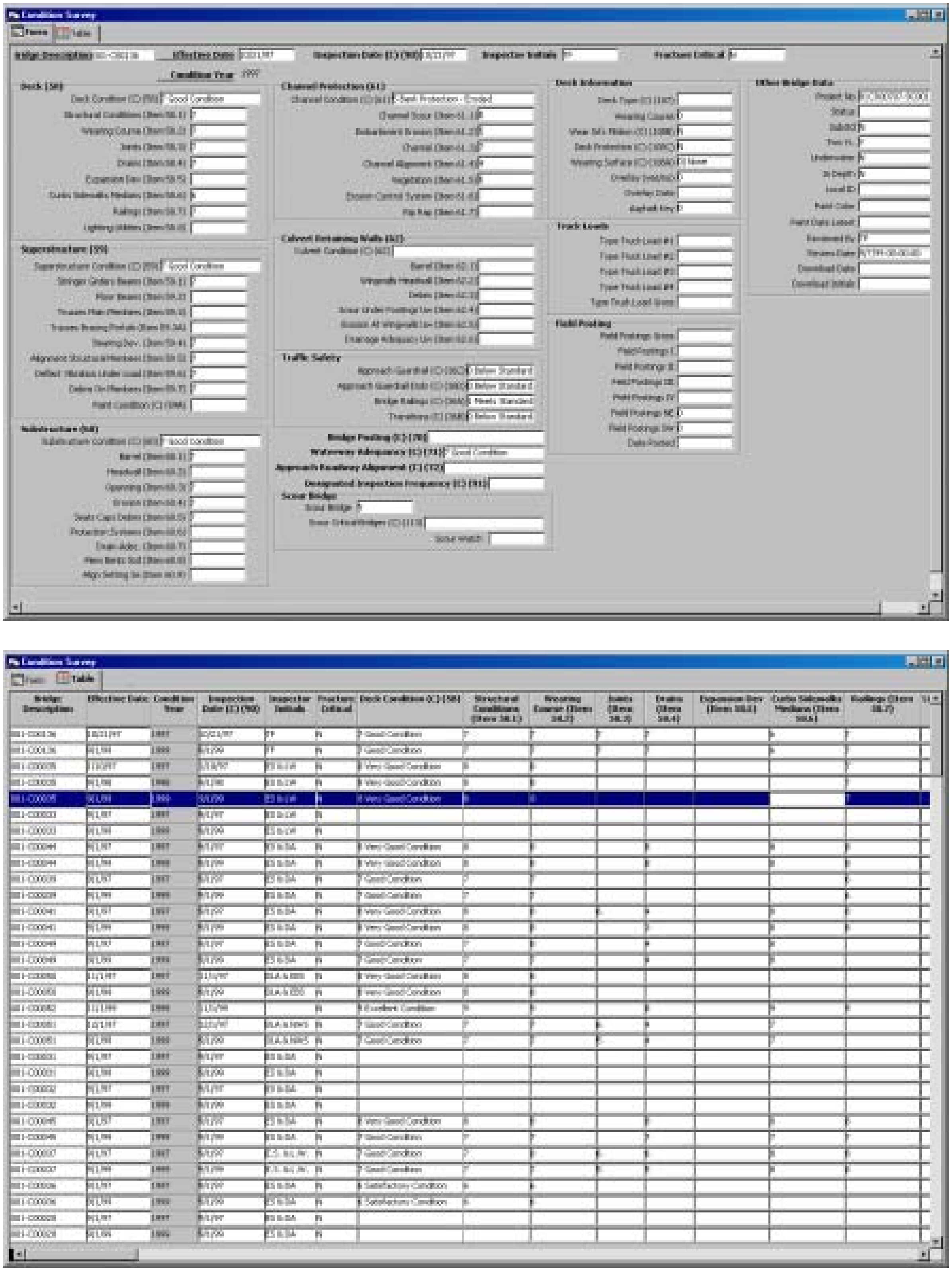


FIGURE 2 Condition survey form and table views.

1. Analysis period (the number of years into the future to be simulated).
2. Analysis method, which can be
   - By priority,
   - By condition, or
   - For all needs.
3. Scope of the analysis (which bridges to include in the analysis).
4. Anticipated future budgets in each year (when the method is by “priority”).
5. Condition goal in each future year (when the priority method is “by condition”).
6. The analysis can be made to include bridges, their treatments and costs as found in the Master Work Program. In that way the analysis engine first reduces future budgets by those bridges with known plans for construction (i.e., they are in the Master Work Program). Those bridges are treated and improved and then, at that time, the remaining budget is allocated and distributed across the rest of the bridge network.
7. Different scenario analyses can use different priority formulae.

Once the analysis scenario inputs as just described are completed, the user invokes the “run scenario” command and the simulation begins. The result of simulation is as follows:

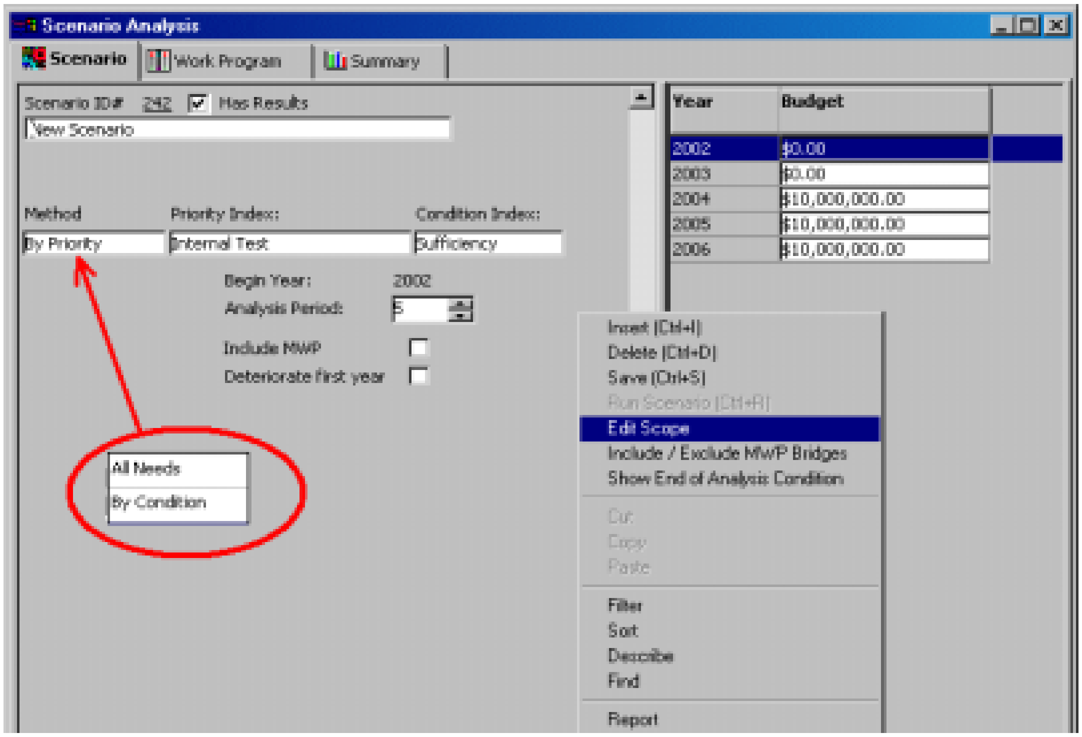
1. A future year’s work plan showing, in each year, the bridges to improve, what their treatments are, and an estimated cost of those treatments.
2. The resultant bridge population condition is a consequence of these bridge improvements, as well as the normal deterioration of unimproved bridges calculated and demonstrated in each future year’s analyses. A portion of the scenario’s output work program is shown in the bottom portion of Figure 3. The resultant condition distribution in all future years is shown in the top part of Figure 4. In this particular scenario no bridges were improved in the first 2 years, which resulted in deterioration of the bridge population. Then, for the next 3 years, sufficient funds were allocated for the bridge population thus stabilizing population condition as demonstrated in the graph. To run simulations that take into account bridge improvement projects that have already been determined, it is necessary for the KYBMS to know those projects including their timing and costs. The bottom portion of Figure 4 shows the KYBMS Master Work Program window. It is here that already-determined bridge projects are entered so that they can (1) stand by themselves and (2) be used as part of future years’ simulation analyses.

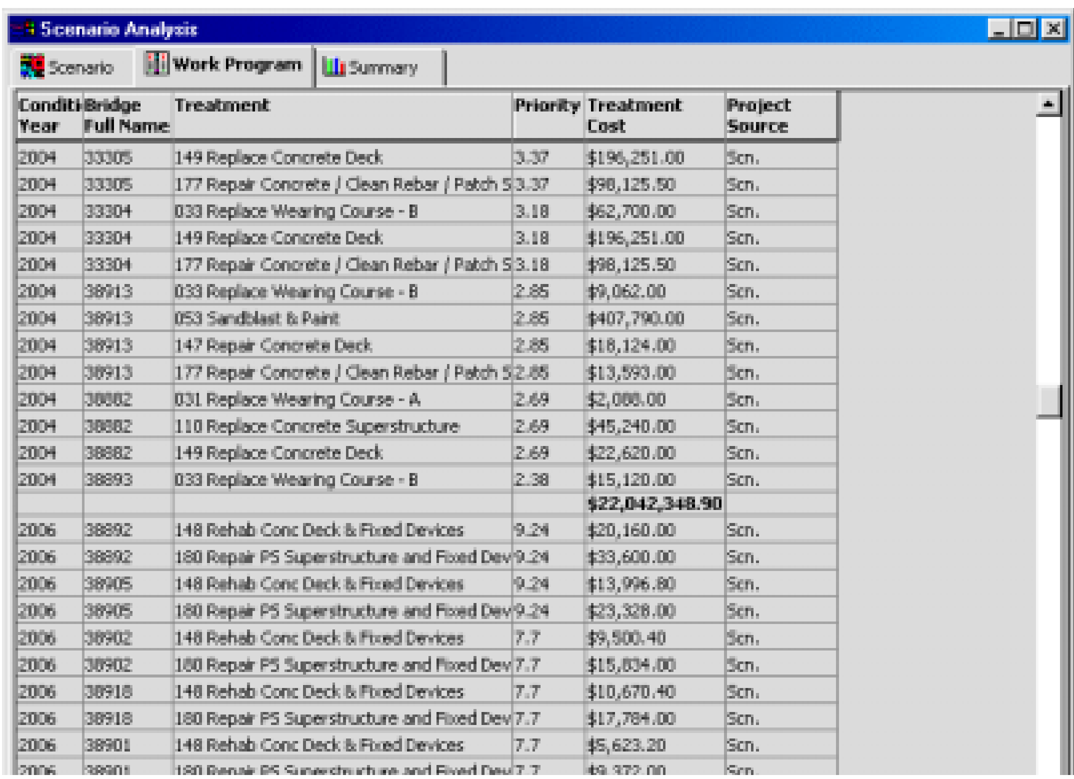
In future years’ analyses there are four steps that are supported by setup data and parameters. On a simulated annual basis, the following occurs:

1. Bridges are deteriorated. Consequently, the rate of deterioration on various types of bridges and various components of the bridge must be estimated and applied.

Based on the deteriorated condition, the need for improvement is assessed, treatments are recommended, and the costs are totaled. Consequently, there is a need for decision criteria by which these treatments and their costs are determined.

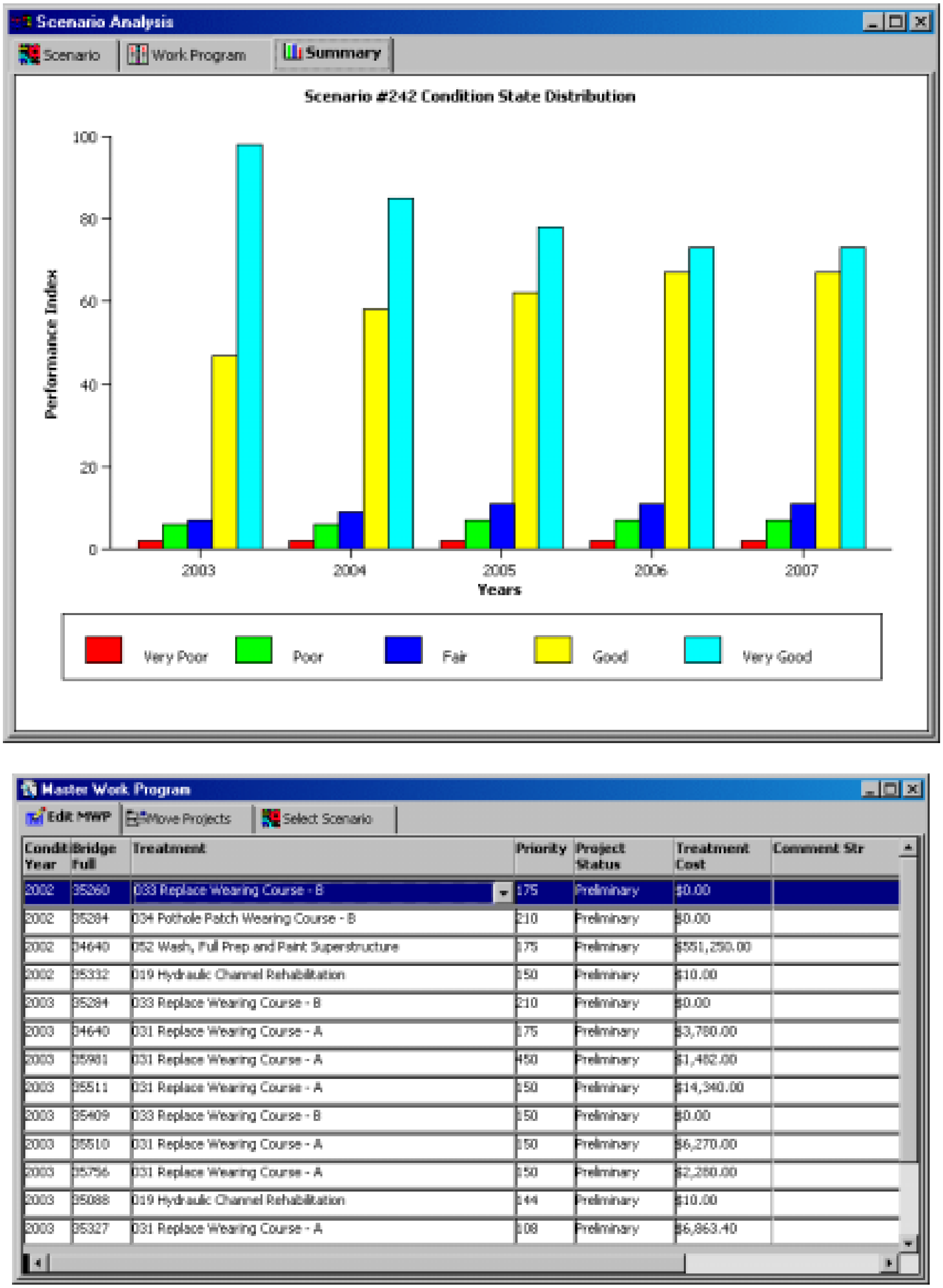
2. Also, based upon the deteriorated condition, bridges and their improvement needs are prioritized, and those highly prioritized bridge improvements whose cost can be accommodated by estimated future budgets are selected for improvement.
3. Finally, the condition of those bridges selected for treatment is improved commensurate to the kind of improvement work applied. Consequently, there is a need to





Conditi Year	Bridge Full Name	Treatment	Priority	Treatment Cost	Project Source
2004	33305	149 Replace Concrete Deck	3.37	$196,251.00	Scn.
2004	33305	177 Repair Concrete / Clean Rebar / Patch S	3.37	$98,125.50	Scn.
2004	33304	033 Replace Wearing Course - B	3.18	$62,700.00	Scn.
2004	33304	149 Replace Concrete Deck	3.18	$196,251.00	Scn.
2004	33304	177 Repair Concrete / Clean Rebar / Patch S	3.18	$98,125.50	Scn.
2004	38913	033 Replace Wearing Course - B	2.85	$9,062.00	Scn.
2004	38913	053 Sandblast & Paint	2.85	$407,790.00	Scn.
2004	38913	147 Repair Concrete Deck	2.85	$18,124.00	Scn.
2004	38913	177 Repair Concrete / Clean Rebar / Patch S	2.85	$13,593.00	Scn.
2004	38882	031 Replace Wearing Course - A	2.69	$2,088.00	Scn.
2004	38882	110 Replace Concrete Superstructure	2.69	$45,240.00	Scn.
2004	38882	149 Replace Concrete Deck	2.69	$22,620.00	Scn.
2004	38893	033 Replace Wearing Course - B	2.38	$15,120.00	Scn.
				$22,042,348.90	
2006	38892	148 Rehab Conc Deck & Fixed Devices	9.24	$20,160.00	Scn.
2006	38892	180 Repair PS Superstructure and Fixed Dev	9.24	$33,600.00	Scn.
2006	38905	148 Rehab Conc Deck & Fixed Devices	9.24	$13,996.80	Scn.
2006	38905	180 Repair PS Superstructure and Fixed Dev	9.24	$23,328.00	Scn.
2006	38902	148 Rehab Conc Deck & Fixed Devices	7.7	$9,500.40	Scn.
2006	38902	180 Repair PS Superstructure and Fixed Dev	7.7	$15,834.00	Scn.
2006	38918	148 Rehab Conc Deck & Fixed Devices	7.7	$10,670.40	Scn.
2006	38918	180 Repair PS Superstructure and Fixed Dev	7.7	$17,784.00	Scn.
2006	38901	148 Rehab Conc Deck & Fixed Devices	7.7	$5,623.20	Scn.
2006	38901	180 Repair PS Superstructure and Fixed Dev	7.7	$9,372.00	Scn.

FIGURE 3 Scenario analysis definition screen (top) and resulting work program (bottom).



Condit Year	Bridge Full	Treatment	Priority	Project Status	Treatment Cost	Comment Str
2002	35260	033 Replace Wearing Course - B	175	Preliminary	$0.00	
2002	35284	034 Pothole Patch Wearing Course - B	210	Preliminary	$0.00	
2002	04640	052 Wash, Full Prep and Paint Superstructure	175	Preliminary	$551,250.00	
2002	35332	019 Hydraulic Channel Rehabilitation	150	Preliminary	$10.00	
2003	35284	033 Replace Wearing Course - B	210	Preliminary	$0.00	
2003	04640	031 Replace Wearing Course - A	175	Preliminary	$3,780.00	
2003	35981	031 Replace Wearing Course - A	450	Preliminary	$1,482.00	
2003	35511	031 Replace Wearing Course - A	150	Preliminary	$14,340.00	
2003	35409	033 Replace Wearing Course - B	150	Preliminary	$0.00	
2003	35510	031 Replace Wearing Course - A	150	Preliminary	$6,270.00	
2003	35756	031 Replace Wearing Course - A	150	Preliminary	$2,280.00	
2003	35088	019 Hydraulic Channel Rehabilitation	144	Preliminary	$10.00	
2003	35327	031 Replace Wearing Course - A	108	Preliminary	$6,863.40	

FIGURE 4 Scenario analysis results summary (top) and developed master work program (bottom).

estimate the condition improvement effect of every bridge treatment across each condition variable.

In the KYBMS, these four sets of parameters are provided as follows.

Deterioration Analysis

In the KYBMS, all numeric condition variables can be deteriorated. For Kentucky, there are more than 50 such variables. If you want to use the condition variable in a future year simulation for improvement decision criteria, priority, or just to see an estimate of its future condition, a user-defined deterioration model is required for that condition variable. As shown in the top part of Figure 5, there are four types of models allowed—linear, inverse linear, multiplicative exponential, and additive exponential. Each of these models requires just two inputs: (1) the amount of condition change over (2) a particular number of years. The user entering the condition when the bridge is new and the condition at any time later in the bridge's life obtains the deterioration rate depicted by the model in the KYBMS.

It is interesting to note that normally a deteriorated condition is signified by smaller condition values. However, at times, depending upon the agency's choice of condition variables, larger condition values may mean deteriorated condition. The KYBMS accommodates both of these situations by including a direction parameter in the deterioration window. When deterioration direction is "down," smaller condition numbers mean deteriorated condition. When deterioration condition direction is "up," larger condition values mean deteriorated condition.

Decision Analysis

The decision criteria assessing bridge improvement needs are handled in KYBMS through user-defined decision trees. For improvement assessment purposes, each bridge in KYBMS is identified by type and then divided into its component pieces. There are four types of bridges to which different decision criteria can be applied—nonmovable bridges, movable bridges, culverts, and tunnels. To each of these bridge types, decision subtrees are assigned, each corresponding to one or more bridge components. This is shown in the bottom of Figure 5 where in the leftmost subwindow the four bridge types are identified as well as those subtrees assigned to each type. Then in the rightmost subwindow are listed all of the subtrees in KYBMS, each of which may or may not be applied to one or more bridge types. For example, the assessment of channel through a decision subtree is applicable to both movable bridges and nonmovable bridges.

Subtrees are assigned to bridge types using this criterion: "if a subtree can apply to one or more bridges of this type, it should be included as part of the decision criteria." For example, since there are some nonmovable bridges that span water, the channel decision criteria should be included. Note that in the case in which there is no water under a nonmovable bridge, channel-related condition variables will not be recorded and, consequently, the decision subtree will not be employed.

Subtrees can be built, edited, and reviewed by the user. The top part of Figure 6 shows a sample subtree to assess improvement response to bridge substructure. Each node is represented by a box containing three pieces of information: (1) the decision variable, (2) the range of the decision variable, and (3) optionally, bridge treatment that this node presumes. To illustrate the concept of how the system can be configured to accurately recommend suitable repairs, one example configuration of a subtree is described. Reading from left to right this decision subtree

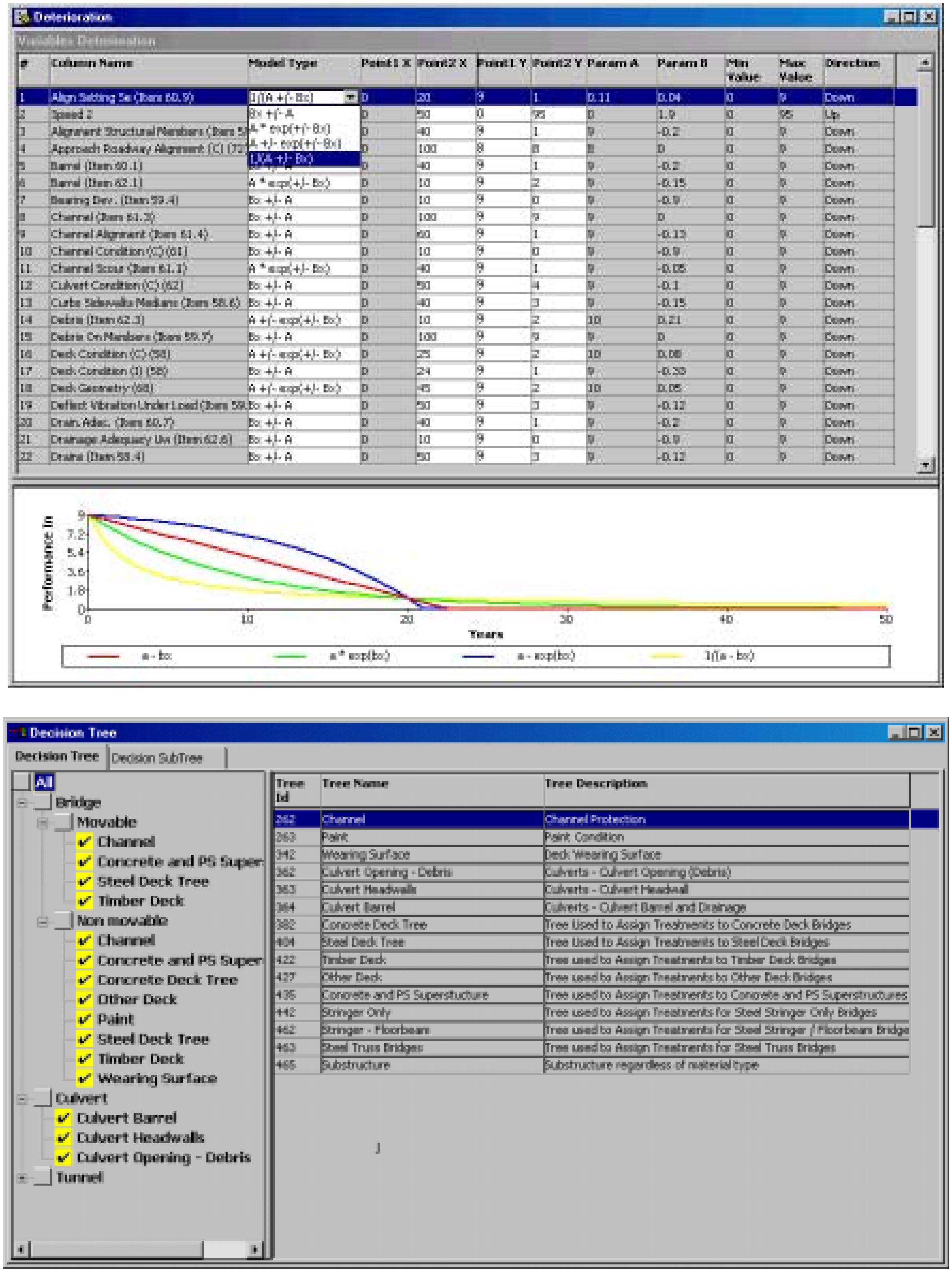


FIGURE 5 Deterioration model setup and review (top) and decision tree setup (bottom).

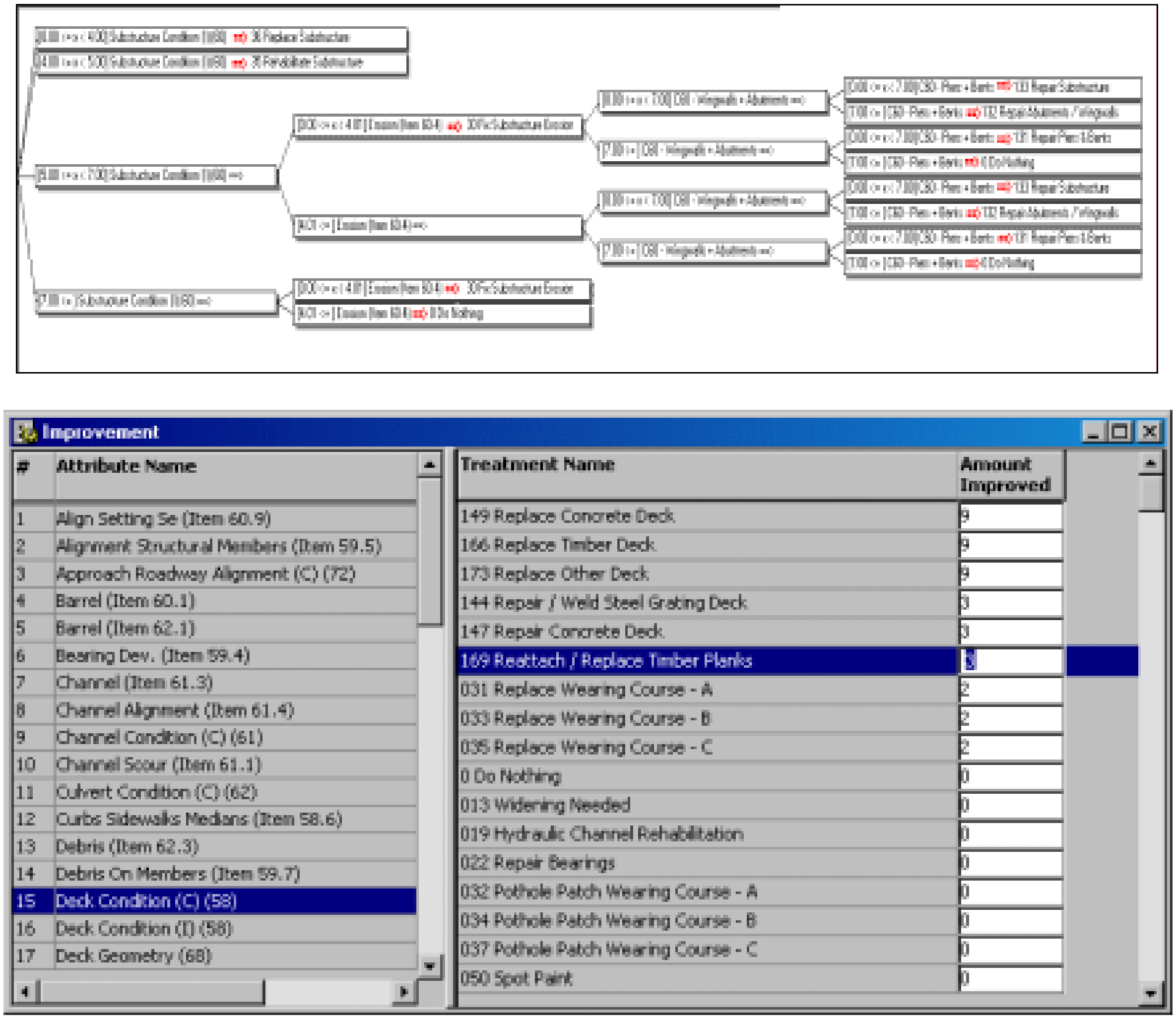


FIGURE 6 Decision tree detail view (top) and bridge improvement models (bottom).

is based upon four variables: a general substructure condition, corrosion problems, wingwall and abutment problems, and pier and bent problems. For these four variables, 9 represents very good condition, while 0 represents poor condition, to the point of bridge closure. Reading from left to right, you can see at the top left of the subtree that poor substructure condition leads immediately to rehabilitation or replacement of the substructure. For bridges with adequate substructure condition, other criteria will determine the need for improvement. Moving to the right in the subtree, you can see that erosion is the next effect. Poor erosion leads to erosion repair. Finally, the last two levels of the decision subtree deal with wingwalls and piers depending on each of their conditions. The specific elements are repaired or, if they are both in poor condition, then the entire substructure is repaired.

In this example, a bridge has substructure condition between 5 and 7, an erosion problem with condition less than 4, pretty good wingwalls with condition greater than 7, and poor piers and bents, which leads to the following conclusions:

- Since the substructure condition is fair, there is no treatment specifically assigned due to substructure condition.

- However, since erosion is a problem, the first treatment to repair that erosion problem is identified at the second level of the decision tree.
- Next, a wingwall condition greater than 7 implies that either the wingwalls are in good condition or that there are none; consequently, there is no rating for them. In any event, since the wingwalls are good or absent, there is no repair assigned to them.
- Finally, at the fourth level, if the piers and bents are in poor condition, they are repaired. If they are in good condition or don't exist, then nothing will be done to them.

That result for the bridge with these four conditions is that its erosion problem will be fixed as well as its piers and bents.

Improvement Analysis

In simulation of future years, once a bridge is improved then its corresponding bridge condition parameters must also be improved. The types of treatments that are applied to the bridge determine the amount that each condition parameter is improved. The bottom part of Figure 6 shows how this is accommodated in the KYBMS. In the leftmost subwindow is a list of every condition variable. By highlighting the condition variable, the amount of improvement due to every treatment is shown in the right subwindow.

Prioritization Analysis

Finally, the KYBMS user, as shown in Figure 7, controls the priority with which bridges are improved. The priority formula is the multiplication of all pertinent priority coefficients. Higher priority coefficients imply higher priority. Also, since the priority formula is multiplicative, a priority coefficient of one essentially is neutral (i.e., it doesn't change the priority at all).

The KYBMS allows a great deal of flexibility in priority definition. Consequently, the KYBMS concept of priority is three tiered; namely, priority coefficients are at the bottom, priority variables are in the middle, and priority indexes are at the top:

- Specifically, in any given future year simulation, one priority index is used; however, many priority indexes can be defined and used differently. For example, a general priority index may be influenced by deck condition; however, a separate priority index would be defined for analyses that are aimed at handling water flow problems under bridges because, in this case, deck condition has nothing to do with resolving or prioritizing that type of problem. This is why KYBMS accommodates multiple priority indexes. As you can see in Figure 7, at the top left four priority indexes are defined.
- The second tier comprises priority variables. Each priority index is composed of one to several priority variables. In Figure 7, this is shown at the top right of each window, which lists five priority variables assigned to the currently highlighted priority index. Different priority indexes will have different priority variables assigned to them.
- Finally, the third tier comprises priority coefficients, which actually are what constitute the priority formula. Each priority variable (second tier) is assigned a set of priority coefficients. This can occur in two ways:

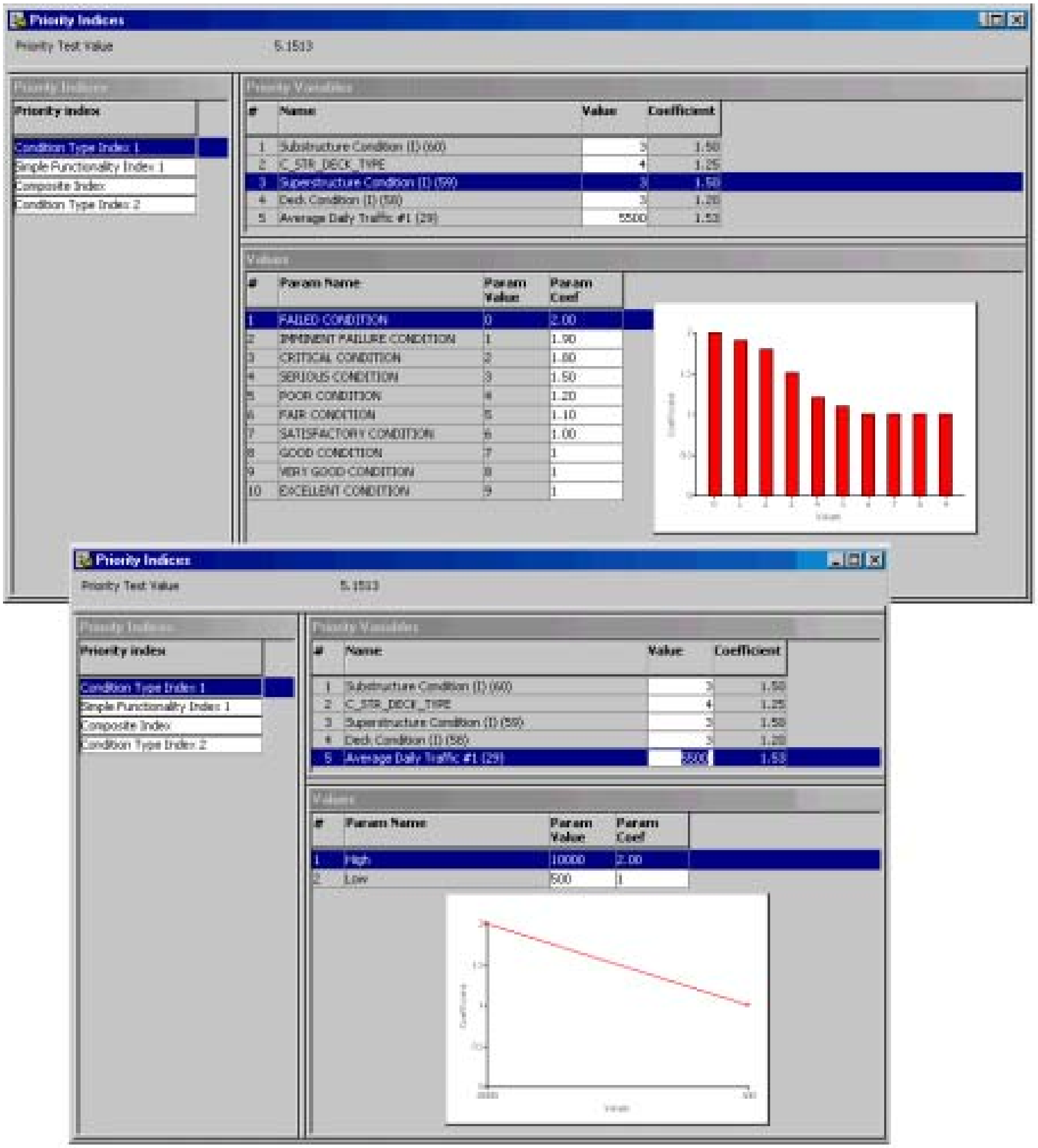


FIGURE 7 Priority indexes definition screen—two types.

1. If the priority variable has specific values, then a priority coefficient is assigned to each of the priority variables' values. In Figure 7 this can be seen in the top screen. Substructure condition is highlighted, which has 10 values from 0 to 9 as shown in the values subwindow. Failed condition is assigned a high priority coefficient of 2, while good to excellent are assigned normal priority values of 1.

2. If the priority variable is a strictly numeric variable, such as average daily traffic as shown in the bottom screen of Figure 7, this is assigned priority coefficients on a linear interpolation basis. In the "Value" subwindow at the bottom, you can see that, for

average daily traffic of 500 and lower, the priority coefficient is 1. For average daily traffic of 10,000 and higher, the priority coefficient is 2, and, for average daily traffic between 500 and 10,000, the priority coefficient is derived through interpolation as shown in the graph.

Bridge Maintenance

In coordination with the maintenance management module of the KYTC asset management system, the KYBMS provides for an efficient interface to recommend, manage, and review bridge maintenance activities. In the KYBMS, the information associated with the work order includes an automatically generated identification number, the bridge name, the treatment, work start and end dates, and the cost of performing the work. The scope of the historical work log is only limited by the bridges in the bridge population and the set of treatments defined by the user in the KYBMS.

The KYBMS can also be used to schedule periodic preventive maintenance work. This maintenance scheduling is performed on a bridge-by-bridge basis. The user highlights the bridge on which periodic maintenance will be performed and inserts from one to many treatments that will be periodically performed. Each treatment for each bridge can follow its own schedule. As the time for periodic maintenance arrives, an alert to treat that bridge appears in the periodic maintenance queue. At that point, the KYBMS user changes that maintenance need into an active work order at which point, programmatically, the bridge and treatment are removed from the maintenance needs queue and the work order is assigned to field crews and completed.

Data Presentation and Reporting

The KYBMS has excellent data presentation capabilities. All types of reports and graphs can be created, edited, and displayed by any KYBMS user.

Tabular Reports

The top part of Figure 8 shows an example of a standard report showing eight variables specified for every bridge in the network. Four bridge condition attributes, three bridge classification variables, and the bridge identification are shown. KYBMS is efficient in that this particular report takes under 10 s to load more than 10,000 bridges in the Kentucky bridge population. If the user views this bridge report today, it will show the data values as of today, and if the report is viewed next month, it will show bridge data updated through that time. The creation of such reports is also easy. From the pick list on the upper left-hand side of Figure 8, it is simply a matter of checking the eight (in this case) appropriate columns from a list of all available columns, then clicking on the Report Design tab to see the resulting report.

This kind of report can easily be changed into a summary report, for example, counting the bridges of a particular functional class and design type. The bottom part of Figure 8 shows such a summary report for all bridges in Kentucky; it shows all the different functional classes (there are 12) and all the different design types (there are 23). For each combination of functional class and design type that exists in the Kentucky bridge population is a count of the number of bridges that fit those two categories. On the bottom right of Figure 8, a pop-up window has been invoked, which identifies that there are 147 functional class and design type combinations in the Kentucky bridge population. All of the 13,000+ Kentucky bridges fit into these 147 category combinations. The effort needed to convert the report list in the top of Figure 8 into the summary report in the bottom takes less than 1 min and is completed as follows:

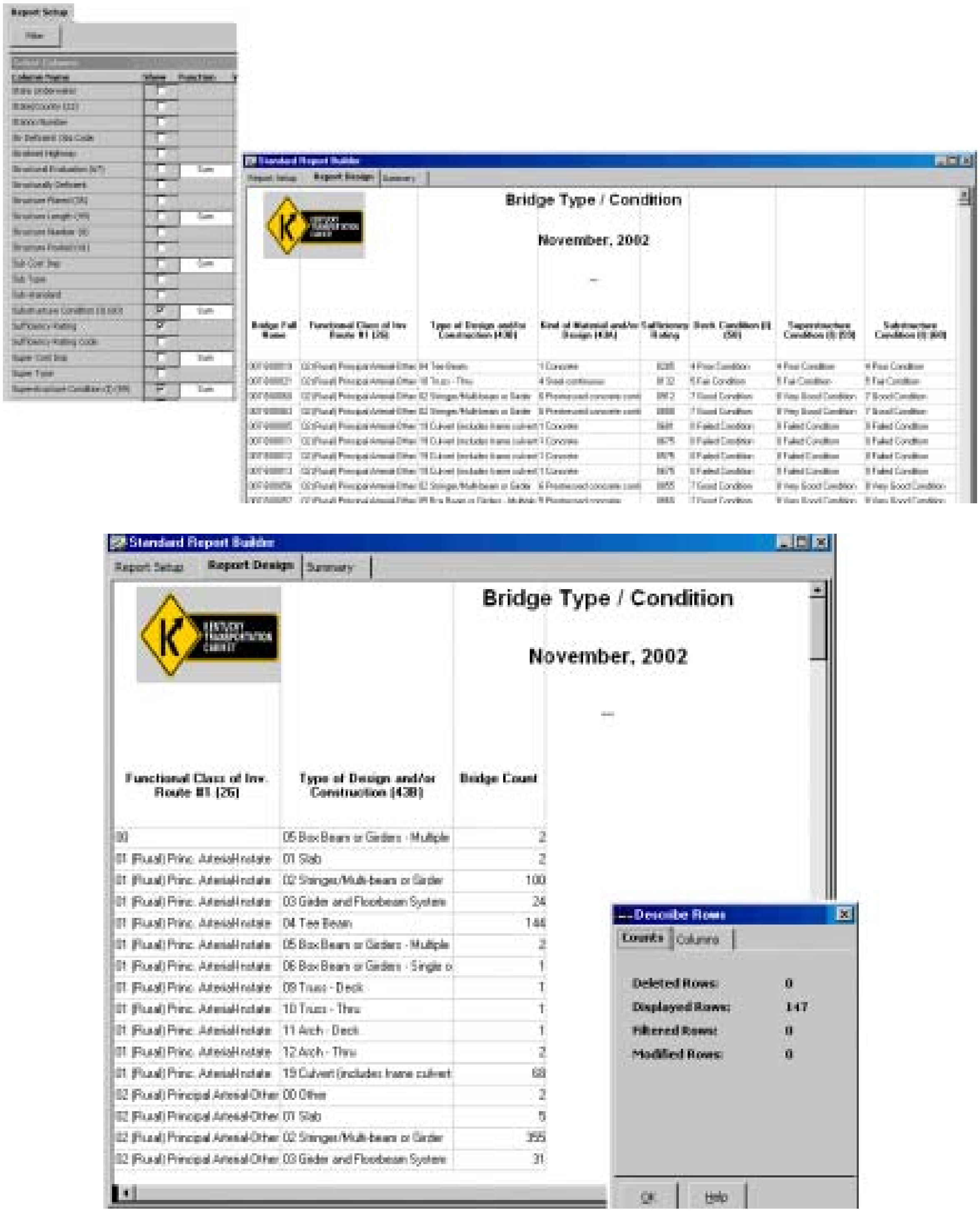


FIGURE 8 Standard reports.

1. In the original report go to the list of all variables and un-check all variables except functional class and design type.
2. Find the “bridge count” variable in this list and check it.
3. Click on the Report Design tab to see the summary report.

The capability to develop both detailed list reports and summary reports is quick, easy, and efficient.

Graphical Reports

Similarly, it is possible to make a large variation of graph reports by using either saved standard reports that were made earlier (listed in Reports: Graph Reports), or the Graph Report Builder in the Report Module. For graph reports it will normally be necessary to limit the number of data in such a way that a readable graph is obtained. Consequently, for these reports frequent use is made of filtering capabilities.

Figure 9 gives examples of Graph Reports. The top example shows the average Sufficiency Index per district for 5 districts (1 through 5) and for 10 types of bridges. This involves a total of 4,322 bridges. The bridge types are listed in the legend box on the right. It is easy to make modifications to this report with the help of the Graph Setup tab given in the upper part of this figure. The setup also shows the graph type that was selected (Multi-variable), the function (average values), and the variables that were chosen for the vertical axis (Value Column), the horizontal axis (Category Column), and the series. The filter button in the upper left was used to limit the data to 5 districts and 10 types of bridges. A report like this can be made within a few minutes, and after it has been saved for future use it pops up immediately by selecting it in the Graph Reports window.

Changing the selections in the Graph Setup window can easily modify a report. But it is also possible to make a new report with the help of the Graph Report Builder. By examining the graphed results in the top example it is apparent that the bridges of type 10 (Thru-truss) show rather low Sufficiency Ratings, and it would be of interest to find out the number and names of bridges involved and the importance of the routes that make use of these bridges. In the Graph Report Builder, in the Graph Setup window, the variables are selected as shown in the bottom example of Figure 9, and the resulting graph shows the results: the bridge name is used on the vertical axis, and the functional class forms the series in the legend box. In this case the filter button was used to limit the graph to “District to No.1”’ with type of bridge as “No.10 (Thru-Truss).” It can be seen that there are only 10 bridges of this type in District 1, but most of them appear to warrant further scrutiny as a result of poor ratings. It took only a few minutes to make this report, and it can be deleted or saved. In the latter case it will be kept in the Graph Reports window, where it will be automatically updated if new information is entered into KYBMS.

SUMMARY

KYTC is implementing an integrated highway asset management system on a statewide basis. An important component of the overall system is the KYBMS.

The software is capable of storing, retrieving, and processing bridge condition and inventory data within a transportation network. It allows users to analyze the current condition and expected needs for the bridge population. The software uses route, location, referencing, identification, traffic, and environmental data common throughout an agency’s database relative

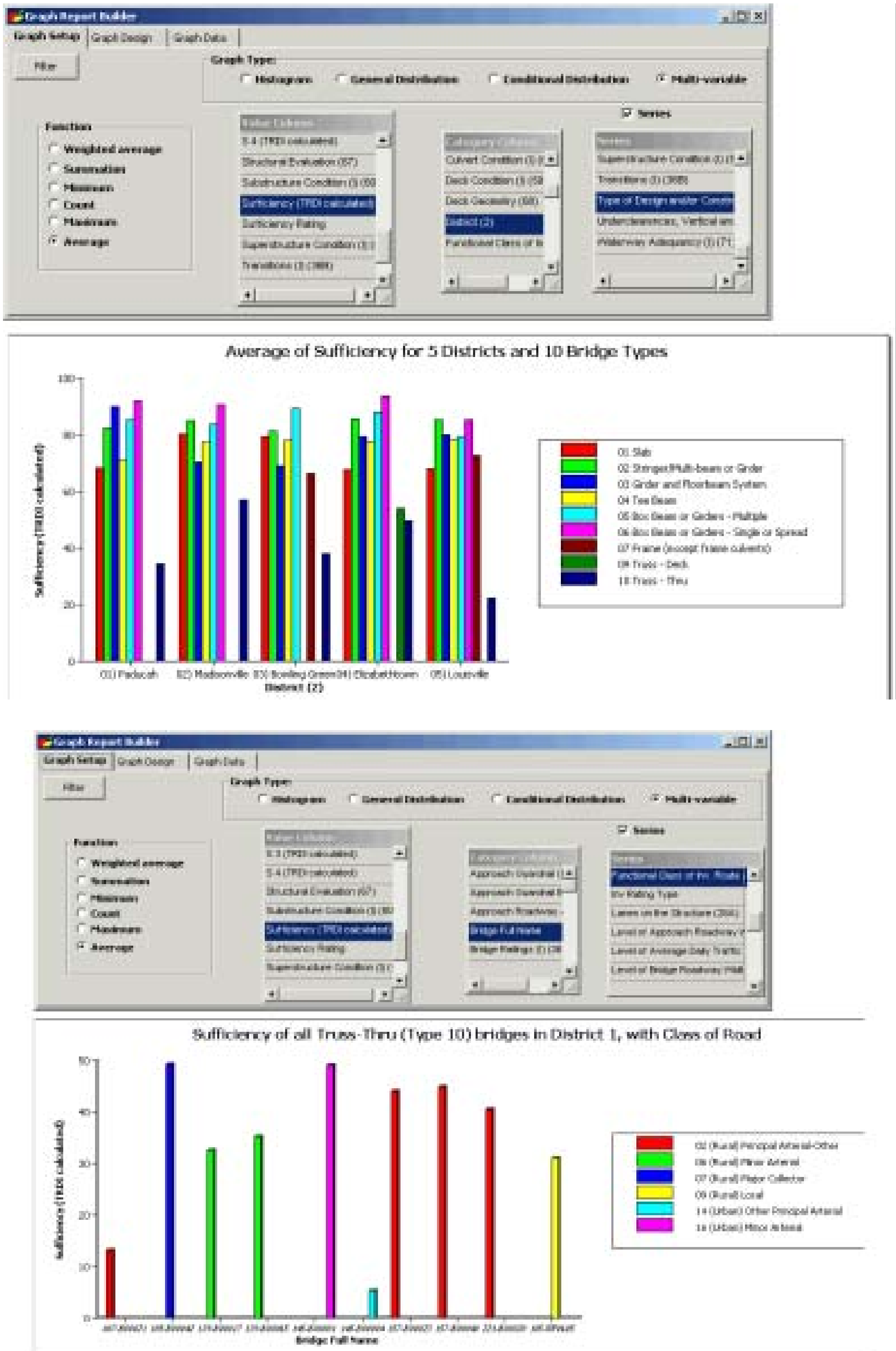


FIGURE 9 Graph report builder.

to its overall transportation network. Visual/BMS is compatible with other TRDI asset management systems such as Visual/PMS and Visual/MMS. It is also compatible with TRDI's TMSView geographic information system/mapping interface.

Visual/BMS provides extensive capabilities in network-level bridge management. In addition to determining consequences of the budget on network condition, the system assists with the selection of the location, recommended treatment, and timing of projects. These analyses are used to formulate the bridge preservation and replacement budget. The budgeting process integrates the actions of maintenance and construction into an effective technique for selecting candidates for bridge preservation and replacement treatments and timings. Visual/BMS offers two approaches for model formulation: multiyear prioritization (BRIDGET methodology) and incremental cost–benefit optimization (replacement for Pontis methodology).

Visual/BMS allows a wide range of predefined and user-defined reporting and graphics capabilities. TRDI develops client-specific reports during each project, thus ensuring information is readily available in formats familiar to each client. For KYTC, an appropriate set of reports was developed and users were trained to quickly develop their own reports and save them in the reports library. The basic formats defined by the users can be edited to produce slight variations and to create secondary types of reports and graphics with similar formats.

Life-Cycle Cost Considerations

Life-Cycle Cost Approach to Bridge Management in the Netherlands

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The Dutch Directorate General for Public Works and Water Management is responsible for management of the national road infrastructure in the Netherlands. Structures such as bridges and tunnels are important structures in the road network and largely determine the functionality of the road network as well as necessary maintenance budgets. A methodology for a probabilistic life-cycle cost approach to bridge management was applied to the concrete highway bridges in the Netherlands. The Dutch national road network contains over 3,000 highway bridges, most of which are 30 years old or more. The annual maintenance cost of these bridges is a substantial part of the total maintenance cost. The question arises of when to carry out bridge replacements. A fundamental solution is to take a life-cycle cost approach with costs of maintenance and replacement and service lifetime as key elements. Maintenance strategies were drawn up for groups of similar elements, such as concrete elements, preserved steel, extension joints, and bearings. The structures were categorized into generic types, each with its own maintenance characteristics. For each structure, the maintenance cost was estimated on the basis of the life-cycle cost analyses of the underlying elements. After aggregation over the entire stock, this process eventually led to the maintenance cost on a network level. To calculate the life-cycle cost, lifetime distributions for concrete bridges were determined, and the expected cost of replacing the bridge stock was computed. The uncertainty in the lifetime of a bridge can best be represented with a Weibull distribution, which can be fit on the basis of aggregating the lifetimes of demolished bridges (complete observations) and the ages of current bridges (right-censored observations). Using renewal theory, the future expected cost of replacing the bridge stock can then be determined while taking into account current bridge ages and the corresponding uncertainties in future replacement times. The proposed methodology has been used to estimate the cost of replacing the Dutch stock of concrete bridges as a function of time.

The Dutch Directorate General for Public Works and Water Management is responsible for the management of the national road infrastructure in the Netherlands. Maintenance is one of the core tasks of this directorate. Structures such as bridges and tunnels are important objects in the road network. They largely determine the functionality of the road network as well as the necessary maintenance budgets. For the management of the structures, a bridge management methodology has been established. The aims of bridge management are

- Effectively managing operational programs,

- Giving a realistic budget estimate at a national level, and
- Tuning bridge management programs with other maintenance programs such as pavement management.

Structures such as bridges are characterized by large investments and a long service life of 50 to 100 years. Although the annual maintenance cost is relatively small compared to the investment cost (less than 1%), the sum of the maintenance cost over the service lifetime is of the same order of magnitude as the investment cost. Therefore, it is not recommended that decisions on maintenance be separated from decisions on investment. The question arises of when to carry out replacements. A fundamental solution to this problem is a life-cycle cost approach. Key elements of this approach are the costs of construction and replacement, the cost of maintenance, and the service lifetime. These three items must be addressed while taking into account the uncertainties involved.

This paper describes the methodology for a probabilistic life-cycle cost approach to bridge management that was applied to the concrete highway bridges in the Netherlands. The next sections describe the features of the Dutch national road network, the annual maintenance cost of the structures, and the replacement value of the structures and their lifetimes. After that, the probabilistic modeling of the total life-cycle cost of the concrete bridges and results are described.

NATIONAL ROAD NETWORK LEVEL IN THE NETHERLANDS

The Dutch national main road network consists of 3,200 km of road, including 2,200 km of motorway. It serves mainly one function, mobility, with traffic safety and environmental aspects taken into account. The network divides assets into four categories: pavements, structures, traffic facilities, and environmental assets. The total number of structures in the network is 3,283. The structures are categorized into generic types, each with its own maintenance characteristics. An overview of the types, their number, deck area, and replacement value (see section on replacement cost) is given in Table 1.

ANNUAL MAINTENANCE COST

A maintenance strategy is drawn up for frequently used elements, such as concrete elements, preserved steel, extension joints, and bearings. Such a strategy requires a description of the minimal acceptable quality or condition, or a description of acceptable defects. Once the strategies are outlined, they can be applied to the stock of structures for the formulation of operational programs, and they can be used to estimate the total maintenance cost. An accurate

TABLE 1 Structures in Main Road Network

Object	Number	Deck Area (m^2)	Replacement Value (€ millions)
Concrete bridge	3,131	3,319,002	6,600
Steel bridge (fixed)	88	301,997	600
Movable bridge	43	347,876	1,100
Tunnel	14	475,228	1,700
Aqueduct	7	86,491	250
Total	3,283	4,530,593	10,250

estimate of the maintenance intervals and the cost of standardized measures are essential but difficult parts of the methodology. This information must be extracted from maintenance experts, since registered data are not yet available. In this process, subjective—and often conflicting—expert opinions must be combined to reach a level of consensus.

For each structure, the maintenance cost is estimated by means of the corresponding maintenance intervals and cost indicators. The results of this cost analysis can be updated by assessing the actual state of the structure through inspections. After aggregation of the maintenance plans of the entire stock of structures and prioritization of the available budgets, this process leads eventually to operational maintenance programs. A typical outcome of a maintenance plan for a single concrete highway bridge is given in Figure 1.

The prognosis of the maintenance cost for elements of structures can be applied to groups of structures. The maintenance cost on a network level can be determined by combining the maintenance costs of these groups of structures and their asset sizes. A similar approach is reported by Das (*1*). Table 2 gives the total annual maintenance cost per structure type.

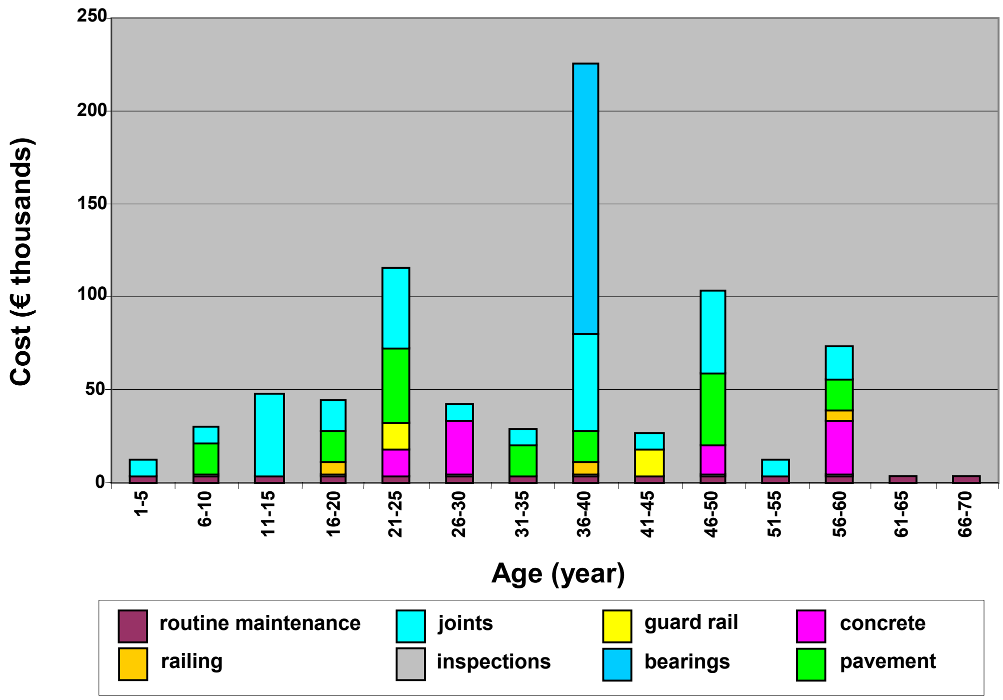

FIGURE 1 Maintenance cost of typical highway bridge summarized over units of time of 5 years.

TABLE 2 Annual Maintenance Cost of Each Structure Type

Structure Type	Total Annual Maintenance Cost (€ millions)
Concrete bridge	37
Steel bridge (fixed)	7
Movable bridge	10
Tunnel	13
Aqueduct	1
Total	68

REPLACEMENT COST

The cost of replacing a structure largely depends on which elements must be replaced and which cost items are to be included. This can easily lead to a variation in replacement cost by a factor 2 to 4. The replacement cost is best based on the bridge length and its functionality. Is it a two-to-three-lane highway, for example. In calculating the expected cost, the bridge is replaced by a standard type of bridge that may not necessarily be the same as the original. The cost of replacement includes all direct and indirect cost items related to the structure itself. The costs of site preparation, demolition, and traffic measures are not included. The cost calculation is based on the same data used in cost calculations for construction projects. The replacement value of the entire stock of structures is presented in Table 1 and plotted against the year of construction in Figure 1. Figure 2 shows a peak in construction activities in the 1970s, which was characteristic for many Western European road networks (*2*).

SERVICE LIFETIME

The lifetime of structures can be assessed in a number of different ways, using the predefined design lifetime, the functional lifetime, and the economical lifetime, which are defined as follows:

- The design or technical lifetime is determined by the design method and the choices on loads and durability of materials and elements. The design codes used in the Netherlands require a design lifetime of between 50 and 100 years. Most highway bridges are designed for an 80-year lifetime.
- The functional lifetime is determined by the structure's use. The design anticipates a certain use, and the functional lifetime ends when the use changes.

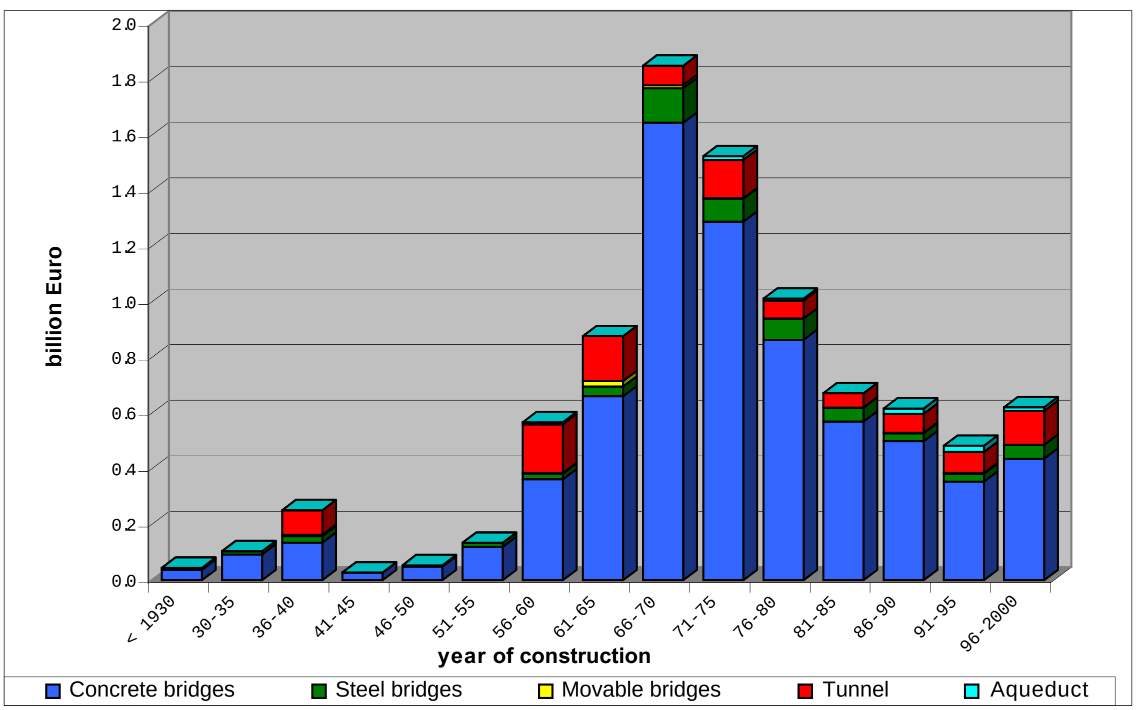

FIGURE 2 Replacement value of structures related to years of construction.

- The economical lifetime ends when the operational cost has become so large that building a new structure is more economical. It should be noted that the term “economical” is not well defined.

Bridges’ observed lifetimes represent a sort of combination of these lifetime types. The approach of using observed lifetimes is more or less common practice for all sorts of structural elements. The application of this approach to bridges is rather innovative and is described in the next sections.

The service lifetime can largely depend on the approach used. In assessing lifetimes, it is important to note that they must be predicted a long time in advance. Uncertainties in future development of road traffic are many. Realizing this, the different approaches can be regarded as scenarios. Accounting for the uncertainty of the lifetime through probability distribution is therefore important.

ESTIMATION OF LIFETIME DISTRIBUTIONS

The uncertainty in the lifetime of concrete bridges can be quantified by performing a statistical analysis on lifetimes of demolished bridges and ages of existing bridges. However, the general opinion on estimating a lifetime solely on the basis of bridge replacement times is that the resulting expected lifetime is often considerably underestimated. Although this underestimation is confirmed by the results found by Dutch study, a statistical analysis is nevertheless useful. The underestimation problem can be resolved by fitting a probability distribution to both the lifetimes of demolished bridges (complete observations) and the current ages of existing bridges (right-censored observations). The estimates so obtained of the expected lifetime of a concrete bridge are more in accordance with the usual design life. Although the right-censored observations do not contain actual lifetimes, they are a valuable source of information. At least it is known that the lifetimes of existing bridges will be longer than their current ages.

The Weibull distribution is recommended for properly modeling the aging of bridges. Using the maximum-likelihood method, a Weibull distribution can be fitted to both complete and right-censored observations [see van Noortwijk and Klatter (*3*)]. An advantage of the Weibull distribution is that the conditional probability distribution of the residual lifetime given the current age can be analytically expressed as the so-called left-truncated Weibull distribution. The observed lifetimes and ages of concrete bridges and viaducts in and over the highway were aggregated. In total, 79 lifetimes of demolished bridges (the complete observations in Figure 3) and 2,974 ages of existing bridges (the right-censored observations in Figure 4) were gathered. Because the right-censored observations were only available in terms of units of time of 1 year, bridge replacement was modeled as a discrete-time renewal process [see van Noortwijk and Klatter (*3*)]. For 157 concrete bridges, the year of construction and/or the length and width are unknown leading to a total number of concrete bridges in the Netherlands of 3,131. These 157 bridges include 19 concrete-steel bridges that are not included in the statistical analysis because they have different aging characteristics than concrete bridges.

A statistical analysis was performed for complete lifetimes and also for the combination of complete and right-censored lifetimes. The general opinion of bridge maintenance managers is that a statistical analysis of replaced bridges is not useful, because the fitted lifetime distribution tends to underestimate the expected lifetime at about 40 to 50 years instead of the usual design life of 80 to 100 years. The main reason for this is that most demolished bridges are not replaced

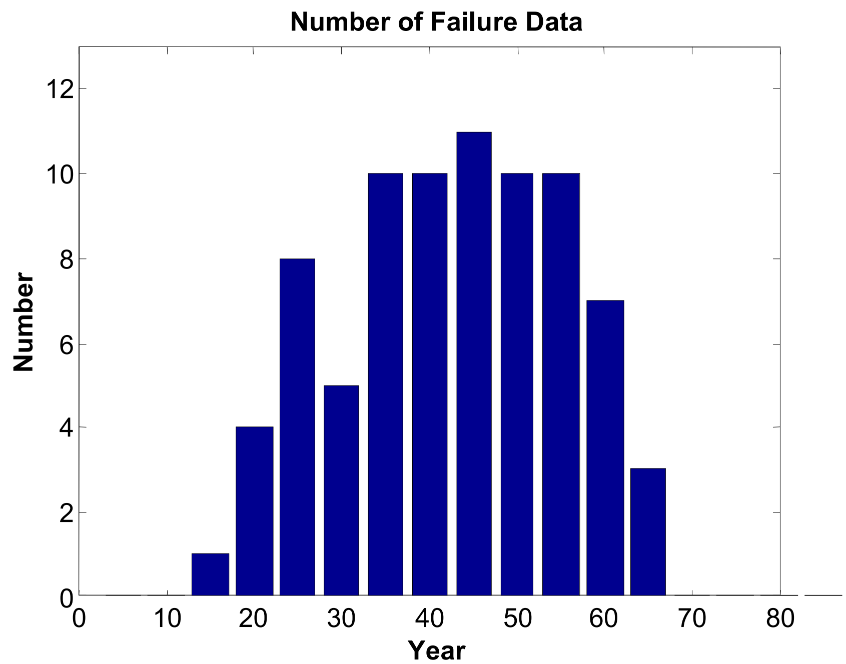

FIGURE 3 Histogram of complete lifetimes gathered in units of time of 5 years.

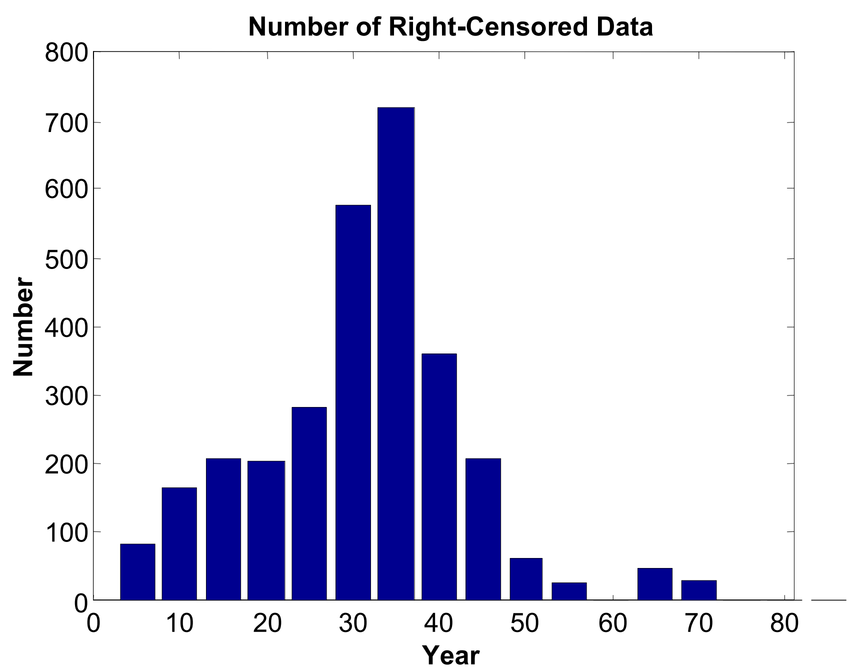

FIGURE 4 Histogram of right-censored lifetimes (current ages) gathered in units of time of 5 years.

as a result of technical failure, but because of a change in functional or economical requirements. Examples are bridges replaced because their load-carrying capacity cannot handle an unexpected increase of heavy traffic. Unfortunately, information has been insufficient to make a distinction between the design, functional, and economical lifetimes. Therefore, the observed lifetimes of demolished bridges can be either of these three, and the bridges were analysed as a whole. Furthermore, possible changes in bridge design over time could not yet be taken into account.

As expected, our statistical analysis of complete lifetimes resulted in an underestimation of the expected lifetime: a mean of 41 years with a coefficient of variation of 0.30. The corresponding maximum-likelihood estimators of the shape parameter and the scale parameter of the Weibull distribution are 3.8 and 45.1, respectively. The resulting Weibull probability density function based on complete observations is shown in Figure 5. However, when the current ages of the concrete bridge stock are included, the results change considerably. The expected lifetime increases from 41 to 75 years! The coefficient of variation changes little, with a value of 0.24. The maximum-likelihood estimates of the shape parameter and scale parameter are 4.7 and 81.8, respectively. The Weibull density function based on both complete and right-censored observations is shown in Figure 6.

CALCULATION OF REPLACEMENT COST

When the lifetime distribution is based on both complete and right-censored observations, the expected replacement cost over a bounded horizon can be computed by means of renewal theory. It is assumed that the replacement cost of a concrete bridge is independent of time. Although an old bridge is seldom replaced by the same type of bridge, it is difficult to accurately assess the cost of a new bridge. The replacement value of the stock of concrete bridges with known years of construction is €6,380 million [€ = US$1.02644 (December 2002)]. The replacement value of the concrete–steel bridges and the concrete bridges with unknown years of construction and/or unknown lengths and widths is €220 million.

In Figure 7, the expected cost per year is shown as a function of time and the bridges' ages (while the ages are gathered in units of time of 5 years). Summing over all the concrete bridges, as well as their corresponding ages and replacement costs, gives the expected cost per year as derived in van Noortwijk and Klatter (*2*) and shown in Figure 8. As expected, the uncertainty in the second replacement time is greater than the uncertainty in the first replacement time. As the time horizon approaches infinity, the expected long-term average cost per year approaches €85 million. Indefinitely far in the future, our delayed renewal process becomes a stationary renewal process for which the expected cost per unit time finds an equilibrium value. To account for the replacement cost of the 157 concrete bridges with unknown years of construction and/or unknown lengths and widths, the expected replacement cost per unit time shown in Figures 7 and 8 should finally be multiplied by a factor 6600 / 6380 = (6380 + 220) / 6380 = 1.03.

The cost of maintenance is not included in Figures 7 and 8. The life-cycle cost can be estimated by combining the cost of replacement with the cost of maintenance. The cost of maintenance can be regarded as constant, averaged over the large number of structures. This assumption will only be valid for an aging bridge stock. The annual cost of replacement after a long time will be approximately €85 million—about twice the annual cost of maintenance of €37 million. The first peak in the expected cost of replacement is three times the annual cost of maintenance. These results can be regarded as a first step in developing a replacement strategy based on life-cycle cost. Different scenarios for replacement, such as preventive replacement or postponed replacement, by extending the lifetime can be assessed on the basis of life-cycle cost.

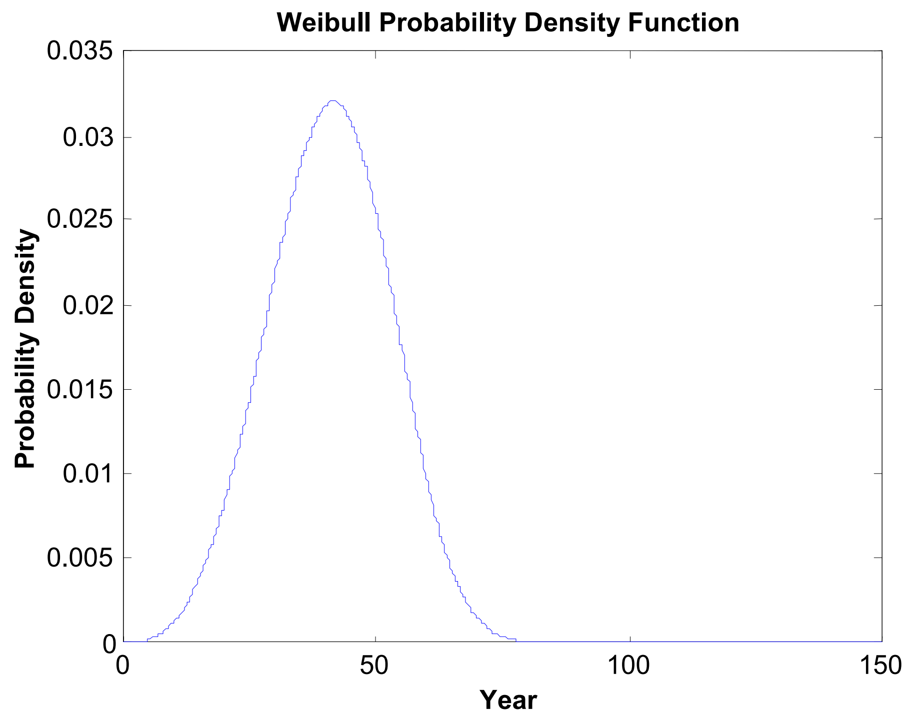

FIGURE 5 Weibull distribution estimated on the basis of complete lifetimes.

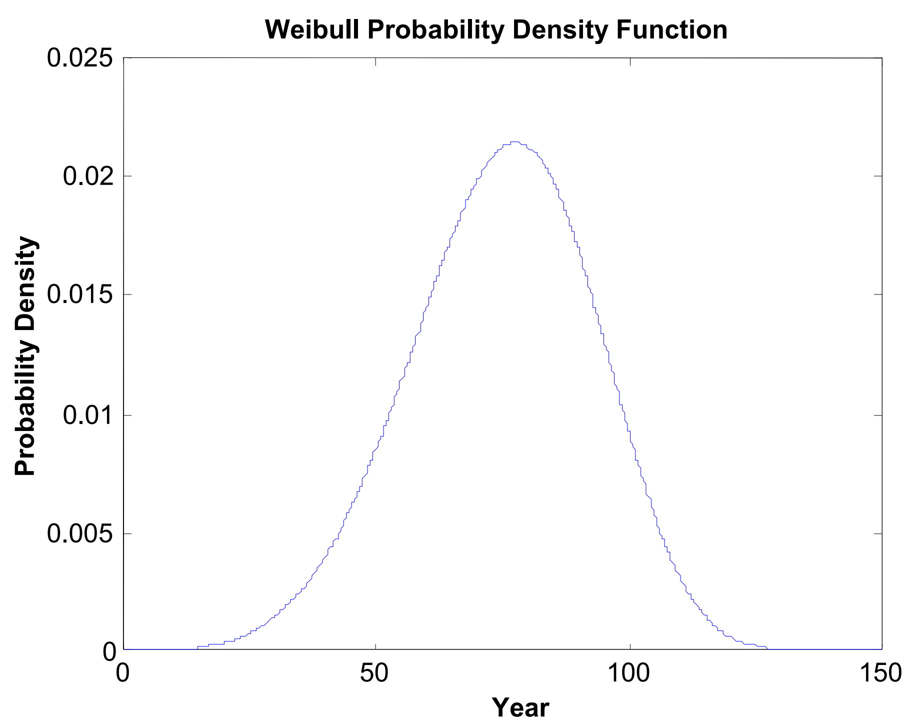

FIGURE 6 Weibull distribution on the basis of both complete and right-censored lifetimes.

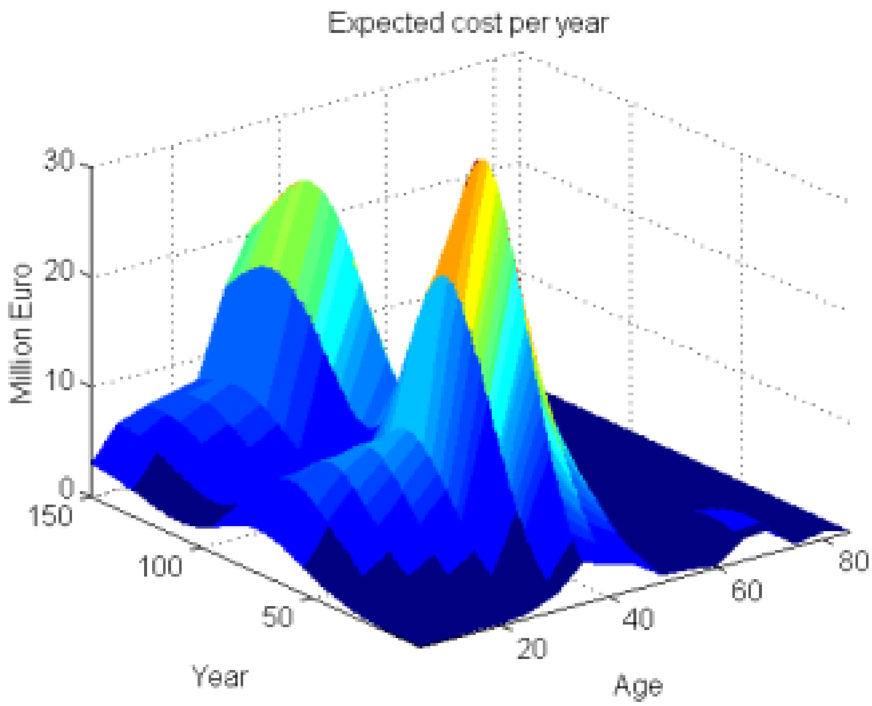

FIGURE 7 Expected cost per year as a function of age, where ages are gathered in units of time of 5 years.

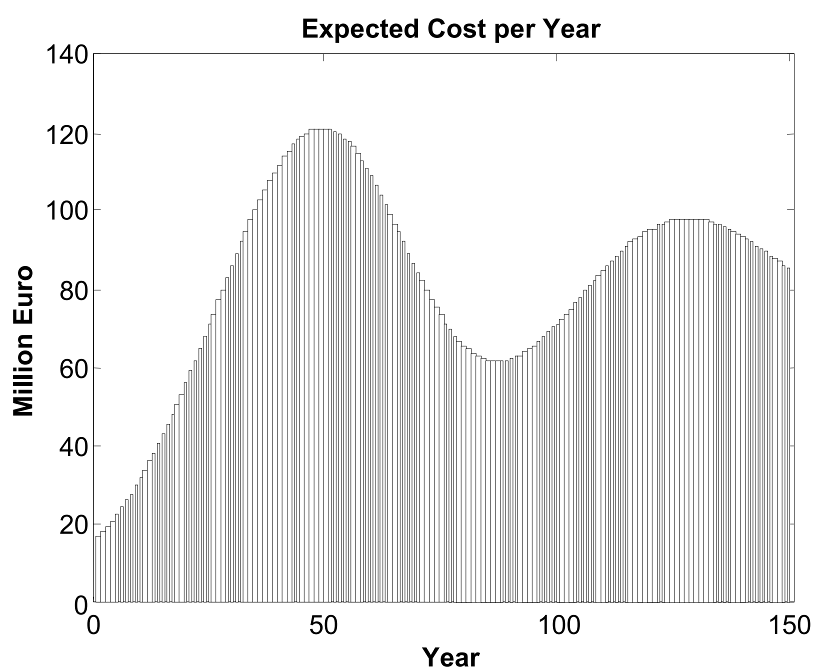

FIGURE 8 Expected cost per year summed over all ages.

CONCLUSIONS

The results presented in this paper demonstrate that cost of replacement and maintenance can be combined into a life-cycle cost approach. For this a systematic assessment of cost data, asset data, and lifetimes is needed. Creating a consistent set of data needs great care.

The service lifetime of a bridge is a parameter that is difficult to predict, but it has major influence on replacement cost. Several scenarios have been developed in assessing bridge lifetime. To account for the uncertainty in the lifetime by a probability distribution is therefore important.

A statistical analysis was used for determining the lifetime distribution of concrete bridges in the Netherlands. A Weibull distribution was fitted to both complete lifetimes of demolished bridges and current ages of existing bridges. Unlike the average value of the observed complete lifetimes, the expected value of the Weibull lifetime distribution was in agreement with the usual design life. Advantages of representing the uncertainty in the lifetime of bridges with a Weibull distribution are the possibility of properly modeling aging and of analytically deriving the conditional probability density function of the residual lifetime when the current age is given.

The so-obtained Weibull distribution was used to determine the future expected cost of replacing the bridge stock. In calculating this cost, the ages and replacement costs of the individual bridges were taken into account. In a case study, the expected future cost of replacement of the Dutch concrete bridges was estimated. Taking into account the uncertainties in the replacement times has the advantage that the cost is more spread out over time than in the deterministic case.

REFERENCES

1. Das, P. C. Inspecting, Measuring, and Optimising the Performance of Structures. In Workshop Optimal Maintenance of Structures, Delft, the Netherlands, Oct. 4–6, 2000, pp. 89–103.
2. Woodward, R. D. Bridge Management in Europe. Transport Research Laboratory, Crowthorne, United Kingdom, 2001.
3. van Noortwijk, J. M., and H. E. Klatter. The Use of Lifetime Distributions in Bridge Replacement Modelling. In *First International Conference on Bridge Maintenance, Safety and Management (IABMAS), Barcelona, Spain, 14–17 July 2002* (J. R. Casas, D. M. Frangopol, and A. S. Nowak, eds.), International Center for Numerical Methods in Engineering (CIMNE), Barcelona, 2002.

Maintenance Strategy to Minimize Bridge Life-Cycle Costs

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Cost–benefit assessment of bridge maintenance strategies is possible if related expenditures can be correctly evaluated over an appropriate life cycle and if they produce a known effect on structural performance. A method for such assessment was proposed by the authors in earlier publications and was based on data from the 762 bridges under New York City management. A program was developed for investigating alternative strategies to see not only which gives the lowest life-cycle cost, but also how the various cost components vary over the life of a bridge. The allocation of limited resources to bridge maintenance in optimal fashion was explored using the program, which provided numerical results and formal optimization techniques.

Increasingly, maintenance has become a discretionary activity within many organizations, because it is often the first victim of a budgetary shortfall. Those responsible for the maintenance and management of infrastructure know this situation all too well. In light of this, these managers need reliable methods for productively allocating the resources they are given.

In the year 2002 the New York City (NYC) Department of Transportation (DOT) was responsible for 762 bridges (of its roughly 1,800 bridges) with roughly 4,700 spans. The average bridge age is approximately 75 years, and the total bridge deck area is $1.43 \times \text{Mm}^2$ ($15 \times \text{Mft}^2$). Annual expenditures for these bridges include approximately \$500 million to \$600 million for capital and component rehabilitation, \$20 million to \$30 million for preventive maintenance, and a similar amount for demand maintenance or repairs of varied urgencies. Bridge conditions are evaluated according to the New York State DOT inspection system (*1*), which identifies 13 bridge components (see Table 1) and gives their relative importance for overall bridge condition. To compute the overall bridge rating, all bridge components are rated in every span on a scale from 7 (new) to 1 (failed) and used in a formula with the assigned relative importance weighting. The average NYC bridge condition rating over the last decade has remained near 4.5, which suggests a steady state.

In the NYC bridge system, as in other bridge systems, quantitative assessment of the role of maintenance in achieving such a steady state, in reducing deterioration rates, and in improving bridge ratings is the conundrum facing managers in justifying maintenance budgets. It is also the indeterminacy bridge managers face in attempting to apportion limited funds to maintenance tasks. Cost–benefit assessments of bridge maintenance strategies are possible if all related expenditures can be adequately evaluated over an appropriate life cycle. Experience in NYC can be drawn upon to record some known effects on structural performance, especially for extreme cases (such as no maintenance), and to quantify the more subjective elements of assessment. Inspection records of NYC bridges, including condition ratings for all structural components and entire structures, have been updated biennially or annually over the last 20 years and form a part of that experience base.

In two Columbia University studies of NYC bridge maintenance (*2*), the NYC experience was examined to identify 15 distinct maintenance tasks and to compile a list of discrete frequencies recommended for those tasks. This recommended level of maintenance is designated in this and previous work as "full maintenance" and appears as M = 1.0 = 100% maintenance. It is not a firm standard, neither for frequency nor for relative task levels. It is a reference standard from experience in NYC deemed to produce good performance. These 15 tasks, with their recommended frequencies and cost, have been reported previously (*3*, *4*) and are summarized in Table 1.

TABLE 1 Bridge Data and Results for M = 1

Components & Repair Costs			**Tasks, Cost At Full Maintenance and Effectiveness**			
Bearings	100,000	$ each		**Cmli**		
Backwalls	100,000	$ each		**$\$/ft^2$**	**$\$/m^2$**	**Kmi/Cmli**
Abutments	100,000	$ each	Debris removal	0.15	1.63	0.775
Wingwalls	100,000	$ each	Sweeping	0.04	0.43	1.186
Bridge seats	100,000	$ each	Clean drainage	0.06	0.61	1.995
	$\$/ft^2$	**$\$/m^2$**	Clean abutments and piers	0.18	1.95	0.568
Primary members	100	1,080	Clean open grating deck	0.00	0.04	0.00
Second members	100	1,080	Clean expansion joints	0.21	2.29	0.505
Curbs	10	30	Wash deck and splash zone	0.09	1.02	0.567
Sidewalks	10	110	Paint	2.35	25.27	0.025
Deck	80	860	Spot paint	1.55	16.65	0.035
Wearing surface	12	130	Sidewalk & curb replacement	0.09	0.93	0.116
Piers	100,000	$ each	Pavement & crack seal	0.15	1.64	0.818
Joints	25,000	$ each	Electrical device maintenance	0.07	0.78	0.000
			Mechanical component maintenance	0.07	0.71	1.222
Input Bridge Data			Wearing surface replacement	0.09	0.97	0.340
	ft^2	**m^2**	Wash underside	0.86	9.25	0.117
Bridge Area	30,000	2,790	**RESULTS**			
Number of spans	10		Maintenance level		100.0	%
Material (steel/conc)	s		% of full maintenance cost		100.0	%
Open gratings (y/n)	n		Deterioration rate used		E	
Deck (mono/overlay)	o		Expected life		150	years
Deck joints (y/n)	y				**$\$/ft^2/yr$**	**$\$/m^2/yr$**
	$\$/ft^2$	**$\$/m^2$**	Maintenance costs		5.96	64.15
User cost –repair based	30	323	Repair costs		5.35	57.58
–rating based	25	269	Replacement costs		12.00	129.17
NYC cost –repair based	25	269	NYC costs		7.24	77.95
–rating based	20	215	User costs		8.77	94.39
Replacement cost	1,800	19,380	TOTAL ANNUAL COSTS		39.32	423.23

LIFE-CYCLE COST MODEL

The model was developed (*5*) to fill the need for a rational method for allocating resources in bridge maintenance. The principal aim of modeling the effect of bridge maintenance in a life cycle is to devise a tool to quantify those effects in an environment of many variables, most of which are intuitive and subjective estimates of maintenance effects. In the model, value is derived from the ability to look at many "what if?" scenarios, rather than from the explicit values presented in the model formulation. The model also represents a quantitative package in which optimization techniques can be applied.

As described in Testa and Yanev (*5*) and elsewhere, the model uses a matrix of influence coefficients extracted from the subjective estimates of several NYC bridge maintenance and inspection personnel. The coefficients are used to estimate the influence of each of the 15 maintenance tasks on the rating of 13 bridge components in the New York State bridge condition formula. Together with the New York State-assigned importance of each component, the estimates of influence were used to compute the following:

- Relative importance of each task (k_{mi}) for the bridge rating,
- Overall maintenance level (M), and
- Weighted effect of maintenance tasks (M_i) on the deterioration rate of components.

Using experience-based limiting rates (*1, 2, 6, 7*) for zero maintenance (r_0) and full maintenance (r_1), the component deterioration rate was taken in the model to depend linearly on the maintenance levels M_i. This is an adequate modeling as long as maintenance levels are not considered to exceed the recommended levels ($M_i = 1$), for then it becomes possible for the deterioration rate to become negative. More realistic dependence of deterioration rates on each of the maintenance tasks is shown in Figure 1 together with the linear case. The illustration is for a component whose deterioration rate decreases by a factor of 4 as maintenance increases from 0 to the full recommended value. The curve S has a sinh dependence, as suggested in Yanev and Testa (*8*), and Curve E has an exponential dependence. The model allows a choice in computing, and in the present examples, the rate E is used.

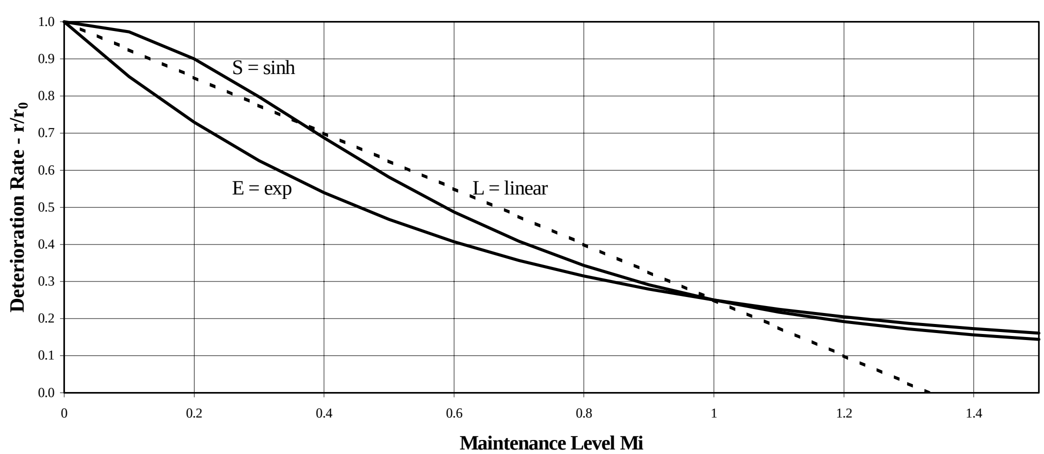

FIGURE 1 Bridge deterioration rate as function of maintenance level.

Given the relationship between bridge deterioration and maintenance level, bridge life can be computed once a repair protocol and a condition for replacement or rehabilitation are specified. Then all costs for maintenance, repairs, and replacement are annualized over that lifespan. These costs may include hidden costs to users and the community, as in the present model. That is not to say such costs are well defined, but rather, that their potential effect can be observed. Total annual costs and annual costs of each of the expenditures can thus be computed as functions of the maintenance level.

Uniform Maintenance Allocation Strategies

The most straightforward allocation strategy with less than full maintenance resources is a uniform fraction of the recommended level for all tasks. Table 1 presents summaries of the standard input bridge data and the results of the model calculation for full maintenance in all tasks. When all tasks are performed at a level other than $M_i = 1$, the overall maintenance level, M, and the percentage of full maintenance expenditure have the same value. Figure 2 represents all the uniform maintenance allocation strategies from M = 0 to the full recommended value M = 1. The total cost and other costs do not vary smoothly with changing maintenance levels. This lack of smoothness is expected, given the discrete nature of the repair protocol, and is most obvious in the curve for replacement cost in Figure 2, which also reflects discrete increases in the projected bridge life, Figure 3.

Two other factors are to be noted. First, the annual cost of maintenance changes directly with the level of maintenance (M) as long as the maintenance tasks are varied uniformly. However, since the tasks vary widely in cost, a given maintenance budget can be apportioned by many nonuniform distributions among the tasks, which results in achieving different maintenance levels. Secondly, for each M, there are many combinations of M_i other than uniform distributions (for which Figure 2 was generated) that generate that value of M, and the allocation that minimizes total cost at that M is sought by a manager.

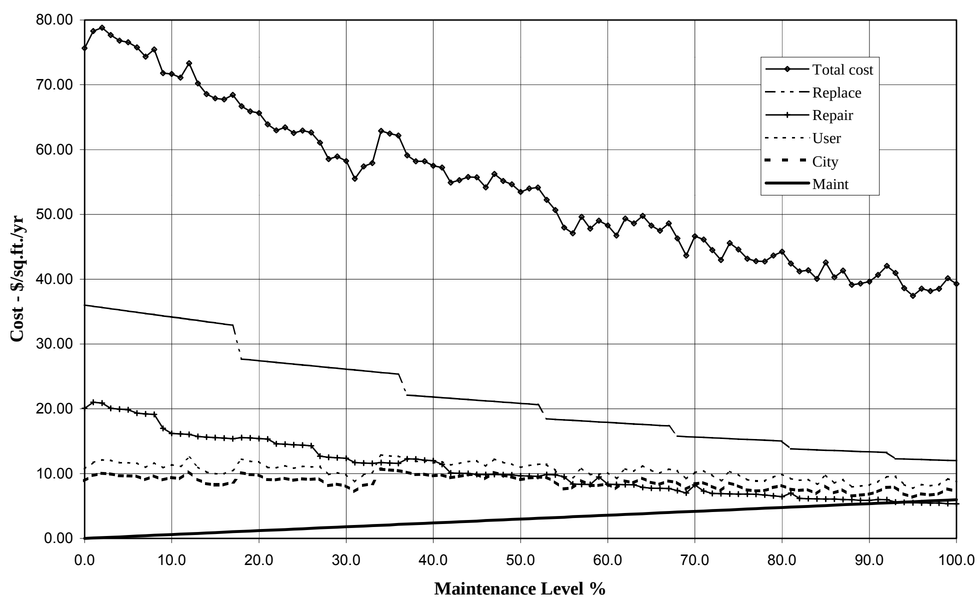

FIGURE 2 Annualized costs per percentage of full recommended maintenance level (M = 100%).

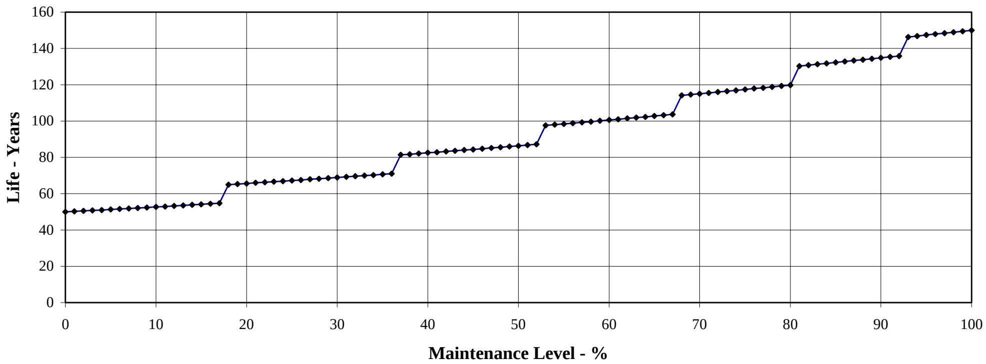

FIGURE 3 Bridge lifespan as function of maintenance level.

Cost-Effective Allocation Strategies

Intuitively, a manager might tend to apportion a limited maintenance budget by considering the cost-effectiveness of each task, which can be expressed as the ratio of the task effectiveness indexes, k_{mi} (computed in the model), divided by the cost of the maintenance task at its full recommended level, C_{m1I}, as outlined in Yanev and Testa (*8*) and as summarized in Table 1. The first two columns of Table 2 show the results of uniform [as suggested in the Columbia University study (*2*)] and cost-effective task allocations, which both produce the same full recommended level (M = 1) of effective maintenance. It is clear that with the cost-effective allocation, a much lower annual expenditure on maintenance—22.6% of the full maintenance expenditure—still leads to a total annual life-cycle cost some 8% lower than with uniform allocation. As might be expected, expenses for repairs are higher, and costs to users for delays and inconvenience are greater. These latter costs are not easily quantified and could be much greater than estimated in this example. However, the result seems to substantiate a cost-effective practice in times of austere budgets when maintenance is sacrificed.

An approach was discussed in Yanev and Testa (*8*) for adjusting the matrix of influence coefficients to render the maintenance frequencies recommended by the Columbia study (*2*) equally cost-effective while retaining the matrix's knowledge-based information. In that case funding at less than 100% level would be allocated proportionally to all tasks, allowing no room for optimization.

Using cost-effectiveness as a criterion for allocating maintenance funding is a rational and appealing approach. Clearly, however, very costly tasks such as painting will be grossly neglected by this approach, as can be seen in Table 2, where the cost-effectiveness level of painting is only 3% to 5% of the recommended full level, compared with 269% for drainage cleaning. This is an argument in favor of treating painting and other tasks of very low frequency and high cost as repairs rather than as maintenance.

OPTIMIZATION STRATEGIES

The maintenance activity combinations generated and discussed above offer meaningful insight into the impact of different maintenance activities upon the total annual costs and lifespan of bridges. Developing more formal approaches to optimizing resource allocation is a natural next step. Optimization methods are routinely employed to allocate resources when constraints are present. Linear programming is the most common and powerful approach, and it is often directed

TABLE 2 Results of Uniform and Cost-Effective Allocations

Allocation	Uniform		Cost Effective		Uniform		Cost Effective		Optimized	
Debris Removal	1.00		1.05		0.54		0.57		0.00	
Sweeping	1.00		1.60		0.54		0.87		1.00	
Clean Drainage	1.00		2.69		0.54		1.46		1.00	
Clean Abutments and Piers	1.00		0.77		0.54		0.42		0.00	
Clean open Grating Deck	1.00		0.00		0.54		0.00		0.00	
Clean Expansion Joints	1.00		0.68		0.54		0.37		1.00	
Wash Deck & Splash Zone	1.00		0.77		0.54		0.42		1.00	
Paint	1.00		0.03		0.54		0.02		0.00	
Spot Paint	1.00		0.05		0.54		0.03		0.00	
Sidewalk & Curb Rep	1.00		0.16		0.54		0.08		1.00	
Pavement & Crack Seal	1.00		1.10		0.54		0.60		0.00	
Elect Device Mnt	1.00		0.00		0.54		0.00		1.00	
Mech Component Maint	1.00		1.65		0.54		0.90		1.00	
Wearing Surf replacement	1.00		0.46		0.54		0.25		1.00	
Wash Undrside	1.00		0.16		0.54		0.09		1.00	
Maintenance Level (M) - %	100.0		100.0		54.3		54.3		54.3	
% of full maintenance cost	100.0		22.7		54.3		12.3		26.5	
Expected Life - years	150		152		98		99		115	
Costs ($/area/yr)	ft.	m.	ft.	m.	ft.	m.	ft.	m.	ft.	m.
Maintenance Costs	5.96	64.15	1.35	14.53	3.24	34.83	0.73	7.89	1.58	16.98
Repair Costs	5.35	57.58	5.85	62.95	9.78	105.28	9.74	104.80	8.63	92.90
Replacement Costs	12.00	129.17	11.82	127.26	18.34	197.40	18.16	195.47	15.67	168.62
NYC Costs	7.24	77.95	7.81	84.03	8.96	96.42	7.98	85.91	6.70	72.06
User Costs	8.77	94.39	9.44	101.59	10.82	116.41	9.66	104.00	8.13	87.50
TOTAL ANNUAL COST	39.32	423.23	36.27	390.37	51.13	550.34	46.27	498.07	40.70	438.07

at "blending" or "transportation" problems, as described by de Neufville (*9*) and Revelle (*10*). Application of linear programming relies on the ability to describe an objective function and all constraints as a series of linear equations, which produces a convex, feasible region for the solution space and an optimum at the edge of this region, specifically at a corner point. These conditions allow identification of a global optimum by means of many available solution algorithms. When the objective function or constraint equations are nonlinear and/or discontinuous, solution procedures become more difficult, and computation times can increase substantially. Nonlinear and discontinuous functions often create inflection points and local minima or maxima, which make identification of global optima even more challenging, particularly for complex problems. In such cases, resorting to piecewise linear approximations and dynamic programming techniques may be necessary.

In the present application, an obvious objective function is the bridge's total annual cost, and a decision-maker may seek to determine the optimal allocation of maintenance levels to minimize this cost subject to the available maintenance budget. In the model, these elements are not linear, and discontinuities exist. Moreover, the model is not purely mathematical, but rather, a computer program developed within a spreadsheet environment, and the variables (tasks M_i) are realistically not continuous but take on discrete values. Even with state-of-the-art search algorithms, the formal optimization to find a global minimum is relatively intractable in this case, and certainly, the built-in spreadsheet tools are inadequate. At this juncture, it is more realistic to explore optimizing combinations of tasks numerically and recognize that identifying a true global minimum must await further analyses.

NUMERICAL RESULTS

The number of possible combinations of task levels for exploration becomes very large if all levels of all 15 tasks, including values greater than 1, are surveyed. However, by considering a selected number of extreme values of M_i and fixing some values for tasks that cannot realistically be completely ignored, a more reasonable number of results can be computed. These can then indicate the cost bandwidth of the results at each M, for other than uniform allocation on the graph of total cost (Figure 2). These numerical results for extreme cases can also indicate what tasks may more consistently lead to reduced costs.

In Figures 4 to 7, the computed results are shown for all combinations of task levels with the following discrete levels of M_i: all tasks may be done at level 0 or 1, with the exception of

- Tasks 12, 13, and 14 (mechanical and electrical component maintenance, and wearing surface replacement), which are always performed at their recommended levels ($M_i = 1$), and
- Tasks 1, 2, and 6 (debris removal, sweeping, and cleaning expansion joints), which are also considered at possible levels 1.5, 2.0, and 1.5, respectively, which is more than recommended levels.

These results show that there is room for optimizing the allocation of maintenance funding. In Figure 4, there is substantial bandwidth about the solid line, which represents the uniform allocation result of Figure 2. The spread is smaller, however, for the repair costs, as shown in Figure 5, and greater for user costs, as shown in Figure 6. These two figures indicate that there is a price to be paid, other than actual expenditures, for the optimization gained by better allocation of maintenance. Figure 6 also shows that user costs do not vary so radically with overall M, as do the other two costs. In fact, the range of user costs varies more with the allocation of maintenance at a given M (bandwidth) than it does with a change in M itself. This may indicate that the maintenance levels for these tasks recommended in the Columbia study (*2*) are more effective in promoting the utility and serviceability of bridges than in minimizing long-term cost.

The use of only extreme values of the task levels results also in a banding of the data points, as seen in Figures 4 to 6. It is most clearly seen in the plot of maintenance costs in Figure 7. The data points in these bands are characterized by the presence or absence of the more expensive maintenance tasks. Thus, for example, the lowest band in Figure 7 is characterized by the absence of expenditures for Tasks 8, 9, and 15 (painting, spot painting, and substructure washing) while for points in the top band of Figure 7, those three tasks are done at their full recommended levels. Further study of the banding in these results may assist in optimizing maintenance.

One data point giving much lower total cost than found with the uniform allocation has been selected from Figure 4 for closer examination. The results for the lowest point at maintenance level M = 54.3% of Figure 4 are shown in the final column of Table 2 for that optimizing allocation. The two columns preceding in Table 2 list the results when uniform and cost-effective allocations are used to give the same maintenance level of 54.3%. This optimal allocation clearly gives not only a lower annual maintenance cost, but also lower repair and user costs. However, the optimal allocation also does away with expensive painting, as was the case

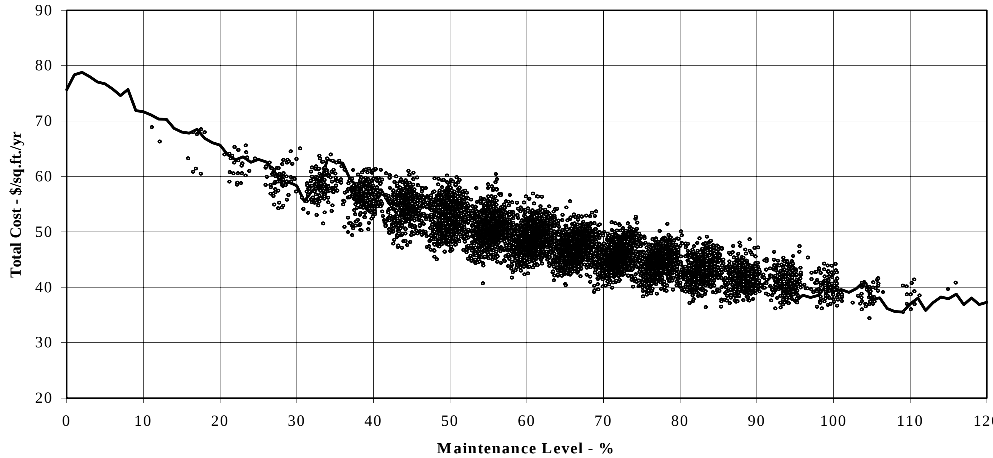

FIGURE 4 Total annualized maintenance costs for uniform and cost-effective resource allocation.

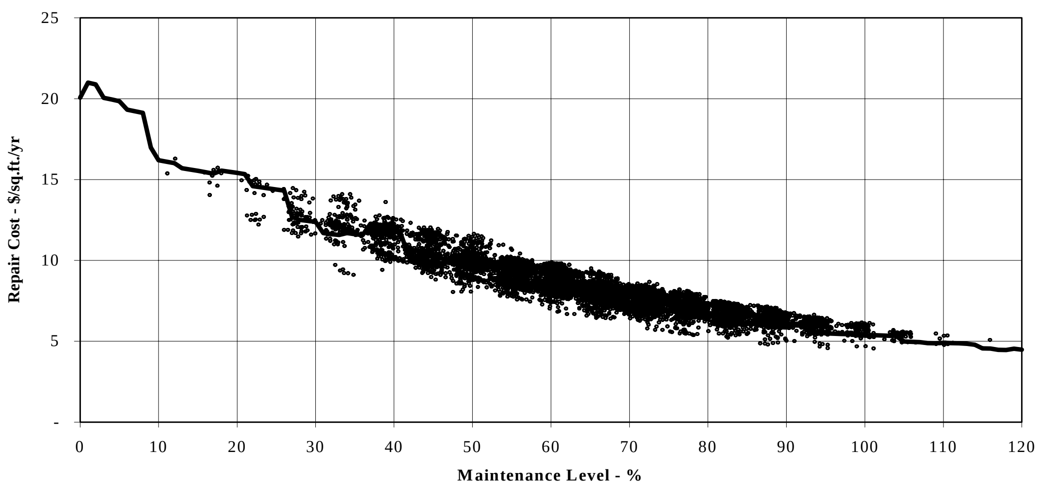

FIGURE 5 Annualized costs of repair for uniform and cost-effective resource allocation.

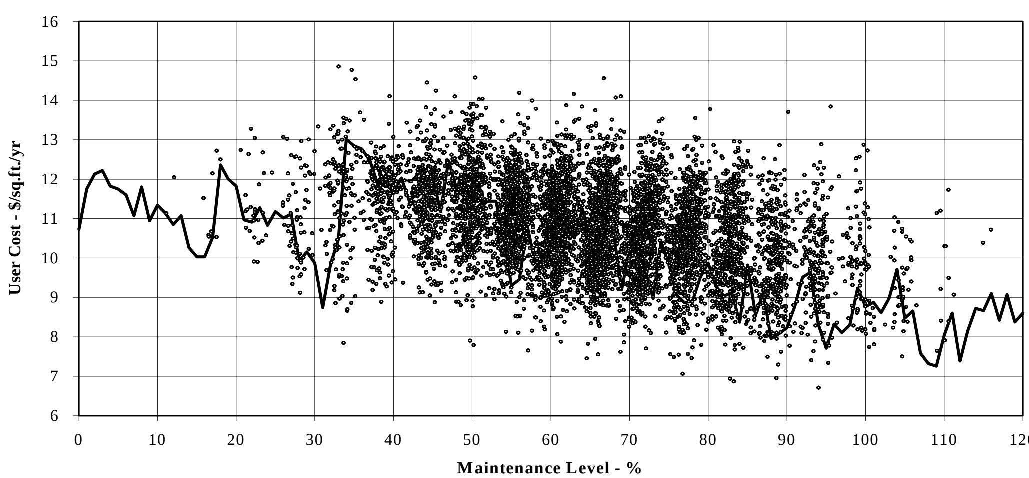

FIGURE 6 Total user costs at uniform and cost-effective resource allocation.

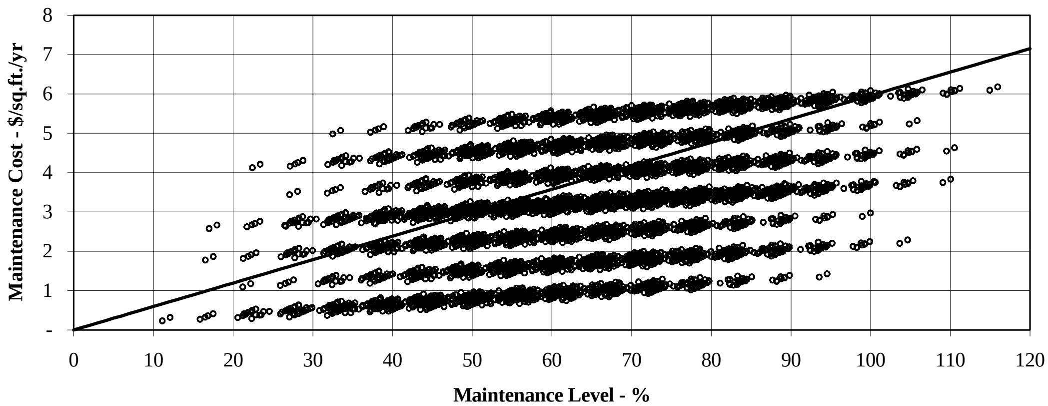

FIGURE 7 Annualized cost of maintenance, with or without more expensive tasks.

in the cost-effective allocation. This raises questions not only about other consequences of allocation strategies, such as esthetics, but also about the appropriateness of treating such high-cost, low-frequency tasks as routine maintenance rather than as repairs.

CONCLUSIONS

The life-cycle cost model developed for bridge maintenance management provides a simple way to explore alternatives and to see the effects of costs, even if they are not so well defined, such as user costs from delays and inconvenience. It gives a vehicle for applying optimization techniques to determine the best maintenance strategy open to a manager. At the same time, it shows a manager what possible downsides could accompany an optimal strategy, such as increased repair costs and user disruptions, if a low-maintenance strategy is followed. More importantly, the model gives substance to possible consequences that were only intuitive until now.

General optimization techniques are not easily applied to the model because of its complexity, but numerical values for extreme choices of the maintenance tasks do indicate the potential for optimal allocation of limited maintenance funding. Both the allocation based on a measure of the cost-effectiveness of each maintenance task and the optimal selection of extreme values (full or none) of maintenance levels show that, in terms of long-term costs, painting tasks are likely to be de-emphasized under budget strictures. These tasks clearly dominate in the cost categories and should be considered as repair items rather than routine maintenance. Further work in this model will explore such a reformulation and application of formal optimization tools.

REFERENCES

1. *Bridge Inspection Manual*. State of New York Department of Transportation. Albany, 1997.
2. *Preventive Maintenance Management System for New York City Bridges.* Technical Report No. 98-1. The Center of Infrastructure Studies, Department of Civil Engineering and Engineering Mechanics, Columbia University, New York, June 30, 1999.
3. Yanev, B. S., and R. B. Testa. Annualized Life-Cycle Costs of Maintenance Options for New York City Bridges. In *Bridge Management 4, Proc., Fourth International Conference on Bridge Maintenance,* Thomas Telford, London, 2000, pp. 400–407.
4. Yanev, B. S., and R. B. Testa. Maintenance Level Assessment for New York City Bridges. Presented at the 79th Annual Meeting of the Transportation Research Board, Jan. 2000, Washington, D.C.

5. Testa, R. B., and B. Yanev. Bridge Maintenance Level Assessment. In *Computer-Aided Civil and Infrastructure Engineering*, Vol. 17, 2002, pp. 358–367.
6. Yanev, B. S. Life-Cycle Performance of Bridge Components in New York City. *Recent Advances in Bridge Engineering, Proc., U.S.–Canada–Europe Workshop on Bridge Engineering*, Zurich, Switzerland, 1997, pp. 385–392.
7. Yanev, B. S., and R. B. Testa. Useful Life of Bridges and Their Components in New York City. *Proc., Eighth International Conference on Structural Faults and Repair*, London, July 1999.
8. Yanev, B. S., and R. B. Testa. Maintenance Level Assessment for New York City Bridges—Part II. In *Life-Cycle Cost Analysis and Design of Civil Infrastructure Systems, Second International Workshop,* Japan, 2001.
9. de Neufville, R. Applied Systems Analysis: Engineering Planning and Technology Management. McGraw-Hill, New York, 1990.
10. Revelle, C., E. Whitlatch, and J. Wright. Civil and Environmental Systems Engineering, Prentice Hall, Upper Saddle River, New Jersey, 1997.

Bridge Load Testing

Bridge Testing to Enhance Bridge Management in Delaware

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In most cases, assessments of bridge condition are based on visual inspections, and bridge ratings are determined by simple analytical methods without use of data on bridge response. Estimates of bridge load-carrying capacity generated in this manner tend, for good reason, to be fairly conservative. Reliable assessments of condition are essential to ensure the safety of the traveling public. Furthermore, because load-carrying capacity is often used to prioritize bridges for repair, rehabilitation, and replacement and funds for these actions are limited, it is more important than ever that these estimates be as accurate as possible. To achieve this goal, researchers at the University of Delaware have been working with engineers at the Delaware Department of Transportation to improve the state's ability to evaluate a wide range of bridges through a variety of field-testing methods. The study produced an overview of field-testing methods and of the impact of testing on bridge management in Delaware.

Bridge ratings are used in prioritizing bridges for repair and replacement. Traditionally, these ratings are based upon simple analytical methods that do not use site-specific data. Estimates of a bridge's load-carrying capacity made in this manner are often overly conservative. In some cases, bridges that have low load ratings, computed from traditional calculations, can be shown to have significantly more capacity when load tested. In these cases, accurate condition assessments and load ratings can prevent unneeded repairs and bridge replacements. By judiciously utilizing bridge load testing, bridge engineers can better allocate limited financial resources and more effectively manage their bridge inventories.

When it comes to determining a safe and accurate load rating for a bridge, the best model of the structure is the bridge itself. During design of a bridge, engineers do not have the luxury of utilizing this resource. However, during subsequent evaluation, it can be employed. For the past ten years, faculty at University of Delaware (UD) have been working with engineers at Delaware Department of Transportation (DelDOT) to integrate bridge field testing, in-service monitoring, and long-term monitoring into the department's bridge management efforts. The tests have been used to make several important decisions on bridge management, the results of which have saved a significant amount of money that otherwise would have been allocated for bridge replacements. The testing also enabled the state to improve its routing of permit vehicles.

BRIDGE FIELD TESTING APPLICATIONS

Years ago, the cost of conducting bridge field tests was prohibitive in almost all cases. However, with advances in sensor technology and in data acquisition systems, it is now economically viable to obtain a wealth of information about a bridge's performance through field testing.

In Delaware, various types of bridge field testing have enhanced bridge management in three distinct situations: (1) bridges with low ratings, (2) bridges with new materials or innovative designs, and (3) bridge rating with site-specific ambient response data.

Assessment of Bridges with Low Ratings

Bridges with low ratings cause problems for the traveling public for two reasons. First, if the ratings are low enough, the bridge will be posted, and some trucks with legal loads will be forced to detour around the posted bridge. Second, bridges with low ratings, even if not posted, will have limited overload capacity. As a result, permit vehicles may be forced to route around these bridges. Because Delaware has a limited number of major routes through the state, posted and low-rated bridges can have serious economic consequences.

As early as 1994, DelDOT and UD began conducting diagnostic load tests on posted bridges. These tests involve instrumenting the bridge with strain transducers and recording bridge response to the effect of known weight trucks. When this method has been used, site-specific load distribution, support conditions, and unintended composite action have been evaluated, and more accurate load-carrying capacities have been determined. The method has been applied to both steel girder bridges and concrete-encased girder bridges. In both cases, results have indicated that previously posted bridges could be unposted. Furthermore, the results have prevented the scheduling of unnecessary bridge replacements.

Another use of diagnostic load testing on low-rated bridges involved a series of tests on short concrete slab bridges on Route 13, the most major north-south highway in Delaware which the state is in the process of upgrading. The upgrading is being conducted in segments through the creation of a new highway called State Route 1 (SR-1). For incompleted portions of SR-1, traffic is being routed onto the existing highway, Route 13. Most of the highway is complete (providing a major truck route), but a single segment of Route 13 will be needed for a few more years. Therefore, several of the slab bridges on Route 13 were being considered for replacement, because they will represent a significant barrier to permit trucks until the final stretch of SR-1 is completed. (Once SR-1 is complete, the bridges, even though their capacity is somewhat low, would be adequate.) Diagnostic tests conducted on six of the short slab bridges indicated that they had considerably more capacity than had been thought. The bridges all had had fill and a previous concrete roadway between the existing roadway and the concrete slab. As a result, these short bridges had much better load distribution than was assumed, and it was shown that the bridges would not limit truck use on SR-1 and did not need to be replaced.

Assessment of Bridges with New Materials or Innovative Designs

DelDOT has been a leader in applying new materials in bridge design and rehabilitation. The state has used advanced composites and high-performance concrete and is in the process of installing MMFX rebar. Because these materials are relatively new, their actual in-situ performance requires evaluation to enable their proper maintenance and management.

DelDOT has used advanced polymer composites for all-composite slab bridges, composite slab-on-steel-girder bridges, and the rehabilitation of steel and prestressed concrete girders. In all cases, diagnostic field tests quantified the performance of the new material and established a baseline against which to measure future performance. In addition, on several of these applications, long-term monitoring systems have been set up to more comprehensively track the bridge performance over time. These efforts are greatly assisting DelDOT in management of these novel bridge applications.

For high-performance concrete, both diagnostic load tests and a long-term monitoring program, including corrosion detection, have been implemented. Similarly, for MMFX rebar, both strain monitoring of overall bridge performance and corrosion monitoring of the new corrosion-resistant rebar will be implemented. The results of these test programs will enhance DelDOT's ability to manage these bridges and will also indicate the effectiveness of the new material and its promise for future application.

Bridge Rating with Site-Specific Ambient Response Data

In addition to diagnostic load tests and long-term monitoring, in-service strain data are being recorded on interstate bridges for the purpose of conducting site-specific load and resistance factor ratings. A histogram of peak strains is generated with the collected data. These peak strains can be used to directly compute the bridge's reliability and indicate its adequacy. The rating methodology is still in development, but results to date show great promise. It is believed that this method can be easily incorporated into the biannual evaluation process. Because the method uses ambient traffic and requires a minimum number of guage locations, it is well suited for widespread application. In fact, this is the only method in Delaware that takes advantage of the new generation of reliability-based rating procedures.

SUMMARY

A bridge load-testing program comprising diagnostic field tests, in-service monitoring, and long-term monitoring has been established in Delaware to help DelDOT more effectively manage its bridge inventory. During the past 10 years, approximately 20 bridges have been tested in Delaware. Results from the testing have allowed DelDOT to remove load restrictions on a bridge on a major truck route and to improve passage of superloads on bridges on two of the major travel routes through the state. The demonstration of load-carrying capacity greater than expected has allowed DelDOT to postpone and reevaluate plans for bridge replacement and to eliminate extensive detour routes around bridges on rural secondary roads. Further tests have been used for

- Verifying nonstandard designs involving highly skewed bridges,
- Evaluating the performance of innovative materials (composites, high-performance concrete, and new types of rebar),
- Monitoring the accumulation of fatigue on a heavily traveled highway overpass, and
- Utilizing in-service strain data to compute bridge reliabilities.

Specifications for the design and evaluation of bridges have, by necessity, been developed to handle a wide range of bridge types. As such, the specifications tend to provide only conservative approximations of actual bridge response. While the results of a bridge test or in-service monitoring must be carefully evaluated, the measured in-situ response offers a great deal of insight into a bridge's actual condition. In Delaware, the implementation of an ongoing bridge capacity evaluation and monitoring program through field testing has already had a significant impact on the state's ability to manage its bridge inventory.

ACKNOWLEDGMENT

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Deterioration and Reliability

DETERIORATION AND RELIABILITY

Determination of Bridge Deterioration Matrices with State National Bridge Inventory Data

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This paper presents the results of a study using Louisiana's National Bridge Inventory (NBI) to determine matrices of the deterioration of bridge components (elements) that can be used in Pontis software. These matrices are for a simplified three-element bridge preservation model the authors developed for the Louisiana Department of Transportation and Development. The results indicate that the deterioration of bridge components does not correlate well with the age of bridges and that the Markov transition probability is a good tool for predicting bridge deterioration. When using state NBI data to generate Markov matrices of bridge components' deterioration, the resulting probabilities will be affected by the average bridge age in the database and the time intervals depending upon which NBI data are initially analyzed. This fact should be considered in the use of these deterioration matrices for predicting future bridge deterioration.

The 2000 NBI indicates that there are 7,075 on-system bridges in Louisiana with a total deck surface area of approximately 129 million ft^2, as shown in Figures 1 and 2. Of these bridges, about 69% are made of concrete, 19% of steel, and 12% of timber, and about 57% are of concrete, 42% of steel, and 1% of timber in the bridge-deck area. These bridges were built over many years, most in the 1950s through 1970s, as shown in Figure 3. This fact led to the question: When will the peak of the need for bridge rehabilitation and improvement occur in the next 20 or 30 years? To answer this question, the Louisiana Department of Transportation and Development (LA DOTD) needed a procedure to predict future bridge preservation requirements for meetzing the state's goals in bridge preservation. The ability to predict the magnitude of future bridge preservation needs is critical for updating the LA DOTD Strategic Plan and the Statewide Transportation Plan.

Similar to many other states' departments of transportation (DOTs), LA DOTD has adopted the Pontis Bridge Management Software (PBMS) for statewide bridge system management. PBMS is a comprehensive system that can assist agencies in allocating resources to protect infrastructure investments, ensure safety, and maintain mobility. It can provide an agency with a rich set of modeling and analyzing tools to support development of projects, budgets, and programs and can help an agency formulate networkwide preservation and improvement policies for evaluating the needs of each structure in a bridge network system (*1, 2*).

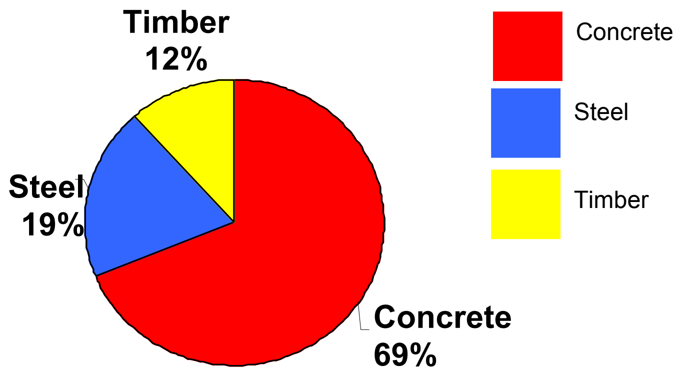

FIGURE 1 Percentage of bridges by category in Louisiana in 2000 (culverts excluded).

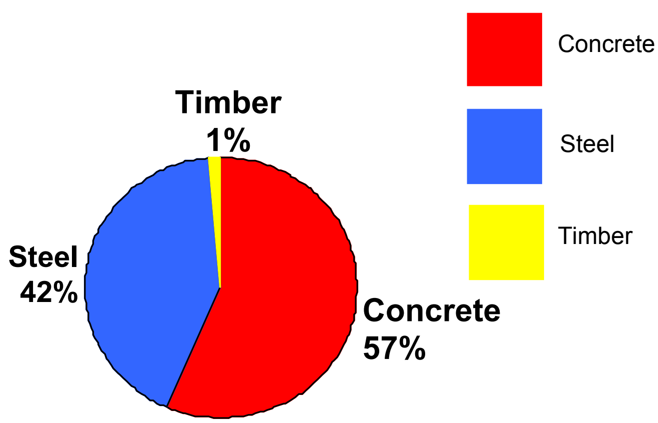

FIGURE 2 Percentage of bridge deck areas by category in Louisiana in 2000 (culverts excluded).

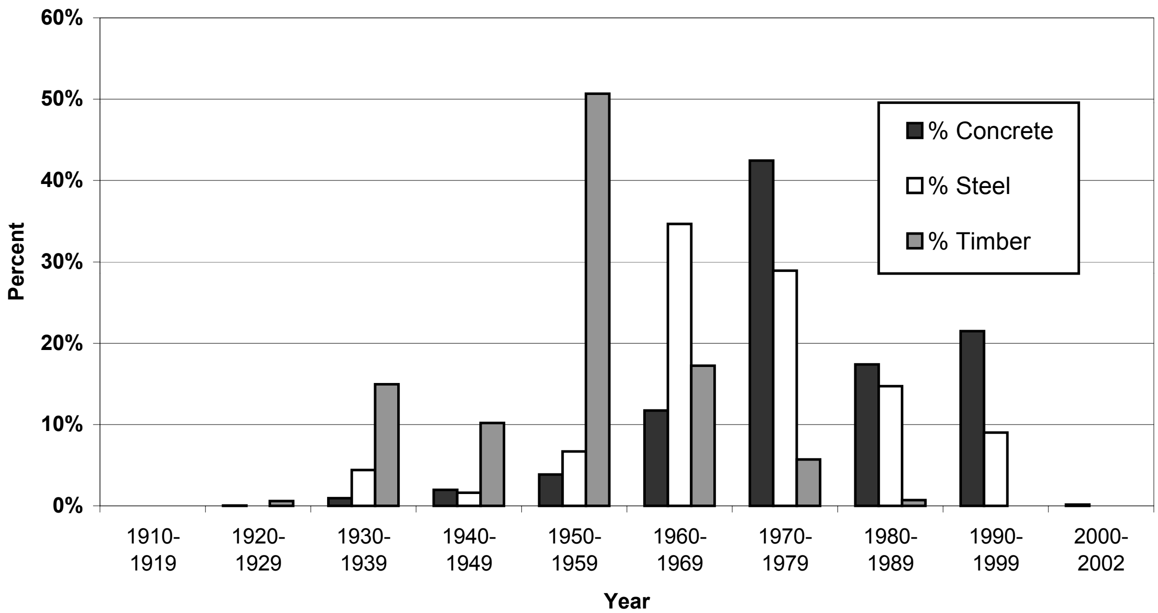

FIGURE 3 Bridge deck areas by different materials and years built in Louisiana.

To conduct its full functions, PBMS requires detailed bridge-inventory data with condition ratings at an element level (*2*). Like many state DOTs, LA DOTD is still in the process of collecting the required data, which is a very expensive and time-consuming task. In fact, there are no completed data available to input into PBMS for predicting future Louisiana's bridge preservation needs, although the data may become available in 3 to 5 years. However, over the years LA DOTD has accumulated current and historical NBI data, which include both bridge inventory and condition ratings. If PBMS could analyze these NBI data to predict future conditions, treatments, and budgets for bridge preservation at a statewide network level, LA DOTD could use this interim tool until the detailed element-level data of bridges are available and PBMS is fully implemented. For this purpose, the authors developed a simplified three-element (deck, substructure, and superstructure) bridge preservation model for PBMS. In this model, the NBI data are reconfigured so they can be directly analyzed by PBMS to predict future bridge preservation needs in line with LA DOTD's preservation goals. This paper will focus on generating the bridge-element deterioration matrices of the simplified model with the NBI ratings.

BRIDGE DETERIORATION

Bridge deterioration was studied by analyzing the historical NBI ratings generated during the past 20 years for all state on-system bridges. An NBI rating is designed to evaluate the conditions of three bridge components: deck, substructure, and superstructure. Table 1 shows the definition of NBI ratings by the FHWA. The research team first examined the quality and consistency of these data. All bridges in the database were divided into 30 subgroups according to structure types and materials, as summarized in Table 2. After initial analysis, the 30

TABLE 1 FHWA Definition of NBI Rating

9	**EXCELLENT CONDITION**
8	**VERY GOOD CONDITION:** No problems noted.
7	**GOOD CONDITION:** Some minor problems.
6	**SATISFACTORY CONDITION:** Structural elements show some minor deterioration.
5	**FAIR CONDITION:** All primary structural elements are sound but may have minor section loss, cracking, spalling, or scour.
4	**POOR CONDITION:** Advanced section loss, deterioration, spalling, or scour.
3	**SERIOUS CONDITION:** Loss of section, deterioration, spalling, or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.
2	**CRITICAL CONDITION:** Advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present, or scour may have removed substructure support. Unless bridge is closely monitored, closing the bridge may be necessary.
1	**IMMINENT FAILURE CONDITION:** Major deterioration or section loss present in critical structural components or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic, but corrective action but back in light.
0	**FAILED CONDITION:** Out of service, beyond corrective action.

TABLE 2 Louisiana Bridge Structure Inventory

No.	Type of Material	No.	Type of Constructure	Deck Area (kft2)					Number of Structures				
				1980	1985	1990	1995	2000	1980	1985	1990	1995	2000
1	Blank	1	Blank	-	-	-	54	1,887	-	-	-	13	85
2	0-OTHER	2	00-OTHER	259	4,809	5,830	12	6	131	174	141	4	2
3	1-CONCRETE	3	01-SLAB	12,734	11,159	12,303	13,017	13,987	2,392	2,586	2,753	2,886	3,049
		4	02-STRINGER	22,416	19,330	13,445	-	-	564	426	423	-	-
		5	04-TEE BEAM	6,068	4,200	4,126	3,695	3,627	474	356	343	326	308
		6	05-BOX BEAM	158	466	783	-	-	7	21	36	-	-
		7	11-ARCH	10	6	6	31	31	5	2	2	4	4
		8	18-TUNNEL	45	45	45	45	45	3	3	3	3	3
		9	19-CULVERT	2,391	2,507	3,054	3,180	3,211	1,086	1,117	1,225	1,278	1,334
		10	22-CHNL BEM	-	-	-	2	4	-	-	-	1	2
4	2-CONCRETE	11	01-SLAB	133	129	146	239	328	18	17	19	29	33
		12	04-TEE BEAM	161	285	673	646	833	9	28	54	67	73
		13	05-BOX BEAM	-	-	-	938	991	-	-	-	44	47
5	3-STEEL	14	00-OTHER	58	161	155	6	6	19	21	18	5	5
		15	02-STRINGER	44,031	39,492	28,434	22,723	25,985	538	543	547	825	789
		16	03-GIRDER	-	-	-	4,233	4,162	-	-	-	97	85
		17	09-TRUSS	500	-	-	94	598	2	-	-	1	3
		18	10-TRUSS	1,226	12,256	12,389	1,857	1,526	16	47	45	37	29
		19	15-MOV-LIFT	744	1,327	1,440	1,195	1,176	22	56	56	38	36
		20	16-MOV-BASC	1,214	1,296	1,294	945	934	9	11	10	8	7
		21	17-MOV-SWNG	211	1,047	1,070	1,088	1,134	13	63	63	70	71
		22	19-CULVERT	179	250	306	434	453	106	141	148	188	257
6	4-STEEL	23	02-STRINGER	546	5,358	11,826	3,362	4,316	41	97	131	97	115
		24	03-GIRDER	-	3,300	3,265	12,295	6,441	-	47	46	159	143
		25	05-BOX BEAM	-	-	-	3,950	4,402	-	-	-	29	52
		26	10-TRUSS	-	-	-	3,657	3,581	-	-	-	24	24
		27	14-STAYED	-	-	-	254	254	-	-	-	1	1
7	5-PRESTRES	28	02-STRINGER	48,842	40,239	45,022	35,717	43,525	705	782	953	1,055	1,102
8	6-PRESTRES	29	02-STRINGER	-	-	-	7,434	9,463	-	-	-	157	257
9	7-TIMBER	30	02-STRINGER	3,957	2,671	2,435	2,236	1,771	1,838	1,350	1,213	1,077	835
Sub-total				######	######	######	######	######	7,998	7,888	8,229	8,523	8,751

subgroups were aggregated to 4 large categories by type (materials and structure) and similarity in deterioration behavior. The four aggregated groups are concrete, prestressed concrete, steel, and timber bridges. The prestressed concrete bridges were separated from the general concrete bridges because of differences in their load-carrying mechanisms.

The deteriorations of bridge deck, substructure, and superstructure components were first explored by examining the correlations of their NBI ratings with bridge ages. Figure 4 provides an example of the correlations and shows that the conditional rating of bridge components does not correlate well with time. Therefore, a concept of Markov transition probability was used, and its empirical (sample) transition probabilities were determined by comparison with the corresponding NBI ratings at either 1- or 5-year intervals.

Figure 5 illustrates empirical (sample) transition probabilities for a concrete bridge deck at 5-year intervals. The transition probabilities were generated for bridge deck, substructure, and superstructure of different materials at 1-year or 5-year intervals. These are the probabilities of projected future ratings from any existing rating for different bridge components of different structures and materials. They are the combined results of historical events that occurred to bridges in the NBI database. These events include both the deterioration of bridges and past LA DOTD rehabilitation actions. These empirical (sample) probabilities reflect Louisiana's past practice and reveal information on bridge deterioration in the state.

TIME EFFECT ON TRANSITION PROBABILITY

As pointed out earlier, the conditional rating of bridge components does not correlate well with time, although time does affect bridge deterioration. Since many bridges in Louisiana are relatively young (see Figure 3), the results will be skewed to an underestimation of deterioration, if the transition probabilities used to predict future bridge preservation needs are based on NBI ratings from all bridges in the NBI database. Therefore, bridges older than 30 years were separated from the database, and their (sample) empirical transition probabilities were determined as shown in Figure 6.

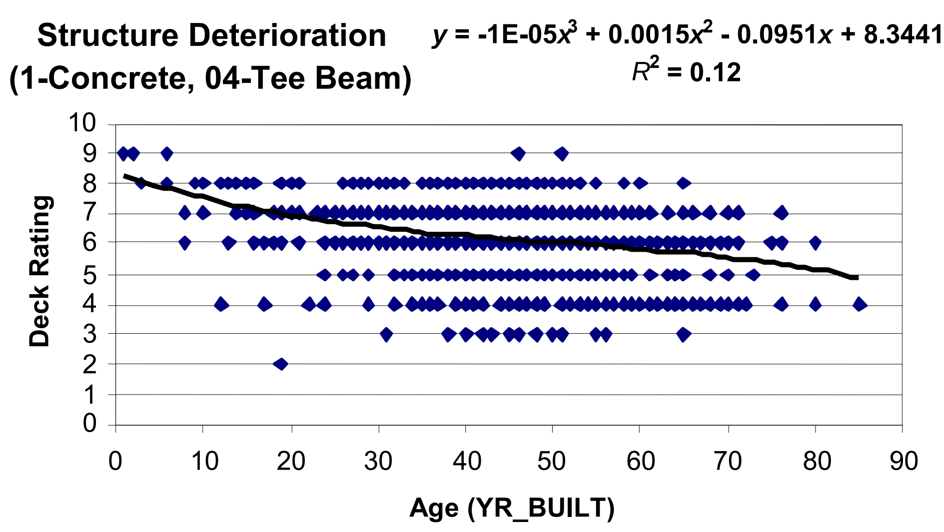

FIGURE 4 Correlation between bridge deck rating and age.

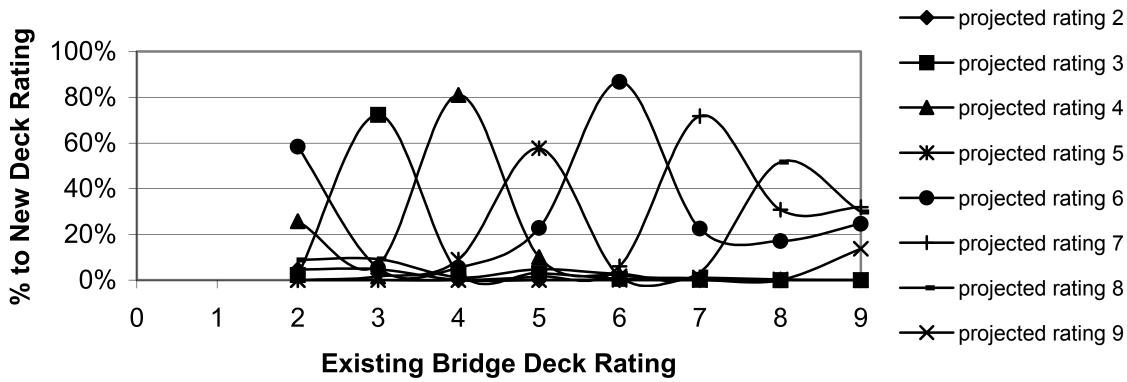

FIGURE 5 Empirical transition probabilities for concrete bridge deck at 5-year intervals.

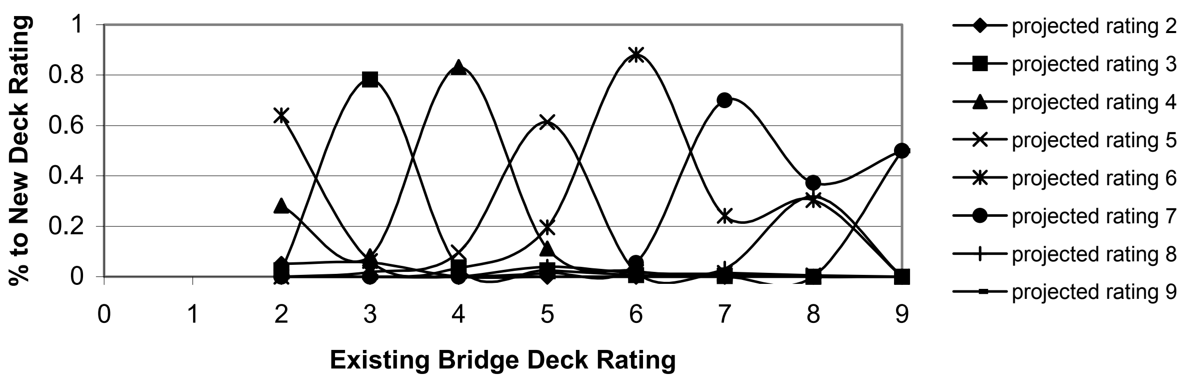

FIGURE 6 Empirical transition probabilities for concrete bridge deck for older bridges at 5-year intervals (data before 1970).

The difference between the transition probabilities of all and those older bridges became obvious when the probabilities were used to predict future bridge deterioration in Louisiana, assuming that LA DOTD practice remains the same. Figures 7 and 8 represent the 30-year projections for rating of decks for steel bridges using these two sets of transition probabilities at 5-year intervals. They indicate that older bridges deteriorate much faster than all bridges. Table 3 shows a summary of the comparison, which indicates that the total bridge-deck area with NBI ratings of ≤4 is about 5.6 million ft^2 if the transition probabilities from all bridges are used. However, the projected deck area is 19.2 million ft^2 if older bridges are used, a 245% increase.

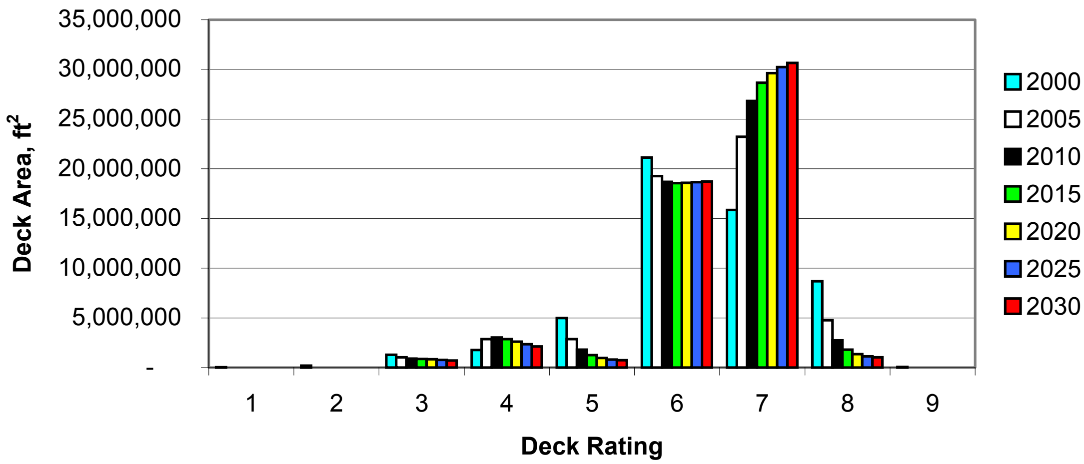

FIGURE 7 Deterioration of steel bridge deck over 30 years on basis of all bridge transition probabilities.

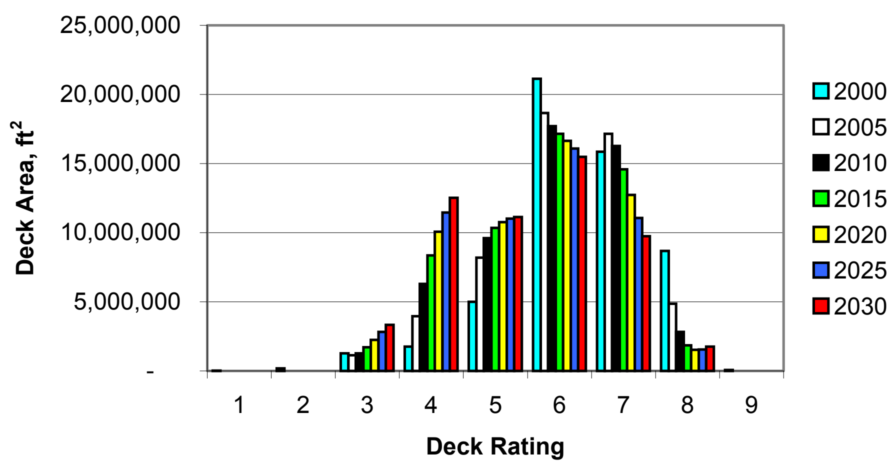

FIGURE 8 Deterioration of steel bridge deck over 30 years on basis of older-bridge transition probabilities.

TABLE 3 Comparison of Predicted Bridge Deck Deteriorations

	Deck Area with NBI Rating ≤4, 30-year Prediction		
Material Type	**Deterioration Based on Bridges Built Before 1970**	**Deterioration Based on All Bridges**	**Difference (%)**
Concrete, ft^2	2,391,000	1,853,000	29.0
Prestressed Concrete, ft^2	114,000	48,000	143.8
Steel, ft^2	15,867,000	2,844,000	457.9
Timber, ft^2	842,000	821,000	2.6
Total, ft^2	19,214,000	5,566,000	245.2

DEVELOPMENT OF DETERIORATION MATRICES

So far, the discussion has focused on bridge transition probabilities obtained from the NBI data. In this section, a method for developing deterioration matrices required by the PBMS simplified three-element preservation model is introduced. First, NBI ratings must be transformed to PBMS element condition ratings, as defined in Table 4. Figure 9 gives an example of such transformed transition-probability matrices on the basis of the transition probabilities shown in Figure 5. A corresponding deterioration matrix can then be developed through the steps described below.

Theoretically, a transition-probability matrix from transformed NBI ratings is:

$$P_t = \begin{bmatrix} p_{11} & p_{12} & p_{13} & p_{14} & p_{15} \\ p_{21} & p_{22} & p_{23} & p_{24} & p_{25} \\ p_{31} & p_{32} & p_{33} & p_{34} & p_{35} \\ p_{41} & p_{42} & p_{43} & p_{44} & p_{45} \\ p_{51} & p_{52} & p_{53} & p_{54} & p_{55} \end{bmatrix}_t$$

where $\boldsymbol{P}_{ij}$ is the transition probability with which the rating of a bridge component changes from i to j on a t-year interval. The diagonal probabilities $\boldsymbol{P}_{ij}$ ($i = j$) are the probability that bridge components stay with the same ratings after the t-year interval. The lower triangle ($i > j$) of the matrix indicates an increase in rating as a result of bridge maintenance or replacement (when j =1).

TABLE 4 Transformation Between NBI Rating and PBMS Element Condition Rating

Deck Element		Substructure and Superstructure Elements	
NBI Rating	Element Condition Rating	NBI Rating	Element Condition Rating
7–9	1	7–9	1
6	2	6	2
5	3	5	3
3–4	4	1–4	4
1–2	5		

$$\begin{bmatrix} .8243 & .1559 & .0084 & .0109 & .0005 \\ .0435 & .9052 & .0251 & .0253 & .0009 \\ .0390 & .1984 & .5919 & .1458 & .0249 \\ .0255 & .0249 & .0680 & .8705 & .0111 \\ .6765 & .1436 & .1287 & .0320 & .0192 \end{bmatrix}_{t=5}$$

FIGURE 9 All-bridge transition matrix of concrete bridge deck on 5-year interval (t = 5).

On the other hand, an element deterioration matrix is of the form:

$$\boldsymbol{D}_t = \begin{pmatrix} \boldsymbol{d}_{11} & \boldsymbol{d}_{12} & \boldsymbol{d}_{13} & \boldsymbol{d}_{14} & \boldsymbol{d}_{15} \\ 0 & \boldsymbol{d}_{22} & \boldsymbol{d}_{23} & \boldsymbol{d}_{24} & \boldsymbol{d}_{25} \\ 0 & 0 & \boldsymbol{d}_{33} & \boldsymbol{d}_{34} & \boldsymbol{d}_{35} \\ 0 & 0 & 0 & \boldsymbol{d}_{44} & \boldsymbol{d}_{45} \\ 0 & 0 & 0 & 0 & \boldsymbol{d}_{55} \end{pmatrix}_t$$

where $\boldsymbol{d}_{ij}$ is the probability with which a bridge component (element) deteriorates from a rating $\boldsymbol{i}$ to a rating $\boldsymbol{j}$ on a t-year interval. The deterioration matrix $\boldsymbol{D}_t$ was developed in this study from the transition probability matrix $\boldsymbol{P}_t$ as follows:

$$\boldsymbol{D}_t = \begin{pmatrix} \boldsymbol{p}_{11} & \boldsymbol{p}_{12} & \boldsymbol{p}_{13} & \boldsymbol{p}_{14} & \boldsymbol{p}_{15} \\ 0 & \boldsymbol{p}_{11} + \frac{1}{2}\boldsymbol{p}_{21} & \boldsymbol{p}_{23} + \frac{1}{6}\boldsymbol{p}_{21} & \boldsymbol{p}_{24} + \frac{1}{6}\boldsymbol{p}_{21} & \boldsymbol{p}_{25} + \frac{1}{6}\boldsymbol{p}_{21} \\ 0 & 0 & \boldsymbol{p}_{33} + \frac{3}{5}\sum_{1}^{2}\boldsymbol{p}_{3\boldsymbol{j}} & \boldsymbol{p}_{34} + \frac{1}{5}\sum_{1}^{2}\boldsymbol{p}_{3\boldsymbol{j}} & \boldsymbol{p}_{35} + \frac{1}{5}\sum_{1}^{2}\boldsymbol{p}_{3\boldsymbol{j}} \\ 0 & 0 & 0 & \boldsymbol{p}_{44} + \frac{7}{10}\sum_{1}^{3}\boldsymbol{p}_{4\boldsymbol{j}} & \boldsymbol{p}_{45} + \frac{3}{10}\sum_{1}^{3}\boldsymbol{p}_{4\boldsymbol{j}} \\ 0 & 0 & 0 & 0 & \boldsymbol{p}_{55} + \sum_{1}^{4}\boldsymbol{p}_{5\boldsymbol{j}} \end{pmatrix}_t$$

This is an empirical formula whose justification is based upon the consideration that historically, Louisiana had no bridge preservation program in place, and most actions on bridges were done through the federal bridge replacement program or by local maintenance crews in the state's different engineering districts. When a bridge is replaced, its components' new PBMS rating should be 1. In most cases, bridges were replaced when they were in very poor condition. On the other hand, interviews with local bridge inspectors indicate that routine maintenance could only raise the NBI rating by 1 in most cases; therefore, it could reduce the PBMS rating by 1. The suggested formula has simulated these two factors.

Another reason for adopting this formula is that it was developed through a trial and error process. Mathematically, it produced a reasonable convergence between 1-year and 5-year deterioration matrices that were developed independently of the original NBI rating data.

Figure 10 represents the deterioration matrix of concrete bridge decks on the basis of all concrete bridges at 1-year intervals. Figure 11 shows the corresponding 1-year deterioration matrix calculated from the 5-year transition probability matrix shown in Figure 9. A comparison of the results shown in Figures 10 and 11 indicates that the same source of data will not guarantee the same deterioration matrix. Therefore, the way of analyzing the NBI data also plays a role here. One reason for this phenomenon may be the quality of the NBI database. For

$$\begin{pmatrix} .9579 & .0382 & .0019 & .0015 & .0005 \\ 0 & .9724 & .0120 & .0118 & .0038 \\ 0 & 0 & .9498 & .0432 & .0070 \\ 0 & 0 & 0 & .9824 & .0176 \\ 0 & 0 & 0 & 0 & 1 \end{pmatrix}_{t=1}$$

FIGURE 10 All-bridge deterioration matrix of concrete bridge deck at 1-year interval (*t* =1).

$$\begin{pmatrix} .9620 & .0347 & .0015 & .0018 & .0000 \\ 0 & .9849 & .0076 & .0062 & .0013 \\ 0 & 0 & .9402 & .0444 & .0154 \\ 0 & 0 & 0 & .9905 & .0095 \\ 0 & 0 & 0 & 0 & 1 \end{pmatrix}_{t=1}$$

FIGURE 11 All-bridge deterioration matrix of concrete bridge deck at 1-year interval (*t* =1) calculated from 5-year deterioration matrix (*t* = 5).

$$\begin{pmatrix} .8065 & .1658 & .0119 & .0117 & .0041 \\ 0 & .8694 & .0512 & .0586 & .0208 \\ 0 & 0 & .7730 & .1882 & .0388 \\ 0 & 0 & 0 & .9150 & .0850 \\ 0 & 0 & 0 & 0 & 1 \end{pmatrix}_{t=5}$$

FIGURE 12 All-bridge deterioration matrix of concrete bridge deck at 5-year interval (*t* = 5) calculated from 1-year deterioration matrix (*t* = 1).

instance, each bridge in the system was inspected every other year. Although the difference is not very large at 1-year intervals, in some cases it can be quite dramatic at 5-year intervals, as shown in Figures 12 and 13. This fact must be considered when using deterioration matrices. For comparison, Figure 14 gives the deterioration matrix of concrete bridge deck on the basis of older concrete bridges at 1-year intervals, whereas Figure 15 demonstrates the corresponding 1-year deterioration matrix calculated from a 5-year transition probability matrix. Similar deterioration matrices were developed for bridge deck, substructure, and superstructure components of different materials, as defined previously.

$$\begin{pmatrix} .8243 & .1559 & .0084 & .0109 & .0005 \\ 0 & .9267 & .0327 & .0326 & .0080 \\ 0 & 0 & .7347 & .1931 & .0722 \\ 0 & 0 & 0 & .9534 & .0466 \\ 0 & 0 & 0 & 0 & 1 \end{pmatrix}_{t=5}$$

FIGURE 13 All-bridge deterioration matrix of concrete bridge deck at 5-year interval (*t* = 5).

$$\begin{pmatrix} .9483 & .0452 & .0036 & .0024 & .0005 \\ 0 & .9714 & .0124 & .0125 & .0037 \\ 0 & 0 & .9495 & .0437 & .0068 \\ 0 & 0 & 0 & .9835 & .0165 \\ 0 & 0 & 0 & 0 & 1 \end{pmatrix}_{t=1}$$

FIGURE 14 Older-bridge deterioration matrix of concrete bridge deck at 1-year interval (*t* = 1).

$$\begin{pmatrix} .9453 & .0485 & .0029 & .0033 & .0000 \\ 0 & .9846 & .0078 & .0064 & .0012 \\ 0 & 0 & .9403 & .0444 & .0153 \\ 0 & 0 & 0 & .9924 & .0076 \\ 0 & 0 & 0 & 0 & 1 \end{pmatrix}_{t=1}$$

FIGURE 15 Older-bridge deterioration matrix of concrete bridge deck at 1-year interval (*t* = 1) calculated from 5-year deterioration matrix (*t* = 5).

CONCLUSIONS

This paper introduces and explains the process of developing a simplified three-element bridge preservation model for PBMS in terms of bridge deck, substructure, and superstructure components. The process has followed the principles introduced by the PBMS manual (*2*), and the application has been tailored to Louisiana's situation. The following conclusions can be drawn from the above discussion:

- The deterioration of bridge components does not correlate well with the age of bridges. Therefore, the Markov transition probability is a good tool for predicting bridge deterioration.

- Historical state NBI data can be used to reasonably estimate future preservation needs for bridges on a statewide network level, assuming that a state DOT continues its past preservation practice for the highway bridge system. Markov transition probabilities determined with the state NBI data can be utilized to predict the future status of the bridge system within given budgetary constraints.
- When using state NBI data to generate Markov bridge component (element) deterioration matrices, the resulting probabilities will be affected by the average bridge age in the database and the time intervals, depending upon which NBI data are initially analyzed. In other words, there are no universal Markov deterioration matrices for bridge components, even within the same database. This may be true even for a future total PBMS element-level database after it is accumulated over time. This fact should be considered when these deterioration matrices are used to predict future bridge deterioration.

ACKNOWLEDGMENTS

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REFERENCES

1. *Transportation Research Board Circular 498: Presentations from the Eighth International Bridge Management Conference—Volumes I and II.* TRB, National Research Council, Denver, Colo., April 26–28, 1999.
2. *Pontis User's Manual, An AASHTOWare^{tm} Proprietary Computer Software Product, Release 4.1.* Cambridge Systematics, Inc., Mass., Aug. 13, 2001.

Synthesis of Element Condition Data for Aggregate Analyses in Bridge Management Systems

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DAVID MARKS
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In bridge management systems such as Pontis, owned by AASHTO, and the National Bridge Investment Analysis System (NBIAS), owned by FHWA, a preservation policy specifies the optimal action to be taken in each condition state for each structural element that may exist on a bridge. Application of an optimal preservation policy requires element-level data for the bridges in an inventory. Working with FHWA, Cambridge Systematics has developed NBIAS to analyze bridge needs at a national level with an element-level approach. To take advantage of the strengths of the element-level approach without the benefit of complete element-level data, Cambridge Systematics and FHWA have developed a set of models for synthesizing element-level data for individual bridges from information recorded for these bridges in the National Bridge Inventory. These synthesis, quantity, and condition (SQC) models use the available data to predict which elements are likely to exist on a bridge, the approximate quantities of each element, and their condition. The SQC models used in NBIAS were developed by combining engineering judgment and statistical analysis of a sample set of approximately 14,000 bridges. The results of this research may enable agencies to perform aggregate analysis of bridge needs without complete element-level data and to validate their element-level data collection programs.

A critical aspect of bridge management systems such as Pontis, owned by AASHTO, and the NBIAS, owned by FHWA, is their reliance on the development and application of an optimal element-level preservation policy. These systems' modeling approaches have been documented previously (*1–3*). In Pontis and NBIAS, the preservation policy specifies the optimal action to be taken in each condition state for each structural element that may exist on a bridge. The policy is formulated as a Markov decision problem and solved independently for each element. Figure 1, reproduced from the Pontis user's manual (*4*), shows how a bridge may be represented as a set of structural elements.

Application of an optimal preservation policy requires element-level data for the bridges in an inventory. Although many states in the United States collect inspection data at the element level, element-level inspections are not mandatory for compliance with National Bridge Inventory (NBI) standards and are not performed in all states.

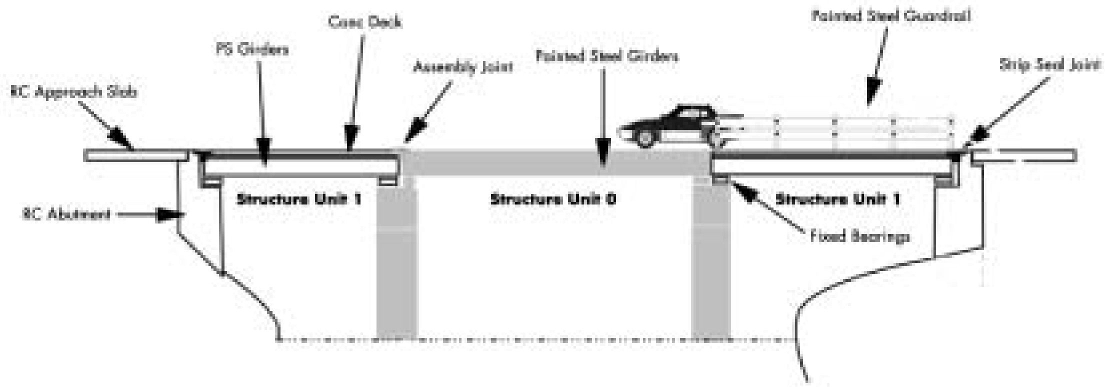

FIGURE 1 Representation of elements of structure (*4*).

Working with FHWA, Cambridge Systematics has developed NBIAS to analyze bridge needs at a national level with an element-level approach. To take advantage of the strengths of the element-level approach without the benefit of complete element-level data, Cambridge Systematics and FHWA have developed a set of models for synthesizing element-level data for individual bridges from the information recorded for these bridges in the NBI. These synthesis, quantity, and condition (SQC) models use the available data to predict which elements are likely to exist on a bridge and their approximate quantities and condition.

This paper details the development of the SQC models for NBIAS. The following sections detail the approach taken for developing the SQC models, describe the methodology for predicting the elements existing on the bridges in an NBIAS database, and offer a set of conclusions on the basis of the research.

NBIAS OVERVIEW

NBIAS is an analytical system that helps generate optimal long-term budgeting policies for bridge maintenance and improvement. Since 1999 FHWA has used the system to help determine national bridge needs for its biannual report on conditions and performance of U.S. highways, bridges, and transit (*5*). Although NBIAS was designed for analyses at a national level, the system has successfully performed network-level analyses at the state level. However, the system was not designed as a substitute for a full-featured bridge management system and is not intended for use in recommending bridge-level work.

NBIAS' distinctive features include

- Input of standard NBI data provided to FHWA by each U.S. state,
- A modeling approach similar to that used in the Pontis system to develop an optimal policy for maintenance, repair, and rehabilitation—preservation—at the element level with enhancements to better characterize user costs (*2*),
- SQC models for synthesizing bridge element data unavailable in NBI,
- Ability to evaluate needs for functional improvements (e.g., widening, strengthening, raising, or replacing a bridge) at the bridge level with the same approach as Pontis,
- Ability to predict more than 70 measures of effectiveness over a 30-year period for a range of different budget assumptions,

- Ability to consider the bridge network as a whole, stratifying its bridge population by functional class and classification on the National Highway System (although NBIAS does not generate bridge-specific project recommendations), and
- A powerful What-If Analysis module for varying budgets, measures-of-effectiveness, and the period of interest for a selected stratification of bridges in real time (*1*).

NBIAS versions 1.0 and 2.0 model preservation needs at an aggregate level and functional needs at the individual bridge level. This approach was taken largely to conform to the original system design's limitations in memory and processing speed. In 2001 Cambridge Systematics began designing NBIAS version 3.0 for FHWA, which was seeking to improve the quality of the system's results by modeling preservation needs at the bridge level. This change required significant additional memory and processing speed and also development of a data model with a relational database (rather than a set of binary and text files) to manage the additional data. However, this approach facilitates a more realistic evaluation of the trade-offs between preservation and improvement work, allows for more accurate calculation of user costs related to preservation, and allows FHWA to add measures of effectiveness in its analyses.

A prerequisite for supporting NBIAS bridge-level analysis was development of a new set of SQC models. The following section describes the SQC models developed for NBIAS 3.0.

SYNTHESIS, QUANTITY, AND CONDITION MODEL FORMULATION

Model Development Methodology

NBIAS SQC models predict whether an element exists on a bridge (synthesis), its quantity and its condition. Before version 3.0, NBIAS relied primarily on a set of discrete choice and statistical models for element synthesis and quantity prediction (*6*). The system uses AASHTO Commonly Recognized (CoRe) structural element definitions for its modeling (*7*).

In revising the synthesis and quantity models for NBIAS 3.0, a major objective was to develop models that

- Are statistically valid at an aggregate level,
- Provide a reasonable picture of the elements likely to exist on a bridge given limits of available data, and
- Yield results consistent with engineering judgment.

The original models, though valid at an aggregate level for predicting quantities of individual elements, are not suited for providing a logical picture of the combination of elements on a bridge. This limits the system's ability to simulate conditions and actions (which may involve work on a combination of elements) at the bridge level.

New synthesis and quantity models were developed using a 14,000-bridge database with NBI and Pontis element data for a representative sample of bridges from different regions of the United States. This database was developed as a separate effort and has been used for previous NBIAS analyses (*1*). The models were developed through statistical and empirical analyses and presented in tabular format as detailed in the technical design (*8*). In identification of causal variables, determination of how elements should be grouped for model development, and final review of the models, the analyses were informed by engineering judgment. For some elements, the sample database could not be used to develop an adequate model, if the quantity of the element in the database was too small to support model development, for example, or the predictive power of the resulting model was poor. In those cases, statistical analyses were supplemented by developing rules on the basis of engineering judgment.

A fundamental constraint is that only the data available through the NBI can be used for the SQC models. Table 1 lists the NBI data items found to be significant in developing the synthesis and quantity models. Definitions for these items are detailed in the NBI Coding Guide (*9*).

Table 2 lists the element groups for which models were developed and shows which NBI items were used for predicting synthesis and quantity for each element group defined for the analysis. Within each element group the models predict quantities for relevant CoRe elements. The table lists

- The variables used in predicting what elements exist on a particular bridge within each group,
- The variables used in predicting element quantities, and
- The units of measure identified in the element definitions for measuring quantities.

For instance, several types of joints are defined in the CoRe elements. The table shows that the models predict what joints exist on a bridge, if any, based on the main span design and material. Given this information, the quantity models predict joint quantities (measured in lineal meters of joints) on the basis of deck width, number of main and approach spans, and skew.

As indicated in table, main span design and material are the variables most commonly used for the synthesis models. Structure length, deck width, and the number of spans are most commonly used in the quantity models. In some cases (for girders, stringers, and bearings) main span design and material are used in the quantity models to determine the spacing of beams or girders, which is a key factor in the quantity of superstructure elements and bearings. The reviewers indicated that the absence of this variable in NBI was a significant limitation in model development.

TABLE 1 NBI Data Items in Synthesis and Quantity Models

NBI Item Number	NBI Item Name	SQC Variable Name
27	Functional classification	FUNCCLASS
34	Skew (degrees)	SKEW
42A	Type of service on bridge	SERVTYPON
42B	Type of service under bridge	SERVTYPUNDER
43A	Main span design type	DESIGN
43B	Main span material	MATERIAL
44	Number of approach spans	APPSPANS
45	Number of main spans	MAINSPANS
49	Structure length	LENGTH
52	Deck width (out-to-out)	DECKWIDTH
92A	Fracture critical	FRACCRIT
107	Deck structure type	DECK STRUCTURE TYPE
108A	Wearing surface protection system	SURFACE
108B	Type of membrane	MEMBRANE
108C	Deck protection	PROTECTION

TABLE 2 Explanatory Variables Used by Element Group

Element Group	Variables for Synthesis Models	Variables for Quantity Models	Quantity Units
Decks and Slabs	Main span design type Deck structure type Deck protection Wearing surface protection Type of membrane	Structure length Deck width	Square meters
Joints	Main span design type Main span material	Deck width Number of main spans Number of approach spans Skew	Meters
Girders, Stringers, and Floor Beams	Main span design type Main span material	Main span design type Main span material Structure length Deck width Fracture critical Skew	Meters
Trusses	Main span design type Main span material	Main span design type Main span material Structure length	Meters
Arches	Main span design type Main span material	Main span design type Main span material Structure length	Meters
Cables	Main span design type	Main span design type Structure length	Each
Railings	Main span design type Main span material Functional classification	Structure length	Meters
Pier Walls	Main span design type Main span material Service type under	Number of main spans Number of approach spans Deck width	Meters
Columns and Caps	Main span design type Main span material Service type under Number of main spans Number of approach spans	Main span design type Main span material Number of main spans Number of approach spans	Each (columns) Meters (caps)
Abutments	Main span design type Main span material Service type under	Deck width Skew	Meters
Bearings	Main span design type Main span material	Main span design type Main span material Deck width Number of main spans Number of approach spans Fracture critical	Each

Condition models were developed through statistical analysis of the element conditions that typically occur given a particular NBI deck, superstructure, or substructure rating. This approach is conceptually similar to that taken by the FHWA NBI Translator to predict element conditions (*10*). Models developed for previous versions of NBIAS with this approach were reviewed for NBIAS 3.0 and are detailed in the system technical design (*8*).

Synthesis Models for Decks and Slabs

Although providing detailed documentation on the resulting SQC models is beyond the scope of this paper, this section presents the synthesis models for decks as an example of the resulting models. These synthesis models are based on the assumption that each bridge contains a deck or slab of one type. Table 3 lists the deck and slab elements modeled in NBIAS. Table 4 details the synthesis logic used to determine the deck or slab most likely to exist on a bridge given its deck structure type, deck protection, wearing surface protection system, type of membrane, and main span design.

It is necessary to refer to the NBI data items listed in Table 1 and elements listed in Table 3 to interpret Table 4. For instance, if the deck structure type is a 3 (open grating), then the deck element is determined to be Element 28 (steel deck open grid). If the deck structure type is a 1 (concrete cast-in-place) and the deck protection is a 4 (cathodic), then the deck element is either 53 (concrete slab—protected with cathodic system) if the main span design is a slab, or 27 (concrete deck—protected with cathodic system). The model for the quantity of the deck is not shown in the table, but is a function of structure length and deck width.

TABLE 3 Deck and Slab Elements in SQC Models

Element Number	Element Name
12	Concrete Deck—Bare
13	Concrete Deck—Unprotected with asphalt concrete (AC) Overlay
14	Concrete Deck—Protected with AC Overlay
18	Concrete Deck—Protected with Thin Overlay
22	Concrete Deck—Protected with Rigid Overlay
26	Concrete Deck—Protected with Coated Bars
27	Concrete Deck—Protected with Cathodic System
28	Steel Deck—Open Grid
29	Steel Deck—Concrete-Filled Grid
30	Steel Deck—Corrugated or Orthotropic, etc.
31	Timber Deck—Bare
32	Timber Deck—With AC Overlay
38	Concrete Slab—Bare
39	Concrete Slab—Unprotected with AC Overlay
40	Concrete Slab—Protected with AC Overlay
44	Concrete Slab—Protected with Thin Overlay
48	Concrete Slab—Protected with Rigid Overlay
52	Concrete Slab—Protected with Coated Bars
53	Concrete Slab—Protected with Cathodic System
54	Timber Slab
55	Timber Slab—With AC Overlay

TABLE 4 Synthesis Logic for Decks and Slabs

Deck Structure Type	Protection	Surface	Membrane	Design	Predicted Element
1, 2	1, 2, 3	All		1 (slab)	52
				Other	26
	4 (cathodic)	All		1 (slab)	53
				Other	27
	Other	0, 1, 2, 8, 9, N	All	1 (slab)	38
				Other	12
		6	0, N	1 (slab)	39
				Other	13
			All except 0, N	1 (slab)	40
				Other	14
		3, 4	All	1 (slab)	48
				Other	22
		5	All	1 (slab)	44
				Other	18
		7	All	1 (slab)	48
				Other	22
3	All				28
4	All				29
5	All				30
6	All				30
7, 9	All			1 (slab)	38
				Other	12
8	All	0, 7, 8, 9, N	All	1 (slab)	54
				Other	31
	All	1, 2, 3, 4, 5, 6	All	1 (slab)	55
				Other	32
N	All				None

Application of Models to Sample Bridges

Tables 5 and 6 demonstrate the application of the resulting models to two sample bridges selected for illustrative purposes. Each table shows the predicted and actual quantity of each element on the bridge. Note that elements not modeled in NBIAS, such as smart flags and piles, have been excluded from the tables.

Table 5 shows predicted and actual quantities for a concrete continuous T-beam bridge. As shown in the table, the model predictions closely match actual results for the deck, open girder/beam, and column elements. The model incorrectly predicts that the bridge has concrete railing (it actually has a railing of type “other”) but provides a plausible estimate of railing

TABLE 5 Predicted and Actual Element Quantities for Concrete Continuous Tee Beam Bridge

Element	Units	Predicted Quantity	Actual Quantity
12—Concrete Deck Bare	Square meters	861	860
110—Reinforced Concrete Open Girder/Beam	Meters	413	413
205—Reinforced Concrete Column or Pile Extension	Each	20	24
215—Reinforced Concrete Abutment	Meters	31	0
234—Reinforced Concrete Cap	Meters	63	0
301—Pourable Joint Seal	Meters	108	42
331—Reinforced Concrete Bridge Railing	Meters	138	
333—Other Bridge Railing	Meters	0	157

TABLE 6 Predicted and Actual Element Quantities for Steel Stringer/Girder Bridge

Element	Units	Predicted Quantity	Actual Quantity
26—Concrete Deck Protected with Coated Bars	Square meters	1,948	1,950
107—Painted Steel Open Girder/Beam	Meters	274	549
205—Reinforced Concrete Column or Pile Extension	Each	12	6
215—Reinforced Concrete Abutment	Meters	28	27
234—Reinforced Concrete Cap	Meters	43	40
301—Pourable Joint Seal	Meters	71	0
302—Compression Joint Seal	Meters	0	77
311—Moveable Bearing	Each	8	20
313—Fixed Bearing	Each	8	12
331—Reinforced Concrete Bridge Railing	Meters	274	262

quantity. The model incorrectly predicts that the bridge has abutments and caps and significantly overpredicts the quantity of joints.

Table 6 shows predicted and actual quantities for a steel stringer/girder bridge. The model correctly predicts the existence of all the bridge elements, with the exception of the joints. For joints the model predicts the bridge has pourable joint seals, but the bridge actually has compression joint seals. The predicted quantities are plausible for the deck, abutment, caps, joints, and railings. The predicted quantities are high for the columns and systematically low for the open girder/beams and bearings. The underprediction of these elements' quantities results from an incorrect prediction of girder/beam spacing.

When applied to the 14,000 bridge sample database, at the aggregate level the SQC models closely match actual element quantities. Further, the models result in predictions of needs in NBIAS 2.0 very similar to those resulting from the prior set of models. Qualitatively speaking, the new SQC models are a significant improvement over the previous SQC models, because they tend to plausibly predict what elements exist on a bridge from the available NBI data, without compromising predictions of element quantities at an aggregate level. In particular, the models tend to perform excellently in predicting data for bridge decks and slabs, the elements that are typically most significant in determining element-level preservation needs. However, for elements besides decks and slabs, the quantities predicted for individual bridges can vary significantly from the actual quantities. In developing NBIAS 3.0, additional work will quantify the impact of the differences in the predicted and actual quantities of elements on network-level bridge needs.

CONCLUSION

This paper describes the development of models for synthesizing element conditions used in NBIAS. The models developed for NBIAS 3.0 result in aggregate predictions of element quantities and conditions consistent with previous models developed for the system. Further, the SQC models developed for NBIAS 3.0 represent a significant improvement over prior models for synthesizing element conditions at the bridge level. Additional analysis will be required to determine how well needs modeled in NBIAS 3.0 compared to needs modeled using real rather than synthesized element data.

Element synthesis models are no substitute for collection of element-level data for supporting bridge management. However, the models described here can be used to compare actual and predicted element quantities. States could also use the models as part of a quality assurance program to identify cases in which element data may require further review. In addition, appropriate synthesis models of bridge-level elements enable future design of an NBIAS version that can accept element-level data as input where available and use element synthesis models as an alternative when such data are unavailable.

ACKNOWLEDGMENTS

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REFERENCES

1. Gurenich, D., and N. J. Vlahos. Network-Level Bridge Management Systems for National Road Administrations. In *Transportation Research Circular 49: Proc., Eighth International Bridge Management Conference, Denver, Colorado, April 26–April 28, 1999*, TRB, National Research Council, Washington, D.C., 2000.
2. Robert, W. E., D. I. Gurenich, and C. Comeau. Consideration of User Costs in Markov Decision Processes Used for Bridge Management. In *Bridge Maintenance, Safety and Management*, International Association for Bridge Maintenance and Safety, International Center for Numerical Methods in Engineering, Barcelona, Spain, 2002.
3. Cambridge Systematics, Inc. *Pontis Release 4 Technical Manual*. AASHTO, Washington, D.C., 2001.

4. Cambridge Systematics, Inc. *Pontis Release 4 User's Manual.* AASHTO, Washington, D.C., 2001.
5. *1999 Status of the Nation's Highways, Bridges and Transit: Conditions and Performance—Report to Congress*. FHWA and FTA, U.S. Department of Transportation, 2000.
6. Cambridge Systematics, Inc. *National Bridge Investment Analysis System (NBIAS) Technical Design.* Technical report prepared for FHWA. U.S. Department of Transportation, 1997.
7. Subcommittee on Bridges and Structures of the Standing Committee of Highways. *AASHTO Commonly Recognized (CoRe) Structural Elements*, AASHTO, Washington, D.C., 1998.
8. Cambridge Systematics, Inc. *National Bridge Investment Analysis System (NBIAS) Version 3.0 Technical Design.* Technical report prepared for FHWA. U.S. Department of Transportation, 2002.
9. *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges*. Report No. FHWA-PD-96-001. FHWA, U.S. Department of Transportation, 1995.
10. Hearn, G., J. Cavallin, and D. M. Frangopol. *Generation of NBI Condition Ratings from Condition Reports for Commonly Recognized Elements*. Technical report prepared by the University of Colorado. Colorado Department of Transportation, Denver, 1997.

Updating Time-Dependent Reliability of Highway Bridges with Pontis Bridge Management System Inspection Results

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Time-dependent reliability analyses are increasingly used to predict structural performance and develop an optimum lifetime maintenance strategy for structural systems, including highway bridges. These analyses require a significant amount of input data and predict structural behavior over several decades. The input data must be updated through inspection. While targeted nondestructive evaluation inspections are preferred, they can be expensive and are not available for all bridges. Biannual visual inspection programs are in place for the nation's highway bridges, and states use bridge management systems (BMSs) to ensure public safety and prioritize the use of scarce maintenance funds. To update the reliability analysis of a highway bridge, the data in the current Pontis BMS must be revised for use of data from visual inspection. Suggestions include a segment-based inspection that gives the analyst the location of defects on the bridge. Also proposed are conservative assumptions, numerical quantification of condition-state descriptions, and linear deterioration of condition states over time. The suggestions were illustrated on a sample highway bridge with defects that can be observed and those that cannot. The strengths and limitations of the proposals were assessed.

The 1991 Intermodal Surface Transportation Efficiency Act requires state transportation departments to implement bridge management systems (BMSs) to more efficiently plan maintenance, monitor bridge conditions, and allocate resources. The results for the nearly 600,000 bridges and culverts in the National Bridge Inventory are reported to FHWA. Most states have adopted the Pontis BMS, which relies on biennial visual inspections of every bridge. Pontis assigns condition ratings to bridge elements, such as railings, joints, or decks; types of materials such as concrete, steel, or timber; and other relevant information such as protected or unprotected decks, open or closed girders, and painted or unpainted stringers (*1*).

At the same time, analysis and design methods for structures have become more rigorous as uncertainty of loads and resistances are quantified to a greater degree. Both component and system reliability methods are gaining in acceptance and usage in evaluating structures. These two trends have progressed simultaneously and independently. A time-dependent reliability analysis for a structure requires assumptions about the load model, resistance model, and deterioration model. These models project the future and are the basis for optimum life-cycle inspection and repair planning. The models must be updated over time to revise the optimum maintenance strategy on the basis of how the structure actually behaves.

The best sources of data are targeted nondestructive evaluations (NDEs) taken at optimum intervals that investigate the defect or parameter that requires updating. Because regular visual inspections are conducted and the data are recorded, can these results be used to update the reliability of the structure? Currently the answer is no, largely because visual inspections were developed for a different purpose and different parties have not communicated. This paper addresses how the current Pontis BMS data can be revised so visual inspection data can be used to update a bridge's reliability analysis. A segment-based inspection is required that gives the analyst the location of the bridge's defects. A reliability analysis can be updated with conservative assumptions and numerical quantification of condition-state descriptions. These proposals are illustrated below on a sample Colorado highway bridge, and the strengths and limitations of the proposals are assessed.

COLORADO HIGHWAY BRIDGE

The results of a Pontis bridge inspection were used to update the time-dependent reliability of Bridge E-17-AH, a typical simply supported, nine-girder, standard-shape highway bridge in the metropolitan area of Denver, Colorado. Figure 1 shows the cross section of the bridge superstructure. The nine steel girders supporting a concrete deck are classified as interior (I), exterior (E), and interior-exterior (I-E). The bridge was modeled as a series-parallel system of 16 separate failure modes. An optimum strategy for inspection and repair was developed for the bridge from a time-dependent system reliability analysis that used an extreme-value live-load model and a deterioration model for chloride penetration of the concrete slab and atmospheric corrosion of the steel girders. Estes and Frangopol (*2*) present the complete details of this analysis.

Corrosion on Steel Girders

In the Pontis system, each element is visually inspected by a trained inspector and classified into one of five condition states, although some elements have fewer condition states (*1*). Element 107 (*3*), shown in Table 1, is used to determine the amount of corrosion on the steel bridge girders. Although the degree of thickness loss is helpful, two problems emerge in updating reliability with this inspection data. The results report linear feet of girder in each condition state but do not specify on which girder or where on a girder the corrosion is found. If the corrosion is located on the web near the support, the shear failure mode is most affected. If the corrosion is on the flanges in the middle of the beam, the moment limit state is most affected. Secondly, the corrosion should be assessed probabilistically if it is to be used effectively in a reliability analysis. Although this is not possible from visual inspection, some conservative assumptions can be drawn that will at least supply a worst-case reliability assessment and may prompt a more detailed and focused NDE inspection.

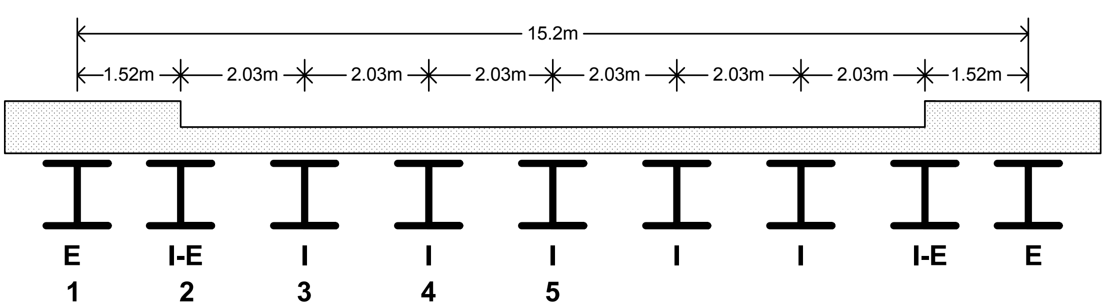

FIGURE 1 Layout of girders in Bridge E-17-AH.

TABLE 1 Colorado Department of Transportation Suggested Condition State (CS) Ratings for Element 107, Painted Open Steel Girders

CS	Description	Rust Code	% Section Loss[a]	Density Distribution[a]
1	No evidence of active corrosion; paint system sound and protecting the girder	—	0–2	Lognormal
2	Slight peeling of paint, pitting, or surface rust, etc.	Light R1	0–5	Normal
3	Peeling of paint, pitting, surface rust, etc.	R1	0–10	Normal
4	Flaking, minor section loss (<10% of original thickness)	R2	10–30	Normal
4	Flaking, swelling, moderate section loss (>10% but <30% of original thickness). Structural analysis not warranted	R3		
5	Flaking, swelling, moderate section loss (>10% but <30% of original thickness). Structural Analysis not warranted due to location of corrosion on member	R3	>30	Lognormal
5	Heavy section loss (>30% of original thickness); may have holes through base metal	R4		

[a] Not part of Pontis definition; created to quantify observed corrosion.

To improve the current Pontis inspection, Renn (*4*) suggested a segment-based inspection by which condition-state ratings are applied to specific locations on the bridge structure. The revision is totally compatible with Pontis condition states and flags, requires little additional documentation, and adequately addresses the deficiency of location. The bridge is divided into small, easily defined segments, and each segment is rated separately. The condition rating for each segment is recorded on a drawing of the structure, so the location of a defect is part of the inspection report. A complete segment-based inspection was conducted on Bridge E-17-AH, and the partial results for corrosion of the girders (Element 107) are shown in Figure 2.

Assuming linear condition-state deterioration over time and normal distribution of deterioration intensity, a probabilistic, yet still conservative, model was developed. It was assumed that a bridge element is initially at the halfway point of a specific condition state and progressively shifts to the right over time. The standard deviation of the distribution is determined by the quality of the inspection program. The study considered three possible inspection programs with inspectors qualified as very experienced, experienced, and inexperienced with 95%, 85%, and 75% expected correct ratings, respectively (*5*). The quality of the inspection program was determined by seven criteria (*6*). For very experienced inspectors, the density distributions associated with CS1 through CS5 for girder corrosion (Element 107) are shown in Figure 3. The condition state definitions for CS1, CS2, and CS3 were modified to reflect section losses of 0% to 2%, 0% to 5%, and 0% to10%, as shown in Table 1.

With the randomness of the corrosion parameters conservatively defined, the section loss was estimated, and the area of the web and plastic section modulus at the time of inspection were computed. The results were used to revise the girders' shear and moment capacities and update the girders' reliabilities. Table 2 shows the reliabilities of selected shear and moment limit states on the girders on the basis of the original deterioration model and the results of a segment-based

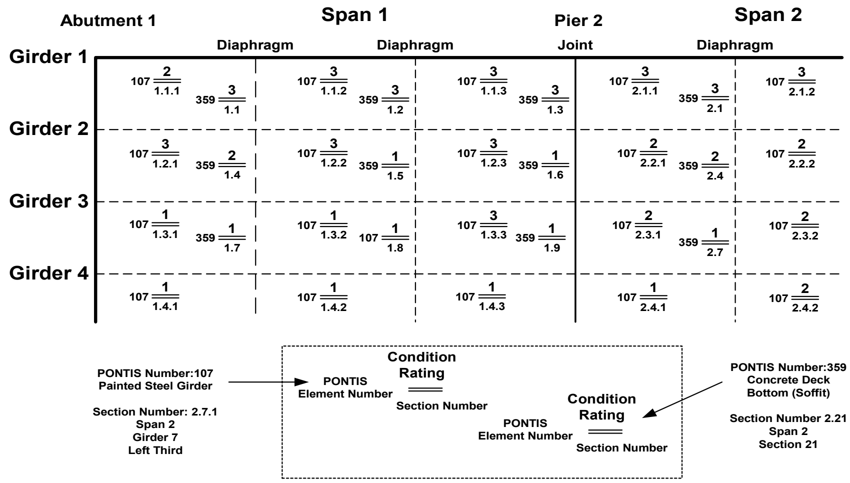

FIGURE 2 Segment-based sample inspection results for Bridge E-17-AH: bottom of concrete deck and superstructure.

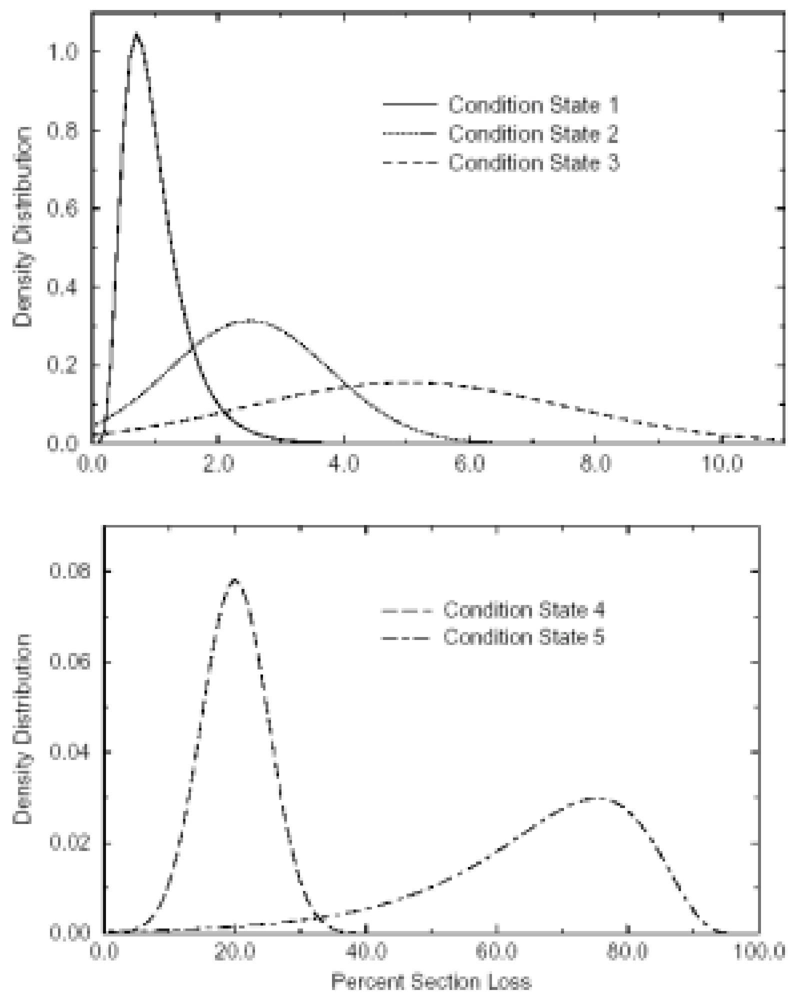

FIGURE 3 With very experienced inspectors: probability density distributions for CS1 through CS5 for girder corrosion.

TABLE 2 Comparison of Girder Reliability Index, β, on Bridge E-17-AH with Inspection Results and Deterioration Model Prediction

Girder Number and Failure Mode	Condition State	Girder Type	Reliability Index			
			Model Prediction β	Inspection Results		
				Very Experienced β	Experienced β	Inexperienced β
V-1	3	E	1.73	6.81	6.67	5.76
V-2	3	I-E	1.57	5.62	5.28	4.60
V-3	3	I	5.88	5.36	5.02	4.43
M-1	3	E	4.11	4.38	4.38	4.37
M-2	3	I-E	2.60	2.78	2.78	2.78
M-3	2	I	2.55	2.54	2.53	2.53

inspection with very experienced, experienced, and inexperienced inspectors. The notations V-1 and M-1 indicate the shear and moment failure modes on Girder 1 as labeled in Figure 1. The deterioration model for the interior girders tracks well with the inspection results for both shear and moment. The deterioration model for shear on the exterior girders clearly must be revised, as the girders are not deteriorating as quickly as expected.

Concrete Slabs and Girders

Concrete slabs and girders pose a greater challenge. A common failure mechanism is corrosion of the embedded reinforcement as a result of a critical concentration of chlorides that have penetrated the concrete. The reinforcement cannot be directly observed. The only option in a visual inspection is to infer the randomness of section loss of the reinforcement from the observed areas, observing number and width of cracks, degree of efflorescence, and percentage of surface spalls.

Thoft-Christensen (*7*) has developed a relationship between crack width on the surface of a concrete slab or beam and the section loss of the corroding reinforcing member. Specifically, the relationship between the increase in crack width Δw over a period of time can be related to the reduction in reinforcement diameter ΔD by the following equation:

$$\frac{1}{2}\left(\frac{D/2}{D/2+c}+1\right)c\Delta w = (\alpha - 1)\pi D\frac{\Delta D}{2} \tag{1}$$

where

D = original bar diameter,

c = distance from the concrete surface to the steel reinforcement, and

α = ratio of density of the corrosion rust product to the density of the reinforcing steel (typical values are between 2 and 4).

The Pontis condition states for concrete beams are sufficiently quantified that with some conservative assumptions, the amount of section loss can be inferred from the inspection results. The deterioration model can be updated for a revised reliability of the concrete structure. Table 3 shows the condition states for cracks in mildly reinforced concrete girders, Element 105 (3).

With the same assumptions made earlier for inspectors' experience levels, entering the condition state at the halfway point, and linear condition-state deterioration over time, each condition state was quantified probabilistically given the width of the observed crack. Table 4 describes these distributions for a concrete member in flexure for very experienced, experienced, and inexperienced inspectors. In this table, μ is the mean deviation and σ the standard deviation of the crack width.

Using the relationship described in Equation 1, the condition states can be probabilistically quantified for the reduction in bar diameter ΔD for a specific structure. Such results were obtained for the concrete bridge deck on Bridge E-17-AH for the bar diameters and concrete cover shown in Figure 4. Monte-Carlo simulation was used with the crack width Δw, the concrete cover c, and the relative material densities α all random variables. Figure 5 shows the resulting probability distribution for the four condition states of this bridge deck. With this information, the deterioration model was revised and the reliability updated. As expected, the uncertainty associated with this bar loss is high. A high degree of uncertainty is associated with both the condition-state definition and the model that relates crack width to section loss. The result will be an overly conservative reliability rating that would likely trigger an NDE inspection.

In the typical model for this deterioration, chlorides penetrate through the concrete until a critical concentration is reached at the steel reinforcement and corrosion ensues. The steel reinforcement corrodes at a predicted rate, and the resulting section loss progressively reduces the structure's capacity. A visual inspection yields no information until a crack appears in the concrete. If chloride penetration time has been overestimated, a visual inspection offers no information that can update the model. The engineer only knows that a crack should have appeared if the model was correct. Useful information is only available only at a late point in the deterioration process when cracks can be observed.

TABLE 3 Pontis Condition State Definitions for Concrete Slabs and Beams for Observable Crack Width (w) (mm) in Mildly Reinforced Concrete Girders

Type of Crack	None	$\leq$0.8 mm	$0.8 < w \leq 2$	$2 < w \leq 2.5$	$2.5 < w \leq 3$	$w > 3$ mm
Shear	1	2	2	3	4	4
Flexure	1	1	2	3	4	4
Diagonal	1	2	2	3	3	4

TABLE 4 Probabilistic Definitions of CS for Concrete Slabs and Beams Subjected to Bending for Inspectors Classified as Very Experienced, Experienced, and Inexperienced

Concrete Slabs and Beams (Flexure)						
Condition State	Crack Width Range (mm)	μ mm	σ mm Very Experienced	σ mm Experienced	σ mm Inexperienced	Density Distribution
1	0–0.8	0.4	0.204	0.278	0.348	Lognormal
2	0.8–2	1.4	0.306	0.417	0.522	Normal
3	2–2.5	2.25	0.128	0.174	0.217	Normal
4	>2.5	3.25	0.383	0.521	0.652	Lognormal

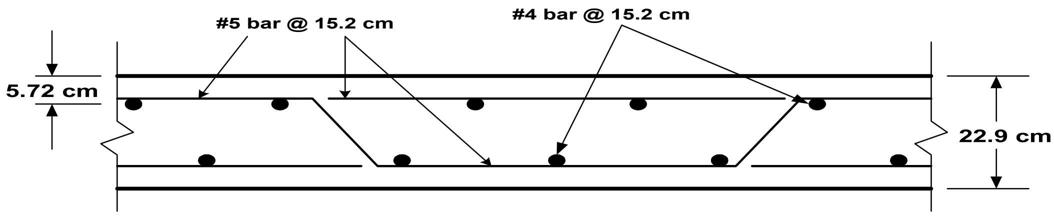

FIGURE 4 Concrete deck for Bridge E-17-AH.

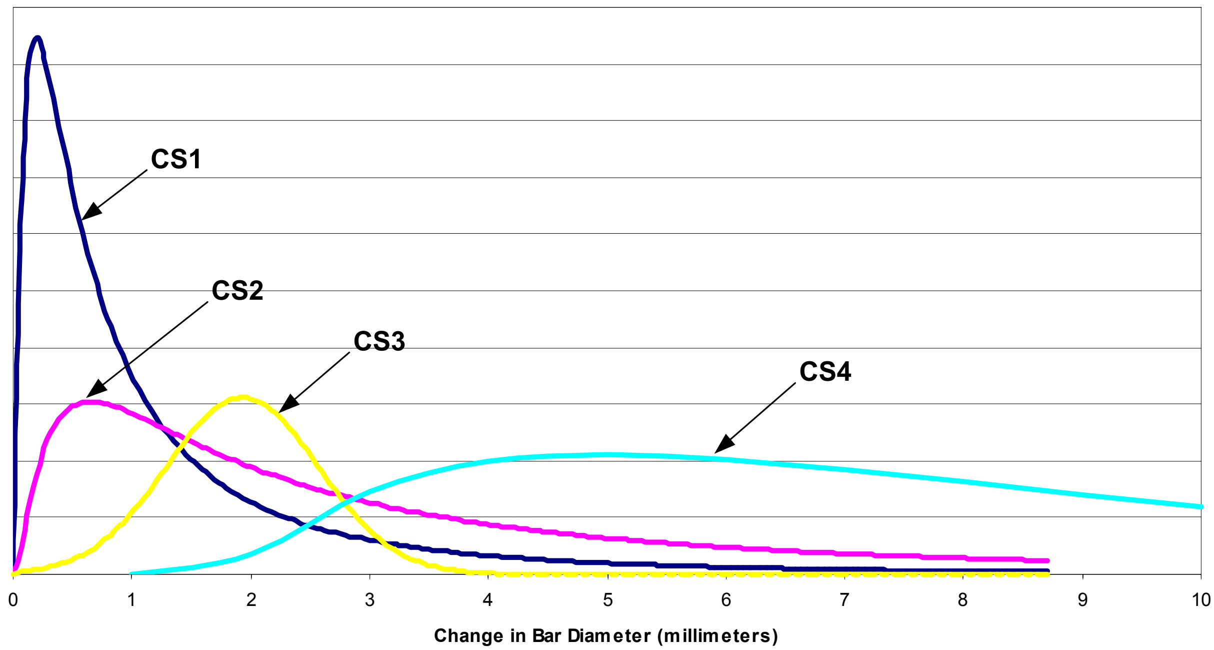

FIGURE 5 Probability density distributions for reduction in bar diameter for concrete deck in Bridge E-17-AH from visual inspection of concrete crack width at deck surface.

CONCLUSION

This paper has shown how data from visual inspections can be used to update deterioration models in probabilistic terms for the purposes of revising a reliability analysis on the basis of actual structural performance. Some deterioration can be directly observed (steel girder corrosion), whereas some must be inferred from observed results (concrete reinforcement corrosion). Other deterioration mechanisms, such as fatigue cracking, are worthy of future study. Fatigue has the same limitation as the chloride penetration model; nothing can be observed early in the deterioration life. Once a fatigue crack is seen, a crack growth model can be applied, but that occurs very late in the structure's usable life.

Although information from visual inspection is not a valid substitute for an NDE inspection, it can be used to conservatively assess and update a reliability model in some cases. Because inspections are conducted every 2 years, a consistent performance record is available for each bridge that, if used wisely, can be extrapolated to predict future performance. The key to success is numerically quantified definitions of condition state. Conservative assumptions can then be made to quantify probabilistically the condition states. Communication between those who develop condition-state definitions and those who use that information is important. Given its ready availability, visual inspection information should be used effectively whenever possible.

ACKNOWLEDGMENT

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REFERENCES

1. Thompson, P. D., E. P. Small, M. Johnson, and A. R. Marshall. (1998). The Pontis Bridge Management System. *Structural Engineering International*, Vol. 8, No. 4, 1998, pp. 303–308.
2. Estes, A. C., and D. M. Frangopol. Repair Optimization of Highway Bridges Using System Reliability Approach. *Journal of Structural Engineering*, Vol. 125, No. 7, 1999, pp. 766–775.
3. *BMS/Pontis Bridge Inspection Manual*. Colorado Department of Transportation, Denver, 1995.
4. Renn, D. P. *Segment-Based Inspection for Load Rating Within Bridge Management Systems*. Masters thesis. University of Colorado, Boulder, 1995.
5. Estes, A. C. *A System Reliability Approach to the Lifetime Optimization of Inspection and Repair of Highway Bridges*. Ph.D. thesis. University of Colorado, Boulder, 1997.
6. Frangopol, D. M., and A. C. Estes. Lifetime Bridge Maintenance Strategies Based on System Reliability. *Structural Engineering International*, Vol. 7, No. 3, 1997, pp. 193–198.
7. Thoft-Christensen, P. Corrosion Crack Based Assessment of the Life-Cycle Reliability of Concrete Structures. In *Proc., Eighth International Conference on Structural Safety and Reliability June 17–22, 2001* (R. B. Corotis, G. I. Schueller, and M. Shinozuka, eds.), A. A. Balkema Publishers, Newport Beach, California, 2001.

Utilizing the Internet

UTILIZING THE INTERNET

Web-Based Bridge Management System

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A web-based bridge management system (BMS) is a system built by combining the geographic information system (GIS) and the Internet. The latest technological advances in hardware and networking have led to a tremendous increase in Internet usage in the United States. Considering the popularity of the World Wide Web, a web-based system is an ideal means for information dissemination. The stakeholders, elected officials, and the public need to have access to information, and this Internet-based system is an efficient way to reach a large number of users at a low cost. Evaluated here is the use of ArcIMS, an off-the-rack software package, to build a web-based BMS. ArcIMS helps in distributing geographic information over the Internet, which allows for real-time integration of data. A web-based system helps departments share data, potentially paving the way for a comprehensive asset management system.

Bridge management systems (BMSs) are decision support tools developed to assist agencies in determining how and when to make bridge investments that will improve safety and preserve existing infrastructure. BMSs combine engineering principles with sound business practices and economic theory, and they provide tools to facilitate a more organized, logical approach to decision making. The need for an efficient BMS is emphasized by increasing system demands, personnel constraints of transportation agencies, and pressures arising from the constraints on availability of funds. Another key issue facing the agencies is accountability to the public. Increased public skepticism and a preference for using private-sector management approaches in the public sector have forced the agencies to be more accountable (*1*). The stakeholders, state government officials, managers concerned with day-to-day operations, private-sector organizations, and the general public need access to information. Though BMS does act as a fact-based dialogue between system users and information-seekers, the demand for information is ever increasing. A web-based BMS, with a combination of the geographic information system (GIS) and the Internet, can prove to be a dynamic tool for information dissemination.

WHY THE INTERNET?

The Internet is the most effective source to disseminate information in a timely and cost-efficient manner. With the appearance of the World Wide Web in the 1990s, the Internet has become the largest repository of information. Ease of use has increased its user base tremendously over the last few years (Figure 1). The following facts give further proof of the impact of the Internet (*2*):

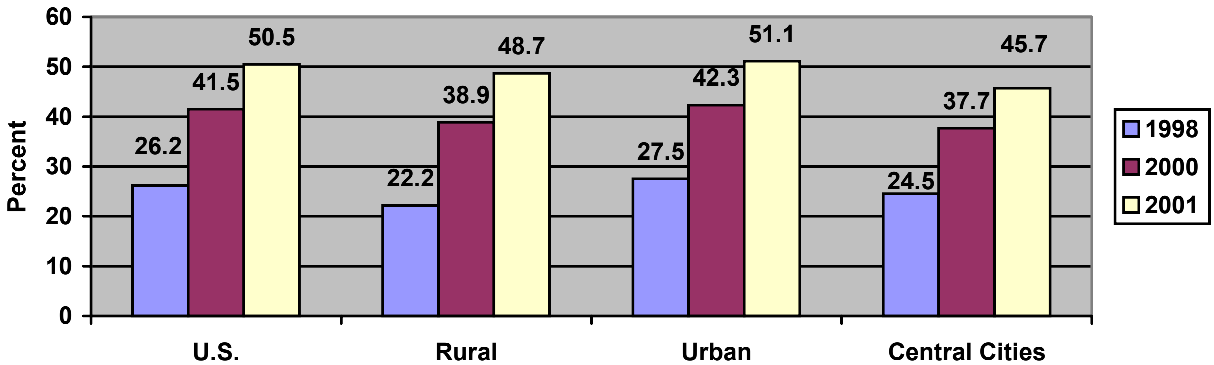

FIGURE 1 Percentage of U.S. households with Internet access.
(Source: U.S. Department of Commerce; U.S. Census Bureau Population Survey)

- The number of Internet users in the United States rose from 122 million to 165 million, an increase of 15%.
- Internet users per capita (number of users per 1,000) has increased twofold from 203.4 to 406.5 in a span of 3 years (1997–2000).
- Sixty-eight million Americans have used government websites to research policy issues and send comments, 28 million more than 2 years ago.
- Cable subscription to high-speed Internet access in the United States is expected to reach 10 million by the end of year 2002 (Figure 2).

GIS and Internet

By definition, GIS is an organized collection of computer hardware, software, geographic data, and personnel designed to efficiently capture, store, update, manipulate, analyze, and display all forms of geographically referenced information (*3*). GIS basically contains a database and spatial analysis programs for data manipulation. The spatial programs handle user requests to produce the results in maps and tables to facilitate decision making. A GIS contains two types of information: attribute data and spatial data. The attribute data are assembled and added to a database that can be manipulated to perform database operations and analysis as well as create graphic output in the form of reports. The spatial data use a coordinate system and three graphic objects: points, lines, and polygons. Because of the amount of data processed by GIS, the data were restricted initially to mainframe computers. But the latest advances in technology in the field of microprocessors have brought GIS to personal computers and workstations (*4*).

A robust BMS must not only contain information on bridge characteristics but also the relationship of each bridge to the overall infrastructure system. Consequently, an important attribute of a BMS is the geographic location of a bridge and other associated infrastructure systems. Nowadays data can be gathered more quickly with higher quality and spatial accuracy than ever before. Data collected using the Global Positioning System accurately give the spatial component required for analysis.

Although dissemination and analysis of information over the Internet are efficient and time-effective in reaching a multitude of users, it was a problem to perform these operations with transportation data in graphical format. A GIS-based system allows the data to be shared within

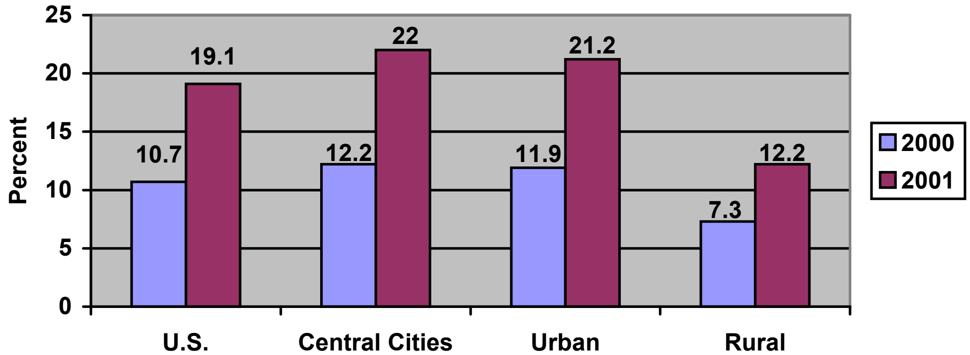

FIGURE 2 Higher speed Internet connection by geographic area as percentage of total Internet households. (Source: U.S. Department of Commerce; U.S. Census Bureau Population Survey)

and among various departments. Each bridge has many attributes and associated pieces of information attached to it. It is vital that all the information regarding the bridge be managed and coordinated efficiently. The graphical display of information using the Internet facilitates information sharing, communications, and management.

Web-Based BMS

The technologies used to create a web-based management system can be classified into two broad categories: off-the-rack software packages and application development modules. This paper discusses the use of ArcIMS, an off-the-rack package created by the Environmental Systems Research Institute (ESRI) for the BMS (*5*). The use of a software package instead of application development greatly reduces the start-up costs and time. With the advances in the software packages there is no compromise in the functionality of the system.

ArcIMS helps in distributing geographic information over the Internet, which allows for real-time integration of data. Users can combine data and information accessed over the Internet with local data for display, query, and analysis. ArcIMS provides a diverse set of mapping, location analysis, and routing capabilities for location service developers (Figure 3).

ArcIMS operates in a distributed environment that consists of both client-side and server-side components. Typically, the client requests information from an Internet or intranet server. Then the server processes the request and sends the information back to the client viewer.

Key Features of ArcIMS

According to ESRI (*5*), the following are the key features of ArcIMS:

- Standard data types. ArcIMS supports industry-standard GIS data types, which include coverages, shapefiles, and a variety of graphic images.
- Multiple available clients. ArcIMS offers a variety of ready-to-use templates and an option to customize existing ArcIMS clients.
- Highly scalable. ArcIMS allows serving data through single or multiple servers, which makes the system highly scalable.
- Multiplatform. ArcIMS runs on Windows NT 4.0 and Sun Solaris operating systems,

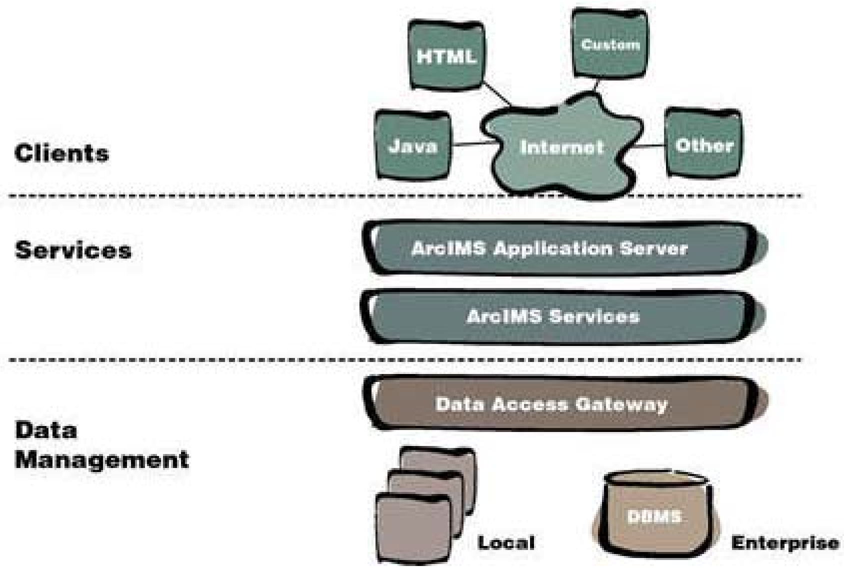

FIGURE 3 ArcIMS architecture. (Source: www.esri.com)

which makes integration easy.

- Easy to install. Simple instructions guide people through the authoring and publishing process.
- Serve data as you want. Delivered using HTML or Java; ArcIMS gets the best data and services as needed.

Making a Basic ArcIMS Site

A simple, three-step process creates maps for the web that have GIS functionality (*6*). ArcIMS has three predefined templates—one HTML and two Java based—which are fully customizable. ArcIMS contains four applications: Author, Administrator, Designer, and Manager. Author, Administrator, and Designer can be used sequentially to quickly set up a website. Author creates an AXL file that is used by Administrator to start a MapService. The MapService is used by Designer to generate the website. Manager, available on Windows NT, is a web-based application that incorporates Author, Administrator, and Designer for remote use of ArcIMS.

Step 1—Authoring a Map Configuration File

In Author, data from shapefile, ArcSDE layers, and image files are compiled into map layers. Scale dependencies, feature rendering, geocoding, and provisions for stored queries are set in this step. The output of the Authoring process, a Map Configuration file, contains HTML-like tags that provide all instruction necessary to render the map so that it can be published on the Internet.

Step 2—Starting a MapService

In the second step, Administrator takes the output from Author (the ArcXML file) and creates a MapService. A MapService makes the contents of the Map Configuration file (i.e., the data layer content and symbology) accessible through the web.

Step 3—Designing the Website
The wizard interface in Designer queries the user for website preferences and generates the HTML and JavaScript files needed to support the site. Three predefined templates—one HTML viewer and two Java viewers—come with ArcIMS. The final output from Designer is a website directory containing HTML files, images, JavaScript, and metadata information.

SUMMARY

Graphic display formats effectively communicate vast amounts of information on the projects. The availability of information on the Internet allows the public to understand the department policies and gives them an opportunity to voice their opinions. Rehabilitation and maintenance work on the bridges are invariably accompanied by traffic delays. Display of such information on the Internet allows people to look for alternate routes and avoid delays. The web-based BMS could be extremely helpful in case of weather-related events. In such circumstances, analysis with GIS could identify potentially hazardous structures and warn people to avoid them. Providing timely information can help reduce delays and enhance the safety of the public. A web-based BMS could be a significant step by the agencies in building a comprehensive asset management system encompassing all infrastructure.

REFERENCES

1. *Asset Management Primer*. FHWA, U.S. Department of Transportation, 1999.
2. How Many Online? http://www.nua.com/surveys/how_many_online/index.html. Accessed Sept. 20, 2002.
3. *Understanding GIS: The ARC/INFO Method.* ESRI, Redlands, Calif., 1990.
4. Ran, B., B. P. Chang, and J. Chen. Architecture Development for Web-Based Geographic Information System Applications in Transportation. In *Transportation Research Record: Journal of the Transportation Research Board, No. 1660,* TRB, National Research Council, Washington, D.C., 1999, pp. 114–121.
5. *ESRI ArcIMS-GIS for Information Technology Departments*. ESRI, Redlands, Calif., 2001.
6. Kieren, T. D. Creating GIS_Enabled Websites Using ArcIMS. http://www.esri.com/news/arcuser/1000/arcims.html. Accessed Sept. 18, 2002.

Bridge Management Through the Internet with DANBROweb

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RAMBØLL

The DANBROweb bridge management system (BMS) focuses on managing the day-to-day activities related to bridge maintenance. Consequently, access to continuously updated information is essential. DANBROweb has been developed as a 100% web-based client–server system that permits all parties involved in the management of the structures to access the same data. The Inventory Module provides relevant technical and administrative data on the structures, supplemented with photos, technical drawings, specifications, and other kinds of digital information. The structures are defined in a six-level element hierarchy. Technical and administrative properties can be registered on all element levels, tailored to each type of element. The Principal Inspection Module holds information on the condition of the bridges and their individual components, thus providing an overview of the condition of the entire bridge stock and identifying the bridges in need of rehabilitation. The Activity Management Module keeps track of all inspection, maintenance and rehabilitation works, budgets, and costs. The general idea of activity management is that a job goes through different stages of execution and the person responsible for each stage changes the status of the job accordingly. The value of immediate access for everybody to continuously updated information is significant in two areas. In activity management it is essential that everybody involved has access to updated information on job statuses. In a distributed organization the concept ensures that the centrally located administration has a continuously updated overview of the structures, their condition, and the funding needs.

Bridge management involves many kinds of activities, ranging from simple registering of inventory data to priority ranking and long-term budgeting using advanced deterioration models and optimization algorithms. Typically, the advanced functions such as optimization and long-term budgeting are used by a limited number of specialists in bridge administration, and the basic inventory and activity management modules are used by a large number of persons with varying educational and practical backgrounds.

DANBROweb is a bridge management system (BMS) intended to be a flexible, practical tool for everyone involved in the maintenance management of bridges or other structures. This paper introduces the web technology and describes the three main modules of DANBROweb—Inventory, Principal Inspection, and Activity Management—and the integrated document management. The paper provides an example of an easy-to-use BMS that covers the needs of everyone involved in the day-to-day maintenance management of bridges, by providing continuously updated information accessible to all.

WEB-BASED BRIDGE MANAGEMENT

The general idea of web-based management is that all users access the same database, as illustrated in Figure 1, using the Internet or an intranet for communications. DANBROweb comprises a detailed user administration giving each user access to see and edit exactly the information needed for accomplishing tasks.

Through the World Wide Web (WWW) a standard client program called a web browser can be used to access all kinds of data on servers throughout the world. Today, all users have computers with web browsers (such as MS Internet Explorer or Netscape) and a more or less permanent connection to the Internet.

The most common use of the Internet is to provide and to acquire information. Companies, institutions, authorities, and individuals put information at the disposal of anyone who wants it. Users throughout the world seek and read the data. However, as the use of the WWW has increased, more advanced programs have been developed that permit two-way communication between browsers and server applications. This means that the users at the client side of the network are not restricted to reading the data made available by the servers; they can change existing data and add new data to the server database.

DANBROweb is a 100% web-based system. Since the client program—the browser—is already on the user's computer, nothing needs to be installed before the system can be used. Or more precisely, the client program—a Java applet—is downloaded to the client computer via the browser every time the system is opened, ensuring that the user is always working with the newest updated program version. And, since the database is located on the server, there is no need to worry about exchange or backup of data. In other words, as seen from the viewpoint of the common user, program updating, data exchange, and backup are automatic.

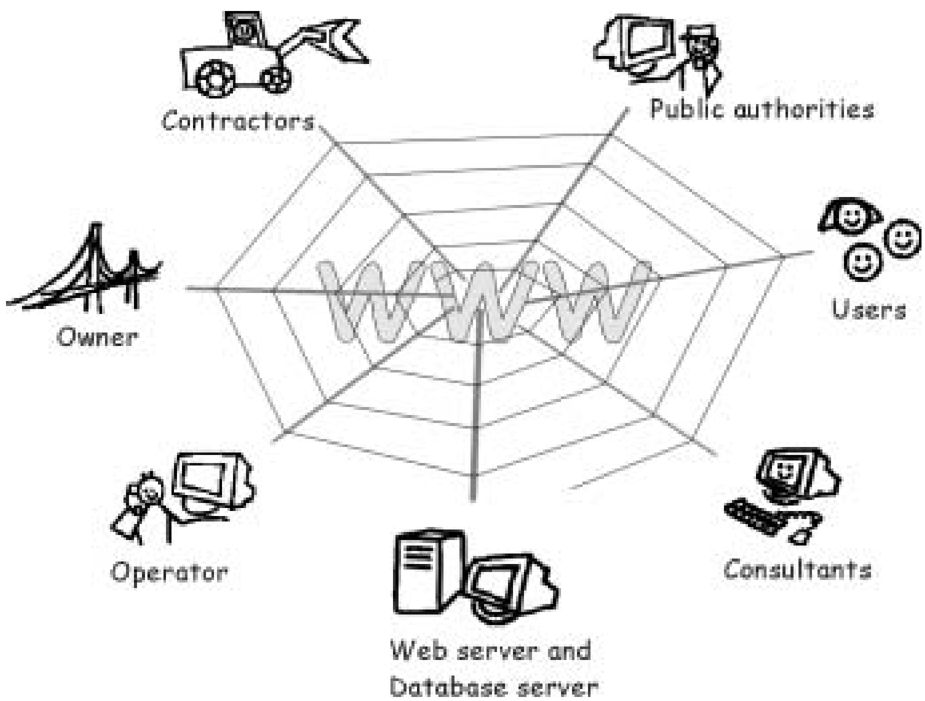

FIGURE 1 All users access the same program and database through the Internet or an intranet if external access is not necessary or wanted.

INVENTORY

The structures to be managed by the system are defined in a six-level element hierarchy as illustrated in Figure 2. The hierarchy may be used to register a number of small bridges (as shown in the figure) or to register a large infrastructure complex, e.g., a large suspension bridge, including approach bridges, toll collection station, and installations.

The actual element hierarchy is based on a parallel hierarchy of prototypes defining which data to register on different element types. Figure 3 shows an example of the properties that can be registered on a normal bridge. (In addition to the data shown, other data fields are to be found on the other tab sheets, Administrative Data and Passage Data). The bridge can contain subelements with their own registered properties, providing different data fields for different element types. The prototype concept makes it possible to manage different types of structures (e.g., minor bridges, large complex bridges, and tunnels) with different properties—in the same installation.

Documents (in a broad sense meaning all kinds of digitally stored information) can be attached to any element on any level. The documents are divided into categories such as photos, inspection reports, drawings, and specifications. The documents can be viewed on screen or printed. This means that the Inventory Module can act as the complete archive for all information on the bridges. Files can be uploaded to the DANBROweb server, or links can be established to existing files in their original location. Finally, a document can be a link to an internal or external website, e.g., the home page of a supplier.

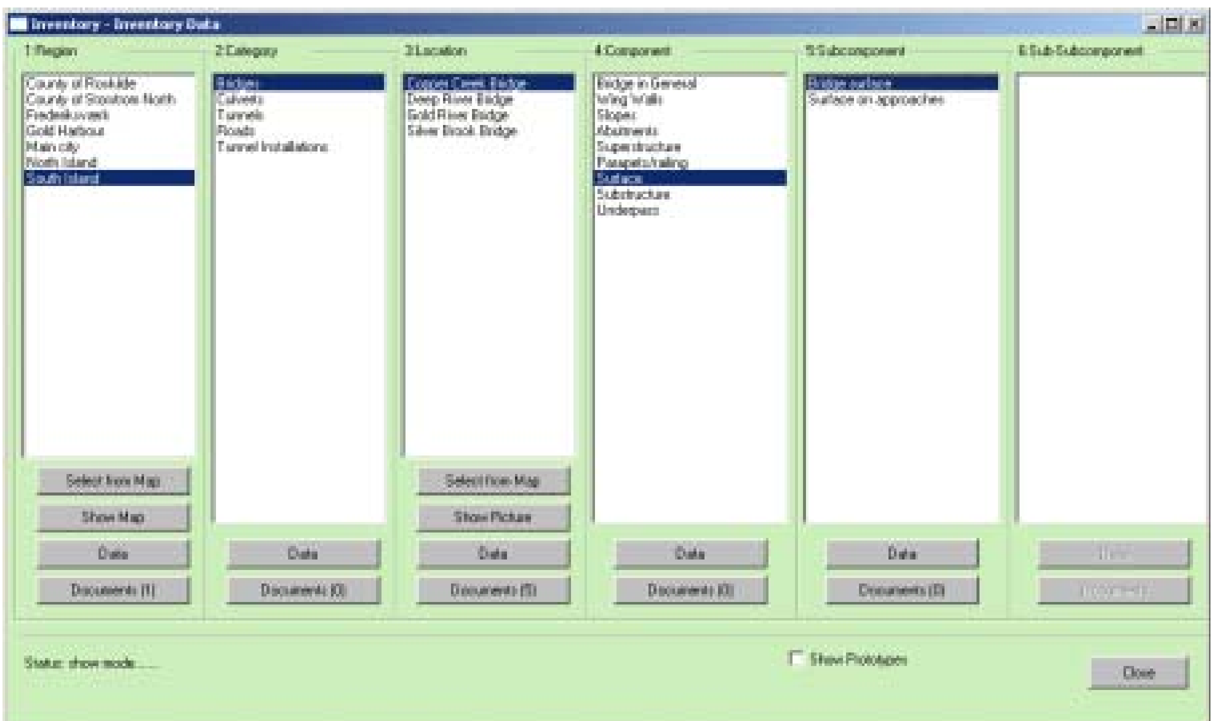

FIGURE 2 Six-level hierarchical element structures. Data and documents can be attached to elements on all levels by means of the buttons under each column.

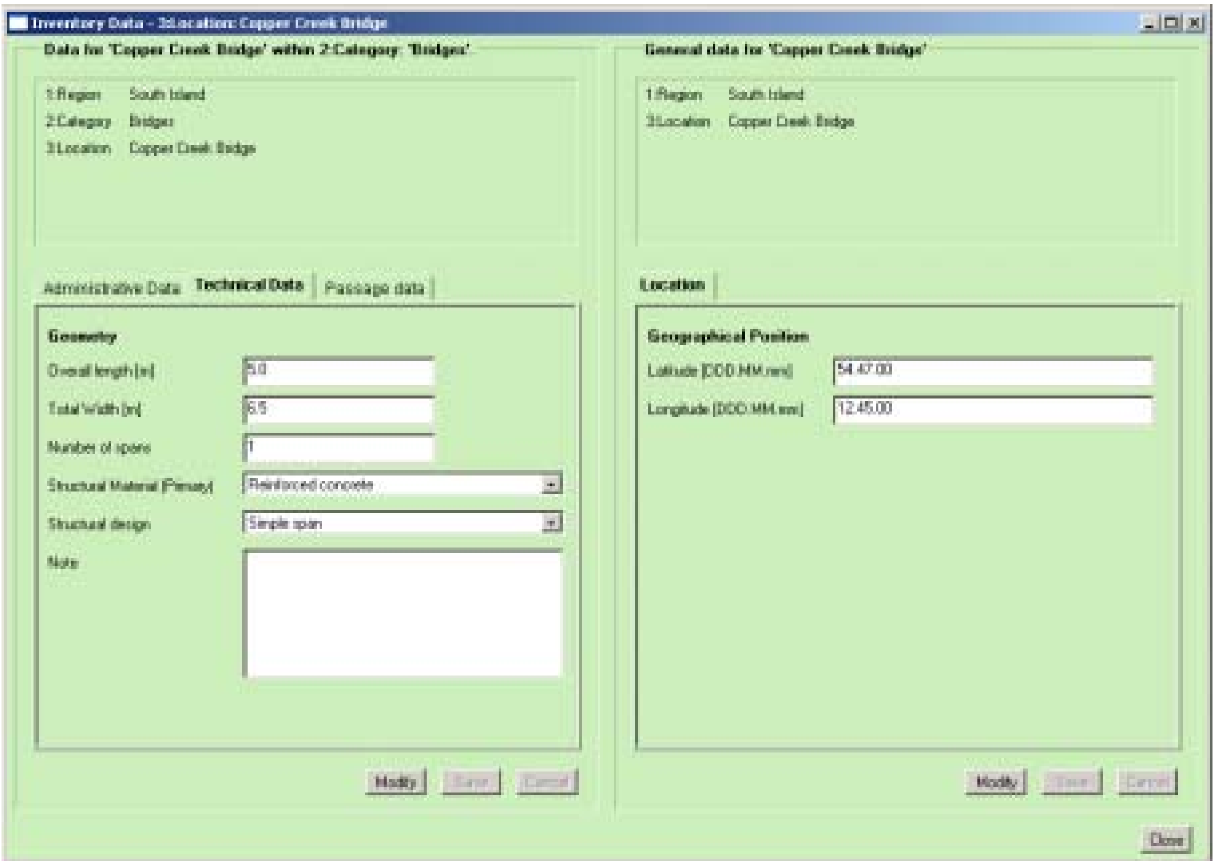

FIGURE 3 Sample inventory data for a small bridge.
Tab sheets and data fields are defined on user level.

PRINCIPAL INSPECTION

The principal inspection—a visual inspection of all accessible parts of the bridge—is the key activity in monitoring the condition of bridges. The purpose is to maintain an overview of the general condition of the entire stock of bridges, and to reveal significant damage in due time, so that rehabilitation works can be carried out in the optimum way and at the optimum time.

In the Principal Inspection Module of DANBROweb (see Figure 4), each bridge and each of its main components is assigned a condition rating ranging from 0 (no damage) to 5 (ultimate damage and complete failure of the component), and when it is relevant, a short description of damage and one or more photos are added. Through shortcuts to the Activity Management Module, needs for inspections and repair works are reported as requested jobs (see the following section on activity management). All activities on the bridge or a user-specified selection of them can be shown in the principal inspection screen. Thereby the screen provides an overview of the condition of the bridge and its components and the activities planned to remedy the registered damage.

ACTIVITY MANAGEMENT

In this context, a "job" is a specific piece of work that needs to be carried out in connection with the maintenance of bridges. It could be routine maintenance, such as cleaning of drains or removal of growth; it could be a major rehabilitation work, such as replacement of bridge

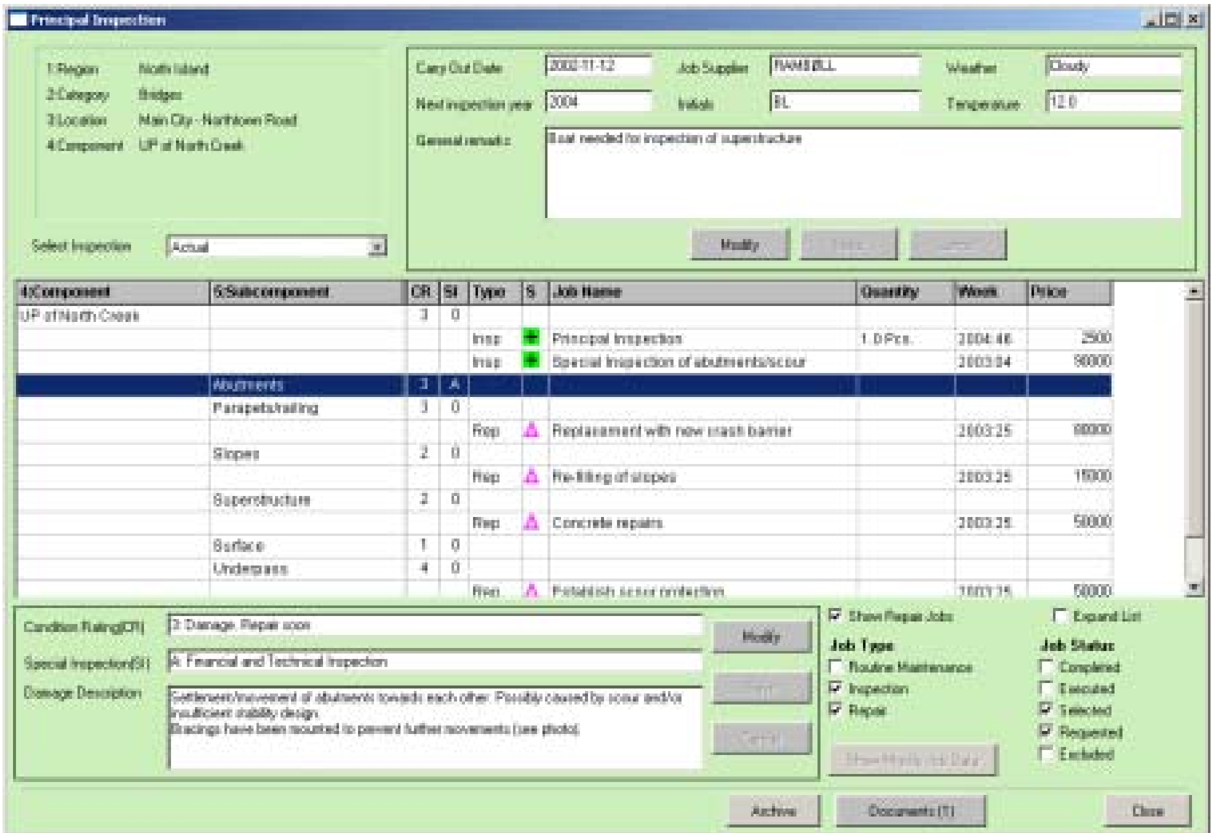

FIGURE 4 Principal Inspection screen giving overview of condition ratings and requested jobs. The status indicators in the "S" column are explained in Figure 5.

columns; or it could be an inspection job. Through its lifetime, the job goes through various statuses, as illustrated in Figure 5.

Request for a Job

Normally the need for a job is identified at an inspection—a routine inspection, a principal inspection, or a special inspection. In any case, the inspector can enter the requisition directly into DANBROweb. If the organization permits, contractors working on the bridges, or even the public, can be given access to require jobs. Everybody can be allowed to request jobs, since no job is executed before the manager has selected it. (The only risk is that one might get flooded with job requests if maintenance standards are poor.)

The job may be taken from a list of standard works, or the user may define it. A description of the job may be added, and supplementary information such as photos, drawings, instructions, and specifications can be attached as digital files. A job can be defined as a cyclic job to be executed at specified intervals. A job can be composed of items from a standard bill of quantities related to a maintenance contract.

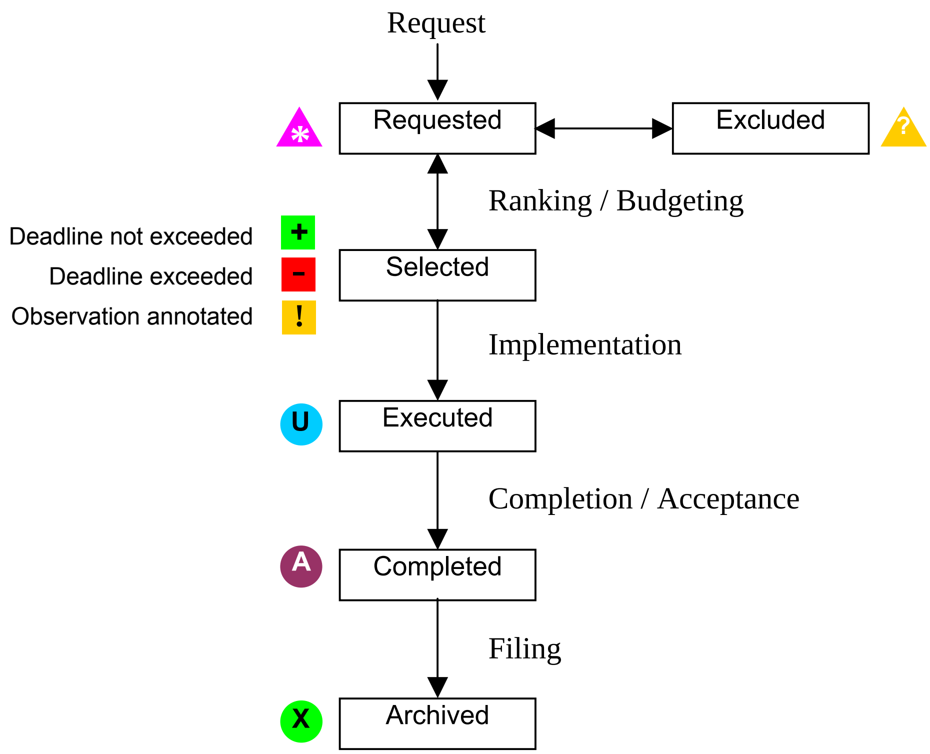

FIGURE 5 Activity management flow. Through the life of a job, it passes through the statuses shown in the boxes. The status indicators shown next to the boxes are used in overviews on screen.

Selecting Jobs to Be Executed

From time to time the manager (the bridge owner or the owner's representative) reviews the reported needs and moves approved works to the current list of due works. A simple priority-ranking system helps in deciding which jobs to approve. When making selections the manager has an overview of all jobs on the screen along with a budget overview that indicates how much is already spent on completed and executed jobs and how many of the remaining requested jobs of each priority category can be selected within the budget (see Figure 6).

Jobs that are not immediately selected can be left for later decision; they can be excluded (stored where they may later be retrieved); they can be postponed until a later year; or they can simply be deleted.

Reporting the Execution of Jobs

At regular intervals, each contractor or consultant who has a maintenance contract with the bridge owner generates a checklist of jobs that each is responsible for carrying out. The checklist looks very similar to the selection list in Figure 6, except that it shows only selected jobs. On the checklist, clear, colored status indicators show which jobs are overdue and which ones are on time. The checklist can be printed and used by the maintenance crew for the execution of the jobs.

In case there is something regarding the execution of the job that the bridge owner or manager or supervisor needs to know, the contractor can add a remark to the job. In that case the

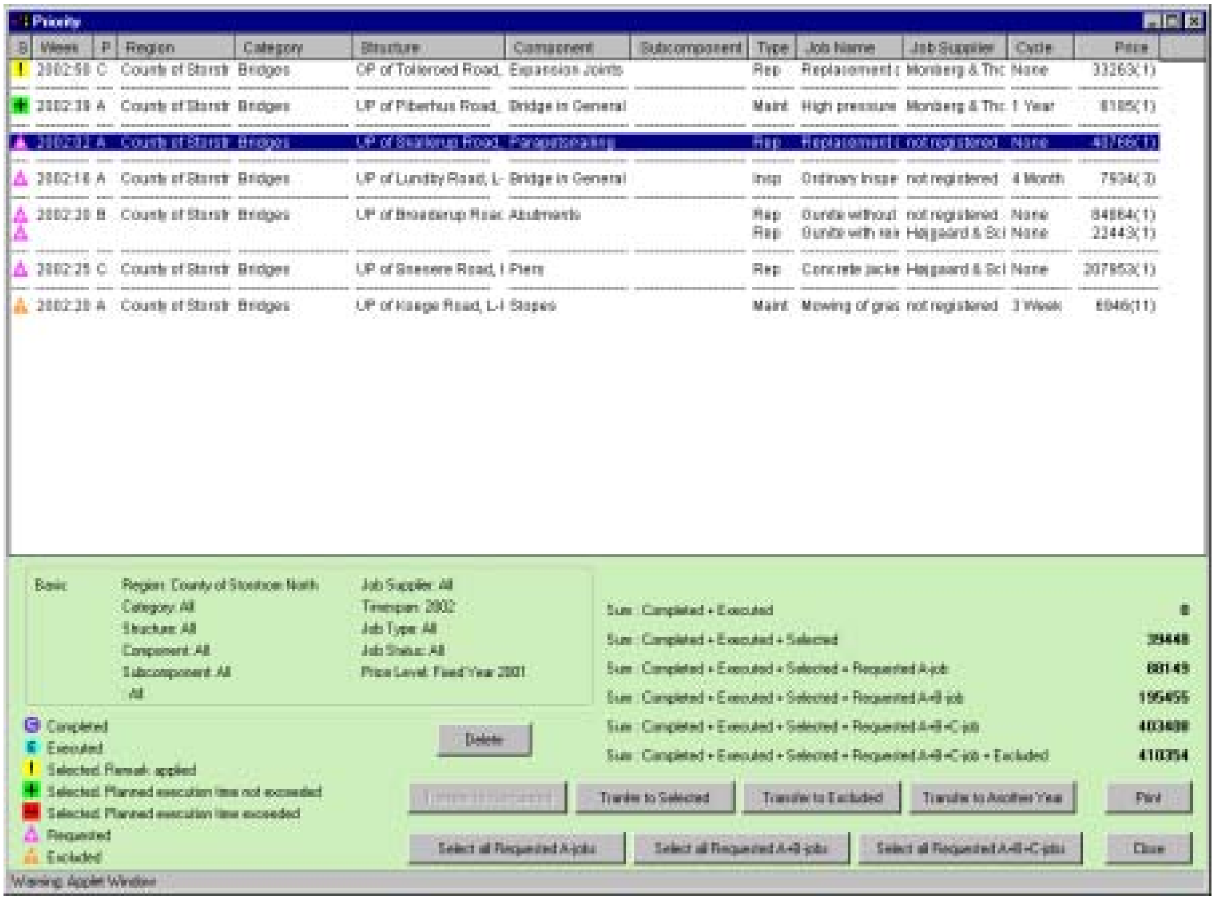

FIGURE 6 Selection of jobs. Each line represents a job. The indicators on the left show the status of each job. The amounts at bottom right show the economic consequences of selecting the various priority-ranking categories.

job will appear with a yellow indicator with an exclamation mark on the manager's job overview window. The manager can open the remark and remove it after having taken action.

When a job has been carried out, the contractor selects it on the checklist, opens the job, enters information on actual execution date and actual quantities and costs, and changes the status of the job to "Executed." The contractor may add notes regarding the execution and can attach files with photos, drawings, and other documentation from the execution. The attached documents will follow the job through the remaining statuses into the archive. When a job receives the status of Executed, it disappears from the checklist. If it is a cyclic job, the next execution is generated and appears on the checklist with the new deadline.

Completion of Jobs

The manager or supervisor reviews the jobs that have been reported as executed by the contractors. When satisfied with the completion of a job, the manager accepts the completion in the system by changing the job status to "Completed." From now on, this job will not appear in the contractor's checklists, but the price of the job will still be included in the budget overview on the manager's selection window (Figure 6).

Filing of Jobs in the Archive

When all the jobs of a fiscal year have been completed, the manager changes their status to "Archived." Now they will not appear on any of the normal job lists, and the costs will not be included in the budget overviews. However, it is still possible to access the jobs from the Archive Module, e.g., to review the job data or look into the attached documentation.

EXAMPLE OF IMPLEMENTATION

DANBROweb has been installed on the Faroe Islands at the central administration for managing the maintenance of a wide range of infrastructure assets, such as roads, bridges, tunnels, culverts, cattle grids, harbors, helicopter platforms, and lighthouses.

Through the prototype concept an individual inventory data set-up is made for each type of structure. Through the user administration it can be secured so that a person dealing with only harbors and lighthouses, for example, cannot see or edit data for other types of structures. Similarly, a geographical distribution with individual user rights can be made. The consequence is that for the individual user in a maintenance district the system is tailored to exactly that user's needs and contains only the data that user needs to work with, while it provides a continuously updated overview of all assets, activities, and budget needs to the central administration.

CONCLUSIONS

DANBROweb is a BMS focusing on the day-to-day management of bridges or other structures. It is meant to be used by many people with very different backgrounds and education. Emphasis has been put on making the system easy to operate and understand. This is achieved in several ways:

- The web concept ensures that for the common user the installation and updating of programs as well as the exchange and backup of data are 100% automatic.
- The user administration makes it possible for each person involved to see only the data and functions relevant for that user.
- The prototype concept makes it possible to tailor the inventory exactly to very different types of structures so that the data screens are not filled with data fields that are not relevant for the actual type of structure.
- The activity management makes sure that a person only sees an individual job when it is at the stage of its life when it has to be dealt with.

Expansion Joints, Coating Systems, Etc.

Econometric Model for Predicting Deterioration of Bridge Deck Expansion Joints

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Bridge deck expansion joints are an important element of bridge structures. When the joints fail to function properly, the performance of bridges can be seriously affected. Numerous research projects have been conducted to investigate the problems caused by the expansion joints and their causes. Some of this prior research was based on field observations while other research utilized in-house laboratory tests. However, a systematic approach to exploring the relationship between the joint condition and the influence of the environmental factors does not yet exist. Also, while the Markov model is frequently used in deterioration modeling, it cannot be applied when the condition rating has only a few levels and is discrete and ordinal. To address these two problems, a methodology is presented that uses the econometric model to predict the joint condition. The joint condition data have three rating levels, i.e., good, fair, and poor; and the ordered probit model is used to estimate the condition by the parameters, including age, traffic volumes, weather, skew angle, etc. Three joint types are investigated in the research: compression seal, strip seal, and integral abutment. The source data are from the 1998 bridge inspection records provided by the Indiana Department of Transportation. The estimation results confirm the findings from the previous field studies as to the parameters' influence on the joint condition. In addition, the results show how the developed deterioration curves can be used for comparing the performance among different types of joints and being incorporated into the life-cycle cost analysis.

Bridge deck expansion joints play an important role in the functioning of a bridge structure. They allow the bridge deck to expand and contract freely, prevent water and debris from penetrating into the bridge substructures, and provide a smooth riding surface. If they fail to function properly, a bridge deck will develop cracks due to its inability to expand or contract; the bridge bearings and seats will deteriorate rapidly because of penetrated water and deicing chemicals; and the impact on vehicles can be substantial because of the rough riding surface. Although bridge joints are a small element of bridge structures, they have significant influences on the performance of bridges.

Several FHWA studies conducted in the 1980s (*1*, *2*) showed that joints seldom achieve the function they intended to perform. The joint seals did not expand or contract as intended and had cracks in the early stages of their estimated service lives. Several studies have investigated the most frequently encountered problems of different types of joints, the possible causes, and their impact on the bridge performance. These research projects consisted primarily of field observations (*3–5*) and some utilized in-house laboratory tests (*6*). A systematic way to investigate the relationship between the joint condition and the influence of environmental factors, such as age, traffic, weather, etc., does not yet exist.

The purpose of this paper is to present a statistical approach, based on the econometric model, to estimate the joint condition by several influential parameters, such as age, weather, average daily traffic (ADT), etc. Three types of joints were investigated in the research, i.e., compression seal (BS), strip seal (SS), and integral abutment (IA). These joints were selected because they are currently used the most and thus have a large amount of data for analyses. 1998 bridge inspection data from Indiana Department of Transportation (INDOT) were used for the research. The remaining paper is organized in the following order. First, the background of three investigated types of joints is briefly introduced. Then the literature review describes how the factors significantly influencing joint condition were verified and how the appropriate modeling approach was selected. Second, the derivation of the ordered probit model is introduced. Third, the history of the inspection data is presented. Finally, the results from the ordered probit model are discussed, and the deterioration curves of each type of joint relating its condition to its age were developed. The performance of each type of joint was compared with the obtained deterioration curves.

BACKGROUND

The SS, BS, and IA joints all serve the same function, as an expansion joint, but they have different configurations and materials (*7*). The BS joint is installed by squeezing and inserting the seal into a preformed joint opening. The largest size seal could provide for a total movement of 102 mm (4 in.). The SS joint is a strip of specially shaped elastomeric material mechanically locked into a pair of extruded metal shapes that are in turn anchored to the edges of deck slabs. The largest size strip can provide up to 127 mm (5 in.) of total movement. The IA joint is structurally different than the previous two types of joints in that the bridge deck is tied to the bridge abutment to accommodate the movement of the concrete slab. The connection between the two slabs was filled with a poured silicone or neoprene seal. The movement range varies depending on the structural design.

LITERATURE REVIEW

Previous Bridge Deck Expansion Joints Studies

In the early 1960s, bridge maintenance requirements were studied nationwide. It was determined that expansion joints are the major source of bridge maintenance problems (*8*). In 1983, a project was initiated by INDOT to examine and evaluate the in-service performance of 97 experimental, watertight joints. In a 5-year evaluation period, more than 60% of the joints were leaking water and the remaining 40% were experiencing problems that would shorten their service lives (*4*). The causes of the joint problems included installation quality, traffic volumes, aging, maintenance, material quality, etc.

Price (*5*) conducted a comprehensive survey of about 250 bridges in Great Britain, which was a detailed study on joint problems and their causes. The possible causes found in the survey included structural movements of the joint, traffic density and axle loading, joint design and materials used, condition of the substrate, weather and temperature during installation and service, site preparation and workmanship, and performance of the bearings. Chang and Lee (*3*) conducted a questionnaire survey of bridge inspectors in Illinois, Indiana, Kentucky, Michigan, and Ohio about the most serious joint problems and their causes. The results were similar to the findings by Price. From these studies, the factors that significantly influence the joint performance were identified and selected as the parameters to estimate the joint condition in the current study.

Previous Deterioration Modeling Studies

The Markov model is the most commonly used methodology for deterioration modeling and there is abundant literature discussing its use (9–*12*). The Markov model is mostly used in pavement and bridge management systems, as it can predict the probability of transition from one condition state to another, and can be used in conjunction with the Markov decision process and the dynamic programming technique. The essential element in the Markov model is the probability transition matrix, which is obtained by minimizing the difference between the average condition values estimated by the linear regression and the Markov chain. However, Madanat et al. (*13*) pointed out that this approach suffered from several limitations when the dependent variable is discrete and ordinal. The assumption of the state independence is also not realistic (*14*).

The econometric model has been used recently in deterioration modeling to overcome the problems of the Markov model stated above. Madanat et al. (*13*) presented a methodology using the ordered probit model to estimate the probabilities of condition changes for bridge decks, which explicitly links the changes of the conditions with the exogenous variables. This approach can be used when the dependent variable is discrete and ordinal, such as bridge and joint condition ratings. Each condition state is estimated by a separate equation. This approach is theoretically appealing, but it requires several estimation equations and enough data to configure a comprehensive transition probability matrix.

In the present study, the ordered probit model is directly used to estimate the joint condition by using the cross-sectional data of bridge joints. This methodology is simple and shows the satisfied results in understanding the influences of environmental factors on the joint performance and in developing the deterioration curves of the joints.

ESTIMATION MODEL

The ordered probit model was developed in the field of biometrics and social science. The earliest literature found is from Aitchison and Silvey's (*15*) work in bioassay, in which the latent continuous variable was an organism's tolerance to some exposure, such as poison. The tolerance could not be observed, but the general state of the organism as unaffected, slightly affected, moribund, or dead, could be assessed. Their model was limited to a single independent variable. Mckelvey and Zavoina (*16*), in a paper written for social scientists, extended the work of Aitchison and Silvey to multiple independent variables. Currently, the ordered model is extensively used in the fields of economics, biology, and other social science fields.

In this study, the bridge joint condition is rated as good, fair, or poor. An example of the distribution of the joint condition with its age can be seen in Figure 1, with poor represented by 2, fair by 1, and good by 0. The application of the ordered model on the joint condition can be explained by the latent variable concept (*17*). The latent variable represents the condition that is unobservable, such as the internal process of the joint deterioration. However, the joint can manifest itself and be observed as good, fair, or poor by the field inspection. Assume that latent performance is represented by a variable, $Y_i^{'}$, which is a function of explanatory variables, X_i, such as the age, ADT, structural types, or other environmental factors. $Y_i^{'}$ is assumed to be a continuous and observable random variable, varying between $-\infty$ and $+\infty$. The structure of the ordered regression model is as follows:

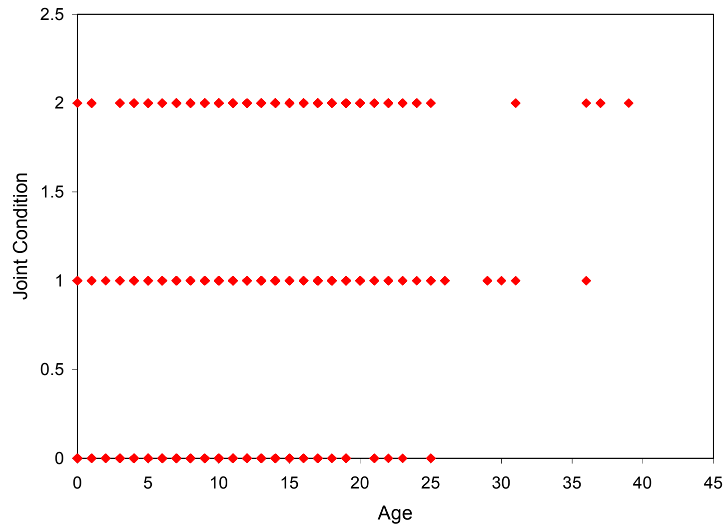

FIGURE 1 Scatter plot of joint condition and age.

$$Y_i' = X_i\beta + \varepsilon_i \tag{1}$$

where ε_i is a random error term. Equation 1 is not directly estimable, since Y_i' cannot be directly observed. To estimate the above deterioration model, a measurement equation that maps the latent variable, Y_i', to Y_i is needed. Y_i' is separated into different categories, each of which is mapped into a unique value of Y_i:

$$Y_i = n \quad \text{if } \lambda_n \le Y_i' \le \lambda_{n+1}$$

where n = 0, 1, 2 (2)

The λ's are the thresholds that separated the intervals. The thresholds λ_0 and λ_3 are defined by open-ended intervals with $-\infty$ and $+\infty$. For illustration, consider the dependent variable used to indicate the different joint condition states. The condition states can be categorized as "good" (G), "fair" (F), and "poor" (P), which correspond to the values of 2, 1, and 0. Assume that this ordinal variable is related to a continuous, latent variable Y' that indicates the actual condition states. The observed Y is related to Y' according to the measurement model:

$$Y_i = \begin{cases} 2 \Rightarrow G & \text{if } \lambda_0 = -\infty \le Y_i' \le \lambda_1 \\ 1 \Rightarrow F & \text{if } \lambda_1 \le Y_i' < \lambda_2 \\ 0 \Rightarrow P & \text{if } \lambda_2 \le Y_i' < \lambda_3 = \infty \end{cases} \tag{3}$$

The probabilities of observing value n in Y given X can be computed as follows:

$$P(Y_i = n \mid X_i) = P(\lambda_n \le Y_i < \lambda_{n+1} \mid X_i) \tag{4}$$

Substituting $Y_i^{'} = X_i \beta + \varepsilon_i$ and subtracting X, β within the inequality,

$$P(Y_i = n|X_i) = P(\lambda_n - X_i\beta \leq \varepsilon_i < \lambda_{n+1} - X_i\beta|X_i) \tag{5}$$

The probability that a random variable Y_i equals to n is the difference between the cumulative probability density function (cdf) evaluated at two different values. Therefore,

$$\begin{aligned} P(Y_i = n|X_i) &= P(\varepsilon_i < \lambda_{n+1} - X_i\beta|X_i) - P(\varepsilon_i \text{ £ } \lambda_n - X_i\beta|X_i) \\ &= F(\lambda_{n+1} - X_i\beta) - F(\lambda_n - X_i\beta) \end{aligned} \tag{6}$$

It is assumed that ε_i has a probability density function given by $f(\varepsilon_i)$ and cumulative density function given by $F(\varepsilon_i)$. If F is the cumulative standard normal density function, then the model is an ordered probit model. If F is the cumulative logistic function, then the model is an ordered logit model. In the research, the ordered probit model is used since the results between these two models are quite similar. The maximum likelihood (MLE) procedure is used to estimate the value of the parameter coefficient β and the thresholds $\lambda_{0,}\ \lambda_{1,} \ldots\ \lambda_{n+1}$ simultaneously. The probability of each value of Y_i can be represented by the following equations:

$$P_i = \begin{cases} P(Y_i = 0|X_i, \beta, \lambda) & \text{if } Y = 0 \\ P(Y_i = 1|X_i, \beta, \lambda) & \text{if } Y = 1 \\ P(Y_i = 2|X_i, \beta, \lambda) & \text{if } Y = 2 \end{cases} \tag{7}$$

The likelihood function of the ordered probit model is as follows:

$$L(\beta, \lambda | Y, X) = \prod_{i=1}^{N} P_i \tag{8}$$

N is the number of observations in the sample. Combining Equations 6 through 8,

$$\begin{aligned} L(\beta, \lambda | Y, X) &= \prod_{n=0}^{2} \prod_{Y_i = n} P(Y_i = n | X_i, \beta, \lambda) \\ &= \prod_{n=0}^{2} \prod_{Y_i = n} [F(\lambda_{n+1} - X_i\beta) - F(\lambda_n - X_i\beta)] \end{aligned} \tag{9}$$

$\prod_{Y_i = n}$ indicates multiplying over all cases where Y is observed to equal n. Taking logs, the log likelihood is

$$\ln L(\beta, \lambda | Y, X) = \sum_{n=0}^{2} \sum_{Y_i = n} \ln[F(\lambda_{n+1} - X_i\beta) - F(\lambda_n - X_i\beta)] \tag{10}$$

Equation 10 can be maximized with numerical methods to estimate the λ's and the β's. In this study, the statistical software, LIMDEP (*18*), is used to estimate the model and it automatically sets λ_1 to 0 to identify the values of other thresholds.

INSPECTION DATA

Since 1978, each state is required by FHWA to inspect bridges every 2 years, and this data record is then sent to FHWA to comprise a subset of the National Bridge Inventory (NBI) database. Each bridge record consists of 70 bridge inventory items, 12 condition and condition appraisal items, and another 8 items describing proposed renovations (*19*). The inspection data includes the ratings of individual components, such as deck joints and bearings, as well as the overall condition of the deck, superstructure, and substructure. In addition to the condition rating, data such as the ADT on each bridge and the year in which the bridge was built, repaired, or reconstructed, are also included.

There are about 5,600 bridges in Indiana and 70% of the joints in these bridges are BS, SS, and IA joints. The quantity of each type of joint used and its condition state are listed in Table 1. Table 2 lists the parameters used to estimate the joint conditions. These parameters were quantitative factors selected from the inspection data by referencing the survey results from Price (*5*) and Chang and Lee (*3*).

RESULT OF ANALYSIS

The results of the ordered probit model for three types of joints are shown in Table 3. From the result, it can be found that the BS and SS joints located in the southern region of Indiana performed better than those in northern Indiana, which is indicated by the positive coefficients. Since the BS and SS joints have larger *t*-statistic values than the IA joint in the southern region, the weather appears to have more influence on the BS and SS joints than on the IA joint. This is

TABLE 1 Number of Observations of Each Type of Joint

Joint Type	Joint Condition			Subtotal
	Good	Fair	Poor	
BS Joint	657	627	522	1806
SS Joint	312	112	56	480
IA Joint	721	194	85	1000

TABLE 2 Parameters Used in Estimating the Joint Conditions

Parameters	Data Format
Structural material	1 for concrete and 0 for steel (dummy variable)
Substructure settlement	Rated from 1 to 9; 1 is the worst and 9 is the best condition (dummy variable)[a]
Skew angle	Continuous variable
Structural Length	Continuous variable
ADT	Continuous variable
Weather	North, central, and south region (dummy variable)[b]
Age	Continuous variable

[a]Since most data are concentrated on condition 4 to 9, this parameter was separated into two groups with a similar number of data points to explore their influences. The first group consists of condition 4 through 7 and the second group 8 to 9. Therefore, the first group is represented by 0 and the second group by 1.
[b]Central and South are used as the dummy variables. Therefore, the north region is represented by Central = 0 and South = 0; the central region is represented by Central = 1 and South = 0; and the south region is represented by Central = 0 and South = 1.

probably due to the IA joint being a saw-cut type of joint that does not have a large seal piece, which is more easily influenced by rain or snow. Structural materials do not appear to have significant influence on all three types of joints, since it shows small *t*-statistic values, and this is also true for skew angle and structural length. However, the skew angle has more influence on the BS joint than on the others because its *t*-statistic value is 1.754 (*p*-value = .079), which is close to the boundary (*p*-value = .05). This is reasonable because the seal of the BS joint is a thick piece of closed-cell rubber that is more prone to be influenced by the skewed angle. Also, it was observed that when the structural length increases, the probability toward the better condition increases. Substructure settlement has significant influence on the BS and SS joints but not on the IA joint. Better substructure condition (less settlement) increases the probability of having better joint conditions, as was expected and indicated by its positive coefficient. The influences of the average daily traffic are significant for all three types of joints. More daily traffic causes faster deterioration of the joints, as the negative coefficient indicates. Finally, aging is a significant variable for all three types of joints. As the joint age increases, the condition usually worsens. These results were as expected and parallel past studies on the joints.

DEVELOPMENT OF DETERIORATION CURVES

After the models were estimated for all three types of joints, a probability plot with the joint condition and its age can be developed. The plot was drawn by varying the age for each condition, i.e., good, fair, and poor, while keeping other independent variables at their desired values. For instance, the deterioration curves could be developed by using specified criteria such as the north region, steel bridge structure, 0 degree skew angle, bridge structural length at 60 m (approx. 197 ft), worse condition of substructure settlement, and average daily traffic at 12,000 vehicles per day. By substituting these values back into equation 6 and varying the age number, the desired deterioration curves were developed. The probability plot of the bs, ss, and ia joints are shown in Figures 2–4. It can be seen from the curves of three types of joints that the probability of the condition "good" decreases and the probability of the condition "poor"

TABLE 3 Results of the Ordered Probit Model

Parameters	BS Joint Coefficient	BS Joint *t*-statistic	SS Joint Coefficient	SS Joint *t*-statistic	IA Joint Coefficient	IA Joint *t*-statistic
Central	-0.4772	5.886	0.6541	0.398	-0.8835	7.717
South	0.5950	6.887	0.4925	2.625	-0.0172	0.126
Structural material	0.0606	0.851	0.0958	0.634	-0.1605	1.041
Skew angle	0.0030	1.754	0.0010	0.457	0.0024	0.754
Structural length	0.0010	0.886	0.0007	0.344	0.0002	0.075
Substructure settlement	0.6143	8.441	0.3333	2.384	-0.0007	0.038
ADT	-0.0002	15.082	-0.0776	0.553	-0.0002	7.289
Age	-0.1063	18.745	-0.0813	7.995	-0.1111	13.020
	L(0)=-1975.574		L(0)=-417.7088		L(0)=-763.5246	
	L(B)=-1527.662		L(B)=-358.0355		L(B)=-594.4125	
	Rho Square = 0.227		Rho Square = 0.143		Rho Square = 0.221	
	$\lambda_0 = 0$	$\lambda_1 = 1.287$	$\lambda_0 = 0$	$\lambda_1 = 0.9718$	$\lambda_0 = 0$	$\lambda_1 = 1.0847$

increases as the age increases. The probability of the condition "fair" increases slowly at the beginning, arrives at its peak at a point in time, and then starts to decrease as the age increases. The plot guarantees that the joint will deteriorate with time, as shown by these curves, which is a feature of the ordered probit model. At any point in time, the probability of good, fair, and poor is added to 1.

By plotting these curves of all three types of joints, a comparison can be made among them. It can be seen from the "good" curves that the IA joint has the slowest deterioration rate and the BS joint has the fastest deterioration rate. It was also found that the success rate of the BS joint is the lowest, only 0.7, which means approximately 30% of the BS joint fail to function properly immediately after installation. By comparing the "poor" curves for all three types of joints, similar conclusions were obtained by comparing the "good" curves. The BS joint had the highest rate of deterioration, then the SS joint, and the IA joint. The comparison of the "fair" curve shows that the peak of the BS joint shifted to the left, compared to those of the SS and IA joints, which indicates that the BS joint arrives at the condition "fair" faster than the SS and IA joints. The results of these plots confirmed the survey made by Change and Lee (*3*). In the questionnaire survey of bridge inspectors in Indiana, the result indicated that the performance of the SS joint and the IA joint was better than the BS joint.

CONCLUSION

With the limitation of the Markov model, the ordered probit model is a good tool to estimate the infrastructure condition when the dependent variables are discrete and ordinal and do not have many levels. In this study, the joint condition rating has three levels and the ordered probit model was used to estimate the joint condition and showed quantitatively how the joint condition was influenced by the environmental parameters. The results were generally confirmed by the previous studies. In addition, this model can be used to develop probabilistic deterioration curves

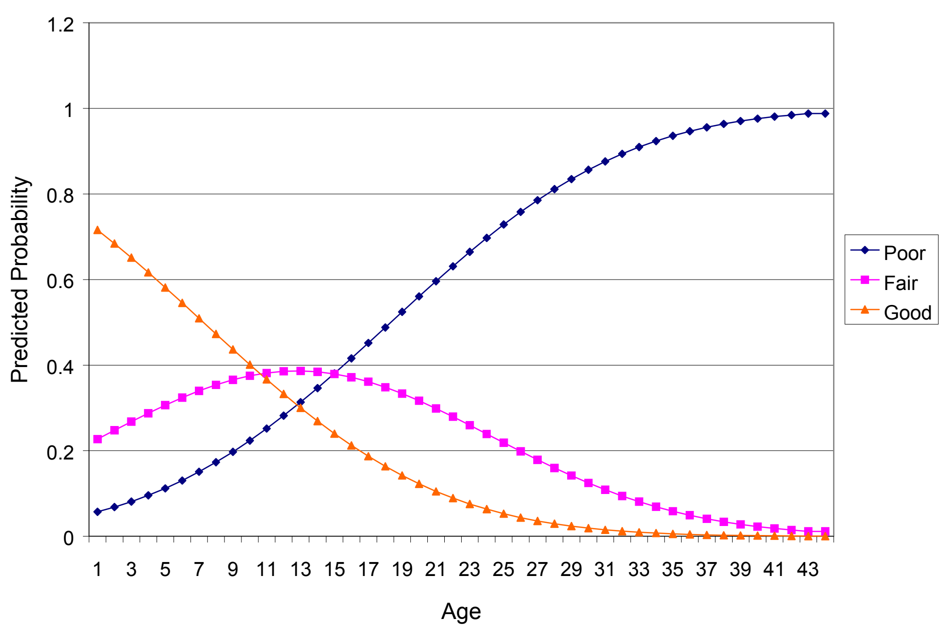

FIGURE 2 BS joint deterioration curve.

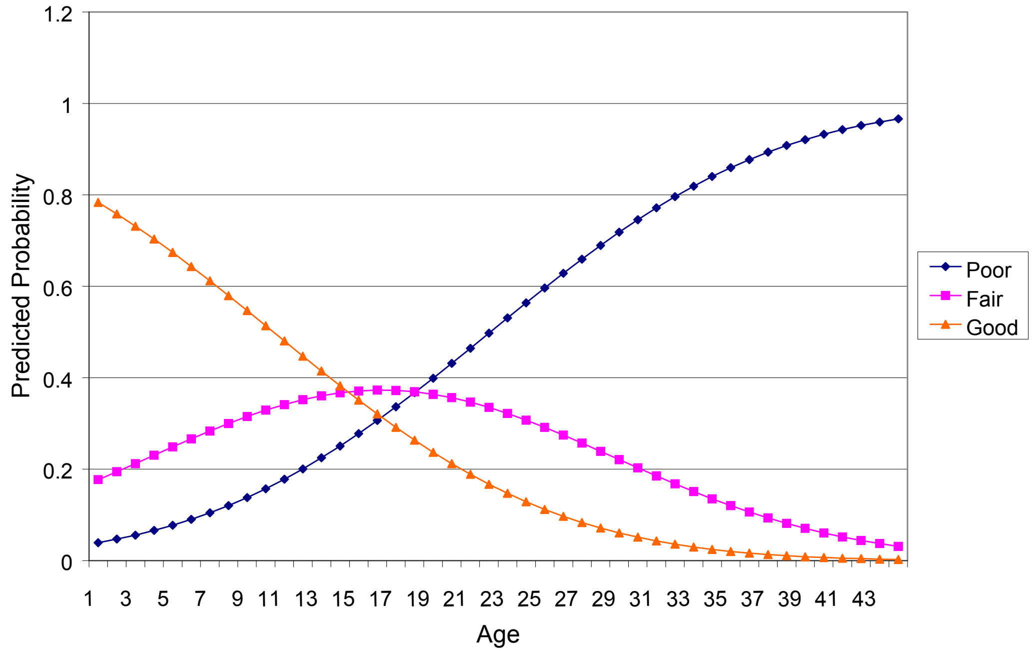

FIGURE 3 SS joint deterioration curve.

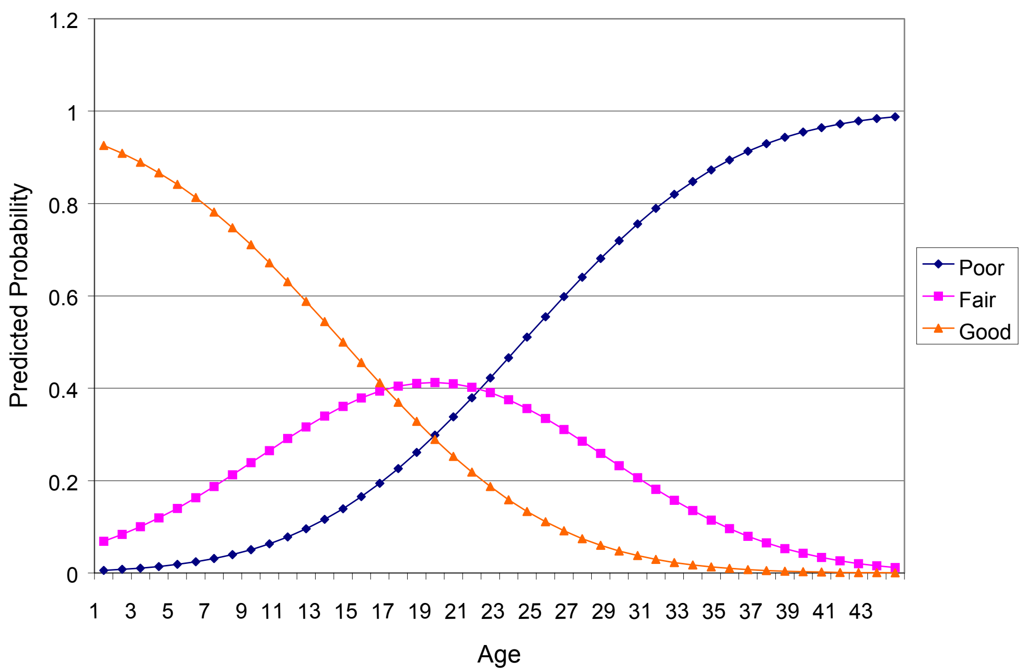

FIGURE 4 IA joint deterioration curve.

and to compare the performance of joints. The probability nature of the curves also allows them to be used in simulation and in making the life-cycle cost analysis. It is expected that this model can provide a simple and effective way to assess the deterioration mechanism of other infrastructure elements.

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REFERENCES

1. Romack, G. P. *Performance of Bridge Deck Expansion Joints*. Summary Report. Report No. FHWA-SA-91-010. FHWA, U.S. Department of Transportation, October 1990.
2. Romack, G. P. *Performance of Bridge Joint Systems Designed for Large Movements*. Publication No. FHWA-SA-92-018. FHWA, U.S. Department of Transportation, March 1992.
3. Chang, L. M. and Y. J. Lee. *Evaluation and Policy for Bridge Deck Expansion Joints*. Final Report. Report No. FHWA/IN/JTRP-2000/1, Joint Transportation Research Program, Purdue University, IN, 2001.
4. Fincher, H. E. *Evaluation of Rubber Expansion Joints for Bridges*. Final Report. Report No. FHWA/IN/RTC-83-1, Indiana Department of Highways, January 1983.
5. Price, A. R. *The Performance in Service of Bridge Expansion Joints*. TRRL Report LR 1104, Transport and Road Research Laboratory, Crowthorne, UK, 1984.
6. Kuo, S. and D. Waddell. Performance of Bridge Deck Expansion Joints by Large-Scale Accelerated Testing Apparatus. Proceedings of the Fourth World Congress on Joint Sealants and Bearing Systems for Concrete Structures, 1996, pp.197-215.
7. Chang, L. M. and Y. J. Lee. Evaluation of the Performance of Bridge Deck Expansion Joints. *Journal of Performance of Constructed Facilities*, ASCE, in press, February, 2002.
8. Walker, J. H. and P. E. Wasserman. Workshop on Integral Abutment Bridges. Pittsburgh, Pennsylvania, November 13-15, 1996.
9. Butt, A. A., M. Y. Shahin, K. J. Feighan, and S. H. Carpenter. Pavement Performance Prediction Model Using the Markov Process. *Transportation Research Record 1123*, TRB, National Research Council, Washington, D.C., 1987, pp. 12-19.
10. Feighan, K. J., M. Y. Shahin, K. C. Sinha,, and T. D. White. Application of Dynamic Programming and Other Mathematical Techniques to Pavement Management Systems. *Transportation Research Record 1200*, TRB, National Research Council, Washington, D.C., 1988, pp. 90-98.
11. Jiang, Y., M. Saito, and K. C. Sinha. Bridge Performance Prediction Model Using the Markov Chain. *Transportation Research Record 1180*, TRB, National Research Council, Washington, D.C., 1988, pp. 25-32.
12. Scherer, W. T. and K. M. Glagola. Markovian Models for Bridge Maintenance Management. *Journal of Transportation Engineering*, ASCE, Vol. 120, No. 1, 1994, pp. 37-51.
13. Madanat, S., R. Mishalani, and W. H. Wan Ibrahim. Estimation of Infrastructure Transition Probabilities from Condition Rating Data. *Journal of Infrastructure Systems*, ASCE, Vol. 1, No. 2, June 1995, pp. 120-124

14. Madanat, S., M. Karlaftis, and P. McCarthy. Probabilistic Infrastructure Deterioration Models with Panel Data. *Journal of Infrastructure Systems*, ASCE, Vol. 3, No. 1, March 1997, pp. 4-9.
15. Aitchison, J. and S. D. Silvey. The Generalization of Probit Analysis to the Case of Multiple Responses. *Biometrika*, Vol. 44, 1957, pp. 131-140.
16. Mckelvey, R. D. and W. Zavoina. A Statistical Model for the Analysis of Ordinal Level Dependent Variables. *Journal of Mathematical Society*, Vol. 4, 1975, pp. 103-120.
17. Long, J. S. *Regression Models for Categorical and Limited Dependent Variables*. SAGE Publications, Inc., California, 1997.
18. Greene, H. W. *LIMDEP- User's Manual*. Version 7.0. Econometric Software Inc., New York, 1998.
19. Federal Highway Administration. *Recording and Coding Guide for the Structure Inventory and Appraisal of the National's Bridges*. Report No. FHWA-PD-96-001. Washington, D.C., U.S. Department of Transportation, December 1995.

Structural Patch Repair of Concrete Structures
Toward the Development of a Risk-Based Implementation Procedure

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A repaired bridge has to fulfill strength and durability criteria to achieve a specified safe and useful design life with expected levels of maintenance. During structural patch repair, the structural behavior of a concrete member can change during the excavation of old concrete and its subsequent replacement with new material, and this behavioral change needs to be considered before carrying out the repair. Currently, little guidance is available on the redistribution of stresses within a reinforced concrete structure during and after such repair. Also, there are different views among engineers on whether a structure should be propped or left unpropped during patch repair. However, structural properties and loading conditions are not the only factors important for achieving a successful repair operation. It is not only the stresses present during and after repair that affect the short- and long-term behavior of a repaired structure but also other aspects, such as environmental conditions and compatibility of repair material with original concrete. As the effects of these many facets of patch repair on the behavior of a structure are different, it is difficult to make decisions by comparing them directly. Thus, a risk-based methodology is suggested to help in decision making related to the implementation of patch repair. Discussed are the importance of a risk-based approach to structural patch repair implementation and the parameters to be considered in developing such a methodology.

The amount of deterioration of concrete bridges has become a major problem in the recent past. Thus, the repair and rehabilitation of damaged concrete bridges have evolved into a major growth sector. Among other considerations, a repaired bridge has to fulfill the strength and durability criteria so that it can achieve a specified safe and useful design life with the expected levels of maintenance. However, repair of repairs sometimes becomes needed because of the insufficiency of the original repair in restoring a structure to a desired level of performance. This can be caused, for example, by poor implementation procedure or from a lack of consideration of behavior and performance in a wider context.

A number of techniques are available for structural repair of deteriorated bridges, the most common of these being patch repair in which the deteriorated concrete is taken out and replaced with new repair material. During structural repair a substantial quantity of concrete section may be removed. For example, in the case of deterioration caused by reinforcement corrosion due to chloride attack, all of the chlorine-contaminated concrete should be taken out to a level below the reinforcing bars. This means that the structural behavior of the reinforced concrete member can change during repair, and this needs to be considered before carrying out the repair.

Currently, little guidance is available on the redistribution of stresses within a reinforced concrete structure during and after patch repair. Also, there are different views among engineers on whether a structure should be propped or left unpropped during the repairs. The technical argument in favor of leaving a structure unpropped is that, when the repair is on the tension side, depropping after the repair would result in the cracking of the repair patch, which can pose durability-related problems. On the other hand, it is argued that without propping (which is expensive) the structural behavior will change. In addition to this, without propping, parts of a structure other than the repair patch may become overstressed, giving rise to cracks and durability problems in those parts. If temporary props are used to relieve load from a member, or to prevent significant redistribution of stresses within the member during and immediately after repair, then the consequences of altered structural behavior may be reduced or eliminated.

Structural properties and loading conditions are not the only factors to be considered for achieving a successful repair operation. This is because it is not only the stresses present during and after repair that affect the short- and long-term behavior of a repaired structure but also other aspects, such as environmental conditions and compatibility of repair material with the original concrete.

Prior work by Canisius and Waleed (*1*) has suggested that the effects on a structure of these many facets of patch repair are different—making it difficult to make decisions by comparing them directly. Thus, this paper suggests that a risk-based methodology, which provides the common denominator of risk, be used to help in decision making related to the implementation of structural patch repair.

OUTCOME OF PREVIOUS WORK

Recently, the British Research Establishment (BRE) completed a desk study related to structural concrete patch repair (*1*). Here, the author defines structural concrete patch repair as a situation in which

- The failure to conduct the repair would affect either structural serviceability or safety, or both, at that time or, due to resulting poor durability, in the future service life.
- The repair (even for cosmetic reasons) could, depending on the process employed, affect the structural behavior (serviceability and safety) either at that time or, due to resulting poor durability, in the future.

The following were the conclusions of the above-mentioned project, which involved a literature survey and a nonlinear, finite element-based analytical study of patch repair implementation, especially in relation to propping or not propping of the structure during excavation and repair:

1. It is only one or a few aspects of the multiaspect repair and structure system that are usually dealt with in the available literature.
2. The conclusions derived from single, or limited in number, aspect studies are inadequate for developing general rules.
3. Even when researchers studied the same aspect, differences may sometimes be seen in their conclusions, and this may be attributed mainly to a lack of a system approach.
4. Available published work deals mainly with simply supported beams, and there is no known study with respect to the patch repair of plate and shell structures with or without propping.
5. The recommendations based on simply supported beams cannot be generally applied to plate and shell structures or even to statically indeterminate beams.
6. There is a need to examine the conclusions and recommendations available in the literature to test their general applicability to conditions other than those considered by the

original researchers and to develop a general set of guidelines. For example, holistic investigations are needed in relation to

- The structural behavior of continuous beams with exposed reinforcement,
- The behavior of beams with exposed reinforcement under uniform load, and
- The repair of continuous beams and plates and shells.

7. In the 1990s there had been a call for a system approach for implementation of patch repair (*2*), although more from a materials point of view. To date, there seems to be no publication that presents the development of such an approach in which the structural consequences and the possibile loss of durability are considered simultaneously in a rational manner.
8. The lack of a system approach may be attributed to a perceived lack of a common denominator that may be used in assessing various alternate options.
9. A risk-based approach offers the common denominator needed to implement a system approach for structural patch repair of concrete.
10. A risk-based approach would also offer a means of evaluating different patch repair techniques and of defining performance requirements of repair.
11. It is necessary to develop a risk-based methodology to assess and implement concrete patch repair.

EFFECTS OF PROPPING OR NOT PROPPING DURING STRUCTURAL PATCH REPAIR

A major item of disagreement and incomplete research was seen with respect to the need for propping or not propping during excavation and repair. Most of the research was with respect to tension-side repair of simply supported beams, and thus there was no evidence of its applicability to continuous beams and plates and shells. The dissimilar conclusions related to the elasticity modulus and other properties required from the repair material (in relation to the substrate). There had also been research suggesting that structures should not be propped during repair because depropping would lead to the cracking of the repair patch—although some other publications referred to the possible transfer of stresses to other regions of a continuous structure. Practitioners have informed us of how, without propping, undesired phenomena, such as failure of reinforcement anchorage and reinforcement distortion, had resulted during the excavation for patch repair.

Figure 1 is a flat-slab structure patch repaired where shown. It is obtained from a not-yet-released BRE report by the author. Figure 2 is a comparison of how the absence of propping during the patch repair could give rise to both further and more severe cracking, within and without the repair region, in comparison to the case in which propping is provided. To obtain the results, the analyses considered the repair process itself, as allowed for by the phased analysis capability of the DIANA finite element software (*3*).

All the above indicates, as concluded during the previous work (*1*), that structural patch repair should be carried out holistically while considering the consequences of actions to be undertaken.

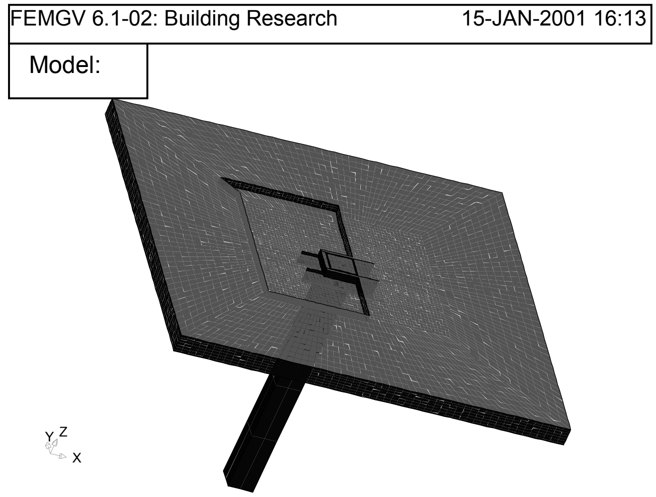

FIGURE 1 Substructure for detailed analysis. Flat slab with a column. Region of excavation for patch repair is shown.

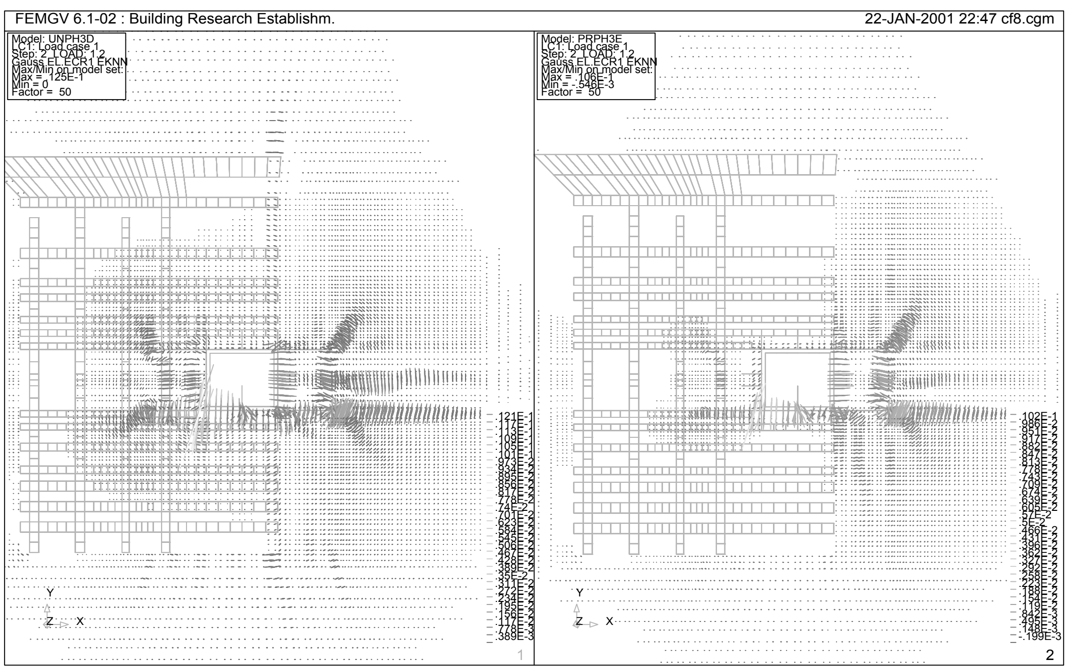

FIGURE 2 Cracking of the repaired structure. Detail near repair region and column head. Comparison of first crack normal strain vectors (in plan) from repair without propping (left) and repair with propping (right) analyses.

COMPLEXITY OF STRUCTURAL PATCH REPAIR

When patch repair has been chosen as the most appropriate rehabilitation option for a concrete structure, the engineer needs to consider many aspects of behavior and requirements that would influence its implementation. These include

- Failure of anchorage of exposed reinforcements due to the stresses within the bars.
- Distortion of stressed reinforcements due to the excavation of concrete around them.
- Cracking of the structure at the re-entrant corners of the excavation.
- Transfer of loads to other parts of the structure and its effects, such as cracking.
- Cracking of the repair patch during hardening or drying and subsequent loading.
- Failure of the repair–substrate interfaces.
- Entrapment of moisture.
- The decision on whether to interrupt the use of the structure, or continue with its use, either partially or completely.
- Duration of the repair action and its effects, for example, on the continued use of the structure. In the case of a bridge, this implies effects on the road network and, in turn, on the socioeconomic aspects of the relevant locality, pollution, etc.
- Occupational health and safety aspects of the workers and others in the vicinity.
- Required service life of the repair.
- Importance of the structure.

The above aspects, and others not detailed, are affected by many conditions related to the structure, the repair, and the general socioeconomic importance of the structure. These include

- Location of the repair—for example, in relation to how it may affect the continued use, the structural aspects, ease of repair, method of application of repair, etc.;
- Nature of the structure—for example, whether it is simply supported, continuous, a beam or a plate, etc.;
- Age of the structure—for example, in relation to the creep that may have occurred and deterioration elsewhere;
- Mechanical properties of the repair material and the substrate, as may affect the performance of the repair and structure system—for example, Young's modulus, tensile strength, and tensile strain capacity;
- Compatibility between repair material and substrate—for example, in relation to thermal properties and electrochemical aspects;
- Propping or not propping of the structure during repair and its implementation;
- Preparation of the substrate surface that would form the interface with the repair material;
- Interface bond between the repair material and the substrate concrete;
- Shrinkage and creep properties of the repair material and, if the structure is new, also that of the substrate;
- Method of implementation of the excavation and the repair material;
- Level to which the repair is carried out—for example, the extent of excavation, whether new reinforcement bars are provided, etc.;
- Quality control provided and the workmanship involved at various stages;

- Vibration of the structure during the hardening period—for example, vibration in a bridge due to its continued use;
- Curing of the repair and its later protection;
- Protection of the repair and structure system against subsequent adverse effects—for example, the use of cathodic protection, coatings, etc.;
- Environment, both micro and macro, of the repair and structure system; and
- Loads on the structure—in the past, during repair, and in the future.

The above list indicates that many things affect or influence the way a repair, and the repair and structure system, perform in both the short and the long term. These make the behavior of a repair system complex to consider, design, implement, and maintain.

PERFORMANCE REQUIREMENTS FOR A REPAIR

Any repair of a structure has to be carried out to satisfy various relevant performances with respect to the whole structure and the repair system. That is, the repair material or the repair system is only a part of this, protecting the structure and helping it to fulfill its requirements. Unfortunately, it is only the performance requirements for the repair material and system that repair engineers often seem to be concerned with. This is reflected, for example, by current repair and protection standards such as the emerging European standard EN1504 (*4*). While these requirements with respect to repair materials or systems are important, they do not reflect the whole picture; it is the performance of the whole repair and structure system that is ultimately important.

Considering a system approach, the performance requirements of a repair and structure system can include the following (some of these requirements are related to each other):

- Required service life;
- Regulations—for example, the Building Regulations of the United Kingdom;
- Codes and standards—for example, Eurocodes and, in the United Kingdom, the standards of the Highways Agency;
- Structural safety, related to the location, the type and importance of the structure, and loads;
- Serviceability of the structure, in relation to, for example, deflections and vibrations;
- Durability of the structure;
- Economical and whole life costs considerations;
- Occupational health and safety aspects;
- Maximum maintenance level desired;
- Environmental friendliness and sustainability of the repair system; and
- Aesthetics of the implemented repair.

HOW TO OBTAIN THE REQUIRED PERFORMANCE?

As can be seen from the above, there are many performance requirements to be satisfied by a repaired structure. At the same time, the performance of the repair and structure system depends on many factors, as detailed previously. Thus, a major question that could naturally arise in the mind of a repair engineer is how to go about selecting the best performance, possibly at a minimal cost—which is now increasingly defined as whole life costs. This, of course, may be achieved by mathematical minimization, or optimization, of the costs over the lifetime of the repaired structure. However, the following should be noted:

- Such optimization would be constrained by the regulations and any other fundamental requirements that cannot be changed.
- Optimization requires various opposing and supporting parameters to be expressed in terms of a common "currency."
- This common currency can be risk, defined as the product of the probability of an undesired event occurring and the expected consequences of that event, usually estimated as a monetary cost.

Sometimes it is impossible, or difficult, to express consequences in monetary terms, for example, the number of deaths due to an adverse event. If the number of deaths is the only major consequence to be considered, then one can try to minimize the number of deaths, constrained by the available funds, etc. However, in such a case, if there are also other types of consequences to be considered, or in cases in which items that cannot be financially expressed (e.g., sustainability and aesthetics) occur, the optimization needs to be a constrained one.

RISK AS THE BASIS FOR REPAIR-RELATED DECISION MAKING

It is a known fact that, for some time now, maintenance and management of highway structures have been moving toward risk-based approaches. These approaches are used to help in decision making with respect to bridges, dealing with, for example, when to inspect, when to repair, whether and when to demolish and rebuild, and, also perhaps, to what extent to repair a structure. However, this does not seem to extend, for example, to the selection of the method of repair or protection (in addition to patch repair), method of implementation of the repair, and consideration of the risks to repair engineers, in addition to those to owners. A risk-based methodology that considers these repair issues can also eliminate the current need to distinguish between structural and nonstructural repair (*4*)—which the repair community finds difficult and, sometimes, impossible to understand. Thus, it is felt that there is a strong argument in favor of using a risk-based decision-making methodology related to concrete repair. However, for such a process to be usable in practical day-to-day decision making, it needs to be a simplified one based upon fully probabilistic models.

CHALLENGES TO RISK-BASED DECISION MAKING

Whatever the desirability, the development of a risk-based methodology would not be very easy. Its development requires relevant knowledge about the various phenomena involved and data on the relevant parameters that influence behavior, so that quantitative information can be derived. For example, information—including statistical information—is required on

- Material behavior;
- Material properties;
- Properties of the structure;
- Quality control and workmanship issues;
- Environments, both micro and macro, in the past and in the future; and
- Loading, past and present.

This would require efforts on the part of repair engineers and researchers to collect data. It is believed that the emerging European standard EN1504 (*4*) for the repair and protection of concrete structures would help in this area. This is because this standard requires extensive testing as a means of quality control, both in factories and on site, for verification and for checking of the implementation.

As mentioned previously, there is also a need to develop simplified risk-based methods, based upon fully probabilistic methods, for the use of engineers in their day-to-day tasks. This means conducting relevant research based on fully probabilistic methods.

Considering that it is cheaper to conduct computer analyses than experiments of the type required for the particular exercise discussed here, the developments can be based on analytical studies, which, however, are to be verified by experiments. Obviously, when information is not available, one needs to make reasonable assumptions and study the sensitivity of results to those assumptions.

BRE EFFORTS TOWARD THE DEVELOPMENT OF A RISK-BASED METHODOLOGY

BRE was to have already started research toward the development of a risk-based procedure for decisions related to bridge repair. However, late funding restrictions within the prospective funding organization has prevented this work program from going ahead at the present time. However, under the sponsorship of the United Kingdom’s Foundation for Built Environment, BRE has recently begun an extensive project to develop such a methodology in relation to buildings and car parks. Obviously, although not all aspects relevant to bridges will be considered within this project, and not all aspects considered here would be applicable to bridges, this study would help to advance the concepts with respect to bridges, too. As suggested above, the study would be based mainly upon analyses, with information from testing under other relevant projects providing the real-life information. For example, BRE is currently involved in a European Union project, REHABCON, together with other partners from the United Kingdom, Sweden, and Spain. This project, on the rehabilitation of concrete structures, is expected to give rise to information necessary for the development of a decision-making methodology for risk-based repair.

CONCLUSION

This paper presented the need to develop a risk-based methodology that can be used for making decisions related to structural concrete patch repair. By considering the different aspects important for the behavior of concrete patch repair, the performance requirements for the repair and the structure, and the need to optimize between various conflicting requirements, it was shown that a risk-based approach would provide the best way forward for repair engineers. It was also suggested that a risk-based approach would take away the need to distinguish between structural and nonstructural patch repair.

BRE is currently involved in the development of a relevant methodology. Its results will be presented to the profession in the future.

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REFERENCES

1. Canisius, T. D. G., and N. Waleed. *The Behavior of Concrete Repair Patches Under Propped and Unpropped Conditions: Critical Review of Current Knowledge and Practices*. FBE Report 3. Foundation for Built Environment, Watford, England, 2002.
2. Emmons, P. H., and A. M. Vaysburd. A Total System Approach of Design and Production of Durable Repaired Structures. *Proc., 5th International Conference on Structural Faults and Repair*, Vol. 1, June 1993, London, pp. 97–107.
3. DIANA Finite Element Program V7.2, TNO, Delft, Netherlands, 2001.
4. CEN. *Products and Systems for the Protection and Repair of Concrete Structures*. European Standard (pr)EN(V) 1504. European Committee for Standardization, Brussels, Belgium, 1999.

Scour Modeling and Experiences

Physical Modeling of Scour in Bridge Management

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The equations and methods for determining local scour depths given in FHWA's Hydraulic Engineering Circulars 18, 20, and 23 give satisfactory answers for the design and evaluation of bridges with noncomplex foundations or waterway conditions. However, the design or evaluation for scour of bridges with complex foundations or waterway conditions may require one- and two-dimensional physical model studies. Model studies are a combination of art and science. This study provided similitude, some requirements for modeling fluvial systems, and case histories of model studies for analyzing scour at highway bridges.

The equations and methods for determining local scour depths in FHWA's Hydraulic Engineering Circulars (HECs) 18, 20, and 23 (*1–3)* give satisfactory answers for the design and evaluation of most bridges. However, the design or evaluations for scour of bridges with complex foundations or waterway conditions often require physical model studies.

Complex piers are defined in HEC 18 (*1*) as piers composed of two or more elements (pier shaft, pile cap or footing, and piles) that by design, long-term degradation, or contraction scour are exposed to the flow. Staggered piles, which may not be aligned with the flow, are also often required for the foundation.

Complex waterway conditions include

- Reaches where two parallel bridges are close together with or without overbank flow,
- Bridges with complex angles of attack at different flow magnitudes,
- Bridges over streams with complex interaction between main channel and overbank flow, and
- Bridges, which because of the location of piers in relation to each other or to an abutment, have flows with indeterminate angle of attack or velocity, or both.

Two- and three-dimensional (2-D, 3-D) physical models have been successfully used to analyze complex foundations and flow conditions. Model studies are involved in the design of the foundations for the Bonner Bridge over Oregon inlet in North Carolina and the Woodrow Wilson Bridge over the Potomac River between Maryland and Virginia. These bridges have complex pile groups supporting a large pile cap, which supports the pier shaft. For both of these bridges, all three foundation components are in the flow for the design discharges.

Physical model studies were needed in investigation into the causes of failure of the Interstate 90 (I-90) bridge over Schoharie Creek in upstate New York in 1987 (*4, 5*), the U.S. Route 51 (US-51) bridge over the Hatchie River near Covington, Tennessee, in 1989 (*6–8*), and the I-5 bridge over Los Gatos Creek (Arroyo Pasajero) in the Central Valley of California in 1995 (*9*). Both 2-D and 3-D model studies were needed for studying the New York bridge failure, because the pier shaft was composed of two columns on a plinth, which rested on a massive footing (*4, 5*). A 2-D model was needed for the Tennessee Bridge, because stream instability exposed a pier shaft, footer, and piles to local scour. A 2-D model was needed for the California Bridge, because maintenance crews introduced changes in the foundation that altered the original design and may have increased scour depths. In all cases equations were inadequate to determine the scour depths.

Physical hydraulic modeling of highway crossings is a combination of art and science. The science entails fluid mechanics similitude theory, and the art entails engineers' experience in determining necessary compromises because the fluid mechanics conditions in the model cannot precisely replicate prototype conditions. This paper will describe some problems in model prototyping and present case histories of model studies to show how art and science were used to overcome these problems.

PHYSICAL HYDRAULIC MODELING

Similitude Requirements

Complete similitude requires geometric, kinematic, and dynamic similarity between prototype and model. Geometric similarity requires complete similarity in the ratios of the linear dimensions between model and prototype. Kinematic similarity requires similarity of motion, which requires that the ratios of the velocity components in the geometric-similar model be equal at all points to those of the prototype systems. Dynamic similarity requires that the forces in the geometric- and kinematic-similar model be the same as those in the prototype systems. Complete similitude is developed from Newton's second law of motion:

$$Ma = \sum F_p + F_g + F_v + F_\sigma + F_\varepsilon$$

where

Ma = mass reaction to acting forces F and is the inertial force,
F_p = pressure (p) force resulting from the motion,
F_g = gravity (g) force on the fluid,
F_v = viscous (v) force,
F_σ = surface tension (σ) force, and
F_ε = elastic compression (ε) force.

For similitude the ratio of the inertial forces, model to prototype, must be equal to the ratio of the active forces. This is achieved if the Froude number (*Fr*), Reynolds number (*Re*), Weber number (*W*), and Mach number (*M*) are equal between model and prototype. The pressure force (*p*) is usually regarded as the dependent variable and does not play a controlling part in similitude. These numbers are defined as follows:

Fr = ratio of inertial forces to gravitational forces = $V / (gL)^{1/2}$

Re = ratio of inertial force to viscous forces = VL / v
W = ratio of inertial forces to surface tension forces = $V / (\sigma / \Delta L)^{1/2}$
M = ratio of inertial forces to elastic compression forces = $V / (\varepsilon / \Delta)^{1/2}$

Water is generally inelastic, and *M* can be disregarded in hydraulic modeling. Surface tension forces are not important if the model is sufficiently large. With free surface turbulent flow, the *Fr* similitude criteria predominate, and normally the *Re* criteria can be ignored.

The *Fr* modeling similitude has the following relation:

$$\frac{V_m}{\sqrt{g_m L_m}} = \frac{V_p}{\sqrt{g_p L_p}} \quad or \quad \frac{V_r}{\sqrt{g_r L_r}} = 1$$

where

V = velocity (ft/s or m/s),
g = acceleration of gravity (32.2 ft/s^2 or 9.81 m/s^2),
L = length (ft or m),
m = model,
p = prototype, and
r = ratio of model to prototype.

Tables giving scale ratios for geometric, kinematic, and dynamic similitude for *Fr, Re,* and *W* are given in some hydraulic engineering publications such as Rouse (*10*) and Richardson et al. (*11)*.

Distorted Scales

Distorted scales are used when it is necessary to depart from geometric similarity, for any one of several reasons. For instance, the prototype's physical size may be beyond the limitations of the modeling facilities, or the size of the model may cause very shallow flows or low velocity flows. Distorted models are often needed for 3-D models.

In scour studies geometrically distorted scales may be necessary for

- Obtaining sufficient tractive force (shear) on the bed to produce bed material movement,
- Exaggerating water-surface slope and wave heights for observation and measurement,
- Fitting the model into the limitations of the physical facilities,
- Increasing the size of the model so that *v* and σ are not a factor, and
- Simplifying the collection of data and operation of the model study.

However, distorted scales have the following disadvantages:

- Velocity, scour, and other measurements may be seriously distorted.
- Construction of the model may be more difficult.
- Waves in the model may be distorted or not detectable.

- Interpretation of the results is not straightforward and depends on the experience of the engineer that is conducting the tests.

Mobile Bed Models

Similitude in mobile bed models requires that the model reproduce the fluvial processes, such as sediment transport, scour of bridge foundations, channel deposition or degradation, lateral channel migration, bed configuration, and boundary roughness. It is generally not possible to faithfully simulate all these processes in a mobile bed model. The problem is scaling or the impossibility of scaling the bed material so that it responds to the forces of the fluid in a way similar to the response in the prototype. However, with the addition of knowledge and understanding of the interaction between the flow and bed material in the prototype, operation of the model can successfully simulate the prototype's conditions and response. An example is a case in which at flood flows the bed configuration in the prototype was plane bed but in the model was dunes for the same *Fr*. Successful results were obtained by operating the model with a bed material size slightly larger than the beginning of motion size and a plane bed.

Often 2-D and 3-D models are used in conjunction in mobile bed model studies. A 3-D rigid boundary model study is a tool to determine the velocity distribution and flow pattern, which values are then used in a smaller-scale 2-D model of a specific area of the flow field around a pier or abutment, for example.

In studies of sediment transport, degradation, and aggradation, bed materials are lightweight, such as walnut shells or plastic beads. In scour studies, however, sand is the preferred bed material. In model studies of scour, the bed form in the prototype is a major consideration.

Bed Material

The size of bed material is the least understood consideration in scour model studies and often is the area for the most difficult modeling assumptions even though there are some general guidelines to follow. Models should represent bed material size, bed configuration, and sediment transport in the prototype, but these features are difficult to model, because bed material properties change drastically when sizes attain cohesive soil ranges. As mentioned earlier, if a plane bed exists in the prototype, then a plane bed must exist in the model with or without sediment transport. Also mobile boundary or rigid boundary 3-D models at small scale (1:50 or less) and even distorted scale are used to determine approximate scour dimensions, flow distribution, and so forth. For more detailed analysis, a larger scale (1:5, 1:10, or 1:15) 2-D model of specific areas of the flow field is then constructed.

Some hydraulic engineers have successfully conducted model studies by maintaining the same ratios between model and prototype of

- Average velocity to fall velocity (V / T) or
- Average velocity to the square root of gravity times a characteristic sediment size D ($V / (gD)^{1/2}$.

Many local pier scour model studies are based on the premise that near-maximum local scour occurs at the incipient motion velocity of the bed material and that a reasonably conservative approximation of local pier scour can be determined without actually modeling the prototype bed material in the lab.

The authors believe there are enough questions about bed material modeling to warrant a synthesis study of practice or a task force of experts to develop systematic guidelines for modelers to best quantify scour from laboratory investigations.

Beginning of Motion
Beginning of motion velocity or shear stress for the model and prototype bed material can be determined by methods given in Vanoni (*12*), Richardson et al. (*11*), Richardson and Davis, (*1*) and other publications.

Bed Configuration
Similarity in bed configuration between model and prototype is an important and often overlooked consideration in mobile bed model studies. The bed configurations in alluvial channels are described in Richardson and Simons (*13)*, Richardson et al. (*11*), and Vanoni (*12*). In scour studies erroneous results will result if the two are not similar. If the prototype has a dune bed and the model has ripples, scour depths in the model have no relation to scour depths in the prototype. The results are also erroneous if the flow in the prototype is plane bed or antidunes and the bed configuration in the model is ripples or dunes. Successful results have been attained when prototype structures on sand bed stream with plane bed configuration were modeled using sand that was just at the threshold of movement and a plain bed.

In spite of problems in modeling mobile beds, such model studies have been successfully conducted and are recommended for site-specific cases. Examples of scour studies of mobile beds are presented below.

EXAMPLE SCOUR MODELS

Schoharie Bridge Model Studies

On April 5, 1987, the 540-ft-long New York State Thruway Authority (NYSTA) (I-90) bridge over Schoharie Creek collapsed during a near-record flood. Ten lives were lost when one tractor semitrailer truck and four automobiles fell nearly 80 ft into the river. Three nonredundant spans, supported by four piers, fell when two piers collapsed. The first pier and two spans collapsed about 90 min before the second pier and one span. Two approach spans did not fall. The National Transportation Safety Board (NTSB) was asked to investigate the cause of the failure. NTSB and NYSTA commissioned Resource Consultants, Inc. (now Ayres Associates) of Fort Collins, Colorado, to determine the role that erosion, scour, and hydraulic forces played in the collapse (*14*). As part of the investigation, a 3-D physical undistorted 1:50 scale model of a reach of the Schoharie Creek, which included the Thruway Bridge, was constructed. Also a 2-D 1:15 scale model of a pier was constructed with the information gained from the 3-D model. Colorado State University civil engineering hydraulics staff conducted the model study (*4, 5*).

The bridge was opened to traffic on October 26, 1954. On October 16, 1955, it was subjected to a larger flood (76,600 ft^3/s, 2,169 m^3/s) than during the April 5, 1987, flood (63,100 ft^3/s, 1,787 m^3/s). The 1955 flood was between a 200- and 500-year event, and the 1987 flood was between a 70- and 100-year event.

The bridge has a long approach, which is across the Schoharie floodplain. It is about a mile upstream from the confluence of the Schoharie Creek with the Mohawk River. The four piers were complex with large footings (5 x 19 x 82 ft) supporting a slightly tapered plinth 16 ft x 9 ft at the top. There were no piles. The top of the footing was set at the top of the glacial drift bed material that was stratified by ice contact. The piers were protected with riprap. The drift was covered with cobbles and boulders. Additional information is presented in reports by NTSB (*14*); Wiss et al. (*15*), Thompson (*6*), Jones (*7*), Jackson et al. (*8*); and Richardson et al. (*4, 5*).

Objectives of Model Study

The objective of the 3-D model study was to determine and document the distribution of the discharge at the bridge, contraction scour (if any), local scour at the two piers, and the magnitude and direction of the flow velocity in the bridge cross section before and after the initial collapse.

The objective of the 2-D model study was to determine and document the scour and velocity field around the first pier that failed. In 1987 there were no equations or methods to determine scour around complex piers.

Construction and Operation of Models

The model bridge was constructed of wood. The floodplain and banks of the river were molded to scale of sand and covered with mortar, as there was no erosion of the banks or floodplain. The model riverbed was composed of coarse sand. A study of the prototype bed and bed material showed that there was no long-term degradation or contraction scour and little if any bed material movement. The bed configuration was plane bed. Therefore, local scour at the piers would be clear water scour. The model bed material size was selected so that the beginning of motion velocity of the coarsest size faction was slightly lower than the average model velocity of the largest discharge to be modeled. Figure 1 shows the model. Flow was established through the model for the 1955 peak discharges using known prototype upstream and downstream water surface elevations. For the 1987 peak discharge, known downstream water surface elevations and the result of a Water-Surface Profile (WSPRO) computer model study were used. Prototype discharges, which were modeled through the bridge, were decreased from the 1955 and 1987 peak river discharges to 73,600 ft^3/s (2,084 m^3/s) and 62,300 ft^3/s (1,764 m^3/s), respectively, because some of the peak discharges bypassed the bridge at the west end of the approach.

FIGURE 1 Looking upstream of Schoharie Creek Bridge model. Piers are numbered from right. Pier 3, which failed first, is second from left (*4*).

Scour depths and geometry, velocity, and discharge distribution were determined for the 1955 and 1987 peak discharges.

The 1:15 2-D model bridge pier was constructed of plastic. It was placed in a flume 8 x 4 x 200 ft at a slight (5°) angle to the flow. This angle was determined from the 3-D model study. The bed material was selected on the same criteria as for the 3-D model to obtain plane-bed, clear-water scour. The appropriate flows for the 1955 and 1987 peak discharges were determined from the results of the velocity traverses in the 3-D model (see Figure 2).

Model Study Results

The scour hole for the 3-D model for the 1987 peak discharge is shown in Figure 3, and the velocity distribution is shown in Figure 2. The model showed that Pier 2 failed, because the bridge debris from the Pier 3 failure caused a jet of water to erode its foundation (see Figure 3). Both models gave a maximum scour depth for the 1987 flood of 15 ft (4.57 m) and showed the same skewness and dimensions of the scour hole. Measured prototype scour depth was 14 ft (4.27 m). The model studies showed a need to develop methods to compute scour depths for complex piers. The Colorado State University *(1, 11)* pier scour equation calculates a scour depth of 22.7 ft (6.92 m) using plinth width and 37.2 ft (11.34 m) using footing width. The models also showed the effect of the angle of attract on the dimensions of the scour hole. In addition, the 3-D model showed that the velocity distribution is greatly skewed for some distance downstream of a bend (see Figures 1 and 2).

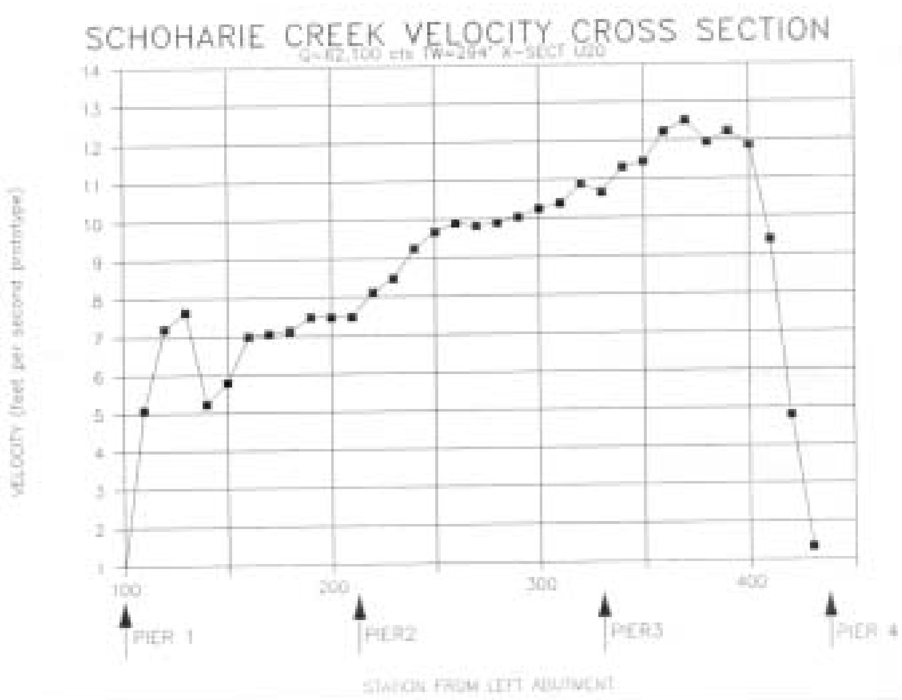

FIGURE 2 Velocity profile for the 1987 flood peak taken at upstream edge of bridge. Peak velocity between Piers 1 and 2 is flow from floodplain (*4, 14*).

FIGURE 3 Model scour holes for 1987 flood discharge. Pier 3, which failed first, is on right, and Pier 2 is at left. Model scour depth for Pier 3 is 15 ft (4.57 m) (*4, 14*).

Hatchie Bridge Model Study

Two years, almost to the day, after the Schoharie Creek failure, two spans of the northbound US-51 bridge over the Hatchie River near Covington, Tennessee, collapsed during a seasonal flood event causing eight fatalities (*6, 8*). Again the NTSB investigated the cause of the collapse, and FHWA deployed a bridge scour inspection boat and several hydraulic engineers to help with the investigation.

This collapse was attributed primarily to a very subtle lateral migration of the main channel to overtake a floodplain pier that was not designed to be in the main channel. This collapse demonstrated the value of plotting periodic stream bed cross sections against a background of the pier foundations. This capability is a feature of the expert system CAESAR (Catalog and Expert Evaluation of Scour Risk and River Stability) later developed for bridge inspectors under an NCHRP project (*16*).

Once the main channel migrated past the first floodplain pier, Bent 70 on the bridge plans, the simple 2-ft-wide pier became a complex obstruction to the flow with the pier stem, the pile cap or footer, and several feet of the piles exposed to the currents. Because no complex pier procedure existed then to evaluate scour from that type of flow obstruction, a 1:20 scale model of Bent 70 (Figure 4) was tested in the FHWA hydraulics lab to determine how much local pier scour might have contributed to the collapse (*7*). The model study showed that local pier scour was probably <4 ft, but even that could have been enough to trigger the collapse after the main channel migrated to the pier, because the friction pile tips for Bent 70 were only approximately 20 ft below the footers.

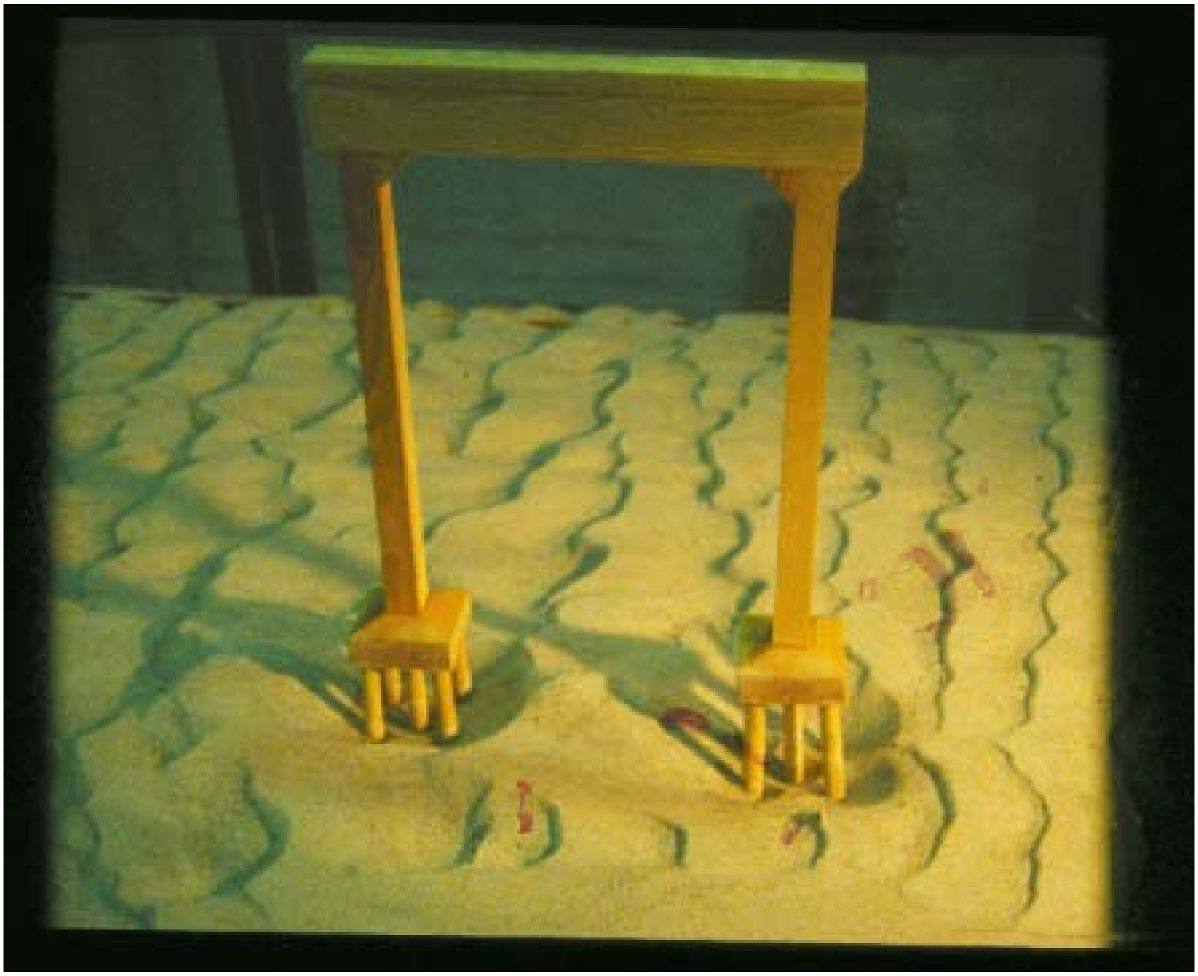

FIGURE 4 Bent 70 was modeled as a complex pier, because channel migration exposed the footers and piles to flow currents.

Arroyo Pasajero Bridge Model Study

The third in a series of catastrophic bridge collapses relating to scour occurred in March 1995 when the entire I-5 bridge (from abutment to abutment) over the Arroyo Pasajero (Los Gatos Creek) collapsed, which caused seven fatalities (*9*). I-5 is the major route connecting Los Angeles and San Francisco, and the catastrophe could have been much worse if the highway had not been closed to all but local traffic because of water over the roadway in another location.

A multitude of events contributed to the collapse. There was general subsidence of the landscape downstream of the bridge. A headcut had already traveled upstream through the bridge opening, which lowered the bed by several feet. A structural webwall approximately 7 ft high had been constructed to tie the foundation columns together for lateral support after the channel had degraded. This webwall presented a bigger obstruction to flow, which increased local pier scour. The upstream approach channel was shifted to one side of the bridge opening, which caused the flow to approach the bridge piers at a severe skew angle (Figure 5). Also a lot of woody debris was in the channel during the flood. The 122-ft-long bridge contracted the flow channel through the opening. Finally the foundation column reinforcement steel stopped approximately 3 or 4 ft below the stream bed elevation as it was in the bridge opening just after

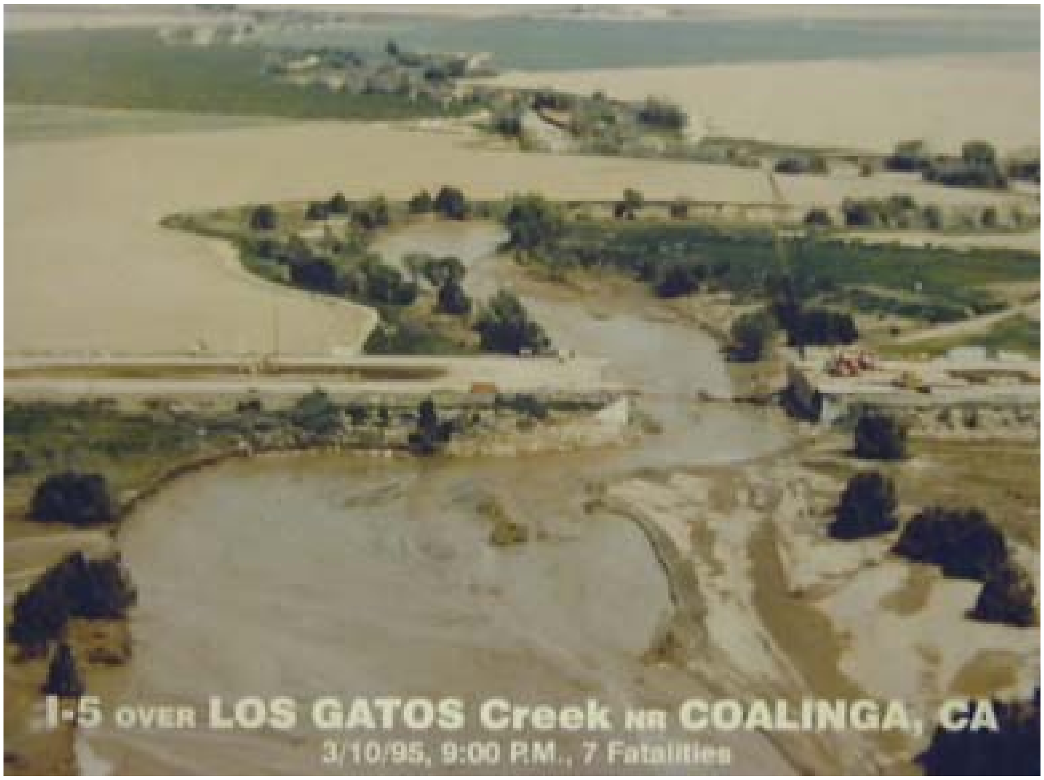

FIGURE 5 View of Arroyo Pasajero looking downstream toward I-5 bridge. Channel has shifted to left and is severely contracted by the bridge opening.

the 1995 flood. The lower 28 ft of the foundation columns were of unreinforced concrete.

Contraction and local pier scour during the flood lowered the bed below the reinforcing steel level. The dynamic forces of currents acting on the webwall and debris that might have collected on the columns could then have snapped the unreinforced columns.

The three-span bridge was supported by four six-column bents, all of which were completely destroyed during the collapse. A 1:20 scale model of one of the was tested in the FHWA hydraulics lab to determine how much local pier scour might have contributed to the collapse. There was special interest in the effect of the webwall, because it was added after the original design.

The model study indicated that local pier scour was probably 6 to 9 ft, depending on the coincidence of contraction scour and pier scour. The webwall contributed as much as 2.6 ft or as little as 0.5 ft, depending on the elevation of the bed during the flood. If the webwall had extended well below the stream bed, it could have added as much as 9 ft to the local pier scour.

Woodrow Wilson Bridge Model Study

The proposed piers for the replacement Woodrow Wilson Bridge, which is currently under construction, were modeled at two laboratories and were evaluated using the Texas A&M University SCRICOS technique (*17*) and Annandale's Erodibility Index method (*18*) to complement the physical model studies.

Small scale 1:100 tests were conducted at the Turner Fairbank Highway Research Center in the 6-ft-wide flume in the FHWA hydraulics laboratory (*19*). The piers were complex geometric designs; the piles were capped near low water surface levels; and the flow obstruction during the 100-year and 500-year floods included the pile groups, the pile caps, and the pier stems. Scour computations were further complicated by the requirement to provide protection against vessel impact for the bascule piers that support the bridge's lift spans. More than 30 physical model scenarios were run to evaluate scour for a variety of combinations of existing and proposed bridge piers. Figure 6 illustrates a typical model for this bridge. Part of an iterative design process, the models were conducted interactively with the design team and provided guidance on the impact of several alternative foundation designs.

Four larger-scale physical model tests were conducted under the direction of Dr. Max Sheppard at the U.S. Geological Survey Biological Research Division hydraulics laboratory at Turners Falls, Massachusetts (*20*). That lab features a 20- x 12- x 200-ft flume with very large flow capacity. The large-scale tests were designed to complement the small-scale results, but they were costly and time consuming and it was not feasible to test the numerous design iterations at that scale.

The modeling technique for the small-scale tests was to scale flow depths' velocities and pier geometry according to the *Fr* scaling ratios. Then a bed material was selected to attain an incipient motion velocity near the model velocity from the *Fr*. Model scour measurements were scaled to prototype dimensions geometrically at the same ratios as the flow depths, and the pier dimensions were scaled.

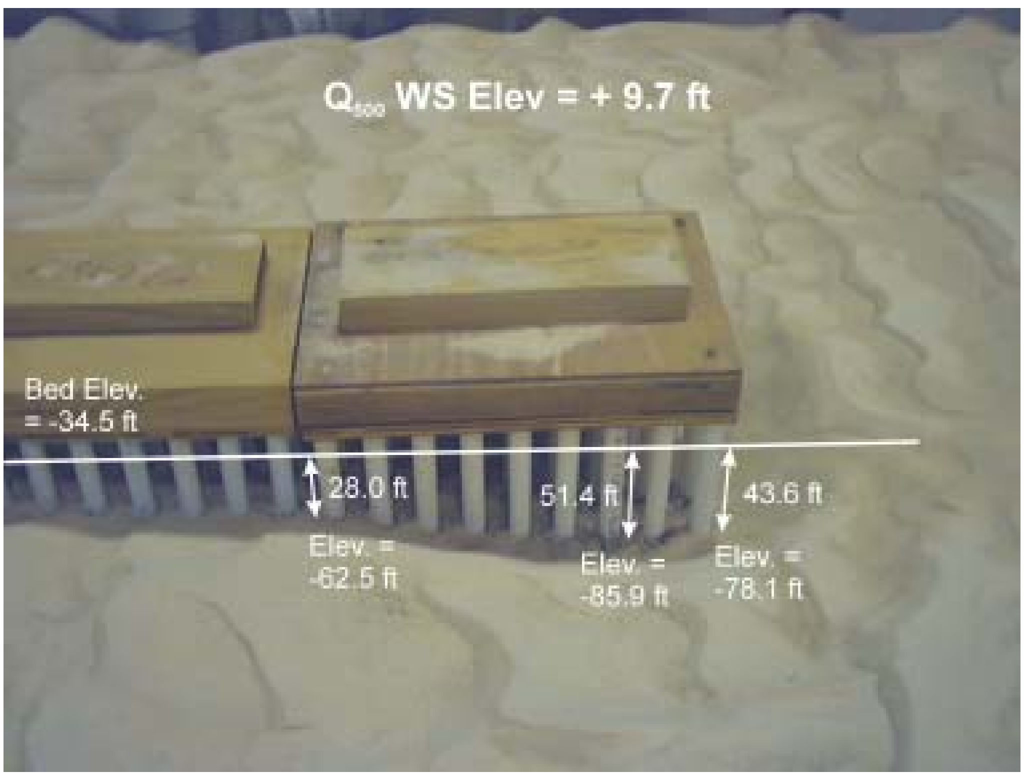

FIGURE 6 More than 30 physical model scenarios were tested for proposed replacement Woodrow Wilson Bridge over the Potomac River between Virginia and Maryland.

FIGURE 7 Concrete dolphins originally proposed for vessel impact significantly increased scour at the bascule piers for the Woodrow Wilson Bridge.

As a laboratory expedient to avoid frequent change of bed material in the flume, a procedure was developed for making fine adjustments in the scour predictions on the basis of the assumption that the HEC-18 pier scour equation had the correct exponents to account for the effects of small changes in flow depths and velocities.

Both the large- and small-scale tests showed that the three 45-in.-diameter concrete vessel impact dolphins shown in Figure 7 caused a jet of flow to focus at the pier and almost doubled the scour. Small-scale tests showed that different arrangements and numbers of dolphins could alleviate that problem, but a fender ring of small diameter piles reduced the pier scour.

The model studies focused on a main channel pier, M1, the bascule pier on the Maryland side of the main channel, and on a secondary channel pier, M9. A number of other piers and combinations of piers were also tested during the design process. The dimensions of the foundation for the bascule piers changed four times during the study period. The footing or pile cap for the M1 foundations for the inner- and outer-loop bridges started as a 56- x 101-ft footing supported by battered steel shaft piles 42 in. in diameter. This footing was later changed to be 78 ft wide with 54-in. battered piles, then to 96 ft wide with 54-in. battered piles, and finally to 87.5 x 126.5 ft with 72-in. vertical steel shaft piles. At each interval of the design process, model study results were used to predict relative impacts on scour of the design changes. Small-scale model results predicted maximum local scour for the final M1 design to be 47.0 ft for Q_{100} and 51.4 ft for Q_{500}. Scour was slightly less when the fender ring was placed around the pier. Maximum local scour at M9 was predicted to be 26 ft for Q_{100} and 29.6 ft for Q_{500}.

Evaluation of scour for this bridge was further complicated by the subsurface soil profile, which included relatively erosion-resistant cohesive soil layers above the predicted scour depths for several of the piers. Samples from soil boring of these layers were tested in the Erosion Function Apparatus developed at Texas A&M University (*21*) under an NCHRP project. Additional soil tests were done as suggested by George Annandale to support the Erodibility Index procedure. Scour in the erosion-resistant soil layers was evaluated by both the Texas A&M SCRICOS method and Annandale's Erodibility Index method. Both methods are described in the appendices of the fourth edition of HEC-18 (*1*).

A scour review team composed of state DOT hydraulic engineers, the consultants responsible for the scour evaluation, geotechnical consultants, structural consultants, and researchers was organized to decipher the results of various scour evaluations for this bridge. The review team considered the HEC-18 and model study results to represent the worst-case assumptions for the erosion-resistant cohesive soil layers and used results from the SCRICOS and Erodibilty Index to moderate scour prediction only when these methods indicated that the implied safety factor was very high.

CONCLUSIONS

Model studies of scour in the analysis of bridge failure and in the design of new bridges have been very effective. In analysis of the Schoharie Creek Bridge failure, the model studies gave a scour depth of 15-ft versus a measured prototype depth of 14 ft. They showed the effect of a bend on the distribution of velocity and that the velocity around the pier was sufficient to remove the riprap.

The model study of the Hatchie River bridge failure established that the combination of stream lateral migration and pier scour caused the failure. Pier scour was not a large component in this failure, but it was probably enough to trigger the failure after lateral migration exposed the relatively shallow foundation for pier bent. This failure, more than any other, demonstrated the value of plotting periodic cross sections against a background of the pier foundation profiles as an underwater inspection expedient.

The model study of the Arroyo Pasajero bridge failure showed that the webwalls that were added for the lateral stability of the columns after an earlier headcut did not add as much local scour as might have been expected. A combination of contraction scour and local pier scour probably lowered the stream bed below the level of reinforcement of the columns, and the plain concrete sections of the columns could not resist the lateral forces. This failure demonstrated the need for an interdisciplinary team for scour evaluations.

Designers used model studies of the Wilson Bridge Replacement Bridge to make more logical decisions in the iterative design process for this major Interstate bridge. Model studies were used in conjunction with new techniques developed to evaluate scour in erosion-resistant cohesive soils.

Model studies have increased our knowledge of scour and improved scour equations. However, research is needed to determine appropriate methods for conducting model studies. Research is needed in selection of the bed material and scale ratios in the model, taking into consideration the need to simulate the bed configuration, sediment motion, beginning of sediment motion, and sediment transport in the prototype.

REFERENCES

1. Richardson, E. V., and S. R. Davis. *Hydraulic Engineering Circular No. 18: Evaluating Scour at Bridges—Fourth Edition.* FHWA-NHI 01-001. FHWA, U.S. Department of Transportation, 2001.
2. Lagasse, P. F., J. D. Schall, and E. V. Richardson. *Hydraulic Engineering Circular 20: Stream Stability at Highway Structures, Third Edition.* FHWA-NHI 01-002. FHWA, U.S. Department of Transportation, 2001.
3. Lagasse, P. F., L. W. Zevenbergen, J. D. Schall, and P. E. Clopper. *Hydraulic Engineering Circular 23: Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance, Second Edition.* FHWA-NHI 01-002. FHWA, U.S. Department of Transportation, 2001.
4. Richardson, E.V., P. F. Lagasse, J. D. Schall, J. F. Ruff, T. E. Brisbane, and D. M. Frick. Hydraulics, Erosion and Channel Stability Analysis of the Schoharie Creek Bridge Failure, New York. Resource Consultants, Inc.; Colorado State University, Fort Collins, Colo., 1987.
5. Richardson, E.V., J. F. Ruff, and T. E. Brisbane. Schoharie Creek Bridge Model Study. In *ASCE Hydraulic Engineering,* Reston, Va., 1988.
6. Thompson, P. L. Hatchie River US-51 Bridge Failure. In *Transportation Research Record 1279,* TRB, National Research Council, Washington, D.C., 1990, pp 24–35.
7. Jones, J. S. Model Study of Hatchie River U. S. 51 Bridge. *Public Roads,* Vol. 54, No. 1, FHWA, McLean, Va., June 1990, pp. 160–165.
8. Jackson, L. E., P. L. Thompson, and E. V. Richardson. Hatchie River and Schoharie Creek Bridge Failures. *ASCE Compendium of Papers from Water Resources Engineering Conferences, 1991 to 1998* (E. V. Richardson and P. E. Lagasse, eds.), Reston, Va., 1999.
9. Richardson, E. V., J. S. Jones, and J. C. Blodgett. Cause of the 1995 California I-5 Bridge Failure. *ASCE Compendium of Papers from Water Resources Engineering Conferences, 1991 to 1998* (E. V. Richardson and P. E. Lagasse, eds.), Reston, Va., 1999.
10. Warnock, J. E. Hydraulic Similitude. In *Engineering Hydraulics* (H. Rouse, ed.) John Wiley and Sons, New York, 1950.
11. Richardson, E. V., D. B. Simons, and P. F. Lagasse. *Hydraulic Design Series 6: River Engineering for Highway Encroachments.* FHWA-NHI 01-004. FHWA, U.S. Department of Transportation, 2001.
12. Vanoni, V. A. *Sedimentation Engineering*. ASCE M & R No. 54. ACME, Reston, Va., 1977.
13. Richardson, E. V., and D. B. Simons. Resistance to Flow in Sand Channels. *Proc., XII Congress of International Association for Hydraulic Research.* Vol. l, No. A 18, Fort Collins, Colo., 1967.
14. *Highway Accident Reports: Collapse of the New York Thruway (I-90) Bridge Over The Schoharie Creek, Near Amsterdam New York, April 5, 1987*. Report NTSB/HAR-88/02. National Transportation Safety Board, Washington, D.C., 1988.
15. Wiss, Janney, Elstner Associates, Inc., and Muser Rutledge Consulting Engineers. *Final Report—Collapse of the Thruway Bridge at Schoharie Creek.* New York Thruway Authority, Albany, N.Y., 1987.
16. Palmer, R., G. Turkiyyah, and P. Harmsen. *NCHRP Report 426: CAESAR: An Expert System for Evaluation of Scour and Stream Stability.* TRB, National Research Council, Washington, D.C., 1999.

17. Briaud, J. L., F. Ting, H. C. Chen, S. R. Gudavalli, S. Perugu, and G. Wei. SCRICOS: Prediction of Scour Rate in Cohesive Soils at Bridge Piers. *Journal of Geotechnical Engineering,* ASCE, Vol. 125, Reston, Va., 1999, pp. 237–246.
18. Annandale, G. W. Estimation of Bridge Pier Scour Using the Erodibility Index Method. *ASCE Compendium of Papers from Water Resources Engineering Conferences, 1991 to 1998* (E. V. Richardson and P. E. Lagasse, eds.), Reston, Va., 1999, pp. 83–97.
19. Jones, J. S. Hydraulic Testing of Woodrow Wilson Bridge Designs. *Public Roads*, Vol. 63, No. 5, FHWA, McLean, Va., March/April 2000, pp. 40–44.
20. Sheppard, D. M., and M. Odeh. *Local Sediment Scour Model Tests for the Woodrow Wilson Bridge Piers*. Maryland State Highway Administration, Baltimore, 1999.
21. Briaud, J. L., F. Ting, H. C. Chen, Y. Cao, S. W. Han, and K. Kwak. Erosion Function Apparatus for Scour Rate Predictions. *ASCE Journal of Geotechnical and Geoenvironmental Engineering,* Vol. 127, No. 2, 2001, pp. 105–113.

Monitoring and Plans for Action for Bridge Scour

Instruments and State Departments of Transportation Experiences

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There are more than 484,000 bridges over waterways in the U.S. National Bridge Inventory. In 1988 the FHWA issued a Technical Advisory (TA) to guide states in evaluating bridges over waterways for vulnerability to failure from scour. The evaluation to date has identified 26,140 bridges as scour-critical. These bridges require replacement or countermeasures to protect them from failure. The TA recommends that a Plan of Action (POA) be developed for each scour-critical bridge. Monitoring a bridge for scour, combined with a POA that details procedures if scour threatens a bridge, is an acceptable countermeasure. Research sponsored by the FHWA and TRB has developed several fixed and portable scour-monitoring instruments. Fixed instruments are low-cost sonar, buried stainless steel pipe with a magnetic sliding collar, and float-out cylinders with radio transmitters. These instruments can use solar panel with battery for power, data loggers to store the data, and telemetry to send the data to central locations so personnel can act if the bridge is in danger of failure. The portable instruments include sounding weights, rods, fathometers, and geophysical devices. This study describes the instruments and experience with their installation.

In the United States 604,279 bridges are listed in the National Bridge Inventory (NBI). These numbers include federal highway system, state, county, and city bridges. Approximately 84% of these bridges are over water. Bridge failures cost millions of dollars each year as a result of not only the direct costs of replacing and restoring bridges but also the indirect costs related to disruption of transportation facilities. Hydraulic and geomorphic factors (scour and stream instability) cause 60% of bridge failures in the United States (*1*). Scour is the result of the erosive action of flowing water that removes bed material from around the abutments and piers that support the bridge. Stream instability is the dynamic action of a stream by which the channel migrates back and forth and up and down, which can also cause bridge failure.

In 1988, as the result of a 1987 bridge failure that claimed 10 lives, the FHWA of the U.S. Department of Transportation (DOT) issued a Technical Advisory (TA) to guide states in developing and implementing a scour evaluation program for all bridges over water for vulnerability to scour and stream instability (*2*). As part of the TA, FHWA issued an Interim Procedure for Evaluating Scour at Bridges (*3*) and recommended that states develop a Plan of Action (POA) for each scour-critical bridge. The 1988 TA and its Interim Procedures were superseded in 1991 by TA 5140.23 and Hydraulic Engineering Circular (HEC) 18 (*4*, *5*). To date, guidance has been expanded on scour in four editions of HEC 18, on stream stability in three editions of HEC 20, and on countermeasures in two editions of HEC 23 (*5*–*7*).

Evaluation results are coded in Item 113 of the NBI following guidance in the FHWA document *Recording, and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges* (*8*). Item 113, Scour-Critical Bridges, uses a single-digit code from 0 to 9 to identify the current status of a bridge's vulnerability to scour. A score of 0 indicates that the bridge has failed and is closed to traffic, and 9 indicates that the bridge foundations are on dry land well above floodwater elevations. In addition, the letter N indicates that the bridge is not over a waterway, U signifies that the bridge's foundations are "unknown," and T indicates that the bridge is over "tidal" waters. All bridges coded 3 or less are scour-critical, and the TA recommends a POA be developed. A scour-critical bridge is a bridge whose foundation elements are determined to be unstable for the calculated or observed stream stability or scour conditions.

EVALUATION RESULTS

Table 1 presents results of states' scour evaluations as of April 2002 (*9*). As indicated, the evaluations of bridges for scour and stream instability are 93.1% complete. However, all bridges over water have been screened. There are 26,140 scour-critical bridges in the United States that require a POA. Another 88,912 bridges have unknown foundations, and some of these may be scour-critical. With limited funds available, not all scour-critical bridges can be replaced or repaired. Therefore, countermeasures against scour and stream instability should be taken until the bridge can be replaced or repaired. However, many bridges may only need a countermeasure [e.g., a road with low average daily traffic (ADT)]. Most scour-critical bridges and many bridges with unknown foundations should be monitored and inspected after high flows. During a flood, scour is generally not visible and during the falling stage of a flood, scour holes generally fill in. Visual monitoring during a flood and inspection after a flood cannot fully determine if a bridge is safe. Instruments to measure or monitor scour can help to assess the scour condition for this uncertainty.

PLAN OF ACTION

"The plan of action should include instructions regarding the type and frequency of inspections to be made at the bridge (scour-critical), particularly in regard to monitoring the performance and closing of the bridge if necessary, during and after flood events. The plan of action should include a schedule for the timely design and construction of scour countermeasures determined to be needed for the protection of the bridge" (*4*).

The POA guides inspectors and engineers on actions that can be taken before, during, and after flood events to protect the traveling public. The POA should contain management strategies for timely

TABLE 1 Results of Bridge Scour Evaluations as of April 2002

TOTAL HIGHWAY BRIDGES AS OF 4/15/02 (5/3/02) **Attachment F**

State	Bridges Over Waterways	Scour Screening										Scour Evaluations		
		Low Risk				Scour	Unknown		Scour	Total		Total	Evaluation	
		Culverts	Screened	Assessed	Total	Susceptible	Foundations	Tidal	Critical	Screened	%	Evaluated	Candidates	%
AK	810	34	0	372	406	0	201	53	150	810	100	556	53	91
AL	14108	5559	0	2222	7781	3386	2797	0	144	14108	100	7925	3386	70
AR	11623	2304	0	3518	5822	0	5548	0	253	11623	100	6075	0	100
AZ	5561	3482	40	956	4478	71	172	0	840	5561	100	5278	111	98
CA	15386	2910	2507	5536	10953	412	3661	29	324	15379	100	8770	2955	75
CO	6793	1339	0	4987	6326	12	38	0	417	6793	100	6743	12	100
CT	2360	572	0	1245	1817	48	82	0	413	2360	100	2230	48	98
DC	94	0	0	93	93	0	0	0	1	94	100	94	0	100
DE	576	181	0	270	451	0	0	0	125	576	100	576	0	100
FL	8258	1711	653	2604	4968	196	2681	149	264	8258	100	4579	998	82
GA	12147	5332	0	732	6064	0	6007	0	76	12147	100	6140	0	100
HI	860	130	50	566	746	24	11	2	64	847	98	760	89	90
IA	23493	3228	642	14724	18594	87	3996	0	816	23493	100	18768	729	96
ID	3209	1073	0	1284	2357	0	587	0	265	3209	100	2622	0	100
IL	21641	3914	160	15633	19707	3	1272	0	614	21596	100	20161	208	99
IN	15903	1001	0	12703	13704	56	444	0	1699	15903	100	15403	56	100
KS	23803	6100	57	15685	21842	1432	93	0	441	23808	100	22226	1484	94
KY	11225	2641	0	8110	10751	11	424	0	39	11225	100	10790	11	100
LA	9891	0	0	3060	3060	810	5473	0	548	9891	100	3608	810	82
MA	2467	281	0	679	960	232	404	1	870	2467	100	1830	233	89
MD	3163	1017	0	992	2009	0	560	0	594	3163	100	2603	0	100
ME	1867	272	0	1039	1311	18	191	112	235	1867	100	1546	130	92
MI	7575	1111	0	2700	3811	2375	709	0	680	7575	100	4491	2375	65
MN	11331	4460	31	5474	9965	377	509	0	480	11331	100	10414	408	96
MO	20912	4026	0	16435	20461	308	18	0	101	20888	100	20562	332	98
MS	14790	2269	0	3137	5406	14	8608	0	762	14790	100	6168	14	100
MT	3578	154	304	1289	1747	34	1746	0	51	3578	100	1494	338	82
NC	14135	4485	49	3227	7761	14	6195	81	88	14139	100	7800	140	98
ND	4132	771	113	1023	1907	20	2132	0	73	4132	100	1867	133	93
NE	14830	2780	13	2616	5409	1237	7762	0	395	14803	100	5791	1277	82
NH	1755	163	80	1388	1631	30	50	0	44	1755	100	1595	110	94
NJ	3551	316	0	2428	2744	49	344	40	367	3544	100	3111	96	97
NM	3001	1563	172	671	2406	73	498	0	24	3001	100	2258	245	90
NV	889	555	31	114	700	33	53	0	102	888	100	771	65	92
NY	12096	1619	0	9474	11093	129	62	133	679	12096	100	11772	262	98
OH	23326	1338	0	16283	17621	5273	241	0	191	23326	100	17812	5273	77
OK	20835	5981	8	14345	20334	0	0	0	501	20835	100	20827	8	100
OR	5480	252	0	1782	2034	17	1890	68	1471	5480	100	3505	85	98
PA	17328	1680	886	7315	9881	1442	461	0	5544	17328	100	14539	2328	86
PR	1605	244	63	758	1065	26	372	33	109	1605	100	1111	122	90
RI	337	30	0	176	206	0	0	0	131	337	100	337	0	100
SC	7784	1054	0	1170	2224	0	3704	155	1701	7784	100	3925	155	96
SD	5373	1000	0	1653	2653	136	2584	0	0	5373	100	2653	136	95
TN	16520	7816	0	6022	13838	391	1236	0	1055	16520	100	14893	391	97
TX	40562	16647	0	12442	29089	757	9923	49	673	40491	100	29762	877	97
UT	1682	435	0	527	962	101	447	0	172	1682	100	1134	101	92
VA	9818	2747	0	7014	9761	2	0	0	55	9818	100	9816	2	100
VT	2304	68	0	1348	1416	373	246	0	298	2333	101	1714	344	83
WA	5157	145	0	3729	3874	72	305	0	906	5157	100	4780	72	99
WI	10689	1682	0	6722	8404	215	2002	0	68	10689	100	8472	215	98
WV	5742	350	4	3404	3758	25	1735	0	225	5743	100	3979	28	99
WY	1931	396	12	1063	1471	32	438	0	2	1943	101	1461	32	98
Nationwide	484286	109218	5875	232739	347832	20353	88912	905	26140	484142		368097	27277	
Percent		22.6%	1.2%	48.1%	71.8%	4.2%	18.4%	0.2%	5.4%	100.0%		93.1%	6.9%	

bridge replacement or repair or countermeasures to protect the bridge from stream instability and scour. It should detail the location of the bridge; its identification number; type of foundation and foundation material; importance of the roadway to the transportation network; ADT; importance of the route to emergency facilities, schools, detour, and evacuation; detour instructions; media alert instructions; maintenance supervisors; state patrol notification; sources of repair materials; notification instructions; and POA author and sign-off authority.

The POA should contain strategies for inspection that detail the type and frequency of inspections. Are the normal 2-year inspection and 5-year general underwater inspection sufficient or should inspections be more frequent? Is there a need for continuous monitoring? Should the bridge be inspected after every flood of a given magnitude or frequency? What kind of inspection will be conducted and how? Who is responsible for additional inspections? Inspection strategies should contain a determination of what constitutes a scour-critical condition and instructions for action when it is reached.

The POA should have closure strategies that identify a person who has authority for closing and reopening a bridge. (States have different guidelines on who can and cannot close a bridge). The criteria for a bridge closure should be established by the bridge owner on the basis of parameters that would make a bridge critical for scour conditions (observed scour, scour calculations, water level, discharge, rainfall, flood forecasting, buildup of ice or debris, movement of riprap, and scour monitoring). The POA should detail any load restrictions, lane closure, or complete bridge closure.

An interdisciplinary team should develop the POA. The team should, as a minimum, include hydraulic, geotechnical, and structural engineers.

A generic POA, developed by the FHWA, is presented in Figure 1 (*10*).

SCOUR-MONITORING INSTRUMENTS

Scour monitoring at a bridge involves either portable or fixed instruments.

Portable instruments are used if only occasional measurements are required, such as after a major flood or for monitoring many bridges relatively infrequently. Fixed instruments are used if measurements are frequent or regular or if ongoing monitoring (e.g., weekly, daily, or continuous) is required. The bridge's physical conditions, such as its height above the water and type of superstructure, can influence the decision to use portable or fixed equipment. For example, it is complicated to use portable instruments from the bridge deck if the bridge is very high off the water or has a large deck overhang or projecting geometries. Taking a measurement from a boat requires a boat ramp near the bridge and clearance under the bridge to permit safe passage of a boat.

Bridges with large spread footings or pile caps or those in very deep water can also complicate the installation of some types of fixed instruments. Stream channel characteristics include sediment and debris loading, air entrainment, ice accumulation, or high velocity flow, all of which can adversely influence measurement sensors in portable or fixed instruments. Traffic safety issues include the need for traffic control or lane closures when fixed instruments are being installed or serviced or when a measurement is being taken from a portable instrument from the bridge deck. Therefore, the selection of the category (portable or fixed) and type of instrument for monitoring is not always straightforward.

Bridge Identification: _____; Location of Bridge: ____; Year Built: ____;
Replacement Plans (if scheduled):_____Foundation Type: _______ Foundation Soils Types: ______
ADT:___________; Service to Emergency Facilities or Evacuation (Y/N): ___________
Sources of Scour-Critical Rating (assessment, analysis, and/or observation):
____
Comments About Rating (e.g., analysis did not account for erosion-resistant material; emergency riprap placed after last flood, etc.):
____
Inspection and Monitoring:

- Increase Inspection Frequency: _______________________________________
- Types (probing, diving, inspection of banklines): _________________________
- Special Inspection Criteria (after bankfull events, during major events): _______

____
Monitoring Type (fixed instrumentation, portable instrumentation): ________________
Criteria for Monitoring: ____
Closure Plans (limit loads; lane closure; full closure): _________________________
____
Criteria for Closure (discharge; floodwater elevation; flood forecast; scour soundings):
____
Authorization for Closure (Bridge Maintenance engineer; Inspector; Police; Statewide Bridge Closure Procedure):
____
Detour Route: ____

Criteria for Reopening Bridge: ____
Countermeasures Considered: (1)____________________; Cost: $ _______________
(2)____________________ ; Cost: $ ______________
(3)____________________ ; Cost: $ ______________
Countermeasure Recommended: __________________________; Status: __________
____
Author(s) of POA:__________________________________; Date: _______________
Concurrences on POA: ________________________, __________________________,

FIGURE 1 Generic POA developed by FHWA.

Fixed Instrumentation

Fixed scour-measuring and -monitoring instruments can be grouped into four broad categories (*11–14*):

1. Sounding rods—manual or mechanical device (rod) to probe the streambed;
2. Fathometers—instruments built around commercially available fathometers;
3. Buried or driven rods—devices with sensors on a vertical support, placed or driven into the streambed; and
4. Other buried devices—active or inert buried sensor (e.g., buried transmitter).

Sounding Rods

The sounding rod consists of a tube fastened to the bridge, a sliding rod inside, and a footpad resting on the bed (*11*). As the scour hole develops, the rod drops downward. Sounding rods can be manually or automatically operated. In a manual reading, the displacement of the rod from a previous reading is noted. An automatic sounding rod has a linear displacement measuring device to track the rod location, which may or may not be recorded by a data logger and transferred by telemetry to another location. Sounding rods are well suited for coarse bed channels and for checking on riprap displacement. In sand-bed streams, sounding rods may penetrate into the sand bed, or binding, which influences their performance and accuracy. They are best suited for piers or abutments where the instrument can be mounted in a vertical orientation. Installations along the slope of a spill-through abutment are susceptible to additional binding forces. Other limitations include a maximum extension limited by the unsupported length of the rod, vibration of the rod in fast moving water, and binding from fine-grained sediments.

Fathometers

A fathometer consists of an instrument box (black box) and a transducer. The instrument box initiates and receives a sound pulse, determines the time between the two, converts the time to distance, records it, and sends it by telemetry if desired. The transducer is in the water and converts the pulse to a sound wave, which it propagates to the bed. The transducer then receives the returning echo. With a sonar fixed instrument, the instrument is typically mounted in an instrument shelter, and the transducer is mounted on the bridge pier or abutment. Commercially available sonar devices range from low-cost fish finders to survey-grade fathometers. A research project sponsored by NCHRP demonstrated that a low-cost sonar could be effectively used as a fixed sonar instrument for scour monitoring (*11*, *12*, *14*).

In a fixed installation, the transducer is aimed at the location where maximum scour is predicted to occur. The signal must not be obscured by debris or ice. Loss of signal associated with the entrainment of air (which was experienced in the laboratory flume) or very high concentrations of sediment can occur, but is not a major concern in many applications. However, there may be cases in the field where highly turbulent, air-entrained flow conditions or suspended sediment will preclude the use of these instruments.

The NCHRP project developed a mounting for the transducer that permits service from the bridge deck or above water. Steel or a polyvinyl chloride (PVC) conduit is bracketed to the bridge substructure to aim the sonic transducer at the most likely location for scour. The transducer is encased in a PVC probe, which is pushed down through a larger diameter steel or PVC conduit. The transducer is positioned so that it protrudes through a fitting located below water at the bottom of the conduit. With this arrangement the transducer is serviceable from above water.

The advantages of low-cost sonar instruments include the ability to fabricate from off-the-shelf components or purchase a complete package (*12*). This type of instrument can provide a time history of scour conditions and tracks both removal and replacement of material as the flood passes. They can also be used in deep-water situations where other fixed instruments cannot. There can be problems with water temperature fluctuations, since the speed of sound in water is a function of temperature; however, within the expected accuracy limits of a scour measurement (typically 0.3 m), this is not a concern for most installations. If necessary, corrections can be made for temperature and, in very limited cases, for salinity (which can also affect the speed of sound) as a postprocessing step.

Buried and Driven Rods

This class of scour-measuring devices includes all sensors and instruments supported by a vertical support member such as a pipe, rail, or column that can be placed vertically in the bed where scour is expected to occur. There is no evidence that buried or driven devices either enhance or reduce scour at the pier. The support column can be installed by driving, jetting, augering, or excavation and burying. Examples of this class of device include the sliding collar device, the New Zealand Scubamouse, and the Wallingford Tell-Tail devices (*13*, *15*, *16*).

In the United States, the most widespread application of buried or driven rods has been with the sliding collar device, which was developed and promoted through NCHRP Project 21-3. The sliding collar has an open architecture with attached 152-mm (6-in.) magnets. It slides down a stainless steel support pipe 51 mm (2 in.) in diameter. In manual operation, the position of the collar is located with a sensor (probe) that consists of a magnetic switch attached to a battery and buzzer on a long graduated cable. In operation, the probe is lowered through the annulus of the support pipe, and the buzzer activates when the sensor reaches the magnetic collar. Collar position is determined by using the graduated cable to determine the distance from an established datum near the top of the support pipe to the magnetic sliding collar.

In automated operation of the magnetic sliding collar, a string of magnetically actuated reed switches is positioned at predetermined intervals in the stainless steel pipe driven into the streambed. Magnets on the sliding collar activate the reed switches as the sliding collar slides down the stainless steel pipe as scour develops. A datalogger provides excitation voltage for a brief sampling period. To protect the stainless steel pipe from damage from debris or ice, often only the head of the device protrudes from the streambed in front of a pier or adjacent to an abutment. A flexible conduit with the wiring for the automated readout carries the signal by a less vulnerable route, such as along the pile cap or pier footer and up the downstream face of a pier to a datalogger.

Other Buried Devices

This class of devices includes cylinders, which are buried in the bed of a river at various elevations. When scour exposes the cylinders, they float out of the scour hole. These cylinders can be untethered or tethered to the pier or abutment. Obtaining scour data from a tethered device can be as simple as visually observing which tethered devices have been removed from the hole by scour. Untethered devices incorporate a motion-activated transmitter. A receiver on the bridge or stream bank senses when a transmitter has moved and activated (*17*).

A buried transmitter float-out device was developed by ETI Instruments for application on bridge piers over ephemeral stream systems. This device consists of a radio transmitter buried in the channel bed at a predetermined depth. When the scour reaches that depth, the float-out device rises to the surface and begins transmitting a radio signal that is detected by a receiver in an instrument shelter on the bridge. The receiver stores the information and can, through telemetry, notify a central location that the cylinder has floated out. Limitations of this type of device include installation difficulties in coarse bed channels or deep water, and the shelf-life of the battery necessary for the radio transmitter in the float-out device. Shelf life is around 10 years. The float-out device can be monitored by the same type of instrument shelter and data logger currently used to telemeter low-cost sonar or automated sliding collar data. Installation requires a conventional drill rig with a hollow stem auger. After the auger reaches the desired depth, the float-out transmitter is dropped down the center of the auger. Substrate material refills the hole as the auger is withdrawn.

Instrument Shelter
The instrument shelter contains the data logger, telemetry, and a solar panel and gell-cell battery for power. The shelter is similar for all fixed instruments—sliding collar, low-cost sonar, or float-out cylinders. The data logger monitors the sliding collar and sonar scour instruments, typically taking readings every hour and transmitting the data once per day to a computer at a central location. Telemetry may be by cell phone, satellite, or landline. A threshold elevation is defined that, when reached, initiates a digital, voice, or data phone call to a pager. The bridge number is transmitted to identify the bridge where scour has occurred. The float-out devices are monitored continuously, and if one of these devices floats to the surface, a call is automatically made to designated personnel.

Portable Instrumentation
A wide variety of portable instruments has been used for making scour measurements. In general, a portable scour measurement can taken by

- Physical probes,
- Fathometers (sonar), and
- Geophysical instruments.

Physical Probes
Sounding poles and sounding weights on a cable are the most common physical probes. Sounding poles are long poles that can probe the bottom. Sounding weights, sometimes referred to as lead lines, typically consist of a torpedo-shaped weight suspended by a measurement cable. Sound weights range from 3.7 to 75 kg (10 to 200 lb). The lighter weights can be used with a hand line. Heavier weights require a crane and reel. They can be used from the bridge or from a boat. Physical probes collect discrete data. Their effectiveness can be limited by large depths and velocity (e.g., during flood flow condition) or accumulation of debris or ice. An advantage of physical probing is that it is not affected by air entrainment or high sediment loads. The method can be effective in fast, shallow water.

Fathometers
Fathometers or acoustic depth sounders, as described in the fixed instrument section, are widely used for portable scour measurements. Fish finders and precision survey-grade hydrographic survey fathometers are used. Low-cost fish-finder type sonar instruments have been widely used for bridge scour investigations. When the measurements are taken from the bridge, transducers are attached to a pole, hand-line, or tethered float, or are attached to a boom. Tethered float platforms include kneeboards and pontoon-style floats. The size of the float is important for stability in fast-moving, turbulent water.

Floating or nonfloating systems can be deployed from a bridge inspection truck. This is particularly useful when the bridge is high off the water. For example, bridges that are >15 m (>50 ft) off the water are typically not accessible from the bridge deck and require this approach for monitoring.

An articulated arm to position a sonar transducer was developed under an FHWA research project (*18*). The system is mounted on a trailer and can be used on bridge decks from 5 to 15 m (16 to 50 ft) above the water surface. On the basis of the angle of the boom and the distance between the boom pivot and transducer, an onboard computer calculates the transducer's position relative to a known position on the bridge deck. An NCHRP project has developed a truck-mounted articulated arm to position a sonar transducer using the same concepts (*19*).

A Sonar Scour Vision system was developed by American Inland Divers, Inc., with a rotating and sweeping 675-Khz high-resolution sonar (*20*). The transducer is mounted in a relatively large hydrodynamic submersible, called a fish, that creates a downward force adequate to submerge the transducer in velocities exceeding 6 m/s (20 ft/s). Given the forces created, the fish must be suspended from a crane or boom truck on the bridge. From a single point, the system can survey up to 100 m (328 ft) radially. Data collected along the face of the bridge can be merged into a real-time three-dimensional image with a range of 90 m (295 ft) both upstream and downstream of the bridge.

Manned boats are used as a platform for scour measurements. They generally require adequate clearance under the bridge and nearby launch facilities. This can be a problem at flood conditions when the river stage may approach or submerge the bridge low chord and boat ramps may be under water. Normally a fathometer is used for depth measurements and Global Positioning System (GPS) for location.

Issues of safety, launching, and clearance have led to development of a prototype for an unmanned boat. It consists of a small flat-bottom Jon boat, an 8-hp outboard motor, fathometer, and GPS with remote controls. The boat has tested successfully in six flood events (*21*).

The advantage of GPS over traditional land-based surveying techniques is that line-of-sight between control points is not necessary. GPS also works at night and during inclement weather, which can be a real advantage for monitoring scour during flood conditions. The disadvantage of GPS is the inability to obtain a measurement in locations where overhead obstructions exist, such as a tree canopy or bridge decks. However, GPS measurements up to the bridge face, without venturing under the bridge, have been successful.

Geophysical Instruments

Geophysical instruments, based on wave propagation and reflection measurements, determine the interfaces between materials with different physical properties. A primary difference between sonar and geophysical techniques is that geophysical methods provide subbottom information, whereas sonar can only "see" the water-soil interface and cannot penetrate the sediment layer. Geophysical techniques differ in the types of signals transmitted and the physical property changes that cause reflections. A seismic instrument uses acoustic signals, similar to sonar, but at a lower frequency (typically 2 to 16 Khz). Like sonar, seismic signals can be scattered by air bubbles and high concentrations of sediment. A ground-penetrating radar (GPR) instrument uses electromagnetic signals (typically 60 to 300 MHz), and reflections are caused by interfaces between materials with different electrical properties. In general, GPR will penetrate resistive materials and not conductive materials. Therefore, it does not work well in dense, moist clays or in saltwater conditions.

The best application of geophysical technology is to determine scour depth after a flood during lower flow conditions in areas of infilling. In general, the cost and complexity of the equipment and interpretation of the data are limiting factors for widespread use and application as a portable scour-monitoring device. These issues have moderated with the development of newer, lower-cost GPR devices with computerized data processing capabilities. However, GPR may still be limited by cost and complexity and often the need for borehole data and accurate bridge plan information to properly calibrate and interpret the results.

STATE EXPERIENCE WITH SCOUR MONITORS

The number of fixed instruments, as described in this paper, that have been installed by ETI is given in Table 2. This is not the total number of installations. ETI has installed fixed instruments on bridges in 16 states to monitor scour. The total number of devices installed is 292.

Among the installed devices are 48 tilts and 3 piezoelectric film devices. Tilts are devices that determine if the pier is tilting to one side from its initial direction. The presumption is that scour has caused the pier to tilt. The meter sends a signal when the pier tilts, and the change is picked up by a monitoring instrument, which records the change and sends a signal to a central location, by telemetry. Piezoelectric films are films placed on thin metal strips that generate a voltage when vibrated by the flow. They are placed at given intervals on a driven rod and connected by wire to the monitoring instrument. When scour exposes the strips of metal with piezo film they flutter, generating a voltage that the monitoring instrument records and, by telemetry, signals to a central office.

California, Arizona, and Nevada Installations

Anticipating that El Niño would drive storm events, states in the Southwest installed a variety of instruments at bridges in late 1997 and early 1998. Five bridges were instrumented in California, five in Arizona, and four in Nevada. The instruments included automated sliding collars, single- and multipier low-cost sonar, and float-out cylinders. Early warning of potential scour problems was provided by defining threshold scour levels and automated calls to pagers when that threshold was exceeded (*22*).

More than 40 float-out devices were installed at bridges in Arizona (four bridges), California (nine bridges), and Nevada (four bridges). Most devices were installed at various levels below the streambed as described above. However, several devices at bridges in Nevada were buried in riprap at the base of bridge piers to monitor riprap stability.

One of the bridges instrumented, the SR-101 bridge over the Salinas River, near Soledad, California, experienced several scour events that triggered threshold warnings during February 1998. In one case the automated sliding collar dropped 1.5 m (5 ft), which caused a pager call-out. Portable sonar measurements confirmed the scour recorded by the sliding collar. Several days later, a float-out device buried about 4 m (13 ft) below the streambed transmitted another pager call-out.

In both cases, the critical scour depth was about 6 m (20 ft) below the streambed, and no emergency action was called for to ensure public and bridge safety. Because pager call-out was ineffective in alerting maintenance personnel during nonoffice hours, a programmed voice

TABLE 2 Number of Fixed Scour Monitoring Instruments Installed as of October 2002

State	Bridges	Sonar	Sliding Collar	Float-Outs	Tilts	Piezofilm
Alabama	4	2		3	18	
Arizona	5	5		28		
Alaska	12	16				1
California	20	12	10	87	30	
Florida	2	2	1			
Hawaii	2	2	3			
Indiana	1	1	1			
Maryland	1	5				
Minnesota	1		1			
Nevada	5	4		11		1
New York	6	25	8			
New Jersey	2	2	2			
Oregon	1					1
Texas	3	1	2			
Rhode Island	1		5			
Wisconsin	1		2			
Total	67	77	35	129	48	3

synthesizer call-out to a human-operated 24-hour communications centers was implemented at other bridges. This illustrates the importance of effective and well-defined communication procedures and the on-going need for comprehensive training in scour monitoring and events at all levels of responsibility.

Alaska Installations

Installations in Alaska were used to evaluate various forms of telemetry. Cell phones provide an easy and accessible method of telemetry in many states and facilitate early warning concepts, such as pager alerts and automated data and digitized voice calls to base computer stations. However, this technology may not be reliable when it is most needed, as during a major storm when telephone circuits can become overloaded. Furthermore, in places such as Alaska, cell phone coverage is limited, and many Alaskan bridges are in remote areas not served by landlines. Given these concerns, there was an interest in evaluating satellite telemetry.

The Eagle River Bridge, located approximately 32 km (20 mi) north of Juneau, is a scour-critical bridge. Monitoring was the preferred countermeasure, rather than constructing a physical scour countermeasure, because the bridge is expected to be replaced within 10 years, and it is on a fairly low-volume road. An ultrasonic stage sensor, a low-cost sonar device, and a piezoelectric-film sensor were installed in January 2001. The sonar transducer was mounted on the nose of the pier on a breakaway bracket. The piezoelectric-film sensor was buried in the riverbed at the site of the scour hole. Telemetry included a Campbell scientific data logger coupled to a cellular telephone, modem, and voice synthesizer and an Orbcomm satellite transceiver.

The stage sensor and sonar transducers have performed well through two moderately high peak discharges. There has been no activity from the piezofilm sensors to date. This is consistent with observations from the sonar depth sounder, which indicate scour has not yet reached the upper piezofilm sensor. Large woody debris has passed through the bridge opening with no apparent damage to the instrumentation.

The Orbcomm satellite works as a repeater and sends digital packet data to a base station that, in turn, sends the data to the user via e-mail. It has worked well for daily transmission of data in a condensed format. However, it can be fairly expensive if detailed data are required. For northern latitudes, the coverage is not 100% of the time, and this occasionally results in a lag of up to 3 hr for receipt of the data. Normal lags have been acceptable at <1 hr. Initial results indicate that, with judicious use of thresholds to vary the frequency of data transmission, this telemetry method can work well for remote locations where cell phone telemetry is considered marginal and landline communication is unavailable.

The cell phone telemetry system has experienced problems with cell saturation and loss of signal. Data transmission via voice synthesizer from the site has worked well. Data transmission via modem has been problematic.

New York Installations

New York installed 33 monitors on 6 bridges. In addition, in 1994, the U.S. Geological Survey and the New York State DOT installed two scour-monitoring devices (*23*). From 1994 to date, no significant scour has occurred, a result of low flow conditions. There was also no damage from ice or debris. The New York Thruway Authority installed sounding rods on its bridges over water to detect movement of the riprap that protects bridge foundations from scour. To date these instruments have worked well.

Texas Installations

Starting in 1994, sliding collar and sonar devices were installed by the Texas DOT. A manual-readout sliding collar device installed on the US-380 bridge over Double Mountain Fork of the Brazos River recorded 1.5 m (5 ft) of scour. Three sonar devices were installed at bridges in the southeastern part of the state. During the test period, no scour occurred. However, these sonar devices demonstrated the urgent need for telemetry (remote data access) on narrow bridges where lane closures were required to reach an instrument box.

Indiana Installations

In 1997, the Indiana DOT installed two automated magnetic sliding collar and two sonar devices. A sliding collar and a sonar device were placed on the same pier on the US-52 bridge over the Wabash River, and the other two instruments were placed on the same pier on the SR-26 bridge over Wildcat Creek.

These installations used line power (instead of solar power) and short-haul modems to transmit the data from the instrument shelter on the bridge to a power pole located on the river bank. Using line power eliminated potential solar panel and battery damage from either environmental conditions or vandalism. The short-haul modems provided the capability for reliable remote access to the data.

Maryland Installations

Five piers of the Wilson Bridge over the Potamac River near Alexandria, Virginia, were instrumented with low-cost sonar scour monitors. The six locations are seviced by one instrument shelter. The shelter contains the sonar, battery, a Campbell scientific data logger coupled to a cellular telephone, modem, and voice synthesizer. A solar panel keeps the battery charged. Elevations of the bed at the six piers can be obtained at any time by placing a phone call to the system. A digitized voice provides the elevations.

SUMMARY

Fixed equipment for measuring and monitoring scour depth has been successfully used under many different bridge, stream, and climate environmental conditions. Their success has required a toolbox approach. The instruments in the toolbox are the magnetic sliding collar, low-cost sonar, piezoelectric film, and float-out cylinders. They have been installed in remote locations with solar panels for power and telemetry for instant access to information on scour depth.

Portable scour measuring instruments are physical probes with a rod or weights, for both sonar and geophysical measurements. Physical probing and sonar are useful for real-time measurment of scour during a flood and for routine measurements of bridge cross sections. Geophysical instruments are better for determing the depth of scour after the flood and the scour holes have filled in.

The development of these fixed and portable scour measuring instruments—along with GPS, remote-controlled boats, and instrumented trucks—and awareness of the need to measure and monitor scour at bridges have significantly improved the scour database, methods of predicting scour depths, monitoring of bridges for scour, and bridge safety.

REFERENCES

1. Shirole, A. M., and R. C. Holt. Planning for a Comprehensive Bridge Safety Assurance Program. In *Transportation Research Record 1290,* TRB, National Research Council, Washington, D.C., 1991, pp. 39–50.
2. *Scour at Bridges.* Technical Advisory T 5140.20. FHWA, U.S. Department of Transportation, Sept. 16, 1988.
3. *Interim Procedures for Evaluating Scour at Bridges.* FHWA, U.S. Department of Transportation, 1988.
4. *Evaluating Scour at Bridges.* Technical Advisory T 5140.23. FHWA, U.S. Department of Transportation, Oct. 28, 1991.
5. Richardson, E. V., and S. R. Davis. *Hydraulic Engineering Circular No 18: Evaluating Scour at Bridges, Fourth Edition.* Report FHWA NHI 01-001. FHWA, U.S. Department of Transportation, 2001.
6. Lagasse, P. F., J. D. Schall, and E. V. Richardson. *Hydraulic Engineering Circular No. 20: Stream Stability at Highway Structures, Third Edition.* Report FHWA NHI 01-002. FHWA, U.S. Department of Transportation, 2001.
7. Lagasse, P. F., M. S. Byars, L. W. Zevenbergen, and P. E. Clopper. *Hydraulic Engineering Circular 23: Bridge Scour and Stream Instability Countermeasures—Experience, Selection, and Design Guidance, Second Edition.* Report FHWA HI-97-030. FHWA, U.S. Department of Transportation, 2002.

8. *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges*. FHWA, U.S. Department of Transportation, 1995.
9. FHWA Office of Bridge Technoloy. http://www.fhwa.dot.gov/bridge/hyd.htm. Acessed Oct. 1, 2002.
10. *Generic Plan of Action.* FHWA, U.S. Department of Transportation, 1995.
11. Lagasse, P. F., E. V. Richardson, J. D. Schall, and G. R. Price. *NCHRP Report 396: Instrumentation for Measuring Scour at Bridge Piers and Abutments.* TRB, National Research Council, Washington, D.C., 1997.
12. Schall, J. D., G. R. Price, G. A. Fisher, P. F. Lagasse, and E. V. Richardson. *NCHRP Report 397A: Sonar Scour Monitor-Installation, Operation, and Fabrication Manual*. TRB National Research Council, Washington, D.C., 1997a.
13. Schall, J. D., G. R. Price, G. A. Fisher, P. F. Lagasse, and E. V. Richardson. *NCHRP Report 397B: Magnetic Sliding Collar Scour Monitor-Installation, Operation, and Fabrication Manual*. TRB, National Research Council, Washington, D.C., 1997b.
14. Lagasse. P. F., E. V. Richardson, J. D. Schall, J. R. Richardson, and G. R. Price. Fixed Instrumentation for Monitoring Scour at Bridges. In *ASCE Compendium of Papers ASCE Water Resources Enginineering Conferences, 1991 to 1998.* (E. V. Richardson and P. F. Lagasse, eds.), Reston, Va., 1999, pp. 505–524.
15. Melville, B. W., R. Ettma, and S. C. Jain. Measurement of Bridge Scour. *Proc., Bridge Scour Symposium.* Cosponsored by FHWA and Subcommittee of Sedimentation, Interagency Advisory Committee on Water Data, Office of Water Data Coordination, U.S. Geological Survey. Turner-Fairbank Highway Research Center, McLean, Va., 1989, pp. 183–194.
16. Waters, C. R. The HR Wallingford Scour Monitoring System. *Proc., 1994 Centenary Year Bridge Conference,* Cardiff, United Kingdom, 1994, pp. 323–328.
17. Zabilansky, L. J. *Ice Force and Scour Instrumentation for the White River, Vermont*. Special Report 96-6. U.S. Army Corps of Engineers, Cold Regions Research and Engineering Laboratory, Hanover, N.H., April 1996.
18. Bath, W. R. Remote Methods of Underwater Inspection of Bridge Structures. FHWA Report RD-99-100. FHWA, Turner-Fairbank Highway Research Center, McLean, Va., 1999.
19. Schall, J. D. Development of Portable Scour Monitoring Equipment. Report for NCHRP 21-07. TRB, National Research Council, Washington, D.C., Sept., 2002.
20. Barksdale, G. *Bridge Scour During the 1993 Floods in the Upper Mississippi River Basin*. In *Compendium of Papers ASCE Water Resources Engineering Conferences, 1991 to 1998* (E. V. Richardson and P. F. Lagasse, eds.), Reston Va., 1999, pp. 551–556.
21. Mueller, D. S., and M. N. Landers. *Portable Instrumentation for Real-Time Measurement of Scour at Bridges.* Report FHWA-RD-99-085. FHWA, Turner-Fairbank Highway Research Center, McLean, Va., 1999.
22. Schall, J. D., and P. F. Lagasse. Bridge Scour Monitoring: Report in Advanced Experimental Techniques in Environmental Fluid Mechanics. *ASCE Journal of Engineering Mechanics,* Reston, Va., 2002.
23. Butch, G. K. Evaluation of Selected Instruments for Monitoring Scour of Bridges in New York. *ASCE Hydraulic Engineering,* Reston, Va., 1996.

Safety and Serviceability

SAFETY AND SERVICEABILITY

Assessing the Safety and Serviceability of Existing Highway Structures

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Although considerable effort has been put into the development of new standards and codes for the design of new structures, comparatively little has been done on the development of documents for the assessment of existing structures. The results are discussed in a European action that addresses that imbalance. The action, known as COST 345, "Procedures Required for Assessing Highway Structures," is supported by the European Commission's COST program (COoperation in Science and Technology) and involves experts from 16 countries. The purpose is to identify the procedures and documentation required to inspect and assess the condition of structures such as bridges, culverts, earth-retaining walls, and tunnels. The action also defines the requirements for future research work, provides information on the stock of highway structures (this can be used as input to budgetary plans for maintenance works and operating cost models and also for establishing recommendations for construction options), and identifies those structures not amenable to simple numerical analysis. In particular, the focus is on the work carried out by Working Groups 4 and 5 within the COST 345 action on procedures and numerical methods for assessing the safety and serviceability of highway structures. These working groups cover issues such as overall assessment procedures and levels of assessment, uncertainty modeling (load, material, and structural response modeling), and reliability analysis and target reliability levels.

Bridges, earth-retaining walls, tunnels, culverts, and the like make up a substantial proportion of the fixed assets of the land-based transportation infrastructure. The stock of such structures has been accumulating in developed countries over the years; some in-service structures on the highway network predate the 20th century, and a number of masonry arch bridges date back to Roman times. It is clear that establishment of principles and procedures to be used for the assessment of existing bridges is needed, because some aspects of assessment are substantially different from new design and require knowledge and procedures beyond the scope of design codes.

In the absence of adequate documentation for inspection and assessment, the only available option for assessing structural adequacy is to use design standards for new structures. However, such an approach may be inappropriate and overly conservative for a wide range of structures, because the marginal cost of adding strength to a structure under construction is small and future loading conditions are uncertain. In some cases, it may lead to the unnecessary replacement or strengthening of existing structures with all the attendant costs of traffic delays. To study a procedure for the assessment of highway structures, a new European action, COST 345, was started in 1999. The COST 345 action aims to identify the features of a procedure of assessment within which whole-life performance of the structures is qualitatively or quantitatively examined against the factors of safety specified in current design standards. A less conservative approach is justified for assessment because uncertainty can be reduced through measurements of actual material strength and loading conditions. This paper describes the techniques for assessing the safety and serviceability of existing highway structures based on the work carried out by Working Groups 4 and 5 within the European COST 345 action.

OVERALL ASSESSMENT PROCEDURES—LEVELS OF ASSESSMENT

A preliminary assessment can be based on visual inspections or on measurements of physical and chemical parameters for the purposes of monitoring highway structures. When an existing bridge is found to be in need of repair (e.g., concrete in poor condition) and there is not a detailed assessment available, the purpose of such repairs would be to bring the bridge to its original state as far as practicable. However, if a bridge does not exhibit deterioration and it is necessary to assess if the actual structure will be able to carry new loading conditions, assessment calculations must be carried out to judge if the level of risk is acceptable or not.

To save structures from unnecessary rehabilitation or replacement, the engineer must use all the techniques, all the methods, and all the information available in an efficient way. He or she should carry out the assessment in stages of increasing sophistication. Thus, a simple analysis can be cost-effective if it demonstrates that the bridge is satisfactory, but if it does not prove the bridge is safe, the engineer should introduce more advanced methods of assessment. Similar to the European BRIME (BRIdge Management in Europe) project (*1*), COST 345 classifies assessment in five distinct levels, numbered 1 to 5, with Level 1 being the simplest and Level 5 the most sophisticated, as follows:

- Level 1 Assessment. This level gives a conservative estimate of load capacity. Only simple analysis methods are necessary, and partial safety factors from the assessment standards are used.
- Level 2 Assessment. More refined analysis and better structural idealization than in Level 1 is employed. Characteristic strengths based on existing available data are used for the structural materials.
- Level 3 Assessment. Unlike Level 2, site-specific loading or material properties, or both, are determined from new tests on the structure.
- Level 4 Assessment. Unlike lower levels—based on code implicit levels of safety for which reliability depends on the bulk of structures of the type concerned—Level 4 allows for any additional safety characteristic of the particular structure being assessed.
- Level 5 Assessment. Reliability analysis is applied to a particular structure. For this purpose, probabilistic data for all the variables defined in the loading and resistance equations in addition to specialist knowledge and expertise are required.

LOAD, MATERIAL, AND STRUCTURAL RESPONSE MODELING

An engineer could determine accurately whether or not a structure is able to survive a period of time if all information about a structure is known, including all material properties in the structure, all loads to which the structure is and will be subjected, and how the structure does and will behave when subjected to these loads. Because it is not possible to know each of these parameters exactly, engineers must make conservative approximations and estimations, which allow structures to be designed and assessed. With each approximation and estimation there is associated uncertainty.

In the assessment of existing structures, engineers do not have to work with the same uncertainties that existed during the design phase. The loads to which the structure is subjected can be measured to give a more accurate calculation of the extreme loads, both existing and future. The material properties in a structure can be measured, and the structural response models can be verified with appropriate testing methods. The uncertainties that engineers must deal with can be classified as follows:

- Inherent variability. These are uncertainties associated with material properties (e.g., concrete or steel strength), element geometry (e.g., dimensions of concrete deck slabs), or applied loads (e.g., snow loads, wind speeds, and earthquake ground-motion intensities).
- Imperfect modeling and estimation error. These are uncertainties contained in the mathematical models used to represent real-life phenomena. The uncertainty of these models has both a systematic component (bias) and a random component. The determination of the parameters of the mathematical model is based on engineering judgment and, frequently, on a number of in situ tests in the case of bridge assessment. There will always be some uncertainty due to estimation error that can be diminished by increasing the number of tests.

Probability and experimental data can be used to model uncertainty as shown by the histogram in Figure 1. The greatest benefit that can be obtained from assessing an existing structure is that much of the uncertainty due to imperfect modeling and estimation error that

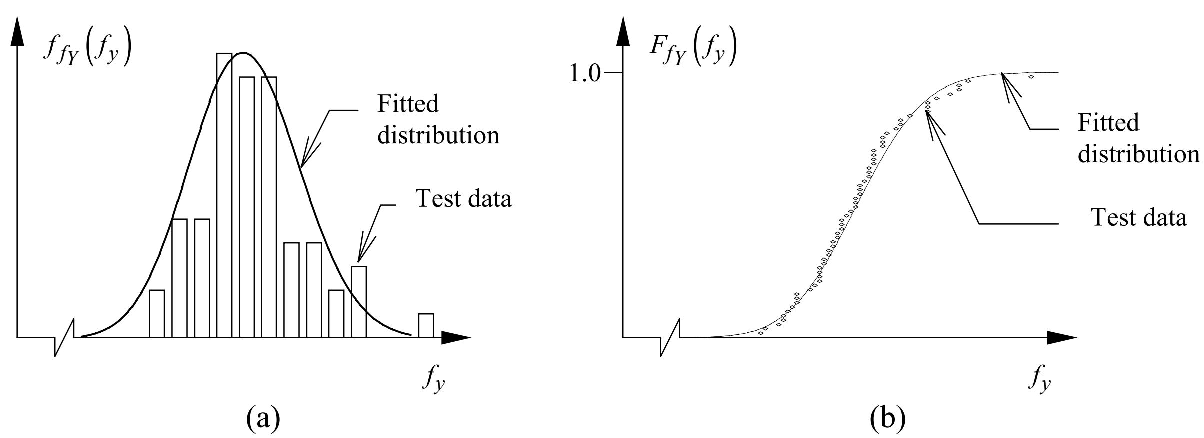


FIGURE 1 Variation of steel yield strength (f_y) represented by (a) probability distribution function and (b) cumulative distribution function.

existed during design can be removed. The reduction of uncertainty when evaluating a bridge can result in an improved reliability and thus may result in the cancellation of a costly and unnecessary intervention.

Load Modeling

An existing structure is subjected to the same type of loads as a new bridge (i.e., dead and superimposed dead load, wind and temperature loading, differential settlement, earth pressure, and traffic loading). However, in the assessment stage, it is possible to have a better description of the loads to which the structure is subjected. For example, the actual thickness of the asphalt layer can be measured, and the superimposed dead load can be calculated more accurately. As a result, a reduction of the partial safety factors associated with these loads is clearly justifiable. There are also new factors to take into account, i.e., the effects of the construction process and subsequent life of the structure, during which it may have undergone alteration, deterioration, or other changes to the as-designed state.

These factors are allowed for through the prescription of partial factors or other code provisions for actual variation in the variables describing actions, material properties, geometric data, and model uncertainty. Thus, if the loads to which an existing structure is subjected are known, the load and resistance models can be updated and the load partial safety factors can be reduced for the various prescribed combinations (which are not envisaged to change from the design code) while maintaining the required safety for the structure.

From an assessment point of view, loads can be time invariant or time variant. Dead load is an example of time-invariant load, for it can be measured accurately, and it can be expected to remain the same for the lifetime of the structure. On the other hand, traffic, wind, and temperature loading are examples of time-variant loads for they are stochastic variables. When assessing a structure, representative records of traffic, wind, and temperature records might be available and the characteristic load effect might be predicted more accurately. Differential settlement, earth pressures, and creep and shrinkage effects are initially time-variant loads, but after some point in time, they behave as asymptotically time-invariant loads.

Additionally, the structural resistance models can be updated through an accumulation of knowledge of the loads that it has successfully carried to date (Figure 2). If a bridge has survived a number of years of service, its resistance is higher than any of the prior imposed loads. Hence, the reliability of bridges improves with proven service.

Load-Monitoring Data Required for Assessment

Time-variant loads such as traffic, wind, and temperature effects require statistical modeling to determine the magnitude of their characteristic effects. Extreme value distributions (e.g., the Gumbel family) are fit to measured data recorded over a period of time, and extrapolation of these yields a value of the given effect for a specified probability exceedance level. Each extreme value distribution has a different tail behavior. The distribution that fits best to the measured characteristic load effect can be obtained by plotting the data on the probability paper corresponding to the chosen distribution. For example, Figure 3 shows the probability paper plots for the Gumbel and Weibull extreme value distributions, respectively, for end support shear in a two-span, two-lane, 80-m-long bridge. It can be seen that the Weibull provides a better fit in the extreme than the Gumbel, and it should be chosen for subsequent extrapolation. The choice of either extreme value distribution is dependent upon the span length but also upon the load effect being considered (*2*).

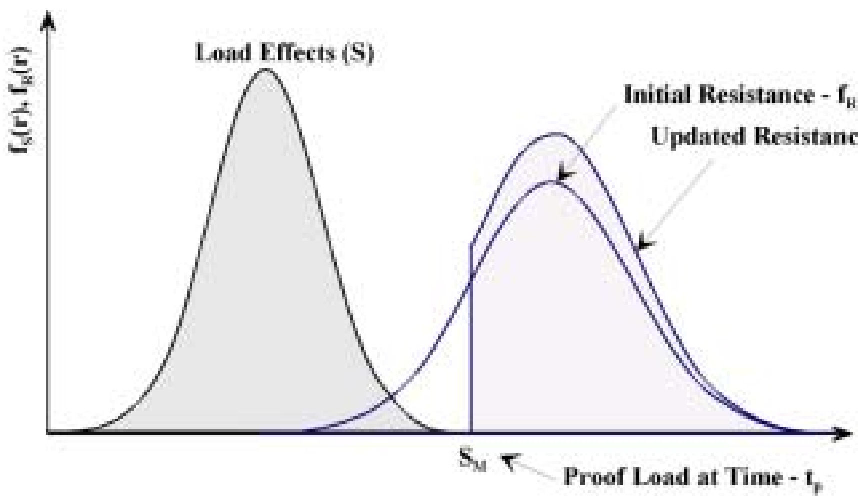
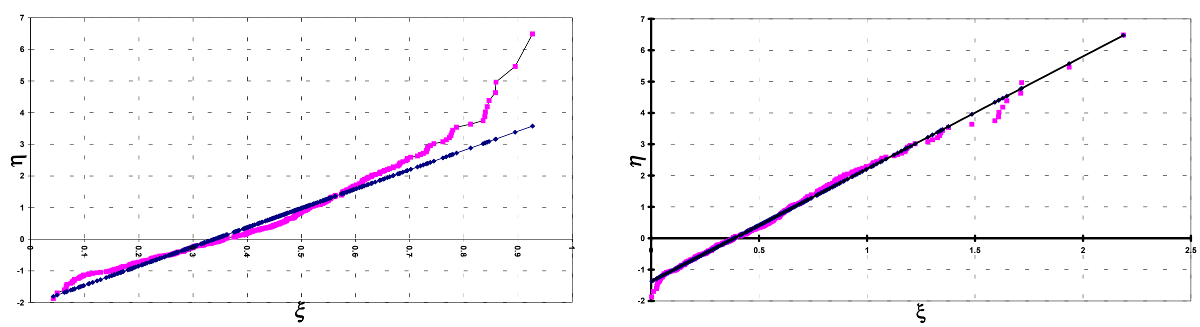


FIGURE 2 Effect of updating load effect [S(t)] and resistance [R(t)] models.



(a) (b)

FIGURE 3 Extreme value approximations for two-span continuous end support shear (total span 80 m): (a) Gumbel I and (b) Weibull.

The duration of the measurements used for extreme value modeling depends upon the effect being determined. For wind and temperature data, maximum and minimum values of the particular effect over a representative period of time (e.g., 50 years) and for a specified sampling frequency (e.g., monthly) should be collected. For traffic data, it is clearly desirable to collect as much data as possible, but 1 or 2 weeks of continuously recorded data can be sufficient for the purposes of assessment (*2*). Traffic data can be obtained using sensors mounted in or on the road pavement or on an existing bridge structure and estimating the corresponding static loads by using appropriate algorithms.

Static Traffic Load Simulation for Assessment

The characteristics of the vehicles that traverse the bridge structure vary widely with respect to their gross weight, axle spacing, distribution of load to axles, location in lane, velocity, and in the likelihood of multiple presence of vehicles on the structure, vary both longitudinally and transversely. Clearly, truck loading is a random phenomenon for which probabilistic models and statistical data are required. For assessment of existing structures, monitoring of traffic data using a weigh-in-motion (WIM) system can provide the necessary statistics to develop site-specific loading models for reliability assessment. Figure 4 illustrates the differences of gross vehicle weight (GVW) distribution depending on the route and, consequently, the importance of collecting WIM data to update loading models. Three types of models can be envisaged:

- Simple load model for assessment. For short-span bridges and culverts, this would be an array of axles corresponding to a meeting event of two trucks. For longer bridges, it would correspond to a traffic jam.
- More complex model for assessment. This might involve a simulation of traffic loading on the structure using traffic with prescribed characteristics (mean and standard deviation of weight and axle spacing).
- Most elaborate model. This could involve a direct simulation of traffic loading on the structure using WIM data measured at or near the site. A number of alternate traffic flow scenarios should be performed for free-flowing, jammed, and mixed traffic on the structure under consideration. A Monte Carlo simulation technique can be used to increase the number of simulated scenarios.

Modeling Materials for Assessment

Material properties play an important role in the determination of the behavior of highway structures. Material properties vary both spatially and temporally. There are spatial variations due to the different combinations of material components within each location (e.g., different combinations of aggregate—cement and water in the case of concrete) and temporal variations due to the loading and physical processes in the materials (e.g., hydration process in concrete that results in increase of concrete strength). There is also a variation between material test specimens and the material in the structure that must be considered. Table 1 gives examples of systematic (bias) and random (cov) uncertainties found in some common material properties.

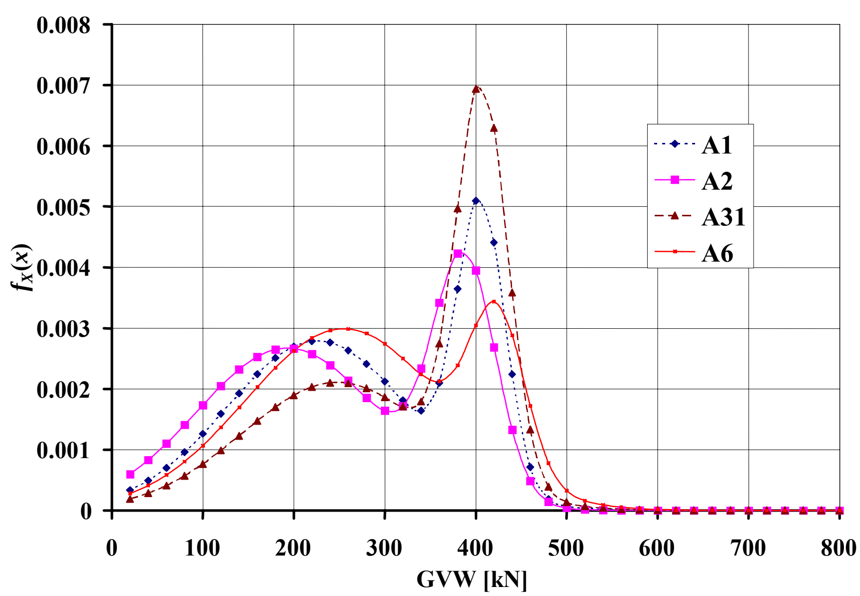


FIGURE 4 Comparison of GVW distributions for a number of alternative sites on the French motorway network.

TABLE 1 Examples of Systematic (Bias) and Random Uncertainties in Material Properties

Variable	Notation	Bias	Cov
Elastic limit of structural steel (welded) (*3*)	f_y	1.25	0.08
Elastic limit of structural steel (rolled) (*3*)	f_y	0.99	0.05
Compressive strength of concrete (20MPa–40MPa) (*4*)	f'_c	1.31–1.19	0.14–0.09
Tensile strength of concrete (20 MPa–40 MPa) (*4*)	f_t	1.47–1.28	0.18–0.16
Modulus of elasticity of concrete (*4*)	E_c	1.18	0.10
Tensile strength of reinforcing steel (400 MPa) (*4*)	f_y	1.22	0.08
Modulus of elasticity of reinforcing steel (*4*)	E_s	1	0

The uncertainties associated with material properties can be taken into consideration by using probabilistic methods. When determining the values of material properties to be used in the assessment of an existing structure, the difference between test values and in situ material properties must be considered, as well as the effects of compliance controls. Mirza et al. (*5*), Bartlett and MacGregor (*6*), the International Organization for Standardization (ISO) (*7*), and the Joint Committee on Structural Safety (JCCS) (*8*) have developed models to determine the concrete strength in compression (in situ). Normal and lognormal distributions are typically used to represent the basic compressive strength of concrete (*9*). Mathematical models of concrete properties can be improved by considering the degree of quality control (*5*, *10*). Madsen and Bazant (*11*) propose models to allow for the drying shrinkage of concrete. Normal or beta distributions can be used to represent yield strength of steel reinforcing properties (*5*, *8*).

Structural Response Modeling

The assessment of a highway structure requires the calculation of the response of a mathematical model of the structure to a complete range of loading conditions. This model should satisfy conditions of equilibrium and produce deformations compatible with the continuity of the structure and support conditions. Separate or interdependent mathematical models of the structure and the soil can be established to determine the structural response. Hence, a particular model for a given structure will be influenced by the assumptions adopted for the foundation and the soil. If it is ensured that the ground can sustain the loading with acceptable displacements or provide adequate stiffness, soil–structure interaction can be ignored in low-level studies. (In bridges, piled foundations have often been employed to provide relatively rigid foundations and allow an analysis of the structure in isolation.) The method of analysis to be used will depend on the following characteristics:

- Behavior of the structural material,
- Structural geometry and boundary conditions, and
- Nature of the applied load.

Methods of analysis are established for different types of structure and for the five different levels of assessment proposed in the report. An assessment at Level 1 is carried out with traditional methods of structural analysis while assessment at higher levels will involve more refined methods of analysis. Traditional methods of structural analysis are based on one- or two-dimensional models with elastic materials, geometric linearity, and static loads. Other available techniques allow for three-dimensional modeling, a variety of nonlinear response actions, and dynamics. In higher levels of assessment, the method of analysis should ideally take account of all the significant aspects of the structural response to loads and imposed displacements.

The levels reflect level of sophistication of the analysis or time available to the assessor. Level 1 assessment corresponds to more simple and conservative methods, while higher levels will be used for more rigorous modeling. The number of parameters required increases with the level of assessment. Therefore, parameters for lower levels of assessment can be based on visual observation, but parameters for higher levels of assessment should be estimated from load testing. The same methods of structural analysis are used for Level 2 and higher levels, but specific material properties and loading can be included in higher levels. Hence, partial factors from assessment standards (smaller than at the design stage) can be used for Level 1, but characteristic strengths for materials must be based on existing data (from the same or a similar structure) for Level 2 and on load tests on the structure being assessed for Level 3 or higher. Level 4 uses modified partial safety factors to account for any additional safety characteristics specific to the structure being assessed, and Level 5 uses structural reliability analysis instead of partial safety factors. Theoretically, the output of higher levels of assessment could be used as a diagnostic tool to prevent weaknesses at localized points and for information on safety values.

All categories are summarized in Table 2. A stability analysis is also to be considered in Level 1. An assessment associated with complex mathematical modeling should be used with considerable caution. The analysis of a special load (e.g., the dynamic response of a bridge to the crossing of a truck) might require some numerical manipulation (i.e., convolution or Lagrange technique) of these structural models.

Integration of Field Data and Structural Models

To represent the structural response correctly, accurate field measurements must be taken. The quality of the output will depend on the quality of the input. Accordingly, complex analytical tools can only be justified if a realistic assessment of the material properties and overall condition of the existing structure can be made. Then, structural models can be improved by measuring dynamic effects (impact factor, damping, etc.) or by measuring other results of load testing, including observed cracks or the true distribution of loads.

The original structural design might have been altered not only due to aging and the application of loads but also to grouting, saddling, guniting, or posttensioning in previous maintenance programs. It is therefore necessary to carry out a visual inspection of the structure. This inspection might reveal scouring of piers or abutment supports, or both; cracks in a section of the structure; quality and condition of the structural material; deformations of the profile; condition of the joints; and damping devices, etc. Calculations can vary as a result of observation. Additionally, a number of reduction factors relating to the condition of the bridge can be adopted based on observation. The structure dimensions should be measured with appropriate surveying equipment on site, and in the case of observed deformations, the new profile may need to be considered in the analysis.

The assessment of a structure might require more data than purely the observation of the visible portion of the structure. Concrete tests include cover depth, rebound hammer, ultrasonics, impact echo, permeability, carbonation, thermography, radar, slot cutting and instrumented

TABLE 2 Analysis Methods Recommended for Each Level of Assessment

<table>
<tr><th colspan="2" rowspan="2">Structure Type</th><th colspan="5">Level of Assessment</th></tr>
<tr><th>1</th><th>2</th><th>3</th><th>4</th><th>5</th></tr>
<tr><td rowspan="7">Bridges</td><td>Not skew Beam</td><td rowspan="4">1-D linear elastic (beam theory or plane frame analysis)</td><td>1-, 2- or 3-D linear or non-linear; elastic or plastic; allowing for cracking</td><td rowspan="7">2- or 3-D; linear or non-linear; elastic or plastic; grillage or FEM (upstand model if necessary); allowing for soil-structure interaction, cracking, surface irregularities and ‘specific’ live loading & material properties</td><td rowspan="12">FEM analysis of specific details of the structure being assessed not considered in previous levels</td><td rowspan="12">Reliability analysis based on probabilistic models</td></tr>
<tr><td>Not skew Slab</td><td rowspan="4">2- or 3-D linear or non-linear; elastic or plastic; allowing for cracking; grillage or FEM (upstand model if necessary);</td></tr>
<tr><td>Not skew Beam & Slab</td></tr>
<tr><td>Not skew Cellular</td></tr>
<tr><td>Skew, tapered and curved</td><td>Frame linear elastic allowing for torsion</td></tr>
<tr><td>Arch</td><td>Empirical or 1-D linear elastic arch frame</td><td>1-, 2- or 3-D linear or non-linear; elastic or plastic; allowing for cracking</td></tr>
<tr><td>Cable Stayed</td><td>1-D linear elastic with modified modulus of elasticity</td><td>1-, 2- or 3-D linear or non-linear; elastic or plastic; allowing for cracking and modeling cable sag more accurately</td></tr>
<tr><td rowspan="2">Culverts</td><td>Rigid</td><td>Frame linear elastic</td><td rowspan="2">2- or 3-D FEM linear or non-linear; elastic or plastic; allowing for soil-structure interaction, cracking</td><td rowspan="2">2- or 3-D FEM, linear or non-linear; elastic or plastic; allowing for soil-structure interaction, cracking, surface irregularities and ‘specific’ live loading & material properties</td></tr>
<tr><td>Flexible</td><td>Frame linear elastic allowing for soil-structure interaction (beam & spring)</td></tr>
<tr><td colspan="2">Earth-retaining walls</td><td>Simple method of analysis</td><td>Beam, 2- or 3-D non-linear FEM on elastic foundation or elasto-plastic continuum</td><td>3-D non-linear FEM, allowing for soil constitutive models and ‘specific’ live loading & material properties</td></tr>
<tr><td colspan="2">Reinforced soil</td><td>Empirical models or 1-D linear elastic</td><td>2- or 3-D FEM of soil</td><td>2- or 3-D FEM of soil in combination with existing structure and ‘specific’ live loading & material properties</td></tr>
<tr><td colspan="2">Tunnels</td><td>Empirical models or beam-and-spring models (non-cohesive soil)</td><td>2- or 3-D FEM; linear or non-linear; elasto-plastic</td><td>3-D non-linear FEM with bedding, fracture planes, … and ‘specific’ live loading & material properties</td></tr>
</table>

Note: FEM = finite element modeling.

coring, and others. Testing of reinforcement corrosion includes half-cell potentials, resistivity and rate of corrosion, chloride concentration and monitoring. Posttensioning tendons can be tested with exploratory hole drilling, radiography, and ultrasonics, or through monitoring. Other tests are related to the determination of in situ stress (*12*).

Load testing must be carried out with caution to protect the structure from further deterioration. The passage of heavily loaded trucks can be used to determine the actual live-load behavior of the structure and to predict maximum traffic-load stresses. Forced vibration or ambient vibration methods are typical dynamic tests to determine the frequencies and mode shapes of vibration of a bridge. As tests on a full scale are expensive and limited, scaled physical models using measurements from tests on the real structure can also be used for assessment purposes. Garas et al. (*13*) verify by testing some of the methods of analysis at realistic scales that cannot be achieved in the laboratory. Measurements can reveal more realistic values for support stiffness, joint condition, restraints, behavior of the cross-section, elastic properties of the structural material, behavior of the foundation, fill and structural material density, road profile, etc. Then, these characteristics can be incorporated into the structural model. Optimization techniques are commonly used for adjusting parameters of the structural models to field measurements. The updated models can be used to more accurately predict and assess the behavior of the structure under different static or dynamic loading conditions. In a structural reliability model, the uncertainties in the design parameters are modeled probabilistically. The process of identifying the behavior of a given structure is described by the ASCE Committee on Structural Identification of Constructed Facilities (*14*).

RELIABILITY ANALYSIS AND TARGET RELIABILITY LEVELS

There are four main formats of reliability analysis: global safety factor format, partial safety factor format, reliability format, and socioeconomic formats.

Global Safety Factor Format

The global safety factor can be selected on the basis of experiments, practical experience, and economic and political considerations. This factor is often associated with elastic stress analysis as described by Equation 1:

$$S \leq R_a = \frac{R_f}{\gamma_g} \qquad (1)$$

where S is the applied stress, R_a is the allowable stress, R_f is the failure stress, and γ_g is the global safety factor. The safety principle consists in verifying that the maximum stresses calculated in any section of any part of the structure under worst-case loading remain lower than the allowable stress. The maximum stresses are calculated by linear elastic analysis, and the allowable stress is based on the mechanical properties of the material used. However, the actual material failure might not be well represented due to discrepancies between actual stress at the critical cross-section in the structure and the result of calculations by this method.

Partial Safety Factor Format

Partial safety factors (load and resistance factor design) are designed to cover a large number of uncertainties and may therefore not be very representative for evaluating the reliability of a particular structure. The semi-probabilistic partial safety factor does not require actual probability calculations as does the reliability format approach. Equation 2 ensures the reliability of the structure by imposing certain requirements for the limit state, the characteristic values [R_k and S_k are characteristic values of the resistance (*R)* and stress (*S),* respectively], and the partial safety factors (γ_R and γ_S).

$$S_k \times \gamma_s \leq \frac{R_k}{\gamma_R} \tag{2}$$

Partial safety factors should be calibrated using probabilistic methods and idealized reliability formats. Although, in most of the countries where this semi-probabilistic approach is applied, the actual values are still influenced by experience as well as economic and political considerations.

Reliability Format

Stress (*S)* and resistance (*R)* are described as stochastic variables. The structure is considered safe if it is possible to verify Equation 3:

$$S \leq R \tag{3}$$

The difference of $R - S$ is known as the safety margin *M*. Figure 5 shows how *M* is also a variable, and it is normally distributed if the variables *R* and *S* are normally distributed. β is the reliability index, given by Equation 4:

$$\beta = \frac{\mu_M}{\sigma_M} \tag{4}$$

where μ_M and σ_M are the mean and standard deviation of *M*.

Reliability formats using probabilistic methods are an important alternative to semi-probabilistic approaches such as the partial safety factor format. Reliability formats are based on

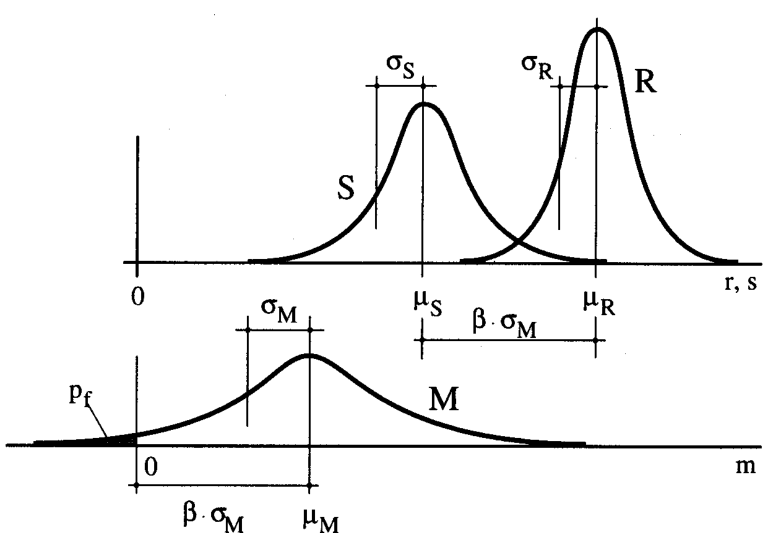


FIGURE 5 Distributions of resistance (*R*), stress (*S*), and safety margin (*M* = *R* – *S*).

the definition of a limit-state criterion, identification of all variables influencing the limit-state criterion, the statistical description of these variables, the derivation of the probability density and its moments for each basic variable, the calculation of the probability that the limit-state criterion is not satisfied, and the comparison of the calculated probability to a target probability (as defined in the following subsection). The evaluation of the probability of failure is a difficult task, except for linear limit states and Gaussian variables. The reliability index methods (e.g., First Order Reliability Method and Second Order Reliability Method) and the simulation methods (e.g., Monte Carlo sampling and Importance or Directional sampling) are two techniques that allow the calculation of probability of failure for complicated functions.

Finally, socioeconomic formats are reliability formats in which failure costs are introduced to determine the required probabilities of failure or reliability indices.

Target Reliability Levels

The target reliability level is the level of reliability required to ensure acceptable safety and serviceability of a structure. There are some differences between the requirements for assessment and for the design of new structures. For example, from an economic point of view, more conservative criteria are used in design standards, because the cost increment of increasing the safety of a structural design is generally very small compared to the large incremental cost between acceptance and upgrading an existing structure. There are also social considerations (such as disruption of traffic as well as heritage values) that do not affect the design of new structures. Finally, there are sustainability considerations, i.e., in the rehabilitation of existing structures. The authorities or bridge owner must specify target reliability levels. These target reliability levels can be explicitly or implicitly specified in a code.

Table 3 compares target reliability levels of various codes and standards currently in use. The engineer dealing with the assessment of an existing structure must decide among the available tables which of the values are most suited and best applied to the solution of the problem at hand, because the estimated probability of failure associated with a project is very much a function of the understanding of the issues, modeling the data, etc. Furthermore, it depends on costs as well as consequences of failure. In the ISO/CD 13822:1999 (*15*), the target reliability levels depend on the type of limit state examined as well as on the consequences of failure, and they range from 2.3 for very low consequences of a structural failure to 4.3 for structures whose failure would lead to very high consequences. In the ultimate limit state, a value of 4.3 would be suitable for most cases. The value β recommended by ISO 2394:1998 (*7*) and the JCCS (*8*) depends on the consequences of a structural failure as well as the costs of a safety measure. In Eurocode 1 (*16*), β depends only on the type of limit state examined, while in the Nordic Committee on Building Regulations (NKB) report (*17*), the failure type and consequence is taken into account in the determination of β. The Canadian Standards Association (CSA) obtains β through an equation allowing for element and system behavior, inspectability, and traffic category (Table 3).

CONCLUSIONS

This paper has described the philosophy behind the methodologies provided by Task Groups 4 and 5 within COST 345 to assess highway structures. The report recommends five levels of assessment varying from simple but conservative to complex but accurate. Then, the following aspects have been reviewed: the unused capacity in highway structures that are not subjected to the full design levels of traffic loading, the processes by which material properties in existing structures can be estimated, and the types of analysis appropriate to each level of assessment. The levels of

TABLE 3 Comparison of Target Reliability Levels

Code	Target Reliability Index β		
ISO/CD 13822:1999 (*15*)	Relatively high cost of safety measure	Small consequences of failure	$\beta = 0.0$
		Some consequences of failure	$\beta = 1.5$
		Moderate consequences of failure	$\beta = 2.3$
		Great consequences of failure	$\beta = 3.1$
	Relatively moderate cost of safety measure	Small consequences of failure	$\beta = 1.3$
		Some consequences of failure	$\beta = 2.3$
		Moderate consequences of failure	$\beta = 3.1$
		Great consequences of failure	$\beta = 3.8$[1]
	Relatively low cost of safety measure	Small consequences of failure	$\beta = 2.3$
		Some consequences of failure	$\beta = 3.1$
		Moderate consequences of failure	$\beta = 3.8$
		Great consequences of failure	$\beta = 4.3$
JCSS Ultimate limit state (*8*)	Relatively large cost of safety measure	Minor consequences of failure	$\beta = 3.1$
		Moderate consequences of failure	$\beta = 3.3$
		Large consequences of failure	$\beta = 3.7$
	Relatively normal cost of safety measure	Minor consequences of failure	$\beta = 3.7$
		Moderate consequences of failure	$\beta = 4.2$
		Large consequences of failure	$\beta = 4.4$
	Relatively small cost of safety measure	Minor consequences of failure	$\beta = 4.2$
		Moderate consequences of failure	$\beta = 4.4$
		Large consequences of failure	$\beta = 4.7$
Eurocode 1:1993 (*16*)	Serviceability	Design working life: bridges 100 years	$\beta = 1.5$
		1 year	$\beta = 3.0$
	Fatigue	Design working life: bridges 100 years	$\beta = 1.5$ to 3.8
	Ultimate	Design working life: bridges 100 years	$\beta = 3.8$
		1 year	$\beta = 4.7$
NKB Report No. 36: 1978 Ultimate limit state (*17*)	Ductile w/ extra carrying capacity failure	Less serious failure consequences	$\beta = 3.1$
		Serious failure consequences	$\beta = 3.7$
		Very serious failure consequences	$\beta = 4.2$
	Ductile w/o extra carrying capacity failure	Less serious failure consequences	$\beta = 3.7$
		Serious failure consequences	$\beta = 4.2$
		Very serious failure consequences	$\beta = 4.7$
	Brittle failure	Less serious failure consequences	$\beta = 4.2$
		Serious failure consequences	$\beta = 4.7$
		Very serious failure consequences	$\beta = 5.2$
CSA Ultimate limit state $\beta = 3.5 - (\Delta_E + \Delta_S + \Delta_I + \Delta_{PC}) \geq 2.0$	Adjustment for element behavior, Δ_E	Minor consequences of failure	$\Delta_E = 0.0$
		Moderate consequences of failure	$\Delta_E = 1.5$
		Large consequences of failure	$\Delta_E = 2.3$
	Adjustment of system behavior, Δ_S	Minor consequences of failure	$\Delta_S = 0.0$
		Moderate consequences of failure	$\Delta_S = 1.5$
		Large consequences of failure	$\Delta_S = 2.3$
	Adjustment for inspection level, Δ_I	Minor consequences of failure	$\Delta_I = 0.0$
		Moderate consequences of failure	$\Delta_I = 0.0$
		Large consequences of failure	$\Delta_I = 0.0$
	Adjustment for traffic category, Δ_{PC}	All traffic categories except permit controlled	$\Delta_{PC} = 0.0$
		Traffic category permit controlled	$\Delta_{PC} = 0.0$

[1] For ultimate limit state, use $\beta = 3.1$, 3.8, and 4.3.

reliability considered appropriate for highway structures and available procedures for full reliability analysis have also been discussed. Practical examples are also provided in the final version of the report, which is expected to contribute to the continued safety and serviceability of the land transport fixed assets in Europe and elsewhere.

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REFERENCES

1. BRIME—Bridge Management in Europe. *Experimental Assessment Methods and Use of Reliability Techniques*. BRIME Deliverables D6. Final Report of the European Commission 4th Framework Program, 1999.
2. O’Connor, A. J. *Probabilistic Traffic Load Modeling for Highway Bridges*. Ph.D. thesis. Trinity College Dublin, 2001.
3. Ellingwood, B., T. Galambos, J. MacGregor, and A. Cornell. *Development of a Probability-Based Load Criterion for American National Standards A58: Building Code Requirements for Minimum Design Loads in Buildings and Other Structures*. NBS Special Publication 577, National Bureau of Standards, June 1980.
4. CEB-FIP. Reliability of Concrete Structures. *CEB Boletin 202*, 1991.
5. Mirza, S. A., M. Hatzinikolas, and J. G. MacGregor. Statistical Descriptions of Strength of Concrete. *Journal of Structural Division*, Vol. 105, No. ST6, June 1979, pp. 1021–1037.
6. Bartlett, F. M., and J. G. MacGregor. Statistical Analysis of the Compressive Strength of Concrete in Structures. *ACI Materials Journal*, Vol. 93, No. 2, 1996, pp. 158–168.
7. ISO 2394: *General Principles on Reliability for Structures*, 2nd ed. ISO, June 1998.
8. JCSS. *Probabilistic Model Code*, 2001. http://www.jcss.ethz.ch. Accessed Dec. 20, 2001.
9. Balaguru, P. Comparison of Normal and Lognormal Frequency Distributions for Representing Variations of Concrete Compressive Strength. *ACI Materials Journal*, Vol. 92, No. 5, March–April 1995.
10. Stewart, M. G. Workmanship and Its Influence on Probabilistic Models of Concrete Compressive Strength. *ACI Materials Journal*, Vol. 92, No. 4, July–August 1995.
11. Madsen, H. O., and Z. P. Bazant. Uncertainty Analysis of Creep and Shrinkage Effects in Concrete Structures. *ACI Materials Journal*, Vol. 80, No. 2, 1983, pp. 116–127.
12. Mallett, G. P. *State-of-the-Art Review: Repair of Concrete Bridges*. Thomas Telford, 1984.
13. Garas, F. K., J. L. Clarke, and G. S. T. Armer. *Structural Assessment Based on Full and Large Scale Testing*. Butterworths, London, 1987.
14. ASCE Committee on Structural Identification of Constructed Facilities*. The State of the Art in Structural Identification of Constructed Facilities*. Draft Report 28. (S. W. Doebling and C. R. Farrar, Los Alamos National Laboratory, eds.), June 1999.
15. ISO 13822: *Bases for Design of Structures—Assessment of Existing Structures*. Committee Draft. ISO, Dec. 1999.
16. Eurocode 1: *Basis of Design and Actions on Structures*, 1993.
17. Nordic Committee on Building Regulations. *Guidelines for Loading and Safety Regulations for Structural Design.* NKB Report No. 36, Nov. 1978.

SAFETY AND SERVICEABILITY

Forecasting Lifetime Safety and Serviceability for Management of a Network of Highway Bridges in Colorado

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A probabilistic framework forecasting the lifetime safety and serviceability of a network of highway bridges located along U.S. Highway 36 in Colorado is presented. In addition to performing a live load rating analysis for each bridge, performance functions defining the failure conditions for superstructure components are derived, in terms of both the ultimate strength and the serviceability limit states, using load and capacity requirements defined by AASHTO specifications. The performance functions are established by grouping the random variables and constant parameters in such a way that the formulas are applicable to any type of bridge having similar superstructure components. Thus, the developed performance functions are applicable to other similar bridges, which creates the possibility of evaluating a larger number of bridges in a wider network area, such as a transportation region within the state of Colorado. Using the developed performance functions, initial reliability indices have been determined for the components of the selected highway bridges. By defining the potential failure of the superstructure through a system model, a system reliability index is calculated for each bridge. Resulting component and system reliability indices are compared with the corresponding load ratings of the bridges. A time-varying truckload model and a material degradation model are used to determine the reliability indices of each bridge component and system over the bridge's lifetime. Both safety and serviceability of each bridge are observed over its lifetime based on the predicted load increase and strength reduction. The resulting reliability profiles form the basis for a network-level management planning study for a group of similar bridges that is based on structural reliability.

Bridges stand as visible signs of an aging transportation system. On the basis of FHWA's inspection data for 2001 (the most recent data available), 28% of the nation's bridges are rated as structurally deficient or functionally obsolete (*1*). The condition of the nation's bridges has improved slightly since 1995 with the percentage of all bridges rated as deficient declining from 32% in 1995 to 28% in 2001. According to The Road Information Program (*1*), the percentage of deficient bridges in Colorado as of 2001 is 18%. Among the deficient bridges, 7% are classified as structurally deficient and 11% as functionally obsolete. With 46 states having higher percentages of structurally deficient bridges, Colorado's bridges are relatively in good condition, keeping the state out of the list of the top 10 states with the highest percentage of bridges rated as structurally deficient in 2001. In light of these facts, it is now generally recognized that if the nation's deteriorating bridges are to achieve their intended service life with an adequate level of safety and serviceability, special-purpose network-level bridge management systems (BMSs) must be used at evaluation and life-cycle cost planning stages. Today, there is a need for bridge-managing agencies to assess and recognize the advancements in structural reliability in bridge engineering and its impact on the service life assessment of bridge networks.

The current state of practice for network-level bridge management in the United States is primarily based on network cost optimization using the Markov decision process with a linear programming solution procedure. Employing these techniques as a software package, Pontis (*2*) serves as the predominant BMS used by many state departments of transportation. Pontis uses condition-based inspection data to generate network-level policies for maintenance, repair, and rehabilitation of a group of bridges. However, a network-level BMS based on structural reliability currently does not exist. An approach presented by Tao et al. (*3*) and Ellis et al. (*4*) to bridge design and life-cycle management combines the Markov decision process with structural reliability theory. Frangopol (5), Frangopol and Ghosn (6), and Frangopol (7) recently proposed a bridge management model based on reliability, without using the Markov process.

The work presented in this paper aims to contribute to reliability-based network-level bridge management by forecasting the lifetime safety and serviceability of a network of highway bridges located along U.S. Highway 36 (US-36) in Colorado. This paper serves as an introduction of an extensive study, detailed results of which will be reported elsewhere. In addition to performing a live load rating analysis for each bridge, performance functions defining the failure conditions for superstructure components are derived, in terms of both the ultimate strength and the serviceability limit states, using load and capacity requirements defined by current AASHTO specifications and guides (*8, 9, 10*). To be able to apply the theoretical models and developed tools to real-life problems, derivations of performance functions are made by strictly adhering to the requirements of the current practice and codes of state and federal transportation agencies. The performance functions are formulated by grouping the random variables and constant parameters in such a way that the formulas are applicable to any type of bridge having similar superstructure components. Thus, the developed performance functions that are applicable to other similar bridges create the possibility of evaluating a larger number of bridges in a wider network area, such as a transportation region within a state.

Using the developed performance functions, initial reliability indices have been determined for the components of the selected highway bridges. By defining the potential failure of the superstructure through a system failure model, a system reliability index is calculated for each bridge. Resulting component and system reliability indices are compared with the corresponding load ratings of the bridges. A time-varying truckload model and a material degradation model are utilized to determine the reliability indices of each bridge component and system over the bridge's lifetime. The safety of each bridge is observed over its lifetime based on the predicted load increase and strength reduction. The resulting reliability profiles form the basis for a reliability-based management system for a bridge network. In such a system, optimal decisions in terms of life-cycle cost are made at the bridge network level by explicitly taking into account the propagation of uncertainties during the entire service life of individual bridges in the network. Such systems will have significant impact on the advancement of intelligent bridge infrastructure management.

HIGHWAY BRIDGE NETWORK

The bridge network, located within Transportation Region 6 of Colorado, is positioned at the northwest corner of the Denver metropolitan area and is within the limits of three different counties—Boulder, Adams, and Jefferson. The entire network consists of 14 mix-type highway bridges. The selection criteria for the bridge network were to choose bridges located within close proximity of each other, preferably along Interstate highways, and within the same transportation and maintenance regions. The latter requirement was established to facilitate a less-complicated data acquisition process. Here, the analysis results for the subnetwork, consisting of six highway bridges located along US-36, as shown in Figure 1, will be discussed. Bridge designation used by the Colorado Department of Transportation (CDOT) consists of a letter followed by a one to two digit number describing the vertical and horizontal coordinates of the bridge, respectively, on a statewide grid. The last two letters are the unique identification symbol for the bridge (*11*).

Bridges in the subnetwork consist of four prestressed concrete girder bridges and two steel-rolled I-beam bridges. All of the bridges have reinforced concrete decks (slabs). Physical characteristics of the bridges are listed in Table 1, in which the number of spans, length and width, the year in which it was built, and the average daily truck traffic (ADTT) are noted for each bridge. The lengths of the prestressed bridges range between 212 ft to 255.5 ft, while the

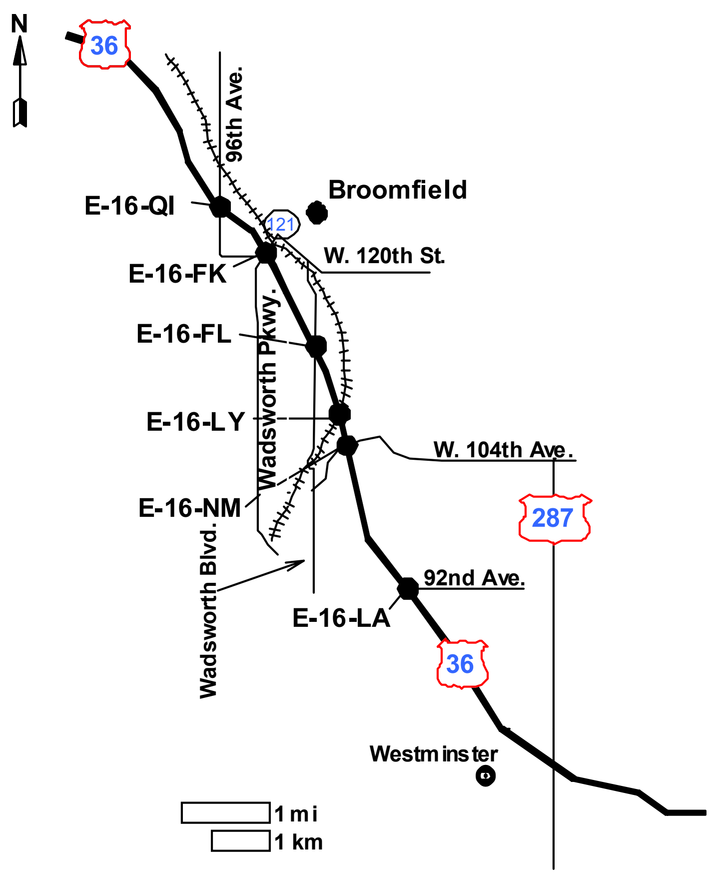

FIGURE 1 Subnetwork of highway bridges on US-36.

TABLE 1 Physical Characteristics of the Two Subnetworks (Prestressed Concrete and Steel I-Beam) of Colorado Highway Bridges Located on US-36

Bridge Name	Bridge Type	CDOT Designation	Number of Spans	Length		Width		Year Built	ADTT (trucks /day)
				ft	(m)	ft	(m)		
E-16-QI	Prestressed Concrete	CPGC	2	243.2	74.1	100.7	30.7	1995	1335
E-16-FK	Steel I-beam	CIC	4	227.0	69.2	34.0	10.4	1951	1370
E-16-FL	Steel I-beam	CIC	4	177.0	54.0	34.0	10.4	1951	765
E-16-LY	Prestressed Concrete	CBGCP	3	243.7	74.3	112.0	34.1	1985	1610
E-16-NM	Prestressed Concrete	CPGC	2	212.0	64.6	92.0	28.0	1991	2955
E-16-LA	Prestressed Concrete	CPG	2	255.5	77.9	128.5	39.2	1983	450

lengths of two steel bridges are 177 ft and 227 ft. The oldest and newest prestressed bridges in the subnetwork were built in 1983 and 1995, respectively, representing a 12-year span between their constructions. Two steel bridges were built in 1951. Currently, the average age of the group of prestressed bridges in the subnetwork is 13.5 years, with oldest and newest bridges being 19 and 7 years old, respectively, while the two steel bridges are 51 years old. Considering the lifetime of a highway bridge as 75 years, the prestressed and steel bridge groups can be viewed as having lived 18% and 68% of their lives, respectively.

Steel bridges in the subnetwork have rolled I-beams carrying the reinforced concrete deck. Prestressed girder bridges, however, have varying cross-sectional geometries. For instance, bridges E-16-QI and E-16-NM are made up of G68 prestressed I-girders having 68-in.-deep sections, while girders of bridge E-16-LA are G72 sections. Bridge E-16-LY, on the other hand, has a rectangular box girder cross section. As a representative sample of the prestressed girder bridges in the subnetwork, side and roadway photographs, together with elevation, plan and cross-sectional views of Bridge E-16-QI, are shown in Figures 2 through 6.

LIVE LOAD CAPACITY ANALYSIS

As the first phase of this study, live load capacities (load ratings) of the individual superstructure components of the bridges in the subnetwork, such as reinforced concrete decks and girders, were determined using AASHTO specifications (*8*), AASHTO's *Manual for Condition Evaluation of Bridges* (*10*), and CDOT's bridge rating manual (*12*). Calculation of live load capacities for the bridges was a necessary and crucial step to identify the code-based load and resistance formulas for each bridge type and, later, to incorporate them into the limit state functions for each bridge component corresponding to each failure mode. AASHTO loads are generally used for design purposes. For rating analysis, AASHTO's manual (*10*), Section 6.7, lists the type of loads to be used in determining the load effects in basic rating equations. The manual specifically instructs that the loads such as wind, earthquake, thermal, and ice pressure should not be considered in calculating load ratings (*10*). Therefore, the network bridges are analyzed under dead loads and truck live loads.

FIGURE 2 Side view of Colorado Highway Bridge E-16-QI (*13*).

FIGURE 3 Roadway of Colorado Highway Bridge E-16-QI (*13*).

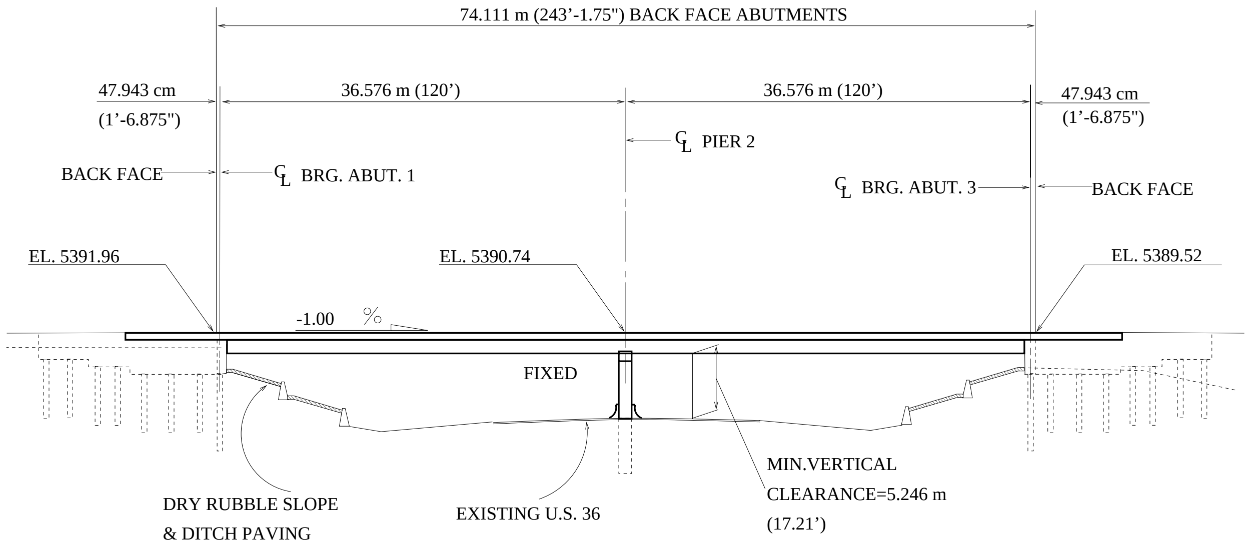

FIGURE 4 Elevation of Colorado Highway Bridge E-16-QI *(13)*.

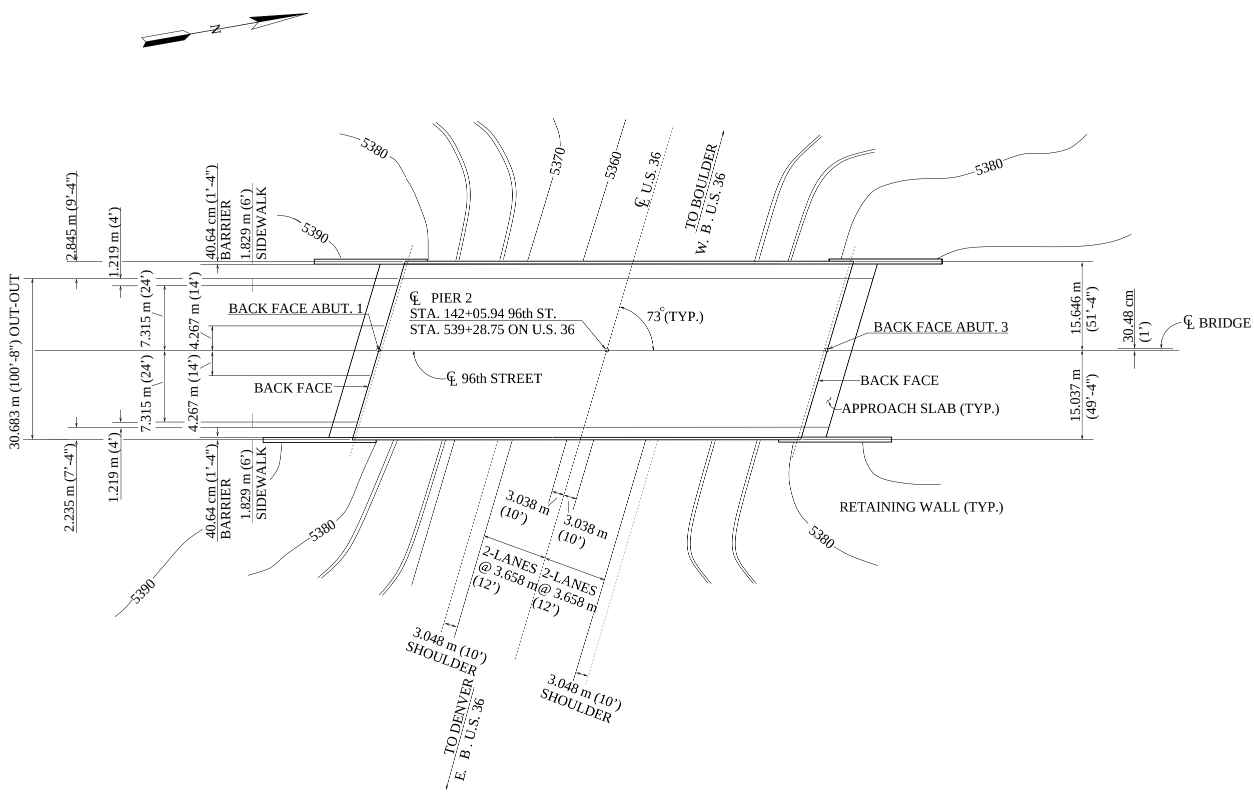

FIGURE 5 Plan view of Colorado Highway Bridge E-16-QI *(13)*.

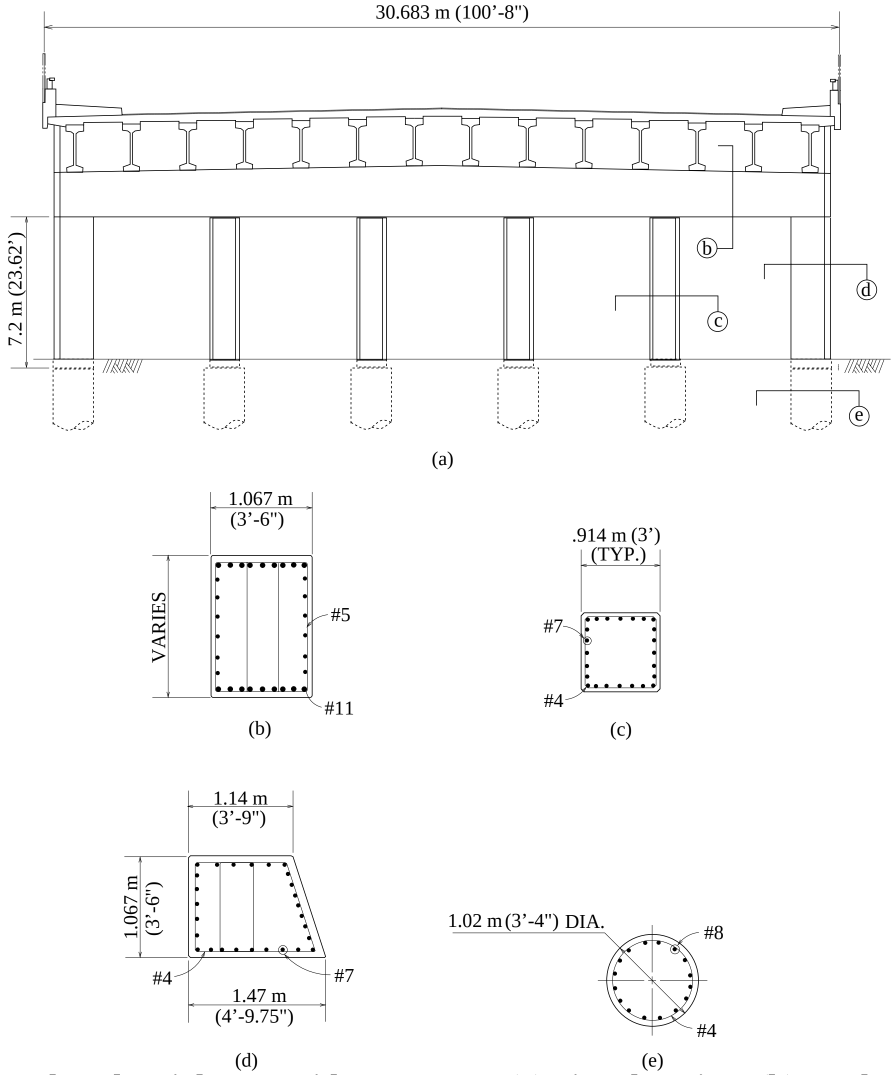

FIGURE 6 Colorado Highway Bridge E-16-QI: (a) pier elevation; (b) cap beam section; (c) and (d) column sections; and (e) caisson section *(13)*.

LIFETIME RELIABILITY ANALYSIS

The second phase of the study included the following steps: identification of random variables for different bridge member types; quantification of random variable data, i.e., determination of descriptors; derivation of code-based limit state equations; investigation and implementation of load and resistance models; and development and utilization of a network-level system reliability analysis program.

Many variables, such as material properties, resistances, applied loads, environmental conditions, and geometry, influence the expected reliability of a bridge component. Random variables used in the developed limit state functions are categorized as resistance, live load, dead load, geometry, and modeling random variables. Nominal values of these variables for each bridge are obtained from bridge plans and documentation. Uncertainty is included by using appropriate dispersion values based on the literature.

Component and System Reliability

Separate system failure models are devised for prestressed and steel bridges in the subnetwork. Furthermore, for prestressed bridges, failure models also differ depending on span continuity. For simple-span prestressed bridges, such as E-16-LA, flexural failure at midspan is treated as the predominant failure mode, while for continuous prestressed girders, flexural failure over the pier support due to negative bending moment is included in the failure model in addition to the flexural failure at maximum moment location. As an example, a system failure model for the prestressed Bridge E-16-QI is shown in Figure 7, in which a moment failure at midspan (m) or at pier (p) is enough for failure of a girder (series), and failure of two adjacent girders at the same time (parallel) or the deck (series) causes system failure.

For steel-rolled I-beam bridges, the system failure model included flexure at the most critical moment location and shear at the critical support shear. Critical locations for moment and shear refer to the positions where maximum load effect is produced by either an AASHTO truck or lane loading. Influence lines for truck wheel loadings are generated to determine the

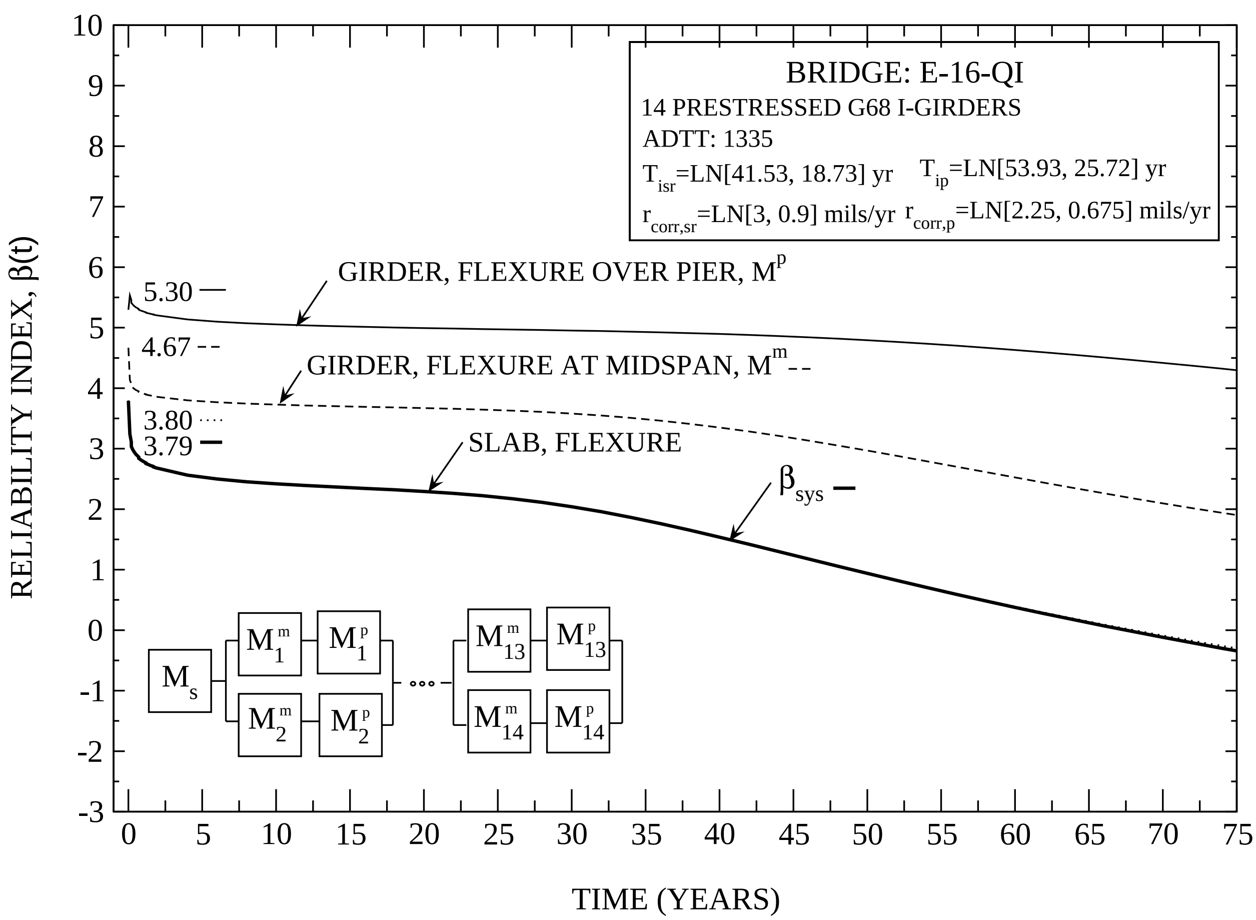

FIGURE 7 Component and system reliability profiles for Bridge E-16-QI. System failure model including the slab.

Reliability Performance Functions

Derived performance functions (limit states equations) are complex in nature due to avoidance of any simplification in AASHTO's resistance and load formulas. Code formulas are directly inserted into the limit state equations, which results in standardized, code-based, generic limit states equations for each bridge component type corresponding to each failure mode. Once randomness is introduced into the limit state equations, verification of accuracy of the resulting reliability index (or the probability of failure) would be highly difficult, if not impossible. Therefore, to guarantee the accuracy of the derived limit state equations, prior to the introduction of uncertainties for reliability analysis, a precise verification procedure was followed. This was achieved by calculating the deterministic quantities representing the differences between resistances and load effects and individually verifying them for each limit states equation by substituting the mean values of random variables into corresponding equations. This was similar to initially treating the problem as a fixed capacity and fixed demand case.

Live Load Increase and Resistance Deterioration Models

In the United States and Canada, a live load model first developed by Nowak (*14*) served as a basis for the development of new design provisions in the United States [AASHTO load and resistance factor design (LRFD)] and in Canada (Ontario Highway Bridge Design Code). In this study, the load model developed by Nowak (*14*) is used, mainly because of its final acceptance for implementation in current AASHTO LRFD specifications.

Fick's law of diffusion equation (*15*) is used as the concrete deterioration model for corrosion for the reinforced concrete decks and prestressed concrete girders of the highway bridges in the subnetwork. Specific surface chloride concentration values C_o for the highway bridges were determined that would realistically reflect the deicing and climatic conditions of the bridges in Colorado. For this purpose, empirical results of a comprehensive study reported by Hutter and Donnelly (*16*) at CDOT were utilized. In their study, concrete decks of a total of 13 bridges in Colorado were periodically measured for corrosion using penetration tests at the level of slab reinforcing. According to Hutter and Donnelly, application of salt for safe public motor vehicle travel varies considerably from one location to another (*16*). Mean values of the diffusion coefficients of individual concrete bridge decks and prestressed concrete girders of the Colorado highway bridges in the network were calculated through the use of a state-of-the-art computer-integrated knowledge system, CIKS, developed by the National Institute of Standards and Technology (*17*).

For deterioration of steel rolled I-beams, corrosion is assumed to penetrate the top and sides of the bottom flanges, in addition to each side of the web. Because of heavier exposure to leaking salt water, corrosion is assumed to occur throughout the web height at the supports, whereas it is assumed to occur only at the bottom quarter of the web height along the rest of the girder length including the midspan location.

ANALYSIS RESULTS

A general-purpose system reliability program for network-level analysis of structures (RELNET: Reliability of System Networks) was developed to calculate the lifetime component and system reliability profiles for different bridge types. The program is constructed based on an algorithm that treats structural systems as a collection of members having time-variant load and resistance functions. Results obtained from the program for the bridges in the subnetwork are briefly presented here, and detailed description of the developed computational platform will be reported elsewhere.

As a representative example, lifetime reliability index profiles for components and the system for Bridge E-16-QI are shown in Figure 7 (with group profiles in Figures 8 and 9). A 75-year time horizon (lifetime) is consistently used for all bridges. These are base profiles, meaning that they show the deterioration of a bridge's components and system throughout its lifetime without any interventions. Any repair or maintenance must be built on top of these profiles, causing discontinuous upward shifts at specific or periodic times. Mean values and standard deviations of the corrosion initiation times for the slab-reinforcing (T_{isr}) and prestressing steel (T_{ip}) are also shown in Figure 7, together with the corrosion penetration rates in terms of mils per year. As shown in Figure 7, for this continuous two-span prestressed girder bridge, having 14 I-shaped girders, flexure over the pier support due to negative moment has the highest reliability, followed by the positive flexure at midspan and the deck. System reliability follows the flexural reliability of the slab starting at 3.79 and decreasing to slightly below zero at the end of 75 years. When the slab is excluded from the system failure model; i.e., if slab failure is not considered as failure of the system, the system reliability profile follows the profile, slightly below, of girder flexure at midspan. In this case, the bridge system deteriorates at a smaller rate with the system reliability index reaching to a higher reliability index (1.99) at the end of its service life.

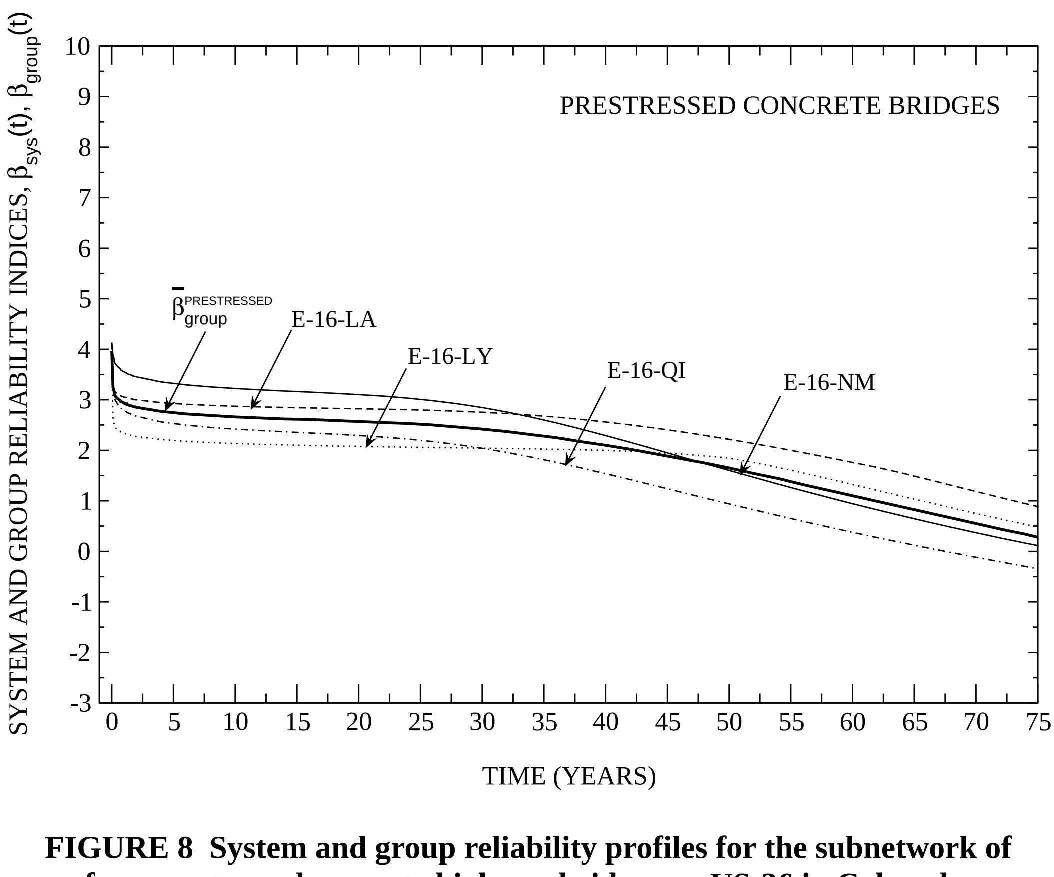

FIGURE 8 System and group reliability profiles for the subnetwork of four prestressed concrete highway bridges on US-36 in Colorado.

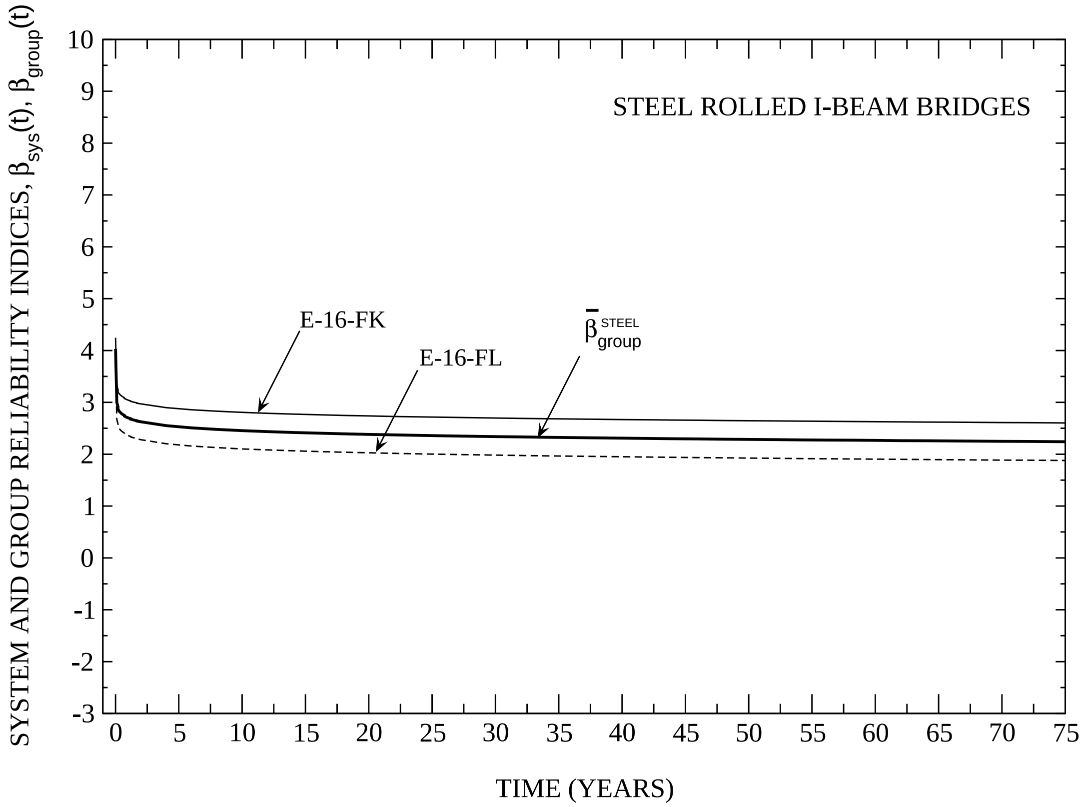

FIGURE 9 System and group reliability profiles for the subnetwork of two steel-rolled I-beam bridges on US-36 in Colorado.

Figures 8 and 9 show the reliability profiles for the four prestressed and the two steel highway bridges in subnetworks, respectively. For comparison purposes, profiles are plotted on an absolute scale, with all bridges starting their lifetime at time $t = 0$. Plotted system reliability profiles are based on system failure models including and excluding the flexural failure of the reinforced concrete decks, for the prestressed and steel bridges, respectively. Reliability profiles of the same bridge types are closely spaced. An average network group reliability profile (β_{group}) is generated for each bridge group based on the system reliability indices of individual bridges within that group.

CONCLUSIONS

An introductory preview of a database containing a network of highway bridges in Colorado, including extensive level of random variable data and a set of performance functions, is presented. Results of lifetime system and network reliability analyses for prestressed and steel bridges located within a subnetwork along US-36 are introduced. Capabilities of a general-purpose computational algorithm, with built-in live load and resistance deterioration models, specifically designed to solve the network level system reliability problem of a group of mix-type highway bridges, are briefly demonstrated. Systematically derived, AASHTO-based, generic limit state equations are applicable to other bridges and can easily be incorporated into code-based system reliability analyses of mix-type bridge networks containing the following member types: reinforced concrete decks, prestressed girders, steel rolled sections, steel plate girders, (steel sections can be compact or noncompact, composite or noncomposite), and reinforced concrete girders. Developed computational platform and methodology are aimed at supporting the advancement of reliability-based, intelligent bridge infrastructure management.

REFERENCES

1. *Showing Their Age: The Nation's Bridges at 40. Strategies to Improve the Condition of Our Bridges and Keep Them in Good Shape.* The Road Information Program, Washington, D.C., May 2002.
2. Thompson, P. D., E. P. Small, M. Johnson, and A. R. Marshall. The Pontis Bridge Management System. *Structural Engineering International*, IABSE, Vol. 8, No. 4, 1998, pp. 303–308.
3. Tao, Z., R. B. Corotis, and J. H. Ellis. Reliability-Based Bridge Design and Life-Cycle Management with Markov Decision Processes. *Structural Safety*, Vol. 16, No. 4, 1994, pp. 111–132.
4. Ellis, J. H., M. Jiang, and R. B. Corotis. Inspection, Maintenance, and Repair with Partial Observability. *Journal of Infrastructure Systems*, ASCE, Vol. 1, No. 2, 1995, pp. 92–99.
5. Frangopol, D. M. Life-Cycle Cost Analysis for Bridges. In *Bridge Safety and Reliability* (D. M. Frangopol, ed.), ASCE, Reston, Va., 1999, pp. 210–236.
6. Frangopol, D. M., and M. Ghosn. Bridge Reliability: Components and Systems. In *Bridge Safety and Reliability* (D. M. Frangopol, ed.), ASCE, Reston, Va., 1999, pp. 83–112.
7. Frangopol, D. M. Bridge Management Based on Reliability States and Whole Life Cost: From Theory to Implementation. *Proc., International Conference on Structural Faults and Repair*, London, England, 2001.
8. *Standard Specifications for Highway Bridges*, 16th ed. AASHTO, Washington, D.C., 1996.
9. *AASHTO LRFD Bridge Design Specifications*, 2nd ed. AASHTO, Washington, D.C., 1998.
10. *Manual for Condition Evaluation of Bridges*, 2nd ed. AASHTO, Washington, D.C., 1994.
11. *Bridge Manual Detailing and Checking*. Vol. II. Colorado Department of Transportation, Denver, 1987.
12. *Bridge Rating Manual*. Colorado Department of Transportation, Denver, 1995.
13. *Plans for Highway Bridge E-16-QI, City of Broomfield Project No. 9595, Plan Archives*. Colorado Department of Transportation, Denver, 1996, pp. B1–B23.
14. Nowak, A. S. Live Load Model for Highway Bridges. *Structural Safety*, Vol. 13, 1993, pp. 53–66.
15. Stanish, K. D., R. D. Hooton, and M. D. A.Thomas. Testing the Chloride Penetration Resistance of Concrete: A Literature Review. Department of Civil Engineering, University of Toronto, Ontario, Canada, 1997.
16. Hutter, W., and D. E. Donnelly. *Rate of Deterioration of Concrete Bridge Decks in Colorado*. Final Report No. CDOH-P&R-R-77-6. Colorado Division of Highways, Denver, Sept. 1977.
17. Bentz, D. P., J. Clifton, and K. Snyder. A Prototype Computer-Integrated Knowledge System: Predicting Service Life of Chloride-Exposed Steel-Reinforced Concrete. *Concrete International*, Vol. 18, No. 12, Dec. 1996, pp. 42–47.

Performance Measures and Reliability

PERFORMANCE MEASURES AND RELIABILITY

Performance-Based Programming of Bridges in New Jersey

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Although all bridge management systems (BMSs) supply a tremendous amount of technical data on an owner's bridge inventory and can be valuable assets in recommending projects and programs, for other than the bridge engineer they fall short of measuring performance. Further, while providing a large amount of information on the condition of any one bridge or the system as a whole, BMSs are a tool based only on those performance measures an agency decides to monitor. For New Jersey, an older state with extremely large traffic volumes on its roads, bridge conditions are a subject of significant concern. As part of an overall exercise for developing a capital investment strategy, bridge staff at the New Jersey Department of Transportation worked with state metropolitan planning organizations; state capital programming, local government aid, and construction staff; and FHWA New Jersey Division Office staff. The objective was to develop a set of performance-based goals, objectives, and performance measures and to devise alternative investment scenarios as programming tools for policy makers for the state's bridges. In this way, the state developed its investment strategy for bridges by setting goals and objectives, measurable performance measures, and benchmarks. With the state BMS serving as a tool to determine the starting point and model various scenarios, a capital plan was developed that is currently driving state investments and focusing decision makers on the needs of the system rather than on individual projects.

Although all bridge management systems (BMSs) supply a tremendous amount of technical data relative to an owner's bridge inventory and can be valuable assets in recommending projects and programs, for other than the bridge engineer they fall short of measuring performance. Further, while providing tremendous information on the condition of any one bridge or the system as a whole, bridge management systems are only a tool in determining what performance measures an agency should employ.

As an older state with extremely large traffic volumes on its roads, New Jersey has significant concern for the condition of its bridges. As part of an overall exercise in developing a capital investment strategy, bridge staff at New Jersey Department of Transportation (NJDOT) joined with metropolitan planning organizations (MPOs); staff involved in capital programming, local government aid, and construction; and staff from the New Jersey FHWA Division Office to develop a set of performance-based goals, objectives, and performance measures and alternative investment scenarios as programming tools by policy makers for the state's bridges. This paper will present the process by which the NJDOT developed its investment strategy for bridges by establishing goals and objectives, measurable performance measures, and benchmarks. The NJDOT BMS was used as a tool to determine the starting point and to model various scenarios. In this way, the current capital plan was developed. It is currently driving state investments and focusing decision makers on the needs of the system rather than on individual projects.

INVESTMENT STRATEGY

Over the years, many tools have become available to help agencies in decision making. Systems such as Pontis contain modules that are extremely capable of generating long-term investment strategies from mathematically derived preservation policies. These systems also can provide functional improvement actions that are based on inputting agency policies into the system. Determining these agency policies remains a difficulty for many agencies, however. For many owners, therein lies the challenge.

In 1996 NJDOT undertook a major effort to redesign its capital programming practices with the objectives of

- Better defining issues of statewide concern,
- Providing a better mix and volume of projects in the program, and
- Better determining needed resources for the various program areas within the agency.

As part of this activity, the bridge office was tasked with recommending strategic goals and supporting objectives with measurable outcomes that could be used to determine the effectiveness of investment. In other words, these measurable outcomes were the performance measures by which it would be determined if the program was successful in reaching its objective or goal. The target date of 2010 was given for completion of these goals and objectives.

Formation of Workgroup and Refining of Problem

In mid-1997, the state bridge engineer's office formed a multidisciplinary committee to develop recommended goal statements and performance measures for consideration by the Capital Investment Strategy (CIS) Task Force for the Bridge Preservation and Capitalized Bridge Maintenance program areas. Participants on the committee included representatives from the following:

- State Bridge Engineers Office, Structural Evaluation unit, Bridge Management section, and Structural Design unit;
- Regional Construction Office;
- Office of Capital Program Development;
- Division of Project Management, Bureau of Scope Development;
- Division of Local Aid and Economic Development;
- Bureau of Maintenance Engineering and Operations;
- New Jersey FHWA Division office; and
- Three MPOs covering the state.

Initially, there was considerable dialogue between the CIS subcommittee and management to refine the task assigned to the committee. Guidance from the department's upper management stated that the goals of a bridge program were twofold: achieving a state of good repair of bridges and ensuring the maximum useful life of the inventory.

One of the first issues the subcommittee tackled was deciding on the unit of measure for assessing conditions and measuring performance. Common practice for measuring success has always been to consider the change in the percentage of structurally deficient bridges year to year. Obviously measuring the number of bridges does not account for the size of the bridge or the costs associated with fixing either a large or small bridge. It was therefore determined that a better measurement would consider bridge deck area in setting targets or assessing performance.

It was also agreed that only structurally deficient bridges would be considered, as it was the department's policy only to address functionally obsolete bridges in corridor improvement projects. As an older state with a very mature infrastructure, New Jersey has many miles of highway built in the 1930s that do not meet today's geometric standards. However, they continue to serve the motorist adequately, and the structures along these routes are in a satisfactory structural condition. With proper management, maintenance, and enforcement, functionally obsolete structures can continue to serve the public for many years.

Finally, it was recognized that the nature of the traffic on these structures was very important. Therefore, bridges were grouped into the following categories:

- State bridges on the National Highway System (NHS),
- State-owned bridges not on the NHS,
- NJ TRANSIT overhead bridges,
- "Orphan" bridges, and
- Locally owned bridges.

Goal Development

In developing goals and performance measures, the committee kept in mind its charge to develop realistic goals that the department could move toward in the real world over time and therefore not be confined to current budget levels. The committee did agree to limit itself by examining what could be done to influence structurally deficient structures only. The committee also agreed to focus goals on those that could be accomplished with the resources normally available to the department. The amount of capital available was considered to be unconstrained.

The BMS was used to generate initial data for the condition of all structures within the system. A list of all structurally deficient bridges was generated. Bridges already in the pipeline for scoping, design, and construction were removed from this pool of structures for the purpose of determining future funding needs where financial commitments were already made to a project. The refined estimates or actual bid costs were used for these projects. Funding needs for the remaining projects were then determined by long-term project development cost.

Projects were then prioritized on the basis of several factors. The first factor was current commitments of a project to an existing construction program or plan. Second was the structural condition. In evaluating condition, priority was given to public safety, ability to provide intended service, and maintenance required to keep the structure in service. Finally, ranking was adjusted on the basis of local need as determined by the MPOs. The BMS was then used again, this time to run several scenarios, including a do-nothing scenario, a scenario assuming current investment levels, and scenarios assuming incremental increases in investment.

The committee then developed objectives for NHS, non-NHS, and local bridges. In reviewing the bridge program area, the committee recognized as most important that the state system have no load-posted bridges interfering with operations. The state system had only two posted bridges, both of which were already in the design pipeline. An obvious recommendation from the committee was that these two projects, which had been lingering because of permitting and environmental issues, be made a priority by the department.

The committee recognized that over time the bridge inventory would continue to deteriorate at a varying rate depending on many factors. No agency could ever expect to have zero structurally deficient bridges in its inventory. However, it was generally agreed that it would be possible to gain ground and reduce the number of structurally deficient bridges with the resources available to the department if the department's management desired. It was thought by the committee that, with the existing staffing and project delivery pipeline, the department could stretch to produce and construct bridges at a rate 2% faster than the bridges were deteriorating.

This preliminary goal was discussed with upper management. On the basis of the committee's analysis, upper management directed that the committee not limit itself to staffing or delivery pipeline constraints but assume they would be addressed. Clarification was provided by three targets for each of the highway operating classes. These targets were to correct the following percentages of structurally deficient deck area by the year 2010 (within 12 years):

- 100% for bridges on the NHS,
- 50% for state-owned non-NHS bridges, and
- 25% for bridges under local jurisdiction.

On the basis of this analysis, it was determined that an increase of 250% in bridge investment was needed to reach these targets. This translated into producing and constructing bridges at a rate 5% faster than they were deteriorating. Upper management did finally adopt these targets as objectives for the state leadership in developing funding for the bridge program.

Determining Detailed Needs

For the purposes of this paper, only selected analysis of detailed needs that the committee considered are presented. Of significant interest was the analysis of the existing capital program relative to projects targeted to address deficient state-owned bridges on the NHS. The performance of the existing capital program was evaluated by projecting the status of remaining bridge deficiencies after implementing rehabilitation and replacement projects scheduled for construction between FY98 and FY02.

Figure 1 shows the performance curve, labeled Total Deficient Deck Area, which illustrates this analysis. This curve identifies the total deficient deck area that was improved as a result of building out the current 5-year bridge program. For example, in FY98 it was estimated that the remaining deficient deck area on the NHS would be about 3.7 million ft^2, assuming that bridge projects scheduled for construction in FY97 were built. At the end of the 5-year period in FY02, a reduction of 800,000 ft^2 of deficient deck area was expected. Again, this assumed that the scheduled bridge projects programmed in the existing 5-year plan would be implemented.

An analysis of the committee goal, based on the same methodology described above, is also shown in Figure 1. As illustrated in this graph, the goal of pursuing a 5% reduction in deficient deck area per year over the 5-year period was very close to the FY98 to FY02 capital plan curve. In fact, the goal would have been achieved in FY98, FY01, and FY02. However beyond FY02, large bridge projects to replace 1.3 million ft^2 costing more than \$100 million were anticipated. Therefore, to offset future anticipated deficiencies, approximately \$142 million of additional funding (above the FY98–FY02 capital plan) was recommended for investment in bridge rehabilitation and replacement. Upon review of this recommendation, upper management directed that the goal be set to eliminate the backlog of structurally deficient bridges on the NHS by 2010.

This recommendation was implemented as shown in Figure 2, which illustrates an upward trend of dollars programmed for bridge improvements in the then-proposed FY98–FY02

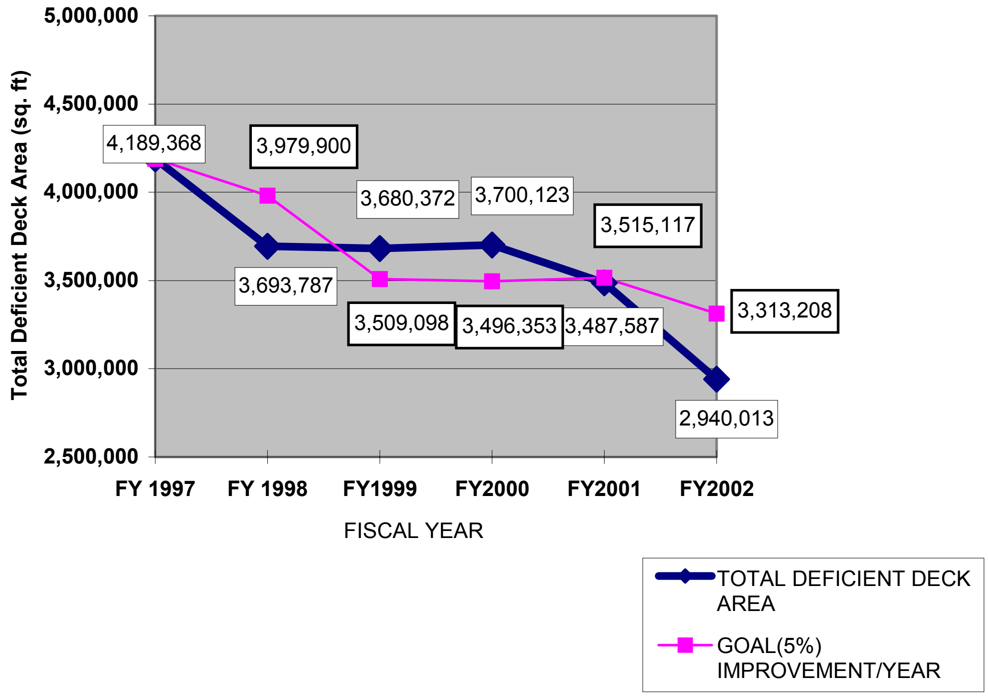

FIGURE 1 Total deficient deck area.

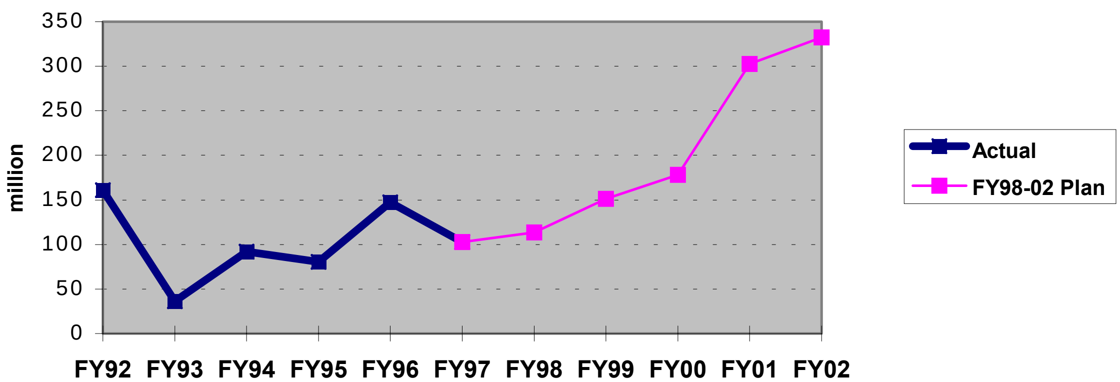

FIGURE 2 Recommended investments.

capital program. In fact the FY98–FY02 program proposed to invest approximately double the amount that was invested in the prior 5-year period.

As a result of the committee's work done, the department formally adopted the following goals and performance measures for this program:

- Goals
  - Eliminating the backlog of structurally deficient bridges on the NHS by 2010, and
  - Replacing the two-posted bridges, Route 9 over Bass River and Route 9 over Nacote Creek, on the state system within the next 5 years.
- Performance measures
  - For NHS highway bridges, the percentage improvement of square meters of total bridge deck area on state highway system bridges, and
  - For load-posted state bridges, the number of load-posted bridges replaced.

LOCAL BRIDGES

Another item of tremendous interest to the department was the condition of local bridges in the state. Although it was one of the first issues identified to be addressed by the department, the committees recommended that a strong message go out to the counties and local governments that their bridges should be made one of their highest investment priorities. The department had made major progress in improving the condition of state structures; however, the statistics for all bridges showed no improvement as a state. The condition of local bridges continued to worsen as improvements were made to state-owned bridges. This resulted in maintaining a status quo in the average condition of all the state's bridges.

At the time, the percentages of structurally deficient bridges in New Jersey were 13% for state-owned bridges and 24% for county and locally owned bridges. This number reflected the large amount of work New Jersey performed on the state system over recent years. Unfortunately, as resources are not equally available, local bridges continued to deteriorate. Therefore, the net effect was no measurable improvement in the general condition of all state bridges. By regulation, 15% of federal bridge rehabilitation (BR) funds must be spent on off-system bridges, 65% for on-system bridges, with 20% to be spent on either category at the state's discretion. New Jersey has roughly equal numbers of county and locally owned bridges, but historically, adequate amounts of BR funds have not been allocated to them. The average spent since 1993 had been $33 million annually in all programs. In addition, it would seem that more funds could be made available from the Transportation Trust Fund strictly for improvements to county and locally owned bridges.

The total deck area of New Jersey's 2,463 county and local bridges is 7,135,000,000 ft^2. Out of that area, records indicate that 1,733,000,000 ft^2, or a little more than 24%, represent deficient bridges. The current number of deficient county and local bridges is 596, which, it is estimated, would require a $12.43 billion investment to correct.

Review of the pipeline indicates that a realistic goal for short-term improvements is 12 bridges, on the basis of projects under way in project management, local aid, or scoping. This would reduce by 2% the total deck area of deficient county and local bridges. This is based on an equivalent deck area of 3,000 ft^2 per bridge (50 × 60 ft) for an annual reduction of 36,000 ft^2 of deficient bridge per year. The committee estimated that achieving this goal would require $24 million annually. Given a priority for local bridge improvements, increasing the output to 30 bridges a year would allow for an annual improvement of 5% of the total deck area of deficient county and local bridges. This assumes the same bridge statistics for an annual reduction of 90,000 ft^2 of deficient bridge per year. It was estimated that achieving this goal would require $60 million annually to achieve.

This category refers to the bridges that are owned by the counties and municipalities in New Jersey that require major rehabilitation or replacement efforts. Recently the department has greatly increased the priority given to these bridges, realizing that they may lie on important secondary road systems that may serve as alternatives to the state road system in their respective areas. The implementation of this policy is illustrated in Figure 3, which shows the steady increase in funding in the 5-year capital plan for local bridges through FY00 compared with the actual programmed amounts in the past years.

As with the bridges on the state highway system, the local bridges were continuing to deteriorate as traffic demands increased. To try to slow down the ever-increasing backlog, the FY98–FY02 capital plan proposed spending more than $80 million in FY00, almost double the amount spent on local bridges in the past. An analysis was performed of the current capital program on the basis of the performance measure the committee developed for local bridges, as presented above. This was done by projecting the status of local bridge deficiency conditions after implementing rehabilitation and replacement projects already scheduled for construction between FY98 and FY02 and also projecting the deterioration rate. Also analyzed for the FY98 to FY02 period was the goal for the local bridges developed by the committee on the basis of the methodology described above. Figure 4 illustrates the results of these analyses.

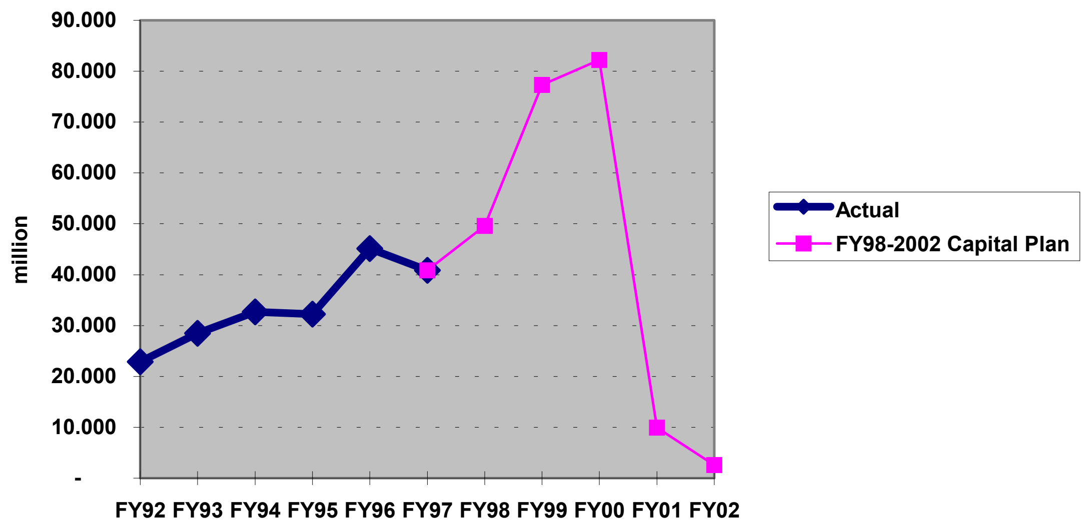

FIGURE 3 Local bridge investments.

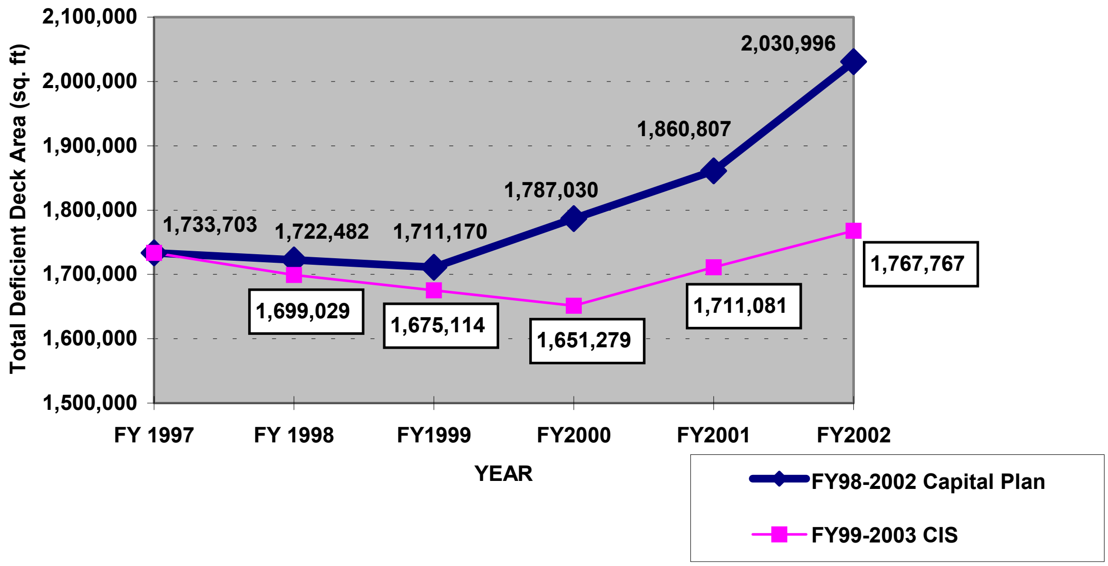

FIGURE 4 Locally owned deficient bridges.

As indicated in Figure 4, the existing 5-year capital program did not meet the committee's goal in any of the fiscal years of 1998 through 2002, and the gap steadily increased between the total amount of deficient deck area and the goal.

Table 1 lists by fiscal year the total deficient deck area, the goal for improvement, the goal as a percentage of the total deficient deck area, and the cost of achieving the goal in addition to what had already been programmed in the 5-year capital program. It shows that an additional $181.3 million was needed over the 5-year period to achieve the goal set by the committee. Figure 5 illustrates the total amount of money needed in each year to reach the goal set by the committee for local bridges. This figure clearly shows that the funding levels set in the existing capital program fell significantly below those needed to achieve the recommended goal, especially in FY00 and FY02. The funding level for FY03 was extrapolated using the short fall of funds from FY02. The additional funds needed to achieve the committee's goal for local

TABLE 1 Targets for Investment in Local Bridge Rehabilitation

Year	Deficient Deck Area (ft^2)	Improvement Goal Per Year (ft^2)	Goal	Cost to Achieve Goal
FY97	1,733,703	1,733,703		
FY98	1,722,482	1,699,029	2% of FY97 area	$16.3 million
FY99	1,711,170	1,675,114	2.75% of FY98 area	$8.8 million
FY00	1,787,030	1,651,279	3.5% of FY99 area	$69.4 million
FY01	1,860,807	1,711,081	4.25% of FY00 area	$9.8 million
FY02	2,030,996	1,767,767	5% of FY01 area	$79.0 million

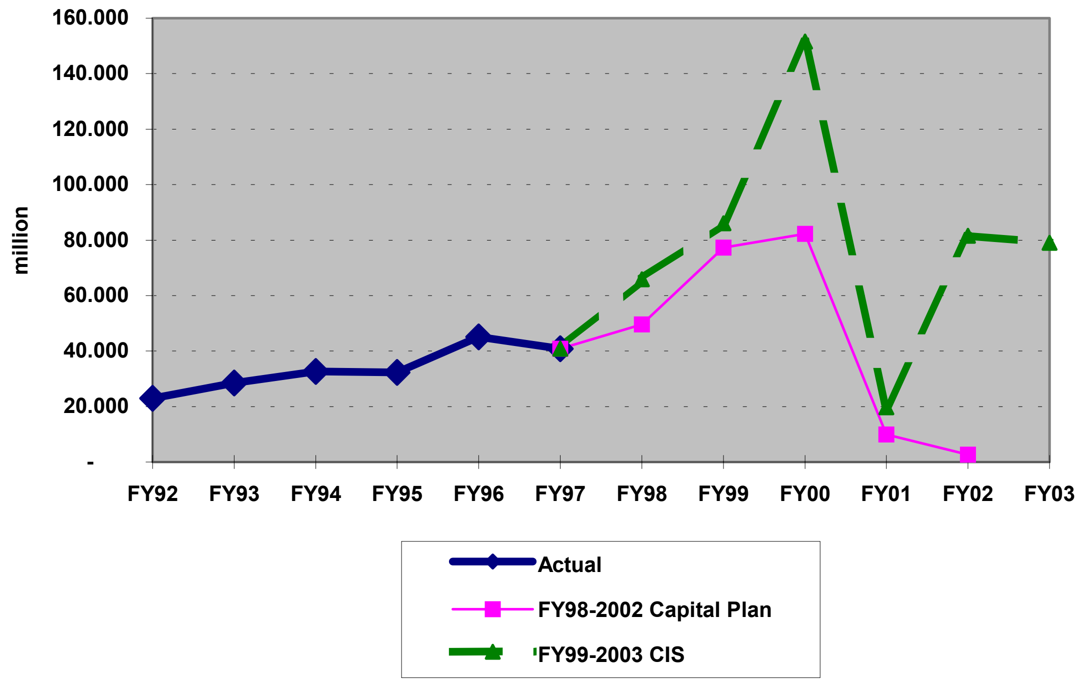

FIGURE 5 Local bridge funding needs.

bridges would have presented a serious difficulty for realistic funding. For that reason it was recommended that funding levels close to the present levels be maintained until an amicable solution to the funding shortfall could be developed.

Largely because of this analysis, a partial solution did present itself. In 1999 the New Jersey legislature passed the Statewide Transportation and Local Bridge Bond Act of 1999. This Act provided $250 million to be targeted for local bridge needs on an expedited basis with streamlined procedures developed by NJDOT and county engineers. Approximately a quarter of these funds have been programmed for local bridge improvements.

The department formally adopted the following goals and objectives:

- Goal. Rehabilitation and reconstruction of structurally deficient county and local bridges must be given priority in New Jersey. The deck area of structurally deficient county and locally owned bridges should be reduced by 2% per year in FY98, increasing incrementally to 5% per year by FY02.
- Performance measure. Percentage improvements of square feet of total bridge deck area on county and locally owned bridges that are listed as structurally deficient was identified as the metric for performance.

ACTUAL PERFORMANCE AGAINST GOALS AND PERFORMANCE MEASURES

While the committee's recommendations were not totally resourced, the goals and performance measures are still being used to measure the effectiveness of NJDOT's capital program. Figures 6 and 7 indicate actual and planned investments and the amount of remaining deficient bridge deck area for NHS bridges. It can be seen that while initial progress was made, the gap in funding has resulted in an increasing backlog of work required to correct deficiencies affecting New Jersey's NHS bridges. NJDOT monitors this data on an ongoing basis and uses it to adjust plans and programs where possible to reach the goals specified.

With respect to the NHS bridge condition, the department recognized it was dealing with a stretch goal to correct that amount of deficient deck area, but it has under construction the replacement of both load-posted bridges and has no other NHS bridges posted in its inventory as of 2002.

CONCLUSIONS

The recommendations of this committee were combined with the work of several other groups in the department on different investment areas to form NJDOT's CIS Document. This work contributed greatly to the passage of the Statewide Transportation and Local Bridge Bond Act of 1999. This document also contributed greatly to the passage of the Congestion Relief and Transportation Trust Fund Renewal Act of 2000. As part of that act, the legislature mandated that updates to CIS Document, along with reports of progress toward meeting the goals and objectives specified, be submitted annually along with the draft transportation capital program.

With respect to the state's investment in bridges, this process has had several effects. First, it greatly focused the capital program decision makers on the need to invest in high-priority, high-payoff bridge projects. It brought to the forefront anticipated needs for major bridge investment that the state will face in the near future. It also focused attention on the major problems of locally owned bridges within the state.

Over the past several years, the department has found the use of performance-based programming of New Jersey's bridge projects to be a tremendous asset for several reasons:

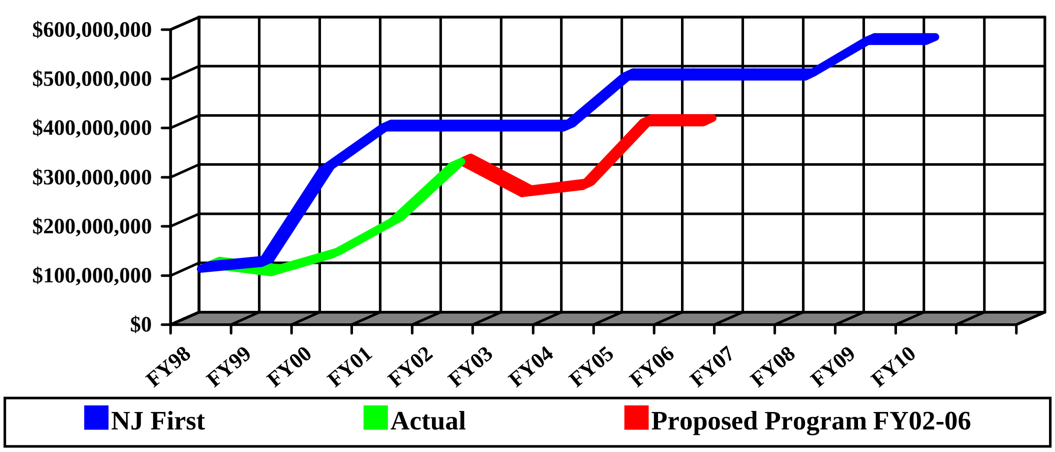

FIGURE 6 Recommended investment versus actual and planned for NHS bridges.

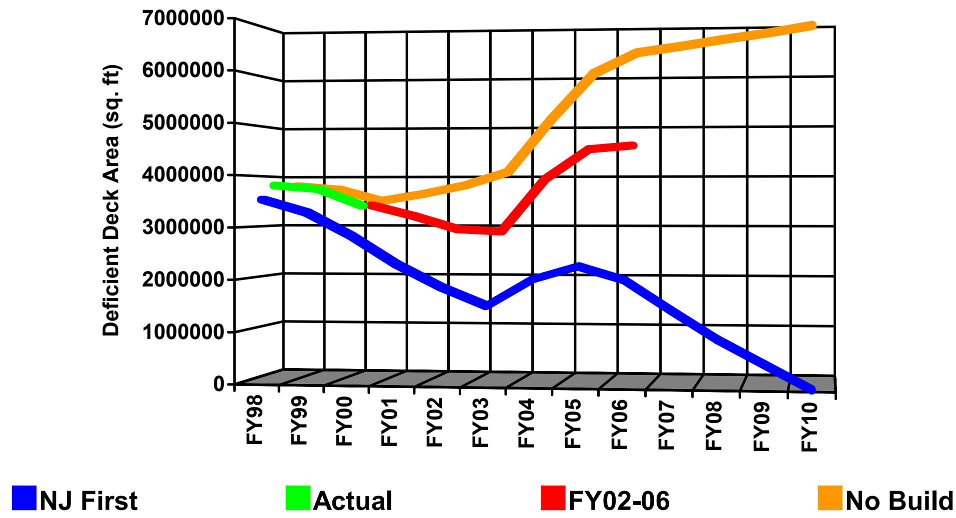

FIGURE 7 Projected deteriorated deck area against proposed, actual, planned, and do-nothing scenarios.

- It has maintained focus on high-payoff projects.
- It has largely prevented diversion of limited bridge resources to special interest projects.
- It has provided a mechanism for measuring the performance of the bridge program.
- Most importantly, it has resulted in better funding for bridge programs in New Jersey by demonstrating in a measurable way both needs and progress to those who hold the purse strings in the state.

ACKNOWLEDGMENTS

A special thanks to Jack Mansfield of Geotechnical Engineering, NJDOT, and to Jose Lopez and Sharon Harrison of NJDOT Structural Engineering for their tremendous support in editing and formatting this paper. Also thanks to Jack Lettiere of NJDOT and Dr. Mark Stout of Capital Programming for their visionary leadership in developing our capital investment strategy.

REFERENCES

1. *New Jersey's Capital Investment Strategy*. New Jersey Department of Transportation, Trenton, 2000.
2. Stout, M. L. New Jersey's Capital Investment Strategy: A Case Study of Performance-Based Programming. Presented at the 81st Annual Meeting of the Transportation Research Board, Washington, D.C., 2002.

New Bridge Performance Measures for Prioritizing Bridges

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The Sufficiency Rating (SR) has been used by the FHWA to establish eligibility for federal bridge funds. Increasingly, the states have viewed this system as needing improvement to better reflect bridge performance. Most states have begun implementing bridge management systems that require collection of element-level condition data that are more detailed than National Bridge Inventory (NBI) condition ratings. Hence, there is a need for new bridge performance measures that provide performance information that is more specific than that of the SR. Task 97 of NCHRP Project 20-07 was initiated by AASHTO in 1998 to develop new bridge performance measures. Four individual performance subindexes and a weighted composite index have been proposed. Databases of eight states with a combined total of 13,392 bridges were evaluated, and results are compared with the NBI SR.

The present reporting system of the National Bridge Inventory (NBI), including the Sufficiency Rating (SR) number, has been used by the FHWA to establish eligibility for federal bridge funds and to apportion those funds to states. Increasingly, the states have viewed this system as needing improvement to better reflect bridge performance. The industry's understanding of bridge safety and vulnerability to failure modes that were not recognized in earlier design codes has advanced significantly since the NBI was implemented. Most states have begun implementing BMS that require collection of element-level condition data that are more detailed than NBI condition ratings. Currently those states must inspect according to both the NBI system and the element-level system or translate element-level data to the NBI format for reporting. The recent introduction of the AASHTO *LFRD Bridge Design Specifications* (*1*) and ongoing NCHRP research to develop reliability-based procedures for bridge assessment advance a significant change in philosophy from current and past practice of bridge evaluation. Hence, there is a need for new bridge performance measures that provide more specific performance information than the SR, that are compatible with bridge management systems (BMS), and that reflect the current state of knowledge and practice of bridge engineering.

In 1998, NCHRP Project 20-07, Task 97, was awarded to Lichtenstein Consulting Engineers, Inc., by AASHTO Committee T18 to develop new bridge performance measures that not only address the aforementioned considerations but are also user-friendly, understandable, and easy to implement with data already mostly available in bridge files (*2, 3*). This project detailed a proposed approach consisting of four individual performance subindexes and a composite index consisting of weighted combinations of the subindexes. Each subindex is a targeted measure that reveals more specific information than the composite index. The four performance subindexes and their functions are the following:

- The Condition subindex is concerned primarily with structural safety.
- The Live Load subindex is concerned with measures of structural safety.
- The Geometric subindex is a measure of traffic safety and bridge functionality.
- The Special Events subindex is concerned with measures of structural safety.

The Bridge Sufficiency Index (SI) is a performance indicator obtained through a weighted combination of the four above subindexes. The bridge databases of eight states with a total of 13,392 bridges were evaluated with the proposed system, and the results were compared with the NBI SR. The results were also used to determine new eligibility criteria for funding that could be used with the proposed performance measures in a revenue-neutral manner. This paper will introduce the new performance measures proposed, discuss their merits, and compare them with current SRs.

PROPOSED INDEXES

The new performance measures are based on a safety perspective, measuring bridge attributes that have a direct or impending effect on safety. Safety includes both the structural safety of the bridge and the traffic safety of the public.

An overview of the proposed bridge performance measures is given in Table 1. The required data for the current and proposed systems are shown in Table 2. The proposed system would require element-level data for the condition index not currently recorded by the NBI. For

TABLE 1 Bridge Performance Measures

Four Individual Subindexes	**Scale**
1. Condition 2. Live Load 3. Geometric 4. Special Events	0 to 1.00 each
Two Composite Indexes	
1. Bridge Performance Index = $\begin{pmatrix} 30 \times \text{Condition Subindex} \\ + 30 \times \text{Live Load Subindex} \\ + 30 \times \text{Geometric Subindex} \\ + 10 \times \text{Special Events Subindex} \end{pmatrix}$	0 to 100
2. Bridge SI = Bridge Performance Index × Essentiality Factor	0 to 100
Where: Essentiality Factor = $1.00 - 0.15 \log \dfrac{(\text{Detour Factor})(\text{Importance Factor})}{200}$	0.85 to 1.0

TABLE 2 Data Items

Bridge Sufficiency Index (Proposed)		Sufficiency Rating (Current)	
Condition Subindex (Element-Level Data)		**Structural Adequacy and Safety**	
	Deck Elements	59	Superstructure
	Superstructure Elements	60	Substructure
	Substructure Elements	62	Culverts
	Other Elements	66	Inventory Rating
	Special Elements		
		Serviceability and Functional Obsolescence	
Live Load Subindex		28	Lanes on Structure
64	Operating Load Rating	29	Average Daily Traffic (ADT)
		32	Approach Roadway Width
Special Events Subindex		43	Structure Type, Main
113	Scour Vulnerability	51	Bridge Roadway Width
	Fracture Vulnerability	53	Vertical Overclearance
	Collision Vulnerability (vessel/truck/train)	58	Deck Condition
	Earthquake Vulnerability	67	Structural Evaluation
		68	Deck Geometry
Geometric Subindex		69	Underclearance
32,51	Roadway Width Differential	71	Waterway Adequacy
36	Traffic Safety Features	72	Approach Roadway Alignment
51	Bridge Roadway Width	100	STRAHNET Highway Designation
53	Vertical Overclearance		
54	Vertical Underclearance	**Essentiality for Public Use**	
55	Lateral Underclearance on Left	19	Detour Length
56	Lateral Underclearance on Right	29	ADT
71	Waterway Adequacy	100	STRAHNET Highway Designation
72	Approach Alignment Rating		
		Special Reductions	
Essentiality for Public Use		19	Detour Length
104	Importance (NHS)	36	Traffic Safety Features
19	Detour Time	43	Structure Type, Main
29	Average Daily Traffic		

NOTE: Numbers shown correspond to NBI item numbers.

states that have not yet implemented element-level inspection procedures, a rough translation of the NBI ratings into an equivalent condition index is proposed. The resulting condition index derived from average ratings of major components, however, is considered a less reliable indicator of bridge condition than that obtained using element-level data. The other subindex values can mostly be calculated from NBI data, but more data collection for special events vulnerabilities will be desirable in the future.

Two composite indexes are presented in this paper:

- The Bridge Performance Index is a straight performance indicator obtained utilizing a weighted combination of the four individual subindexes.
- The Bridge SI is obtained by modifying the Bridge Performance Index with a factor that accounts for the importance of the bridge and its essential function for the public.

It is envisioned that the new Bridge SI will be used as the current SR is used now: to provide a network-level assessment of deficient conditions and for project eligibility standards. The Bridge SI has the same sense as the SR in that a high score indicates an acceptable bridge.

CONDITION SUBINDEX

The new Condition subindex is intended to take advantage of the states' efforts over the past several years to improve bridge inspection standards on the basis of elements and condition states. It is desired to find a method that gives a better indication of condition than the existing NBI deck, superstructure, and substructure ratings, without being closely tied to any particular element inspection standard or bridge management system.

The minimum requirement for direct data input into this Condition subindex is an inspection system that uses individual elements instead of the NBI's broader classifications. The element information must define the percentage of the element that is in a condition that jeopardizes safety. This element information is then summed into element categories that represent the bridge's independent failure modes. For states that have not yet implemented inspections that permit the use of this Condition subindex, a rough translation of NBI ratings into an equivalent condition value is proposed.

Many states have implemented element-level bridge inspection, most of them adopting a condition-state methodology similar to the AASHTO *Guide for Commonly Recognized (CoRe) Structural Elements* standard (*4*). In a condition state inspection, the deterioration of a bridge element is classified into a small number of categories (three to five) that have precise engineering definitions. Only the worst of the condition states indicates the imminent possibility that the element may fail to meet its intended function. Other condition states indicate less severe conditions that may or may not warrant some form of preventive action. The Condition subindex does not measure economic performance and should be limited to those physical conditions that affect safety. A bridge can have many kinds of deteriorated conditions, each with a different impact on the need for replacement, emergency repairs, user costs, and safety. The Condition subindex must have a simple, reasonable way to combine these. BMS typically use life-cycle and user-cost models for this purpose. However, agency and user costs can vary significantly from state to state. An objective of the Condition subindex should be to approximate the effect of deficiencies on bridge and traffic safety, without requiring the input of economic data.

Simplicity is an important characteristic of the Condition subindex. Element-level data are already complex, so there is not much room for adding complexity to the calculation. The simplest methods use the inherent structure of the inspection process and data as much as possible, to minimize the number of adjustments necessary for the subindex to be meaningful.

Categorizing Elements

Each transportation agency has its own inspection manual and its own way of defining bridge elements. Any Condition subindex that includes information specific to individual element types will sacrifice generality. A simpler way is to use element categories. The categories can be defined to meet the following requirements:

- All elements in the category have a similar role in the functionality of the bridge.

- The overall effect of element failure, in agency and user costs, is roughly the same order of magnitude for all elements in each category.
- The types of work and their associated costs for all elements in the category are similar.
- Elements within each category deteriorate at similar rates.
- The worst condition states of the elements within a category have similar meanings.
- Any customized element definitions developed by individual states can readily be placed in appropriate categories, if they are relevant to a safety-related Condition subindex.

The following categories for meeting these requirements are proposed (the categories match the CoRe categories with some exceptions due to the safety-based nature of the Condition subindex):

- Superstructure primary elements. This category includes all primary load-carrying elements spanning substructure elements.
- Substructure primary elements. This category includes all primary load-carrying elements supporting the superstructure. This category includes culverts.
- Decks and slabs. This category consists of decks supported by superstructure elements. It does not include slabs supported directly by the substructure.
- Other safety-related elements. This category includes all elements not included in any of the above categories but that still have some direct or imminent effect on the safety of the structure or traffic safety. Many states have custom elements that could fit in this category.
- Special elements (Smart Flags). The AASHTO CoRe element guide describes a special set of elements called Smart Flags. Smart Flags are intended to highlight a problem that does not fit into the element-level data collection. Because Smart Flags can often indicate a severe defect that has safety implications, they are quite relevant to the Condition subindex.

Recognizing Deficiencies

In general, each element is recognized in the Condition subindex only if a portion of it is in one of the two worst defined condition states. For superstructures and substructures, the worst condition state usually features the language, "[Deficiency] is sufficient to warrant structural analysis to ascertain the impact on the ultimate strength and/or serviceability of either the element or the bridge." For agencies that are not using the CoRe element language, this definition can provide guidance for assigning parts of an element to the appropriate state. The worst element condition state is fully counted in the calculation of the index. The second worst condition state is also considered to have safety implications of a lesser nature. It is counted using the same weighting procedure as is used in the California Health Index, except the weighting decreases with decreasing numerical condition states instead of increasing, as deficiency is being measured by the Condition subindex instead of by economic value. The formula for the second worst condition state weighting factor (WF) is

$$WF = \frac{\text{Condition State Number} - 1}{\text{Number of Condition States} - 1}$$

Superstructure and substructure elements are usually measured in linear units (e.g., meters) or per piece (e.g., each column). Because it would be awkward to attempt to convert all elements to the same measurement system, conditions are indicated in percentages instead. Therefore, the percentage of an element in the worst two condition states is the main input to the Condition subindex calculation. All elements within a category are assumed to have an equal effect on the performance of the category as a whole.

As the percentage of an element in its most deteriorated state increases from zero over time, the importance of the first percentage points of severe deterioration is much greater than the later points. In other words, the movement from 0% to 1%, is of much greater significance than the movement from 50% to 51%. This effect is theoretically nonlinear but impossible to quantify exactly. To keep the methodology simple, the Condition subindex recognizes a linear importance from 0% to 25% and no additional effect beyond 25%. In effect, the percentage in the worst condition state is capped at 25% for each category. The choice of 25% was made for compatibility with the AASHTO CoRe element definitions for bridge decks.

The basic procedure is to calculate a demerit quantity for each element category, then subtract the total demerits from 100, and divide the result by 100 to yield the Condition index. This pattern is similar to the NBI SR and to condition ratings used in pavements and many other applications.

In summary, the Condition subindex evaluates the safety of the bridge on the basis of its element-derived condition. The bridge's structural safety is evaluated from the condition of either the superstructure or the substructure. This measure of safety is then modified by the deck, other, and special elements that have secondary contributions to the structural safety and contributions to the traffic safety.

LIVE LOAD SUBINDEX

The Live Load subindex is primarily a measure of the inherent level of safety in a bridge for live loads that regularly use the bridge.

The standard practice for Live Load assessment of bridges is currently the AASHTO *LRFD Bridge Design Specifications* (*1*). The wide variety of vehicle types commonly found on the nation's highways is usually represented by AASHTO legal loads for load-rating purposes. The rating factor computed for each vehicle type could be considered a direct performance indicator for the bridge under service live loads.

The present standard for load rating bridges is the load factor method. In this method, uniform Live Load factors of 2.17 (Inventory) and 1.3 (Operating) are used uniformly for all bridges irrespective of their traffic exposure conditions. The level of reliability represented by the load factor evaluation is variable. The inventory ratings represent the design standard and can be considered conservative for most evaluations. The operating rating is considered a better indicator of Live Load safety for in-service bridges than the inventory rating. However, the operating ratings, according to the load factor method, with a load factor of only 1.3 may be unconservative for some bridges with high exposure to truck traffic. It is therefore recommended that a modified operating rating be used as the Live Load subindex. A modifier based on site exposure to truck traffic is used to adjust the operating rating factors. The modifier is taken as

$$1.0 - \frac{\text{ADT}}{16600} \geq 0.7$$

This approach is similar to the procedures in the recently adopted AASHTO *LRFD Bridge Design Specifications* (*1*), wherein load factors are selected on the basis of site annual ADT. Bridges that have a rating factor less than 1.0 after modification should be considered vulnerable to damage or failure resulting from live loads. No modification will be required for legal load rating based on the LRFD manual, because it considers exposure to traffic volume in the rating process. Posting or closing a bridge does not affect the bridge's Live Load subindex. Permanent repairs or replacement will change the Live Load subindex as well as all other subindexes, but temporary measures will not.

GEOMETRIC SUBINDEX

Bridge geometrics are major determining factors for traffic safety and serviceability. There are several items in the NBI that document a bridge's geometric characteristics. They are reported as geometric inventory data or appraisal rating data that compare existing geometric features to minimum and desirable criteria. A coding system of 0 through 9 is used. A code of 8 indicates that the bridge has met present acceptable criteria. A code of 4 indicates that the bridge only satisfies the minimum criteria for being left in place as is. The FHWA *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges* (SI&A) (5) provides tables of geometric data for determining appraisal ratings. The geometric requirements are tied to the functional classification and traffic data for the route carried.

A new Geometric subindex that combines NBI items directly relevant to traffic safety and serviceability of the bridge is proposed. The new subindex will be a number between 0 and 1 and can be calculated with data already available in NBI. A Geometric subindex of 1.0 signifies that all key geometric parameters meet minimum acceptable standards, as specified in the 1995 FHWA SI&A Coding Guide. Bridges are neither penalized nor rewarded when they exceed these minimum acceptable standards for being left in place.

Nine geometric items contained in the NBI (or calculable from NBI data) have been identified as central to traffic safety and serviceability of existing bridges (Table 2). They include geometric elements on and under the structure. Features under a bridge may include an underpassing route or waterway or both. Features on a bridge also include approach roadway alignment and traffic safety features.

Relative weights have been determined for each item on the basis of its importance to the bridge's overall serviceability and traffic safety. The relative weights are reflected in the maximum deficiency points assigned to each item. Reduction factors are applied to the maximum number of deficiency points for each item on the basis of the severity of the deficiency and/or the volume of traffic exposed to the deficiency. After reduction factors are applied, the deficiency points for each item are totaled for the structure.

SPECIAL EVENTS SUBINDEX

Many bridges are vulnerable to failure modes that may not have been recognized or adequately safeguarded against in their original design. These failure modes are caused by natural hazards or extreme events and often lead to a bridge's catastrophic failure. Scour-related failures have accounted for more than half of the bridge collapses in the United States. The other three special events listed here are similarly responsible for a large percentage of bridge failures. These special events are becoming a more regular part of design and evaluation and must be represented in the national bridge rating and ranking. Apart from overloads (addressed in the Live Load subindex), four potential failure modes are significant depending on their frequency of occurrence: scour, fracture, collision, and earthquake.

Factors contributing to each failure mode are so unique and diverse that classifying vulnerabilities across failure modes is inherently somewhat judgmental. It is therefore important to develop a system that provides a uniform measure of a structure's vulnerability to failure for each special event. This is done on the basis of the likelihood of the special event's occurring and its consequences. Events with a high likelihood of causing failure with potentially severe consequences present the highest safety hazard to a bridge.

A Special Event vulnerability score is computed for each special event that is likely to occur at the bridge site. The special event with the highest score is deemed to pose the greatest risk to bridge safety. This score is then applied to determine a Special Event subindex for the bridge. The Special Event subindex is a number between 0 and 1. If the subindex is close to 0, the bridge has a high probability of failure as a result of a special event; if the subindex is 1.0 the bridge has a low probability of failure. To compute the Special Event subindex, the Likelihood of Event score and Consequence of Event score must first be determined. The proposed system combines the Likelihood and Consequence by multiplying the two scores to obtain a subindex value.

Likelihood of Event Score

The Likelihood of Event score is determined by classifying the bridge into vulnerability classes for each likely special event (Table 3). Currently the NBI reporting requirements include only the vulnerability of the nation's bridges to scour (Item 113). The rating is determined from observed scour at the bridge site or from the calculated scour potential. The rating reported under Item 113 is a quick indicator of the bridge's vulnerability to scour. Similar coding procedures have not been established for the other special event vulnerabilities in the NBI. It is likely that much of the data needed for classifying bridges for special events may already be available in bridge records or could be obtained with minimal additional effort.

If an evaluation has not been made for a given special event, the bridge will have an unknown vulnerability to that event and should receive a score of 0. This assumption may not have a significant effect on the overall results, as the Special Event score for the bridge is determined by only the governing vulnerability. Generally, a bridge has undergone some prior assessment for the special event for the event to pose the greatest threat to bridge safety at a given site. Three vulnerability classes have been identified to facilitate a uniform system of classifying bridges.

Consequence of Event Score

The consequence to the bridge, if the special event occurs, is evaluated and categorized according to Table 4. A structure's ability to survive an earthquake, a collision, a member fracture, or failure of a pier from scour will vary. The first two special events (earthquake and collision) represent applied displacements and loads. They are not member failures in themselves and may be resisted without significant damage to the structure. The full range of Consequence scores in Table 4 is applicable. The second two special events (fracture and scour failure) indicate the occurrence of a change in the structural condition instead of an external load. The Consequence score for either of these two events cannot be 0 but may not be a 10 (catastrophic failure), depending upon the bridge's redundancy and continuity. If a collapse mechanism results from assuming the affected member failure (as would be the case with almost any scour failure) then a score of 10 is appropriate. Structural damage or minor damage signifies localized failure that may be unnoticed by the bridge's traveling public and would not affect its serviceability and safety.

Currently all fracture-critical bridges are subjected to special inspection procedures targeted to address known fracture vulnerabilities. A member designated as fracture-critical in the NBI would have a consequence score of 10 for the fracture special event.

PROPOSED COMPOSITE PERFORMANCE INDEXES

The Bridge Performance Index is a straight performance indicator that combines the four individual bridge performance subindexes into one composite index. It is a number between 0 and 100. The Bridge SI then includes a modification for the essentiality of the bridge to the region. The two indexes are calculated as shown in Table 1.

Decision making for actions needed in the future would require the consideration of both individual and composite indexes. The composite indexes afford a means to compare all or a subset of bridges, whereas individual subindexes provide more detailed information on key performance issues that may be hidden when they are summed.

The four individual indexes are combined with weighting factors that reflect each subindex's relative importance to overall bridge performance. Prioritizing requires the simultaneous consideration of multiple attributes. Many bridge-related decisions rely on the judgments and opinions of decision makers. The weighting factors in this study reflect the collective judgment of the Project Panel (AASHTO T-18 Committee) and the Project Team. The factors were not calibrated to achieve a specific target value for the composite index.

COMPARISON OF PROPOSED SI TO NBI SR

Figure 1 shows a comparison of the weights in each category for the two systems. The differences in use between the existing and proposed systems are highlighted at the project level, where the greatest benefit from the proposed system can be realized. Bridge ranking is performed after the sufficiency values are calculated for each database. The ranking depends on the relative differences between bridges for each system, as opposed to the comparison of the different

TABLE 3 Likelihood of Event Classes and Scores

High	10
Medium	6
Low	0

TABLE 4 Consequence of Event Categories and Scores

Catastrophic Failure	10
Partial Collapse	6
Structural Damage	2
Minor or No Damage	0

measures between the systems for the same bridges. An examination of the ranking of the deficient bridges by the current system shows that a majority of the bridges that ranked low for the current system will still rank low by the proposed system, although the order of priority will change. The histograms of the two measures are shown in Figure 2 for the combined bridge database in this project. The proposed index has a distribution that more closely resembles a normal distribution and is more incremental. This is expected to produce more accurate relative rankings. The relative rankings by the two systems and the funding eligibility are separate issues.

ELIGIBILITY CRITERIA

The eligibility criteria to use with the proposed bridge performance measures are a policy decision within the domain of the FHWA. This section provides recommendations on the form of the eligibility criteria and values that would result in approximately the same number of bridges found eligible for federal assistance as are currently approved by the NBI system. The proposed criteria were quantified with a combined database consisting of all 13,392 bridges from 8 states evaluated by Lichtenstein and the FHWA. Eligibility for bridge-specific federal aid currently requires that a bridge be classified as either structurally deficient or functionally obsolete and have a SR of ≤ 80. All such bridges are eligible for rehabilitation. Bridges from that selection that have a SR of < 50.0 are eligible for replacement. In almost all cases a bridge with the required SR will also have the required classification. For this reason the comparison will consider eligibility based on the SR versus the proposed bridge performance measures, and the classifications of structurally deficient and functionally obsolete may be included or neglected in the definitions.

The proposed bridge performance measures indicate the safety of a bridge in terms of the four subindexes and two composite indexes. Deficiency in any of the four safety conditions can result in an unacceptable condition of being left in place as is. It is suggested that a bridge be eligible for rehabilitation or replacement whenever any of the four subindexes is less than a threshold value or when the combined SI is less than a threshold value. It is suggested that a bridge be eligible for rehabilitation

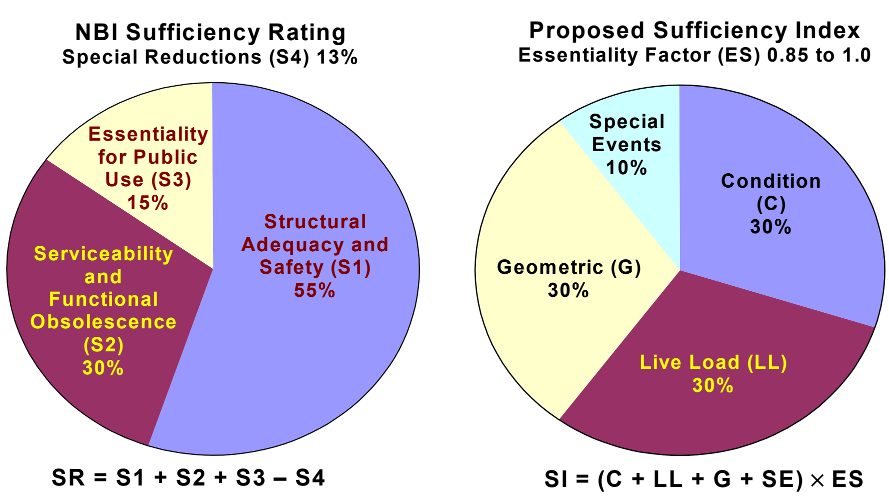

FIGURE 1 Components of bridge performance measures.

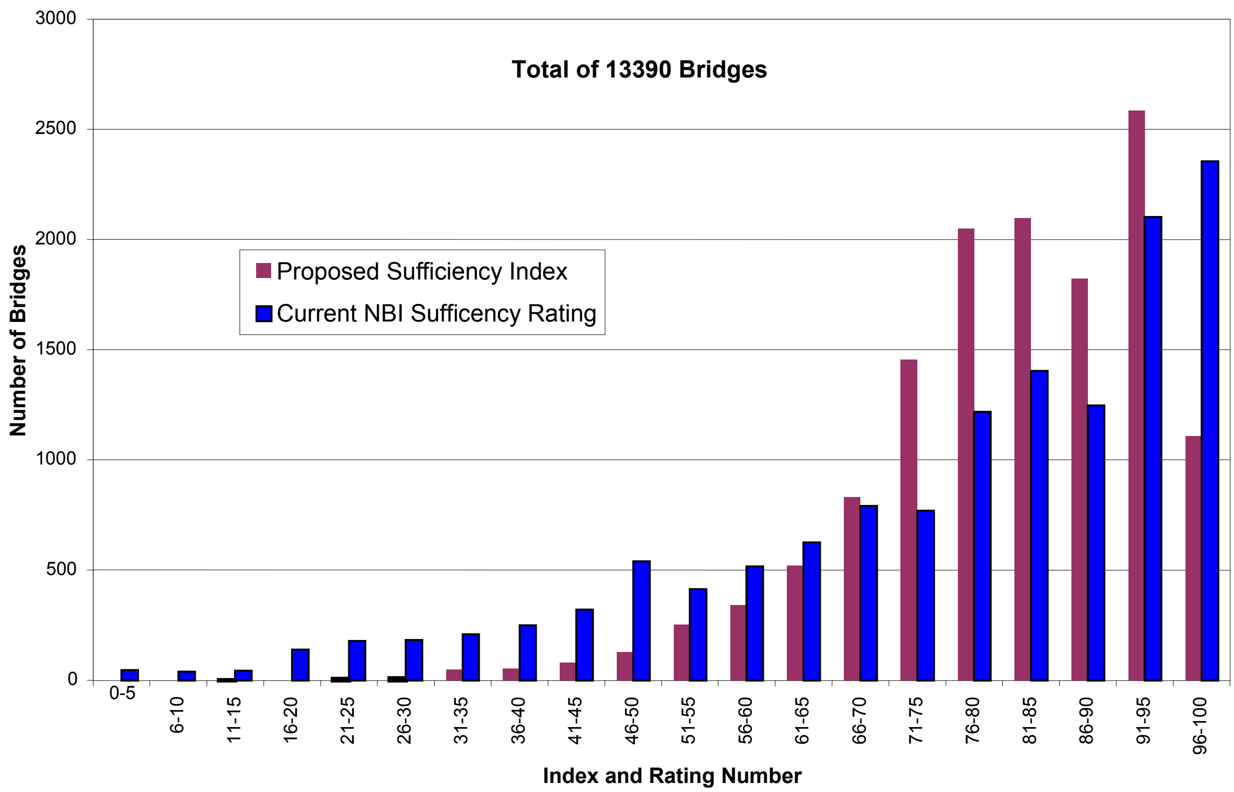

FIGURE 2 Histograms for combined databases from California, Minnesota, New Hampshire, New Jersey, South Dakota, Tennessee, Vermont, and Wisconsin.

funding when the combined SI alone is less than a threshold value.

It is possible that a bridge could meet the requirements set for rehabilitation or replacement without meeting the requirements for rehabilitation alone. This was not true in the old systems, as both requirements were based on the same scale. It may seem appropriate that eligibility for rehabilitation is a stage that a bridge must pass before becoming eligible for replacement. However, neither system actually contains the level of data that would differentiate between a condition that could be alleviated by rehabilitation as opposed to a condition that can only be alleviated through replacement. The proposed system, therefore, has based its primary eligibility for federal assistance on the safety basis of the individual measures. A secondary eligibility is provided to maintain the current level of number of bridges that qualify for a lesser assistance.

Evaluating the database by the current requirements indicates that 6,283 bridges (47% of the database) are eligible for rehabilitation (SR ≤ 80) and 1939 bridges (14% of the database) are eligible for replacement (SR < 50.0) funding. The corresponding threshold value for rehabilitation using the SI is 81.52 (SI < 81.52) for 6,283 bridges. In view of the accuracy of the two systems, a value of 80 for the SI is proposed. This criteria (SI < 80) results in the eligibility of 5,778 bridges (43% of the database) for rehabilitation. It should be noted that not all bridges eligible in a given year receive funding and that the small decrease from using a criteria of 80 would have no practical effect. The comparison of eligibility criteria is shown in the Rehabilitation row of Table 5.

With current requirements, 1,939 bridges (14% of the database) are eligible for rehabilitation or replacement funding. Because the proposed criteria involve two variables for the criteria of rehabilitation or replacement funding (the subindex threshold and the SI threshold), there are many combinations that will result in the same number of bridges by the proposed system. Using a subindex threshold of 0.4 (any of the four subindex values < 0.40) with a threshold of 60 on the SI (SI < 60.0) results in the eligibility of 2,037 bridges (15% of the database) for funding assistance for rehabilitation or replacement. As with the criteria for rehabilitation alone, the small increase in the number of eligible bridges here is expected to have no practical effect. A comparison of the eligibility according to the current system with the proposed system is shown in the rehabilitation or replacement row of Table 5.

These two cutoff values for primary eligibility were selected with consideration of the implications in the meaning of the values and of the fact that the results fit the requirement for the number of bridges eligible. A bridge that has accumulated enough safety defects overall for the SI to fall below 60 without any of the individual subindex values falling below 0.40 is in such general poor shape that it will require major rehabilitation or complete replacement. The subindex score of 0.40 has meaning. For the Condition subindex, major deterioration of either the superstructure or the substructure will result in a Condition subindex value of 0.375, which would trigger the eligibility criteria. For the Special Events subindex, 0.40 is a borderline value. Unlike the other subindexes, the Special Events subindex has six discrete scores that can be taken, as opposed to the more continuous forms of the other, better quantified, subindexes. Using an eligibility criteria of <0.40 will result in only the most extreme case for eligibility for funding on the basis of Special Events alone.

One concern for the selected eligibility criteria was that they yield reasonable and representative results with varied bridge inventories and conditions (i.e., California versus Vermont). There should be enough bridges in the combined databases to be representative of the nation, and none of the individual state inventories should be skewing the results. To test for this, the combined database was first evaluated in its complete form. Then a new database was constructed for each case after the data from a single state was removed from the combined database. Each of these subsets also produced close fits between the number of eligible bridges by the current system and the proposed system using the criteria in Table 5.

TABLE 5 Eligibility Criteria

Funding Level	Current NBI Eligibility with SR		Proposed Eligibility with Proposed Bridge Performance Measures	
Rehabilitation	SR ≤ 80		SI < 80	
	6,283 bridges	47% of total	5,778 bridges	43% of total
Rehabilitation or Replacement	SR < 50		SI < 60 or any of Condition, Live Load, Geometric, or Special Event < 0.40	
	1,939 bridges	14% of total	2,037 bridges	15% of total

CONCLUSIONS

This paper has proposed a system and threshold values based on the same criteria for all of the subindexes. The threshold values were determined so that the number of bridges qualified for each level of federal assistance would remain the same under the proposed and existing systems. This approach was taken because affecting the eligibility or the funding allocations was not the objective of the project. The objectives have been to present a system that has more logical components that can be used by bridge management engineers and are more suited to bridge management software, and to provide a better relative ranking between bridges.

The threshold values for each of the subindexes do not necessarily need to be the same, but at this time no better approach is available. It is important to remember that the subindex values are only decreased when features of the bridge do not meet the minimum safety requirements. This means that any bridge with less than 1.0 subindex values have features of concern. After applying the eligibility criteria to a state's inventory, a list of bridges eligible for federal assistance is generated. The state bridge engineer can then consider this listing with the ranking listing of all bridges by the SI.

The ranking by SI will provide the best comparative tool of the overall safety of each structure. The list of eligible bridges will contain entries that have been overall safety than some bridges not on the bridge list, but these eligible bridges will have specific areas (subindex categories) that are especially unsafe. The bridge management engineer will be able to make improved decisions by easily isolating the problem through the subindex scores and considering overall performance measures.

The approach of the proposed system is consistent with the safety-based goals of the federal bridge program. The results of the proposed system can be essentially revenue-neutral for the individual states, as compared with the current NBI-based system. The new system can be easily programmed into the various bridge management system software packages.

ACKNOWLEDGMENTS

The authors express their appreciation to Paul D. Thompson, consultant to Lichtenstein, for his valuable contribution to this project. The Transportation Research Board with David Beal as project manager funded NCHRP 20-07, Task 97. The authors acknowledge the advice and guidance of AASHTO Technical Committee T-18 with Stan Woods as chairman, the FHWA, and the various departments of transportation that participated in reviewing and testing the new performance measures.

REFERENCES

1. *LRFD Bridge Design Specifications.* AASHTO, Washington, D.C., 1998.
2. Lichtenstein Consulting Engineers. *Bridge Performance Measures*. Final Report. NCHRP, Project 20-07, Task 97, TRB, National Research Council, March 2000.
3. Lichtenstein Consulting Engineers. *Manual for Condition Evaluation and Load and Resistance Factor Rating of Highway Bridges*. Final Draft. NCHRP, TRB, National Research Council, Washington, D.C., June 2002.
4. *Guide for Commonly Recognized (CoRe) Structural Elements.* AASHTO, Washington, D.C., 1997.
5. *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges.* FHWA, U.S. Department of Transportation, 1995.

System Reliability Modeling in Bridge Management

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Most decisions required during the process of bridge management are made under conditions of uncertainty. Uncertainties associated with mechanical and environmental bridge loadings and with bridges' actual load-carrying capacities make it impossible to predict exactly the lifetime performance of these structures. Therefore, realistically, satisfactory bridge performance can be predicted only in terms of a probabilistic measure of assurance of performance (i.e., in terms of reliability). Also, uncertainties associated with future maintenance and user costs make it impossible to predict exactly the whole life costing of bridge maintenance programs. In light of these uncertainties, bridge management systems have to be reliability-based. A framework was developed for modeling system reliability in bridge management. The proposed framework is applicable to both individual bridges and bridge groups. Examples of bridge system reliability modeling under various maintenance scenarios and selection of the optimum solution in a reliability-based context were also developed.

During the past decades, interest in effective management of bridges has increased drastically. As a result, various bridge management systems (BMSs) have been developed to manage limited resources effectively and to maintain bridges in an acceptable condition (*1*). For effective bridge management, the realistic assessment of bridge systems under various uncertainties is essential. These unavoidable uncertainties originate from numbers of sources. Moreover, not only the assessment but also the prediction of future bridge performance is requested in BMSs. Time-dependent factors such as resistance deterioration and time-varying loadings should be included in the prediction procedure. In addition, the effect of future events, such as maintenance interventions, should be considered. In general, uncertainties associated with prediction are larger than those for the assessment of the current condition of bridges. The validity and accuracy of the assessment and prediction results greatly depend on the way the uncertainties are accounted for (*2, 3*).

Recently, the possibility of a better lifetime solution in bridge management has been investigated by considering not only essential maintenance but also preventive maintenance (PM). In general, essential maintenance is applied when the bridge's condition is below a target (minimum acceptable) value. However, PM is applied on a predetermined schedule to maintain a bridge above a target performance level. In practice, a mixed—essential and preventive—maintenance strategy is considered as the best solution in many situations. The effect of a maintenance action depends on bridge types, mechanical and environmental conditions, and properties of selected maintenance actions. Therefore, it is impossible to find an optimum maintenance scenario applicable to all bridge types. To manage bridges effectively at minimum cost and to select the best maintenance scenario, life-cycle cost analysis (LCCA) is necessary (*4*).

This paper provides the framework required for modeling system reliability in bridge management. This framework is used to perform bridge LCCA by using reliability and cost profiles over a given time horizon. It treats uncertainties included in bridge assessment and prediction processes by means of probability and reliability methods. The interaction between cost and reliability can be represented efficiently in this framework. The advantages of the proposed reliability-based framework for BMS are

- Objectivity. The reliability index measure takes into account in an objective manner the uncertainties associated with performance of bridge members and systems. On the contrary, condition states are generally defined by using different scales for different types of bridge members or systems.
- Uniformity. The same measure (i.e., reliability index) can be used for assessing members, subsystems, systems, and occurrence of different limit state conditions. For instance, different failure modes can be considered separately and/or combined in a probabilistic sense. Not only the ultimate performance but also serviceability can be examined by using the same reliability measure.
- Standardization. The same framework can be applied effectively to other types of deteriorating structures. The procedure is identical. Only parameters have to be calibrated independently corresponding to candidate systems. The effects of inspection and maintenance methods can also be considered through a standardized procedure.
- Extensibility. The method can be extended to solve more complex problems requested by modern and future management programs. For instance, the interaction between reliability and cost can easily be accounted for. In addition, mathematical programming optimization methods can be used efficiently to produce the best solution and advanced technologies for inspection and maintenance can be easily accounted for.

This paper illustrates the application of the reliability-based bridge system management to an individual bridge and a group of bridges. Different maintenance scenarios are considered, and their associated reliability and cost profiles are computed. In addition, the extensibility of the proposed method is demonstrated by including the mathematical programming process into the proposed framework. The optimal mean application time of maintenance actions associated with each selected maintenance scenario is evaluated. The reliability and cost profiles corresponding to the obtained optimum mean application times are also evaluated and compared with those resulting from applying solutions suggested by experts.

RELIABILITY PROFILES ASSOCIATED WITH MAINTENANCE SCENARIOS

In modern BMS, evaluating bridge condition is of primary interest and various measures, including health indexes, are used to assess the condition status of bridges. In this study, the condition of a bridge is represented by the reliability index, β. Instead of a single value for the reliability index, however, the probability distribution of the reliability index is defined and used by including uncertainties associated with reliability evaluation processes. The distribution of the reliability index changes over the bridge life cycle as a result of decreasing resistance, increasing traffic volume, and other physical and environmental factors. The time variation of the probability distribution of the reliability index is represented by the reliability profile.

Figure 1 shows an example of the time variation of the distribution of the reliability index. Two reliability profiles based on different maintenance scenarios (i.e., without any maintenance and with cyclic PM) are indicated. The mean reliability index profiles are shown. The maximum reliability deterioration rate, α_{max}, for the case without any maintenance, and the maximum reliability deterioration rate, θ_{max}, for the case with maintenance, were obtained from data on 32 steel/concrete composite bridge reported by Nowak et al. (*5*). The figure shows the reliability index profiles associated with the moment failure mode. However, other failure modes such as shear can be obtained in a similar way. By combining the failure modes the system reliability index profile can be considered. The Monte Carlo method is used to evaluate the distribution of the reliability index profiles. Further details on the reliability deterioration models and random variables associated with the evaluation can be found in Frangopol et al. (*6*).

The reliability index profiles of different bridge members can be probabilistically combined to produce a system reliability index profile. With the reliability index profile, used to the probability can be evaluated that the system down-crosses a specific target reliability value at a certain point in time. The same approach can be used to obtain the appropriate time of rehabilitation (associated with the given target level) and the remaining lifetime of the system. As an example, Figure 2 shows the probability density functions of the time at which the reliability index of a group of bridges with and without PM down-crosses the target reliability index of 4.6. With the results in Figure 2, the time of application of an essential maintenance intervention can be predicted.

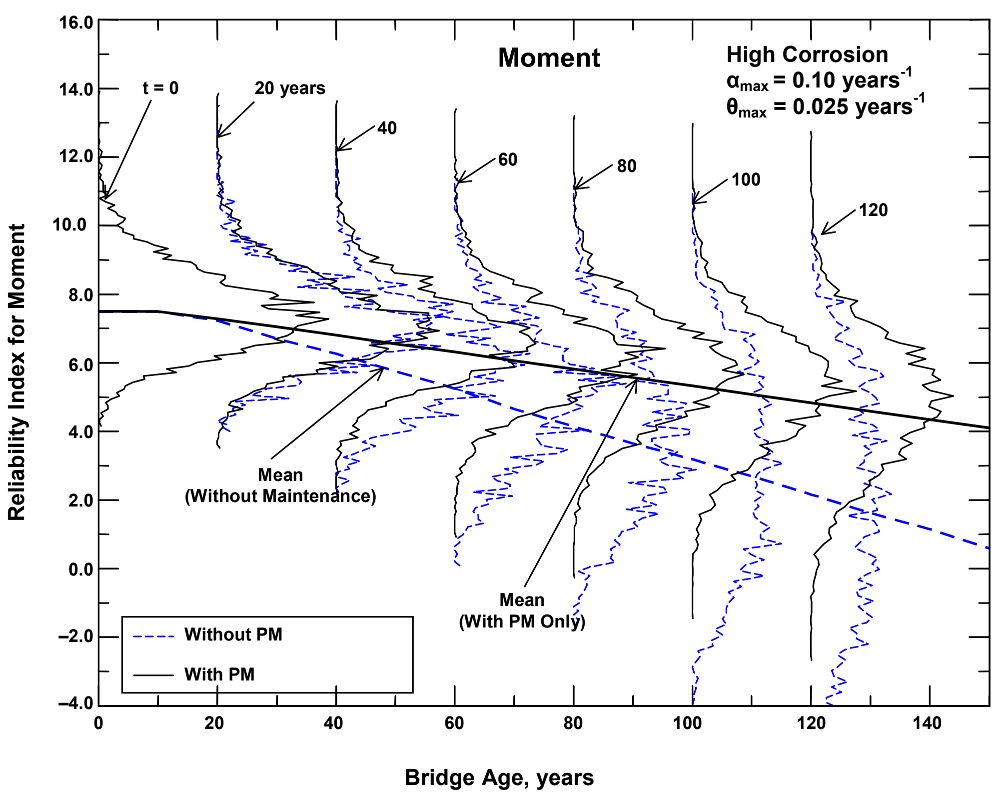

FIGURE 1 Time variation of the distribution of the reliability index for moment.

Within the proposed framework, reliability states can be defined instead of defining conventional condition or health states. For instance, the reliability index range from 3 to 5 can be defined as the state in which a group of bridges needs an essential maintenance intervention. Users can define appropriate maintenance methods for different reliability states to maintain bridges in operating conditions. This framework is applicable for investigating the reliability state of a group of bridges. Therefore, the bridge owner can predict the distribution of bridges belonging to different states and determine appropriate maintenance scenarios. For instance, Figure 3 shows the time variation of the number of bridges in five different reliability states considering PM for a group of 713 bridges built from 1955 to 1998 (*6*). In this case, the reliability states are defined by number as follows:

1. $\beta > 4.6$,
2. $4.6 \leq \beta < 6.0$,
3. $6.0 \leq \beta < 8.0$,
4. $8.0 \leq \beta < 9.0$, and
5. $\beta \geq 9.0$.

The proposed approach can consider more complex maintenance scenarios. As mentioned, the mixed (preventive and essential) maintenance is, in general, cost-effective. Figure 4 is an example of a deterministic reliability index profile produced by a mixed maintenance scenario associated with two essential actions and a single cyclic PM action. In reality, however,

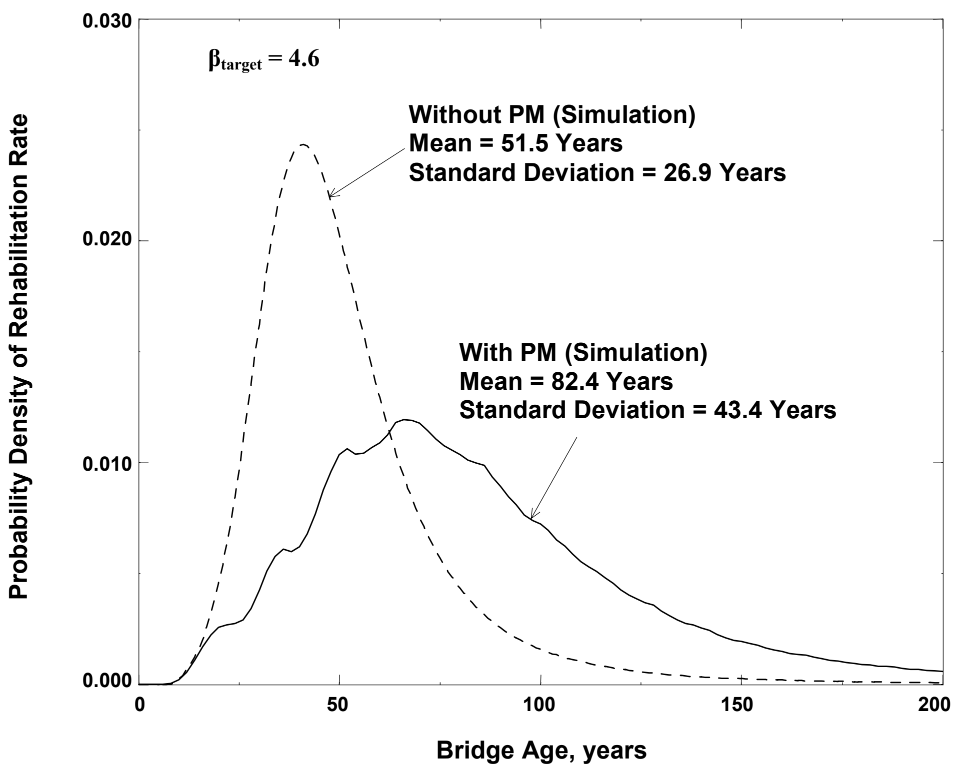

FIGURE 2 PDF of time at which event $\beta < 4.6$ occurs with or without PM.

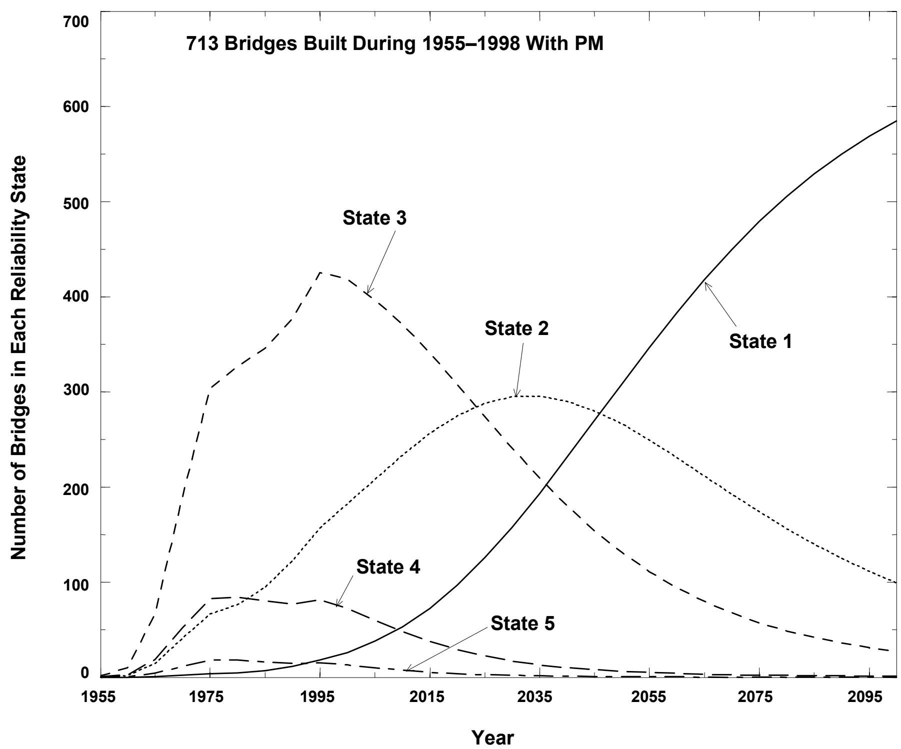

FIGURE 3 Time variation of number of bridges in each reliability state.

the reliability index profile is random because of the uncertainties associated with maintenance actions, such as time of application. Figure 5 illustrates the mean reliability index profiles including the effect of uncertainties, associated with different maintenance scenarios. The eight maintenance scenarios and their definitions considered in Figure 5 are

1. *E0*—no maintenance scenario,
2. *Ei* (*i* = 1, 2, 3, 4)—maintenance scenario associated with essential maintenances,
3. *E0,P*—cyclic PM scenario without essential maintenances,
4. *E1,P*—cyclic PM scenario with one essential maintenance, and
5. *E2,P*—cyclic PM scenario with two essential maintenances.

The properties of the steel-concrete bridge system considered and parameters of applied maintenance interventions can be found in Kong (*7*). The effects of maintenance interventions and the characteristics of different maintenance scenarios can be assessed by using the mean reliability index profiles in Figure 5 (*8*).

COST PROFILES ASSOCIATED WITH MAINTENANCE SCENARIOS

For the life-cycle management of bridges, not only the assessment of structures but also the LCCA is important and essential for selecting the optimal solution. The cost profile corresponding to a maintenance scenario can be constructed during the evaluation of the reliability index profile by considering the cost of each maintenance intervention and the time of

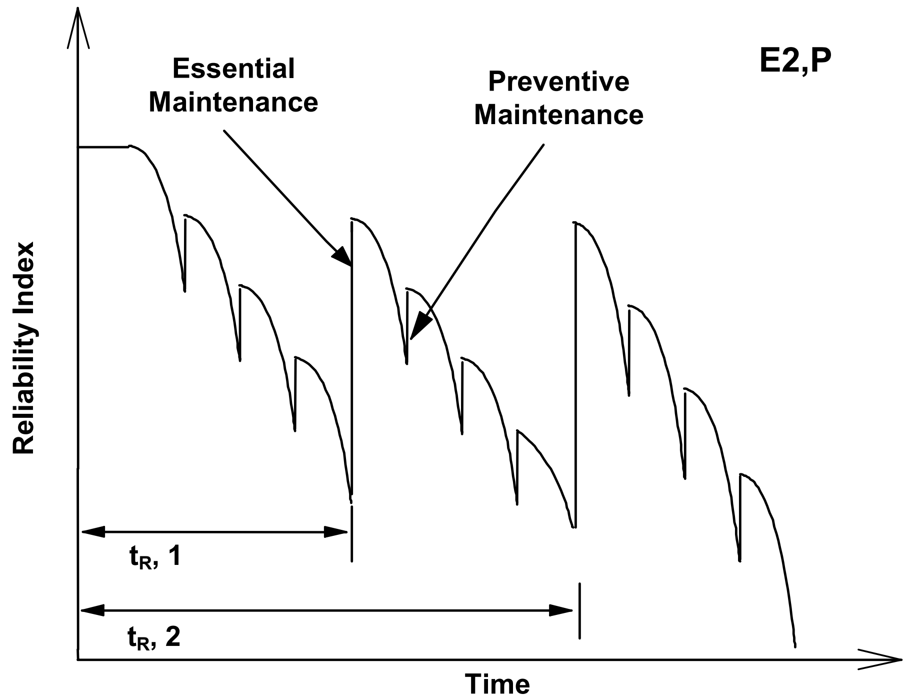

FIGURE 4 Deterministic reliability profile produced by mixed maintenance scenario.

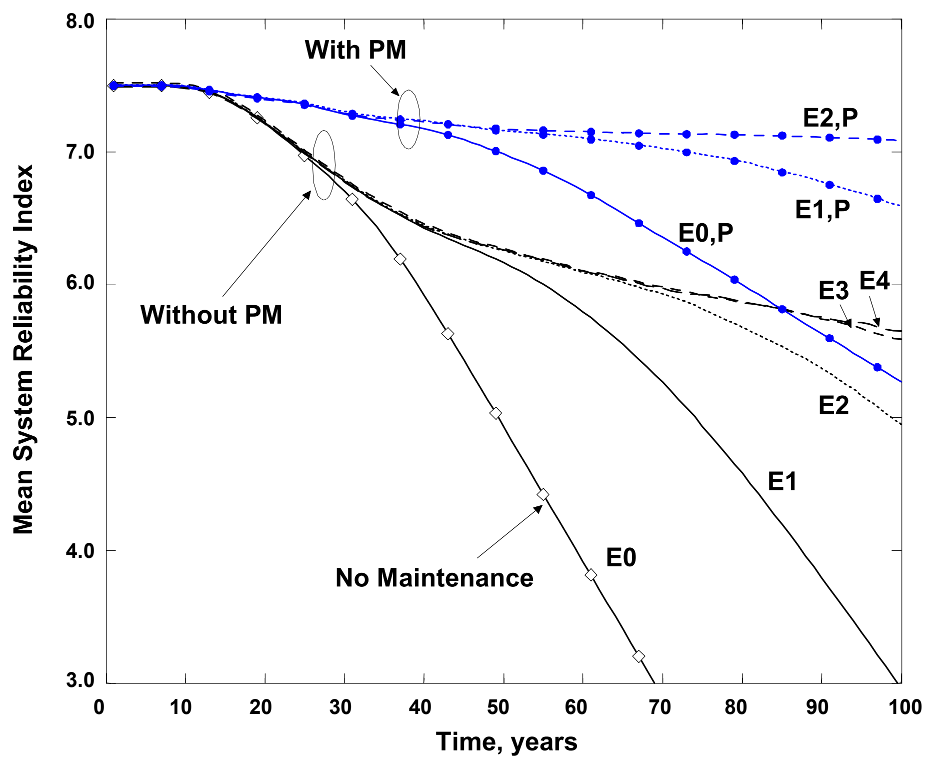

FIGURE 5 Mean reliability index profiles associated with different maintenance scenarios.

its application. For instance, Figure 6 shows the present value of the expected cumulative maintenance cost per unit deck area (m^2) associated with different maintenance scenarios considering a discount rate of money of 6% (*8*). In this case, maintenance cost includes the user cost. As indicated, because of discounting, the rate of cost increase becomes smaller as time goes by. Again, uncertainties included in the cost analysis can be handled during a Monte Carlo simulation process. The cost data for generating the results in Figure 6 are found in a report by Maunsell Ltd and Transport Research Laboratory (*9*). By comparing the reliability index profiles and cost profiles associated with different maintenance scenarios, bridge managers are in a position to choose the best alternative.

OPTIMIZATION ANALYSIS

In general, a bridge management system involves many interdependent parameters. It is common practice to perform an optimization analysis to obtain the best solution that satisfies certain requirements. Various optimization formulations can be proposed. However, the proposed formulation can not always be solved, because it greatly depends on the capability and extensibility of the main computational module included in the bridge management system. For instance, if the application times of maintenance interventions are predetermined, then the times of application cannot be used as design variables. In this case, the appropriate optimization problem is to find the best possible combination of maintenance interventions. For BMS based on lifetime

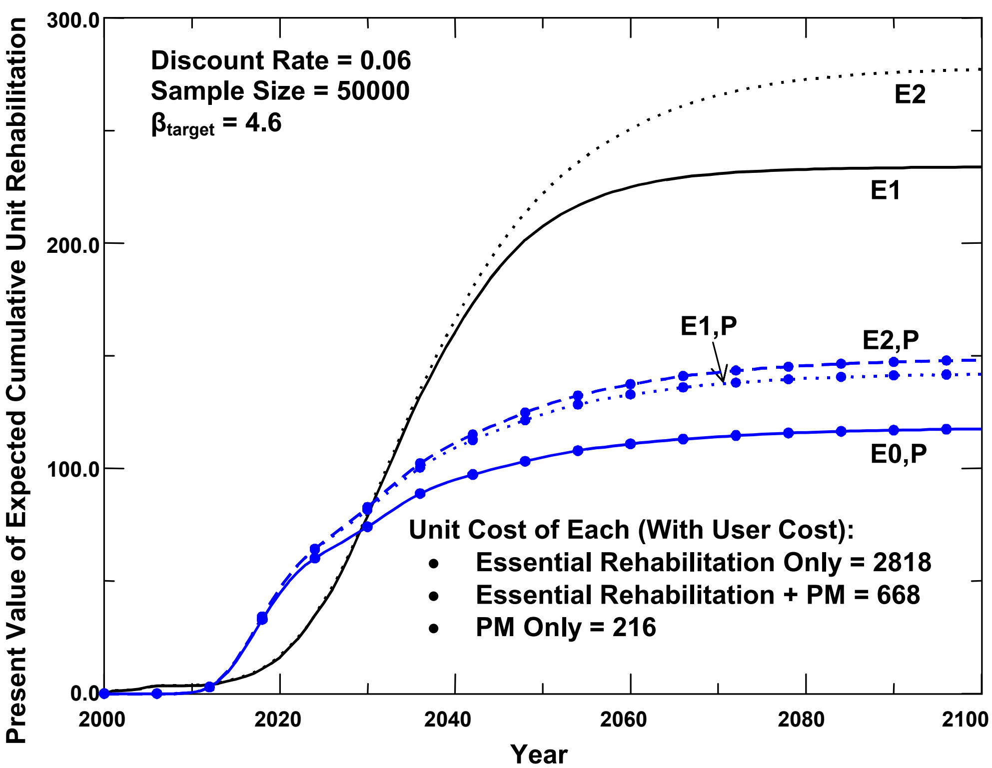

FIGURE 6 Present value of expected cumulative maintenance cost (£) per unit deck area (m^2) associated with different maintenance scenarios.

maintenance cost analysis, the optimal solution is the maintenance scenario associated with the minimum cost. In general, combinatorial optimization algorithms are used for solving this problem. However, this kind of optimization problem can be solved effectively only when the number of possible maintenance interventions is small. In addition, it is impossible to define the interaction between maintenance costs and improvement in bridge reliability with current BMS.

The proposed reliability–cost framework for bridge management can be easily used in the conventional optimization process. In fact, the reliability and cost profiles and the cost-reliability interaction are expressed in terms of functions, and a variety of design variables can be used to construct a proper optimization problem. In this paper, one of the most common optimization problems in bridge management—finding the minimum lifetime cost maintenance scenario—is illustrated based on the proposed framework. In this case, the objective function is the expected cumulative cost at the time horizon, and the design variables are the mean application times of maintenance interventions. Figure 7 shows six maintenance scenarios and the corresponding design variables that represent the application times of maintenance interventions for each scenario. The application time of essential maintenance interventions is described by the log–normal distribution, and the triangular distribution is used for the preventive interventions. The coefficient of variation for each distribution is based on the report by Maunsell Ltd and Transport Research Laboratory (*9*).

Figure 8 illustrates the optimal cost of each maintenance scenario and corresponding optimal mean time of application of maintenance interventions. The time horizon is fixed at 50 years, and the discount rate is set to 6%. Costs are indicated in pound sterling (£) per unit deck area (m^2) for steel/concrete composite bridges. The cost consists of the maintenance and the

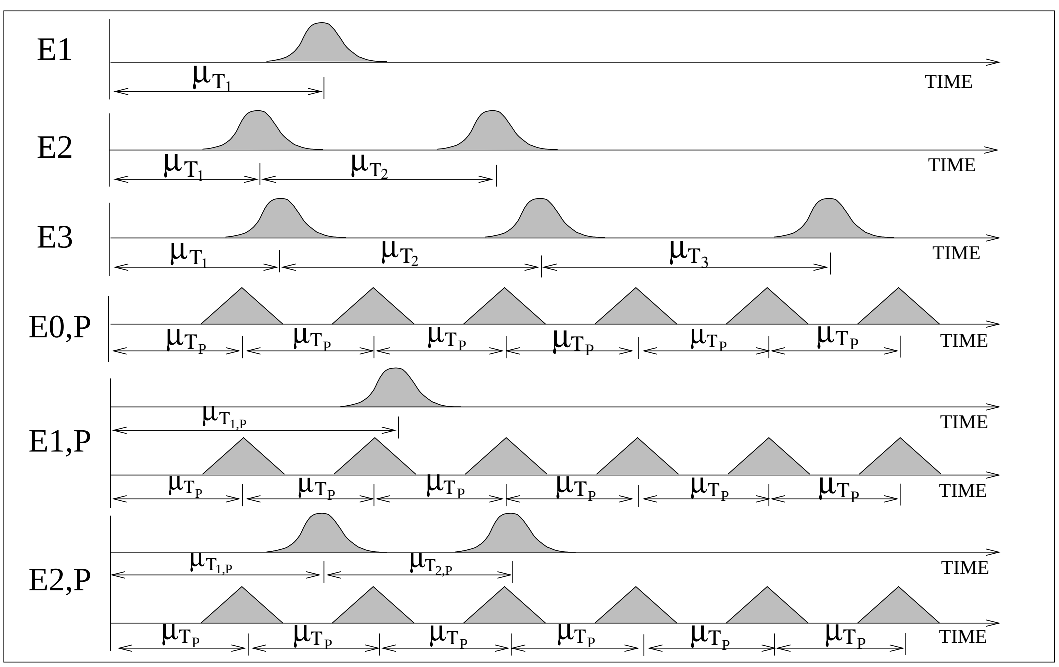

FIGURE 7 Design variables (mean application time) associated with different maintenance scenarios.

failure costs. The maintenance cost includes the user cost. The failure cost is calculated based on the reliability index of the system as

$$c_f \left(\beta_O - \beta_H\right)^2$$

where

c_f = failure cost coefficient,
β_O = mean initial reliability index of the system, and
β_H = mean reliability index of the system at time horizon.

The results show that for the case considered—c_f = 100,000; time horizon = 50 years—the effect of the failure cost on the optimum solution is extremely high. The magnitude of this effect depends on the time horizon, the cost of failure, and the discount rate, among other factors. The additional essential maintenance intervention cost in the total cost decreases as a result of the effect of the discount rate (i.e., costs associated with second essential maintenance in *E2*; costs associated with second and third essential maintenance in *E3*). The time interval μ_{Tp} (see Figure 7) associated with the cyclic PM interventions in *E0,P* is 2.7 years, in *E1,P* it is 6.2 years , and in *E2,P* it is 6.9 years. The minimum cost solutions associated with different maintenance scenarios can be rationally compared by evaluating the mean system reliability index profiles corresponding to these solutions.

Figure 9 shows the mean system reliability index profiles associated with minimum cost solutions. The maintenance scenarios *E0, E1,* and *E0,P* produce relatively low reliability indices at the time horizon compared with other scenarios. Therefore, the scenario *E0,P* can be excluded from selection even if its cost is less than *E1,P*. The scenario *E2,P* represents the best solution for both cost and reliability.

The methods for evaluating the reliability index and cost profiles and optimization maintenance process for individual bridges can be applied to groups of similar bridges. The uncertainties associated with the bridge groups can be considered through random variables (*10*). As an example, a group of 713 concrete/steel composite bridges under various maintenance scenarios is considered.

Figure 10 shows the optimal mean application times and associated costs of seven different maintenance scenarios for the bridge group considered. The base year for discounting is 2000, the discount rate is 6%, the year 2050 is set as the time horizon, and the failure cost coefficient assumed is c_f = 100,000. Given these assumptions, Figure 10 also indicates the maintenance cost of nonoptimal maintenance scenarios with distributions of time of application of maintenance interventions suggested by experts (*9*). Maintenance scenarios associated with data from experts result in higher cost than those associated with computed optimum scenarios with the proposed approach for all maintenance scenarios. For the scenario *E0,P,* the cost ratio between the two cases is 4.25. For the case based on experts' data, the most economical scenario is *E3*. However, the cost of this scenario is 3.5 times larger than the optimal cost result.

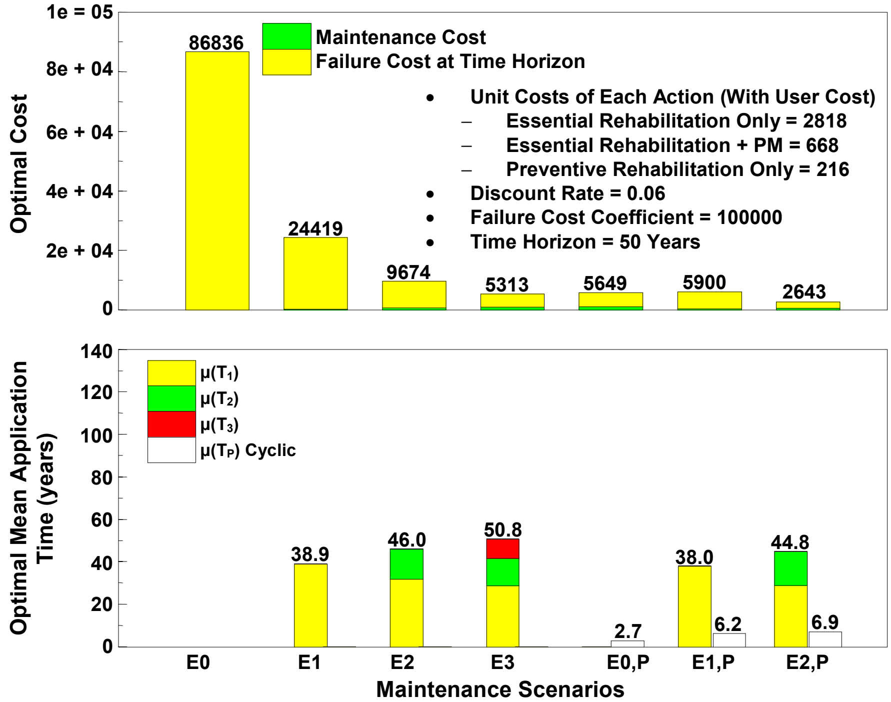

FIGURE 8 Optimal cost of each maintenance scenario and corresponding optimal time of application of maintenance interventions.

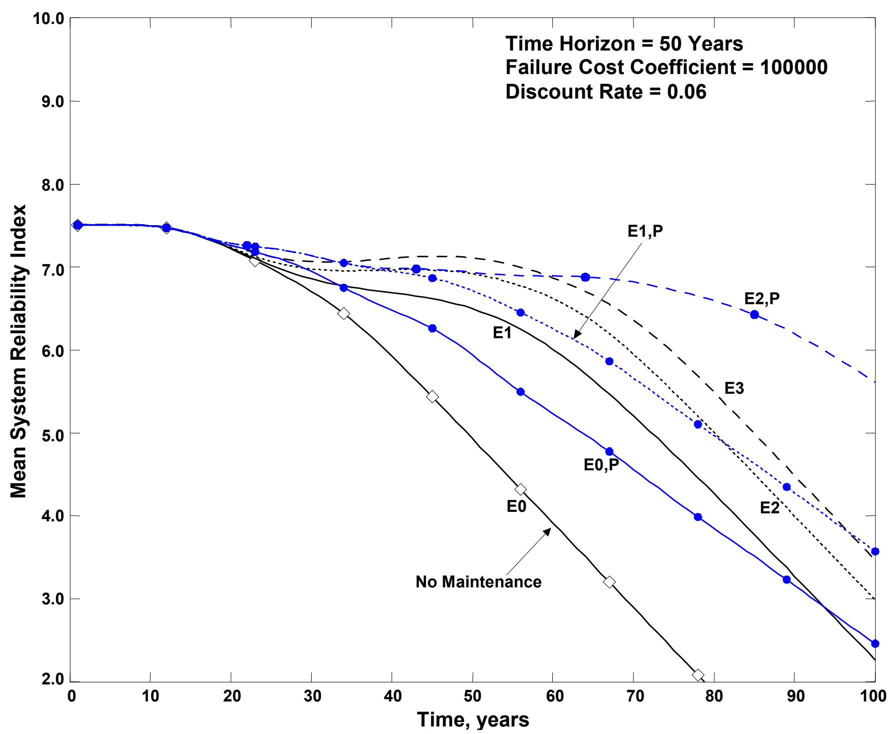

FIGURE 9 Mean system reliability profiles associated with optimization results.

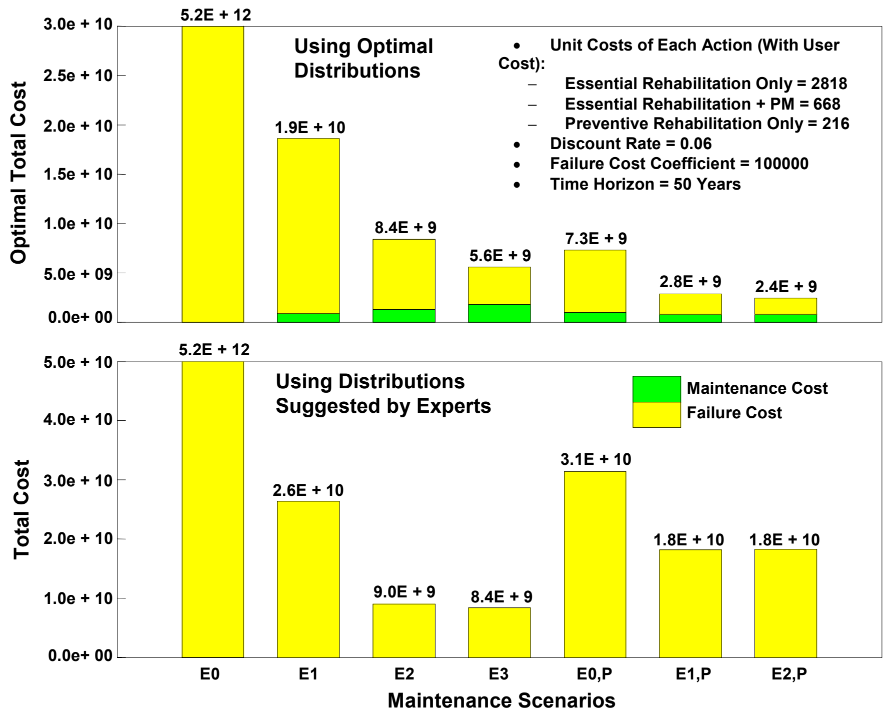

FIGURE 10 Comparison of total costs given optimal times of applications of maintenance interventions (above) and times of applications suggested by experts (below).

CONCLUSIONS

In this study, the reliability-based management framework for bridge LCCA based on system reliability modeling was introduced. The proposed framework uses random variables to deal with unavoidable uncertainties included in bridges' lifetime behavior and constructs the reliability index profile. This profile illustrates the time-varying propagation of the bridge reliability index distribution.

With the proposed approach, various interventions during bridge lifetime, such as inspection and maintenance, can be considered effectively in spite of the great uncertainties associated with them, and cost profiles can be constructed and used for LCCA. The proposed framework provides the basis for lifetime cost optimization of individual bridges and bridge groups. The reliability and usability of the proposed method depend on the quality of data and fundamental relationship between bridge lifetime maintenance interventions and their effects on the reliability of the bridge system.

ACKNOWLEDGMENTS

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REFERENCES

1. *Proc., International Bridge Management Conference, Vol. I and II, Preprints.* TRB and FHWA, Denver, Colo., April 26–28, 1999.
2. Frangopol, D. M. and P. C. Das. Management of Bridge Stocks Based on Future Reliability and Maintenance Costs. *Bridge Design, Construction, and Maintenance* (P. C. Das, D. M. Frangopol, and A. S. Nowak, eds.), The Institution of Civil Engineers, Thomas Telford, London, 1999, pp. 45–58.
3. Das, P. C. Reliability-Based Bridge Management Procedures. *Bridge Management Four (*M. J. Ryall, G. A. R. Parke, and J. E. Harding, eds.), Thomas Telford, London, 2000, pp. 1–11.
4. National Engineering Technology Corporation. *Bridge Life-Cycle Cost Analysis Guidance Manual.* Report NCHRP 12-43. NCHRP, TRB, National Research Council, Washington, D.C., 2002.
5. Nowak, A. S., C.-H. Park, and M. M. Szersen. *Revision of the Bridge Assessment Rules Based on Whole Life Performance: Steel Bridges*. Final Report. The Highways Agency, London, 1997.
6. Frangopol, D. M., J. S. Kong, and E. S. Gharaibeh. Reliability-Based Life-Cycle Management of Highway Bridges. *Journal of Computing in Civil Engineering*, ASCE, Vol. 15, No. 1, 2001, pp. 27–34.
7. Kong, J. S. *Lifetime Maintenance Strategies for Deteriorating Structures*. Ph.D. thesis. University of Colorado, Boulder, 2001.
8. Kong, J. S., and D. M. Frangopol. Life-Cycle Reliability-Based Maintenance Cost Optimization of Deteriorating Structures with Emphasis on Bridges. *Journal of Structural Engineering*, ASCE, 2002 (submitted).
9. Maunsell Ltd and Transport Research Laboratory. *Strategic Review of Bridge Maintenance Costs: Report on 1998 Review*. Final Report. The Highways Agency, London, 1999.
10. Kong, J. S., and D. M. Frangopol. Whole Life Costing of Optimum Maintenance Strategies for Bridge Groups. In *Towards a Better Built Environment—Innovation, Sustainability, Information Technology,* IABSE, Melbourne, Australia (in press).

Expert Systems and Uncertainties

Bridge Expert Analysis and Decision Support System

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Reg Quinton
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Alberta Transportation is in the process of developing an expert system that will support the department's bridge management functions. The system's primary objectives are to facilitate consistent and accurate decisions to optimize the allocation of bridge funds, evaluate system performance, and plan and manage bridge construction, rehabilitation, and maintenance actions. The Bridge Expert Analysis and Decision Support (BEADS) system will be a major component of a larger departmentwide integrated Transportation Infrastructure Management System (TIMS) and will routinely interact with the corporate data repository and other TIMS components. In addition to improvement needs related to condition and functionality, the BEADS system will respond to highway network expansion plans and socioeconomic decisions. The BEADS system consists of individual modules that address bridge structure elements and functional limitations. These include the Substructure, Superstructure, Paint, Strength, Bridge Rail, Bridge Width, Vertical Clearance, Replacement, and Culvert Modules. On the basis of existing and predicted condition and functionality states, the modules identify potential work activities, including their timing and cost, throughout the economic life cycle. The Strategy Builder Module then assembles and groups the identified work activities into feasible life-cycle strategies. A life-cycle cost analysis ranks the strategies. Once the project-level analysis results have been determined, a network-level analysis may be performed to facilitate short-term programming, analysis of long-range budget scenarios, evaluation of network status, and assessment of the impact of policy decisions.

Alberta Transportation is responsible for approximately 30,000 km of provincial highways that include more than 4,100 bridge structures. The department also provides funding for more than 9,800 bridge structures on the province's municipal road system. With a transportation budget in 2001 of Can$1.2 billion (Can$1.54 = US$1.00, 2001), the department must ensure that funds are allocated appropriately to maximize the service life of the highway infrastructure at a minimum life-cycle cost. Tools and systems are being developed to assist in achieving this goal.

Alberta Transportation is in the process of refining and implementing the Bridge Expert Analysis and Decision Support (BEADS) system that will support the department's bridge management functions. The system's primary objective is to facilitate consistent and accurate decisions to optimize the allocation of bridge funds, evaluate system performance, and plan and manage bridge construction, rehabilitation, and maintenance actions.

TRANSPORTATION INFRASTRUCTURE MANAGEMENT SYSTEM

The BEADS system will be a major component of a larger departmentwide integrated Transportation Infrastructure Management System (TIMS). The purpose of TIMS is to justify and rank the development, design, construction, rehabilitation, and maintenance needs of the highway system provincewide to optimize the allocation of funds to ensure long-term value. Along with BEADS, the other expert systems in TIMS include the Roadway Maintenance and Rehabilitation Application (RoMaRa), which addresses pavement management requirements, and the Network Expansion Support System (NESS), which deals with expansion and improvement of the provincial highway system.

TIMS analysis components are supported by a corporate data repository, geographic information system, and reporting tools. Once TIMS development is complete, the BEADS component will routinely interact with the corporate data repository and the other TIMS components. In addition to improvement needs related to condition and functionality, the BEADS system will respond to highway network expansion plans and socioeconomic decisions. Figure 1 outlines TIMS' core work processes.

DEVELOPMENT HISTORY

The development of the BEADS system has been based on the department's existing bridge inventory and inspection systems. The Bridge Information System (BIS) was originally developed in the early 1970s and continues to track and store inventory data on all existing and

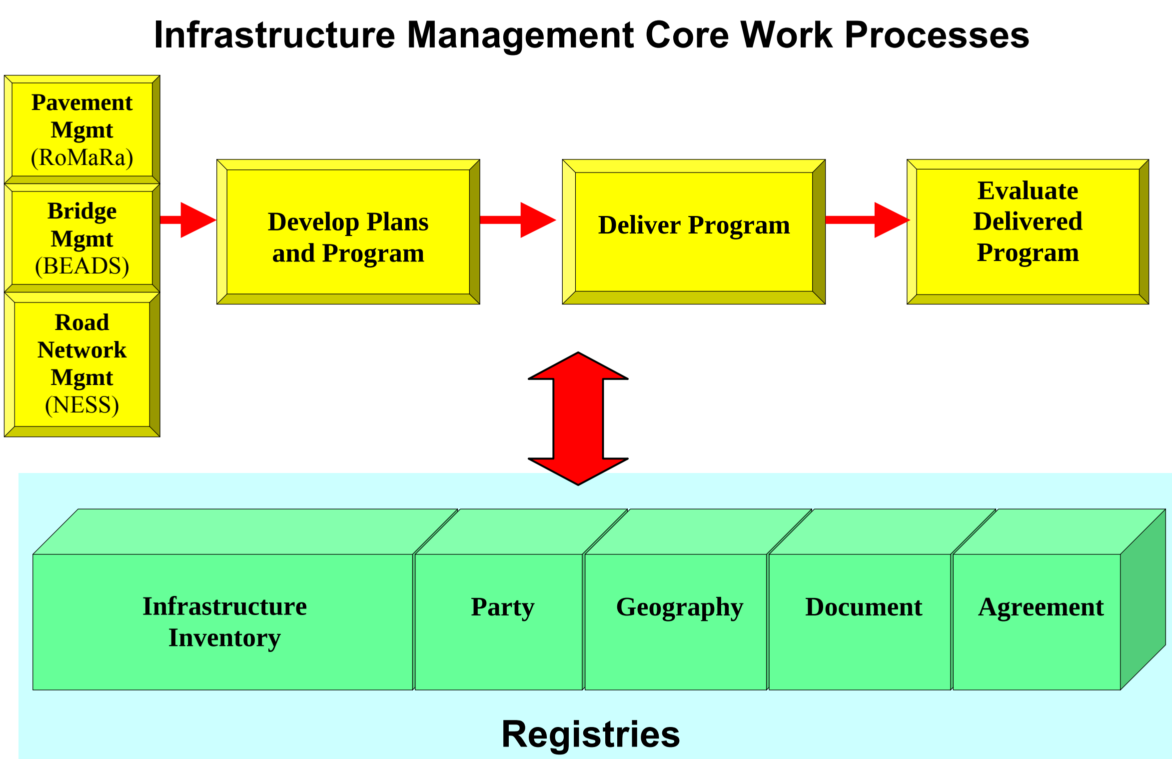

FIGURE 1 TIMS system layout.

proposed bridge structures. The original BIS has been complemented since the early 1970s by the addition of the Culvert Information System (CIS), the Concrete Deck Information System (CDIS), and the Bridge Rating Information System (BRIS), which manage information on bridge-sized culvert structures, concrete bridge decks, and bridge load capacity ratings, respectively. In the mid-1980s, the department initiated development of the Bridge Inspection and Maintenance (BIM) system, which manages inspection of all bridge structures throughout the province. The inspection system was designed to ensure an appropriate level of safety and convenience to the traveling public, identify potential problems at an early stage, and initiate effective maintenance, rehabilitation, or monitoring schemes. With the development of TIMS, the data from these information and inspection systems will be transferred into the central data repository.

Although these systems serve their original purposes, it was recognized that existing information and inspection systems may not be ideally suited to support the analysis function of a bridge management system, particularly regarding the type of information gathered in the inspection system. The BIM system consists of two levels of inspection:

- Level 1. These regular routine inspections are primarily visual inspections that record the worst condition of a bridge element. Quantified inspection data are not gathered in these inspections.
- Level 2. These inspections are carried out at a specified interval or recommended on a one-time site-specific basis. With specialized equipment and expertise, Level 2 inspections gather detailed and quantified information and data on a bridge structure or bridge element and may involve destructive and nondestructive testing and evaluation methods. Level 2 inspections include concrete deck inspection, copper sulphate electrode (CSE) testing, chloride ion content testing, ultrasonic testing of steel trusses, scour survey, steel culvert barrel measurement, timber coring, concrete girder inspection, paint survey, and vertical clearance measurement.

It was expected that the development of any proposed bridge management system or analysis tool would incorporate or utilize the department's existing inventory and inspection systems. Utilizing existing data sources would eliminate or minimize costly changes and revisions to long-established inventory and inspection processes and procedures. In 2000, the department outlined the technical and functional requirements of the proposed BEADS system and, in spring 2001, issued a widely advertised Request for Proposal (RFP) for design and development of the BEADS system. The RFP outlined the system requirements for interfacing with the ongoing design and development of TIMS and the technical and functional engineering requirements of the department's bridge management business. The engineering requirements specified the "what" but not the "how" of the BEADS system. The department was open to a custom-built system or modification of a commercially available bridge management system.

After evaluating the responses to the RFP, the department elected to proceed with in-house design and development of the BEADS system. The rationale for proceeding to a custom-built system instead of modifying a commercially available system was the need to utilize the department's existing bridge inventory and inspection systems and associated expertise, incorporate the department's existing bridge business and management processes, and integrate BEADS within TIMS. These requirements were cornerstones of the RFP but were not satisfactorily met in the submitted proposals. By proceeding with in-house design and development of BEADS, the department's focus changed from determining "what" to determining "how". This included development of system logic, algorithms, formulas, decision trees, and deterioration models.

FEATURES OF BEADS SYSTEM

Supported by Alberta Transportation's existing bridge inventory and inspection systems, the BEADS system provides a project-level (bottom-up) analysis that systematically identifies improvement needs related to condition and functionality with site-specific data. On the basis of existing and predicted condition and functionality states, potential work activities are identified throughout the bridge structure's life cycle, including timing and estimated cost of all actions. Road user costs are included for improvement needs related to functionality but not currently for those related to condition. The work activities are grouped and assembled into feasible life-cycle strategies, which are analyzed and ranked on an economic basis. The strategy with the lowest net present value of costs is the recommended alternative and forms the starting point for further review by department bridge staff.

The BEADS system consists of individual modules that have been developed to address the condition of elements and functional limitations of bridge structures. Condition-related modules include the Superstructure and Paint Modules. Functionality-related modules include the Strength, Bridge Width, Bridge Rail, and Vertical Clearance Modules. The Strategy Builder Module assembles life-cycle strategies from input received from each of these modules. The Substructure and Replacement Modules provide supporting criteria and information to the Strategy Builder Module.

A separate, self-contained module has been developed for bridge-sized culvert structures to identify possible repair actions and determine the culvert structure's year of replacement. Costs of repair and replacement options are also calculated.

The condition-related modules use element condition data, age, and rehabilitation history to identify or trigger improvement needs. The modules include possible work actions for the bridge elements, which are then combined into proposed action plans for the bridge structure's economic life cycle. A cost estimate and timing for each work action are also determined. The output from the condition-related modules is an action plan table that includes

- Year of replacement if no rehabilitation actions are completed, and
- All possible work action plan combinations showing
  - Number of work actions,
  - Life of the action plan,
  - Year, cost, and description of each work action, and
  - Net present value of action plan costs.

The functionality-related modules use inventory and performance data to identify or trigger improvement needs. The modules identify an existing functional need and may also predict the future timing of a functional need. Possible work actions are included in each module to address the functional need. A cost estimate and timing for each work action are also determined. Road user costs are also calculated if possible work actions are not completed. The output from the functionality-related modules includes

- Year functional need will occur,
- Possible work actions to rectify functional need,

- Cost of possible work actions, and
- Annual road user cost if work actions are not completed.

While development of the BEADS system is based on existing inventory and inspection data, some additional inventory data requirements were identified, and it is expected they will be accommodated in future releases of the TIMS corporate data repository. Interim measures have been incorporated in the modules to address information deficiencies. As mentioned earlier, the department's Level 1 inspection process does not currently gather quantified data. Nevertheless, the BEADS system uses this inspection data with certain assumptions and approximations derived from experience and judgment. It is recognized that data limitations will result in impractical recommended strategies for some bridge and culvert structures.

The results from the BEADS system are intended to be a first-cut list of bridge structures with an identified condition or functional need and a recommended strategy. The results may initiate additional inspection, assessment, and expert review by department bridge staff. The first-cut results of the project-level analysis are not intended for short-term programming purposes. Once the project-level analysis is complete, a bridge network analysis may be performed to facilitate short-term programming, analyze long-range budget scenarios, evaluate status of the bridge network, and assess the impact of policy decisions. Figure 2 shows a simple layout of the BEADS system modules. Each module is described below.

SUPERSTRUCTURE MODULE

The rehabilitation of bridge decks is a major expenditure and may have a significant impact on the bridge structure's life-cycle costs. The optimum timing for rehabilitation of a bridge deck must balance the desire to maximize the life of the existing deck and ensure that continuing deterioration does not result in more substantial and expensive actions. The Superstructure Module determines the current and future requirements for the bridge deck, including the protective wearing surface systems and the underlying deck system.

From inspection data and deterioration models, the Superstructure Module estimates the cost and timing for rehabilitation actions. The module is limited to the analysis of cast-in-place, precast and prestressed concrete, and timber bridge decks. The condition of the underlying girder system is reviewed to ensure that the life of the girder system is sufficient to justify any proposed bridge deck actions. Where bridge deck rehabilitation is not recommended because of poor condition or a limited remaining service life of the girder system, the module recommends replacement of the superstructure or bridge structure.

Concrete Bridge Decks

For concrete bridge decks, Level 2 Copper Sulphate Electrode (CSE) testing results and deterioration models are used, if available, to trigger possible action. Some concrete bridge decks cannot be CSE-tested, and assumptions and approximations must be made from deck age and type. Nine possible concrete bridge deck actions are triggered in the module at a particular deck condition state, as follows:

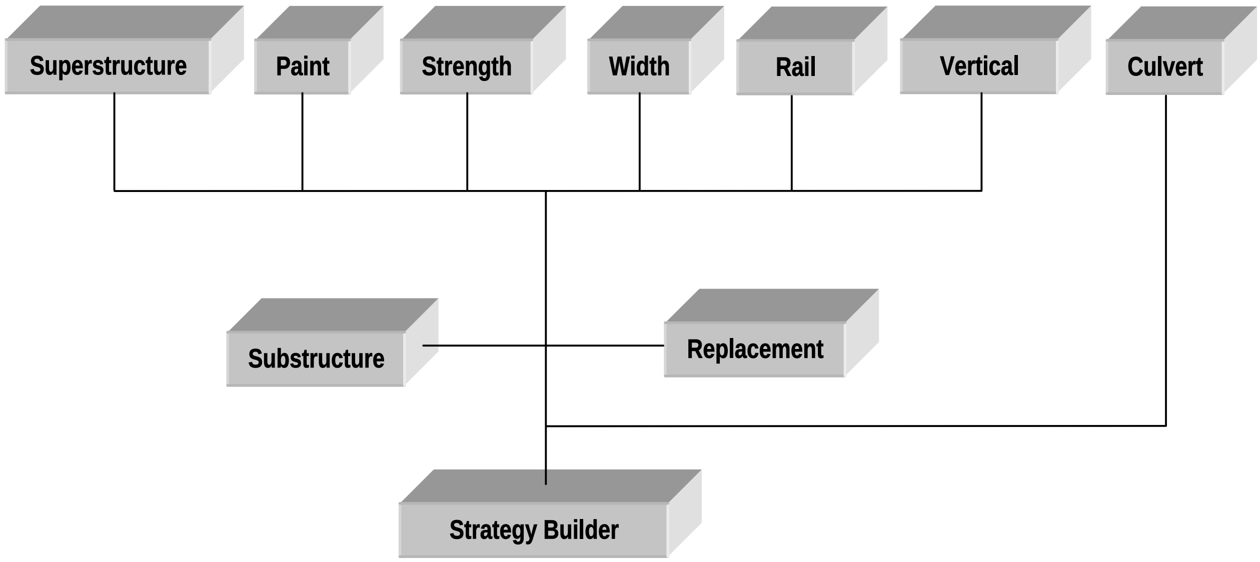

FIGURE 2 BEADS system layout.

- "Do nothing." No bridge deck action is taken on the bridge deck and wearing surface. Instead of bridge deck rehabilitation, it is expected that the bridge deck, bridge structure, or bridge superstructure will be replaced. The module will provide an estimated year for replacement on the basis of the bridge deck's reaching a specified condition state.
- Waterproofing system. The system consists of a hot rubberized membrane beneath two 40- mm lifts of asphalt and is installed over a smooth concrete surface. It is an effective protective system designed to prevent the infiltration of moisture and chlorides. The module assumes that the top lift of the asphalt wearing surface will be periodically replaced during the service life of the waterproofing system.
- Asphalt overlay. This action is taken primarily to improve the rideability of the bridge deck. The asphalt overlay provides little or no protection to the bridge deck against the infiltration of moisture and chlorides. The placement of an asphalt overlay is intended as a possible bridge deck action for an existing waterproofing system.
- Polymer overlay. This is a thin, 6- to 10-mm, epoxy overlay that serves as an effective wearing surface system designed to prevent the infiltration of moisture and chlorides. It is installed over an existing concrete bridge deck, cast to grade, or a concrete overlay.
- Concrete overlay. Typically 65 to 85 mm thick, this overlay consists of a durable and low permeability concrete designed to minimize the infiltration of moisture and chlorides. The concrete overlay may be installed over the deck top of an existing major precast concrete girder or as a replacement wearing surface on a cast-in-place concrete bridge deck.
- Concrete overlay with lateral connection detail. This action is used for bridge structures with channel-shaped prestressed concrete girders connected with grout keys. In this bridge deck action, the concrete overlay is combined with measures to improve the lateral connection between the girder units. These measures may consist of lateral stressing at deck level, lateral stressing through the girder webs, underslung diaphragms, or other connection details.

- Reinforced concrete overlay. Typically 140 mm thick, the overlay consists of a durable and low-permeability concrete designed to minimize the infiltration of moisture and chlorides. This bridge deck action may be installed over single- or multispan precast concrete girders. For multispan bridges, the overlay may include reinforcing steel to provide continuity for live load over the piers. Reinforcing steel is also placed to prevent longitudinal cracking over the lateral connections between the girder units.
- Concrete bridge deck replacement. This action replaces the existing concrete bridge deck and only applies to cast-in-place concrete decks. The bridge decks classified as precast or prestressed concrete usually include the top flange of the girder unit as the bridge deck. The top flange of these precast or prestressed girder units cannot be practically and economically replaced.
- Precast concrete girder replacement. This action replaces the entire girder and deck system instead of rehabiliting the concrete bridge deck and wearing surface. This bridge deck action is only applicable to short-span precast and prestressed concrete girder bridge structures.

Timber Bridge Decks

For timber bridge decks, routine Level 1 inspection data and simple deterioration models are used to estimate the timing of bridge deck actions. In addition to the “do-nothing” action, two possible timber bridge deck actions are triggered in the module at a particular wearing surface and subdeck condition rating:

- Timber stripdeck replacement. This action replaces the longitudinal untreated timber planks that serve as the wearing surface. The planks are typically 75 mm thick and have a relatively short service life that is dependent upon type and volume of traffic.
- Timber subdeck replacement. This action replaces the transverse treated timber planks that transfer the vehicle loads to the girder or stringer units. The thickness of subdeck planks may range from 100 to 150 mm, depending upon the girder or stringer spacing. The timber subdeck planks are treated with creosote to increase their service life.

The Superstructure Module combines the bridge deck actions into possible action plans for the bridge structure’s economic life cycle. An action plan table is forwarded to the Strategy Builder.

PAINT MODULE

The optimum timing for the painting of a steel bridge structure must balance the desire to maximize the life of the existing paint system while ensuring the deterioration of the existing paint system does not result in extensive corrosion and section loss. The Paint Module determines the current and future requirements for the painting of steel bridge structures. The module is limited to analysis of major steel bridge structures. It does not review the painting of short-span steel bridge structures less than 30 m in length or steel substructures, because under current department bridge maintenance practices, this painting is part of annual routine maintenance and rehabilitation programs.

The department has developed a Level 2 paint inspection that gathers detailed and quantified paint condition data. However, these Level 2 paint inspections have not been conducted regularly, and existing Level 2 Paint Inspection data may not be current or available. Therefore, the Paint Module does not currently use Level 2 paint inspection data for the analysis.

It is anticipated that Level 2 paint inspections will become more regular, and data from them will be included in future enhancements and improvements to the BEADS system.

On the basis of routine Level 1 inspection data and simple deterioration models, the Paint Module estimates the timing for paint actions and determines cost estimates. Four possible paint actions are triggered in the module at a particular paint condition rating:

- "Do nothing." No paint action is taken on the existing paint system. It is expected that the entire bridge structure or the bridge superstructure will be replaced, and therefore the existing paint system is not rehabilitated or replaced. The year of replacement is based on the paint system's reaching a specified condition state.
- Top-coating. The superstructure is pressure-washed, and a single layer paint system is applied over the existing paint system. This paint action has a lower cost compared with other paint actions, because it does not require stringent containment provisions to meet environmental requirements.
- Partial paint system replacement. This action replaces the existing paint system over selected areas of the steel superstructure. These selected areas will generally consist of the "splash zones" or areas near the girder ends where the presence of salt-laden water may increase the paint system's rate of deterioration. Full containment of the selected areas is required to capture the spoil from sandblasting operations.
- Paint system replacement. This action consists of replacing the existing paint system over the entire steel superstructure with a new paint system. Full containment of the entire superstructure is required to capture the spoil from sandblasting operations.

The Paint Module combines the paint actions into possible action plans for the bridge structure's economic life cycle. Similar to the Superstructure Module, an action plan table is forwarded to the Strategy Builder.

STRENGTH MODULE

The Strength Module determines if the bridge structure has a functional deficiency resulting from load capacity, if strengthening the existing superstructure is feasible, and if construction costs for strengthening are feasible. It also estimates road user costs associated with bridge strength deficiency.

The functional deficiency is identified by comparing the bridge structure's existing load capacity rating with the current legal limits for the highway type. If a bridge strength deficiency is identified, the module checks the type of superstructure to determine if strengthening is a feasible option. Some superstructure types, such as the older standard precast concrete girders, cannot be economically strengthened, so strengthening is ruled out, and bridge structure replacement is recommended. If this is the case, road user costs associated with the strength deficiency are calculated and included in the life-cycle cost analysis until the year the bridge structure is replaced. Road user costs associated with strength deficiency involve the detouring of overweight vehicles, which increases user time and vehicle operating costs. The module also calculates the initial construction costs associated with strengthening. Results from the Strength Module are forwarded to the Strategy Builder Module.

BRIDGE WIDTH MODULE

The Bridge Width Module builds strategies considering the functional impact of the bridge width. The module determines if and when there is a functional deficiency because of bridge width, if widening the existing superstructure is feasible, and if construction costs for widening are feasible. It also estimates road user costs associated with a bridge width deficiency.

For bridge structures carrying two-way traffic, the functional deficiency is identified by comparing the existing bridge width with the required bridge width for the roadway's current standard, which is based on traffic volumes and roadway classification. As a result, future projections of traffic volumes are required and are calculated by the Bridge Width Module. For bridge structures on twinned-roadways, the TIMS NESS identifies this width deficiency and relays the requirements to the BEADS system. Module screening criteria ensure widening is not recommended for a bridge structure less than 25 years in age. The basis for this screening criteria is recognition that there may have been justification for constructing a bridge structure with a specified width.

If a bridge width deficiency is identified, the module checks the type of superstructure to determine if widening is feasible. Some bridge superstructure types, such as pony and through trusses, cannot be economically widened, so widening is ruled out, and bridge structure replacement is recommended. If this is the case, road user costs associated with the width deficiency are calculated and included in the life-cycle cost analysis until the year the bridge structure is replaced. Road user costs associated with width deficiency involve increased user time and vehicle operating costs as a result of speed reduction and costs of increased risk of collisions. The module also calculates the initial construction costs of widening. Results from the Bridge Width Module are forwarded to the Strategy Builder Module.

BRIDGE RAIL MODULE

The department has installed several standard bridge rail and approach rail systems since the 1950s. Existing bridge rail systems include steel thrie beam, horizontal rail, vertical bar, W-Beam, lattice, double tube, single tube, concrete barrier, and timber rail. The individual bridge rail systems were designed to meet the standard of the day but may not meet today's standards.

The department has no formal bridge rail upgrading program. Instead, bridge rail upgrading is carried out site by site in conjunction with other rehabilitation work or at a site that has a high incidence of collisions or accidents. The Bridge Rail Module identifies a bridge rail need by comparing the existing bridge rail system with the current standard. If a deficiency is identified, the module calculates the initial construction costs to upgrade the bridge rail and the road user costs for not upgrading the bridge rail. Road user costs are based on collision costs, roadway widths, design speed, and average annual daily traffic. Results from the Bridge Rail Module are forwarded to the Strategy Builder Module.

VERTICAL CLEARANCE MODULE

Bridge structures that have a restricted vertical clearance include grade separations, railway overpasses and underpasses, and bridge structure types such as through trusses. In Alberta, the current vertical clearance standards are 5.4 m for roadways and 7.0 m for railways. In special cases, such as local roads, a reduced vertical clearance may be acceptable. The legal height limit for trucks is 4.15 m, and vehicles over the legal height limit but less than 5.0 m require a blanket yearly permit. Vehicles 5.0 m and more in height require a special single-trip permit before traveling on the highway system.

The Vertical Clearance Module identifies bridge structures with a limited functionality because of a vertical clearance restriction. This functionality restriction is determined by comparing the existing vertical clearance with the current vertical clearance standard. If a functionality restriction exists, the module does not recommend any work actions, nor does it calculate road user costs associated with the vertical clearance restriction. The provincial highway system includes a number of high load corridors, and travel of overheight vehicles is tightly regulated and monitored. Overheight vehicles may be accommodated at grade separations using existing interchange ramps, with minimal inconvenience to the traveling public. At present, the Vertical Clearance Module determines if a bridge structure has a functionality restriction because of vertical clearance. This determination is forwarded to the Strategy Builder Module as a flag and does not affect the development of life-cycle strategies.

SUBSTRUCTURE MODULE

Modules within the BEADS system may recommend actions or work activities on the bridge superstructure that require significant expenditures. However, the remaining service life of a bridge structure's substructure elements may govern its overall remaining life. Before development of long-term strategies for the bridge structure by the Strategy Builder Module, it is necessary to estimate the substructure's remaining service life. To ensure the economic benefit of proposed expenditures, this service life should be equal to or greater than the service life of the proposed work activities.

Many substructure elements are not easily inspected or even visible, as in the case of pile foundations and spread footings. Pier elements are located in and over watercourses where access for routine inspections is limited. As a result, it is difficult to estimate the remaining service life of substructure elements with observed condition data and deterioration models. The Substructure Module has adopted a simplified approach that applies a screening factor to determine whether work actions should be considered for the overlying superstructure system.

On the basis of experience and judgment, the department has assumed an expected service life for substructure components constructed from timber, steel, and concrete. The module assumes that no strengthening or rehabilitation actions are taken to extend the estimated remaining service life of the substructure elements. Using the age of the substructure elements and the expected service life, the Substructure Module provides either of the following two recommendations to the Strategy Builder Module:

- Substructure life adequate. The substructure's age is <80% of its expected service life. The economic benefit of major expenditures on the bridge superstructure is expected to be realized during the bridge structure's remaining service life.
- Substructure life not adequate. The substructure's age is ≥80% of its expected service life. The economic benefit of major expenditures on the bridge superstructure is not expected to be realized during the bridge's structure's remaining service life.

REPLACEMENT MODULE

The Replacement Module estimates a bridge structure's expected service life and calculates replacement cost. The expected service life value is used by the Strategy Builder Module to make decisions that may affect the timing of replacement. However, this value does not limit the Strategy Builder Module's ability to develop strategies that exceed the expected service life if condition and rehabilitation actions warrant it. A bridge structure's expected service life is based on superstructure types and road classifications. Values have been developed on the basis of engineering judgement and experience with the department's bridge inventory. The actual condition of bridge structures is not used in determining expected service life.

The replacement cost is calculated as a unit cost, assuming similar dimensions to the existing bridge structure. Although many bridge structures may be replaced with culvert structures, it is very difficult to size the replacement culvert structure without some bridge planning and hydrotechnical analysis. Many existing standard bridges were sized by available girder units and may be either oversized or undersized relative to current engineering practice. For simplicity, it is assumed that bridge structures will be replaced with structures of similar dimensions.

A bridge structure's expected service life is added to its year of construction to determine an estimated reference value of the replacement year. This value is passed to the Strategy Builder Module along with the replacement cost.

CULVERT MODULE

Culvert structures are treated separately from bridge structures because of differences in use, inventory data, inspection data collected, and possible repair options. As a result, a self-contained Culvert Module has been developed that identifies possible repair actions and determines the year of culvert structure replacement. Costs of possible repair actions and replacement are also calculated.

Most culvert structures are isolated from the road by earth fill, and the components that deteriorate are not directly affected by factors such as traffic volumes, loading, and salt application. Deterioration is affected more by environmental factors, geotechnical problems, and corrosive potential and abrasion forces from the stream. The height of fill and soil pressure usually govern design of culvert structures, and therefore, culverts are not rated for load capacity. Small deficiencies in roadway width can be rectified with relatively inexpensive actions, such as installing retaining walls or extending the culvert barrel.

The most important functional aspect of culvert structures is their ability to handle flood conditions, which cannot be adequately predicted without site-specific engineering analysis. The waterway adequacy rating from routine Level 1 inspections is used in the Culvert Module, but the weighting of or reliance upon this value is insignificant because of the difficulty in assigning this rating under nonflood conditions. As a result, the service life provided by culvert structures is determined mostly by the expected life for culvert structure types, considering possible repair actions.

The possible culvert structure repair actions consist of

- Strutting. A line or lines of treated timber struts are installed along the length of the culvert barrel. This type of repair restrains deflection of roof plates or limits the propagation of cracks in longitudinal seams.
- Culvert lining. A new metal pipe is installed inside the existing culvert barrel, and the annular space between the two is grouted. If hydraulically adequate, installation of a liner may significantly extend the service life of the original culvert structure.
- Seam repair. Repairs are performed on circumferential seams that have separated and allow differential movement of adjacent culvert rings. The procedure involves installing plate material inside the barrel section of the culvert structure over the existing circumferential seam.

- Shotcrete beams. Shotcrete beams are installed to repair cracked longitudinal seams. The procedure involves attaching reinforcing steel to the walls of the existing culvert structure and applying a shotcrete material.
- Concrete floors. Concrete floors are installed to protect or rehabilitate a metal culvert structure that has deformed or corroded floor plates.

If recommending no repair actions, the Culvert Module forwards the "must replace year" to the Strategy Builder Module to facilitate development of the base strategies. In addition, if a feasible repair action is identified by decision trees within the Culvert Module (based on Level 1 inspection data), this repair action, along with its cost and associated extension in service life, is passed to the Strategy Builder Module.

STRATEGY BUILDER MODULE

The Strategy Builder Module assembles the actions, costs, and flags from the various BEADS system analysis modules and builds multiple feasible life-cycle strategies for the bridge structure. The module takes advantage of the computing power of present-day computer systems and builds and compares a large number of strategies. All strategies are compared on the basis of the economic life cycle, which eliminates the need to develop complex decision trees or logic functions within the module.

The Strategy Builder Module always considers two base strategies:

- Strategy 1. Do no rehabilitation and replace the bridge structure at the end of its service life, which is the "must replace year" from the Paint, Superstructure, or Culvert Module. All road user costs stemming from functional deficiencies are included in the life-cycle cost until the replacement year. It is assumed that the replacement bridge structure will rectify all functional deficiencies.
- Strategy 2. Do no rehabilitation and close the bridge structure at the end of its service life, which is the "must replace year" from the Paint, Superstructure, or Culvert Module. All road user costs stemming from functional deficiencies are included in the life-cycle cost until the year of bridge structure closure. From that year, all road user costs stemming from the bridge closure are included in the life-cycle cost.

Culvert Structures

In addition to the two base strategies, a third strategy may be developed for culvert structures on the basis of feasible repair action identified by the Culvert Module. This third strategy consists of one of the possible repair actions to extend the culvert structure's service life, followed by replacement.

Bridge Structures

In addition to the two base strategies, up to 13 strategies are developed for bridge structures with an assumed replacement year from Year 5 to Year 65 in 5-year increments (Table 1). The number of strategies may be capped by a service life limitation or "must replace year" produced by the Superstructure, Paint, or Substructure Modules. If there are no service life limitations in the life-cycle period, all 13 strategies are developed and analyzed. Each strategy is populated with the least-cost action plan from the Superstructure and Paint Modules that meets or exceeds the assumed year of bridge structure replacement for the strategy. These strategies do not contain any functional improvement actions, so road user costs associated with strength, width, and bridge rail deficiencies are included, if applicable.

If options to strengthen, widen, or upgrade the bridge rail are available, additional strategies are developed using the 13 rehabilitation strategies as base cases. These strategies include functional improvement actions and road user costs as necessary. For each base case, the functional improvement action is added in 5-year increments between the first year the deficiency appears and the year of bridge replacement. This is done for each category of functional improvement separately and in combination. In cases in which several functional improvement actions are possible, many additional strategies are developed and analyzed.

For example, Strategy 7 calls for bridge replacement in Year 25 and includes a width deficiency that appears in Year 15. In addition to the costs of rehabilitation actions proposed by the Superstructure and Paint Modules, the life-cycle costs include road user costs associated with the width deficiency applied from Year 15 to replacement year, Year 25.

Strategy 7-A modifies Strategy 7 and still assumes bridge structure replacement in Year 25. This strategy calls for bridge widening in Year 15. The life-cycle cost includes widening in Year 15, all other work actions up to Year 25 (year of replacement), and no road user costs associated with width deficiency.

Strategy 7-B further modifies Strategy 7 and still assumes bridge structure replacement in Year 25. This strategy includes bridge widening in Year 20. The life-cycle cost includes widening in Year 20, all other work actions up to Year 25 (year of replacement), and road user costs associated with width deficiency from Year 15 to Year 20.

Figure 3 shows the system layout of the Strategy Builder Module.

Evaluation of Strategies

A net present value analysis of all costs for each strategy is then performed. The analysis is based on the specified discount rate (currently set at 4%) and the life-cycle period (currently set at 50 years).

TABLE 1 Strategies with Assumed Replacement Year

Strategy No.	Strategy
3.	Replace in 5 years
4.	Replace in 10 years
5.	Replace in 15 years
6.	Replace in 20 years
7.	Replace in 25 years
8.	Replace in 30 years
9.	Replace in 35 years
10.	Replace in 40 years
11.	Replace in 45 years
12.	Replace in 50 years
13.	Replace in 55 years
14.	Replace in 60 years
15.	Replace in 65 years

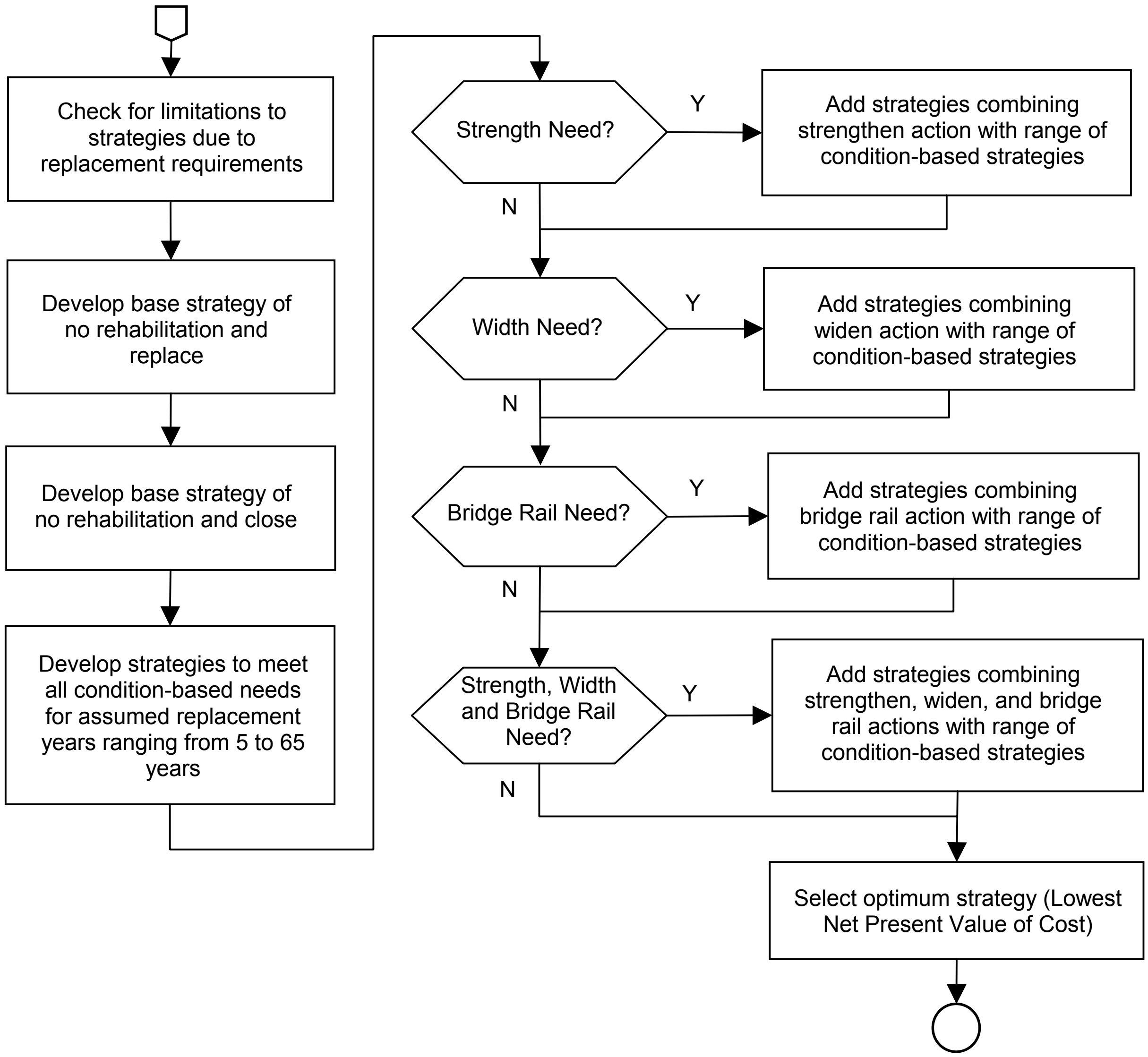

FIGURE 3 Strategy builder module layout.

For all strategies with a replacement action identified within the life-cycle period, a residual value is calculated and included at the end of the life-cycle period as a negative cost. The residual value is calculated as the replacement cost times the ratio of the remaining service life to the total service life for a new bridge structure. A residual value is also included for strategies with no replacement action within the life-cycle period if there is still service life remaining in the existing bridge structure as determined in the Replacement Module.

The strategy with the lowest life-cycle present value of costs is then identified as the optimum strategy for that bridge structure.

BEADS APPLICATION

Results from the Strategy Builder Module may initiate additional inspection, assessment, and expert review of bridge and culvert structures. The optimum strategy from the modules may be manually revised or overridden following expert review or additional assessment. The optimum strategy and a base reference strategy for each bridge and culvert structure are forwarded to TIMS, where bridge priorities are ranked against other requirements for highway network funding. Project-level analysis results may also be used to facilitate short-term programming, analyze long-range budget scenarios, evaluate status of the bridge network, and assess the impact of policy decisions.

BEADS STATUS

Alberta Transportation has developed a working prototype of the BEADS system using Visual Basic for Applications within Microsoft Excel. Testing, calibration, and documentation of the BEADS system continue. The department also continues with the design and development of TIMS with plans to incorporate BEADS in TIMS in the near future.

Addressing Uncertainties in Bridge Management with Influence Diagrams

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A bridge management system is designed to provide information and useful data for analysis so that bridge engineers can make more consistent, cost-effective, and defensible decisions for the maintenance, rehabilitation, and preservation of bridges. In a bridge management system, problem setting often involves many uncertain, interrelated quantities, attributes, and alternatives derived from information of highly varying quality. Recently, there has been considerable interest in addressing uncertainties in bridge management. This paper proposes the use of a type of Bayesian influence diagram known as a directed acyclic graph (DAG). DAGs express outcomes in terms of combinations of primitive events. The graphical structure of these models also captures the dependency structure among events, which enables the decision maker to exploit conditional independence to reduce specification and computation. The influence diagrams give users a clear view of the variables within the framework of a bridge management system and the relationships among them. Influence diagrams provide an appealing knowledge representation that can express uncertain knowledge, beliefs, and preferences in both qualitative and quantitative forms. With influence diagrams, the question can be addressed of perfect and imperfect information in bridge management system decision making.

Bridge management systems (BMSs) are a relatively new approach developed after the successful implementation and application of pavement management systems. The federal mandates for bridge management are outlined in the AASHTO *Guidelines for Bridge Management* (*1*). These guidelines suggest that a BMS should include four basic components: a database (data storage), cost and deterioration models, optimization models for analysis, and updating functions. Thus, a BMS is a decision support system for the bridge management process to help engineers make systematic decisions about a bridge. Bridge management includes all activities related to planning for design, construction, maintenance, and rehabilitation. It consists of the following essential elements:

- Data collection on bridge inventory and condition through survey;
- Information management system with a database and data storage;
- Analysis scheme for determining bridge condition and predicting performance;
- Decision criteria for ranking bridge projects for maintenance, rehabilitation, and repair (MR&R); and
- Strategies for implementing bridge MR&R decisions.

These activities collectively form the structured BMS methodology that can aid bridge personnel by generating optimal solutions for managing the bridge network. Just as pavement management systems, BMSs are generally divided into network and project levels. The network level consists of the nontechnical aspects of a BMS, including budgeting, standard setting, and preservation. The project level, which addresses the engineering and technical aspects of bridge management, consists of inspection, action recording, and project planning. As in pavement management systems (*2*), a third level can be proposed: the project selection level, which ranks candidate projects within the constraints of the available budget. A good BMS should be able to present the engineering and economic trade-offs among alternatives in life-cycle costing.

Decision making in a BMS involves uncertainties, subjective judgments, and risk, which leads to uncertainty about the selection of a rehabilitation action appropriate for a given bridge at some future time. To improve the quality of maintenance and rehabilitation decision making, analytical tools are needed to capture uncertainties, subjective judgment, and risk as experienced in BMS decision making.

Efforts have recently been made to include some aspects of uncertainty analysis and incomplete information in the BMS. Sanford (*3*) proposed the use of the Bayesian influence diagram as a tool to address uncertainty in data collection in BMS decision making. Adams and Sianipar (*4*) presented a tool for analyzing the sensitivity of recommended MR&R action and optimal policies to uncertainty in MR&R costs, transition probabilities, and discount factor. Their approach is also based on probabilistic analysis. Sobanjo (*5*) discussed the uncertainty in estimating costs with reference management and presented types of uncertainties involved in life-cycle cost analysis. The author divided uncertainties into two forms—statistical data randomness and subjectivity—and proposed three methods of handling these uncertainties—the fuzzy sets approach, probability theory, and the hybrid fuzzy–probability approach.

Thomas and Merlo (*6*) proposed a comprehensive BMS update for Ontario, Canada. The BMS objective was general but with an additional requirement that the new BMS fit well within the evolving framework for future asset management requirements. Another unique feature of this update is knowledge-based models in the form of decision trees for selecting rehabilitation methods. The deterioration models are based on the Markovian process.

Bien (*7*) developed an expert function approach based on a neural network paradigm and analytic functions. This approach appears to accurately incorporate the dynamic nature of bridge data collection and rehabilitation decisions. The approach consists of a group of expert functions, such as data compatibility expert function, bridge evaluation expert function, and load capacity expert function. The author proposed an improvement to the expert function approach using fuzzy logic.

This brief overview of uncertainty and risk analysis indicates that the predominant approach in a BMS is conventional probabilities. However, the probability approaches have several shortcomings for infrastructure management (*8*).

The causes of uncertainty in a BMS include

- Lack of information, usually in the form of quantitative information;
- Abundance of information, which overwhelms the limited ability of human beings to simultaneously perceive large amounts of data;
- Conflicting evidence;
- Ambiguity, when certain linguistic information has an entirely different meaning, or

mathematically speaking, when there are no one-to-many mappings; and

- Measurements whose output is accompanied by some uncertainty, such as ground-penetrating radar in bridge deck assessment.

The present state of the art of bridge management calls for tools that can effectively model the decision-making process. The tool should be capable of addressing aspects of the decision-making process, including information from different sources, uncertainty, subjective judgment, and risk.

A framework with the potential to overcome most shortcomings of the existing approaches and with qualitative features is the Bayesian influence diagram. Attoh-Okine presented a general framework and potential use of Bayesian influence diagrams in pavement management system decision making (*9*) and for application in maintenance management decision making (*10*). A comprehensive application of Bayesian influence diagrams was presented by Attoh-Okine and Roddis (*11*) in the area of flexible pavement design applications. The aim of this paper is to apply Bayesian influence diagrams to address uncertainties in bridge management.

INFLUENCE DIAGRAM

Influence diagrams (*12*) provide a graph–theoretic framework for modeling the probabilistic dependence of information among uncertain variables, decision options, and utility functions in complex decision systems. Influence diagrams complement more traditional representations such as decision trees and tabular listings of joint probability distribution and outcomes for each action. An influence diagram is often not as complex as a decision tree, whose branches can easily reach into the hundreds. An influence diagram is more easily constructed and expanded (*13*). Influence diagrams explicitly represent probability dependence and independence in a manner accessible to decision makers and computers. The representation eases the assessment of coherent prior distribution.

An influence diagram is an acyclic graph whose nodes are connected with directed arcs. The four kinds of nodes (see Figure 1) have different kinds of information stored in them, as follows:

- Probabilistic—an uncertain variable (also chance) in a model, drawn as a circle;
- Deterministic—a variable whose value is determined given the values of its condition variables, drawn as double-rounded circles;
- Decision—a variable whose value is under the control of the decision maker, drawn as a rectangle; and
- Expected value—the objective function (or payoff), drawn as double rectangles.

A directed arc into a decision node or expected value node is called an information arrow. A directed arc into a probability or deterministic node is called a relevance arrow. Further interpretations of node and arrow configurations are listed in Figure 2. Arcs leading into decision nodes represent information available at the time of a decision. Arcs leading into probabilistic

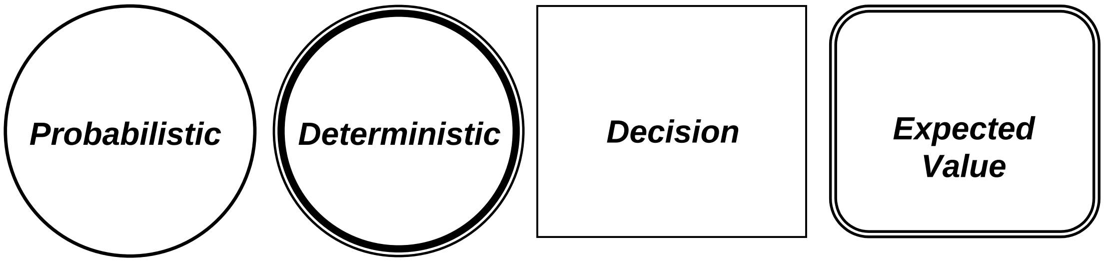

FIGURE 1 Nodes.

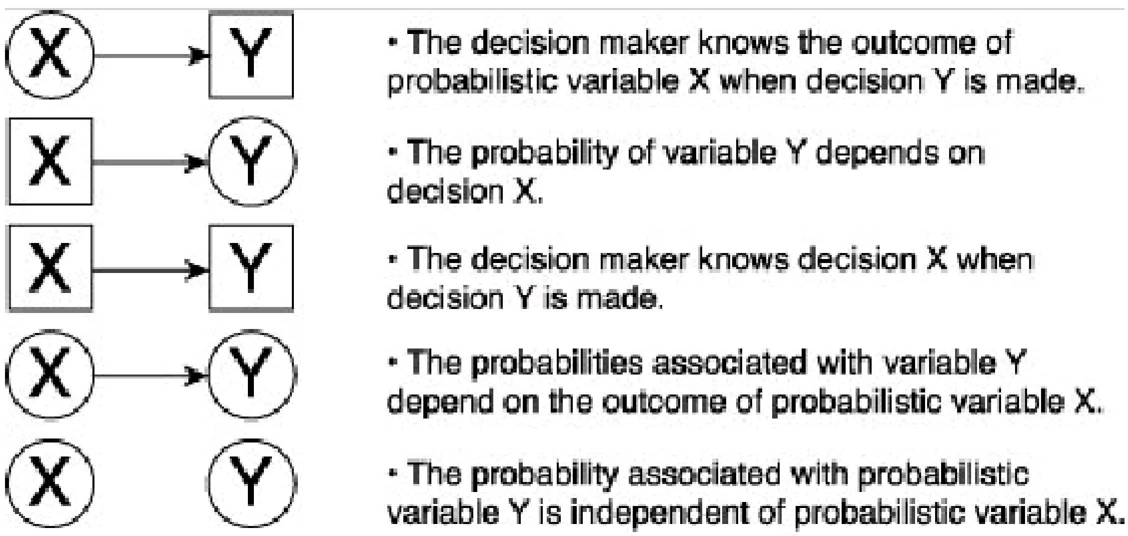

FIGURE 2 Interpretation of arrows.

nodes indicate that the likelihood of the chance event depends (or is conditional) on the outcome at the decision or probability node from which the arc originated. "No forgetting" arcs are placed between decision nodes to signify that decisions are sequential in time and the value of a past decision is remembered. In general, arrows in influence diagrams obey the following rules:

Rule 1. There are no directed cycles in the graph.
Rule 2. The decision nodes are linearly ordered by information arrows.
Rule 3. Each decision nodes inherits any information arrows from preceding nodes.
Rule 4. There are no arrows from expected value nodes.

The mathematical interpretation between state nodes can be deterministic or probabilistic. Assignment of probability distributions presents a more compact description of the problem than a decision tree. Influence diagrams also represent the relationship between two variables equally, whether those variables are discrete, continuous, or a mixture of both. Influence diagrams have an additional advantage over decision trees because they are able to succinctly represent conditional independence between variables in a problem.

Consider two variables *A* and *B*. In the example represented in Figure 3*a*, the discrete probability assignment is a vector $\{A_i / C]$; brackets enclose the probability assessment. *A* represents the variable to which the probability assessment will be assigned, and *C* represents the state of information. The relevance is represented in the matrix $\{B_j / A_i\ C\}$. The product of the unconditional vector and the conditional matrix results in the unconditional probability vector that is assigned to $\{B_j / C\}$:

$$\{A_i / C\} * \{B_j / A_i, C\} = \{\mathrm{B}_j / C\}$$

In Figure 3*b*, *B*, conditioned on the state of information, *C*, can be determined as

$$\{B_j / C\} * \{A_j / B_j, C\} = \{A_i / C\}$$

With respect to a node, the following definitions are of importance:

- A path from one node to another is a set of arrows connected head to tail that forms a directed line from one node to another.
- The predecessor set of a node is the set of all nodes with a path leading to a given node.
- The direct path predecessor set of a node is the set of nodes with an arrow connected directly to the given node.

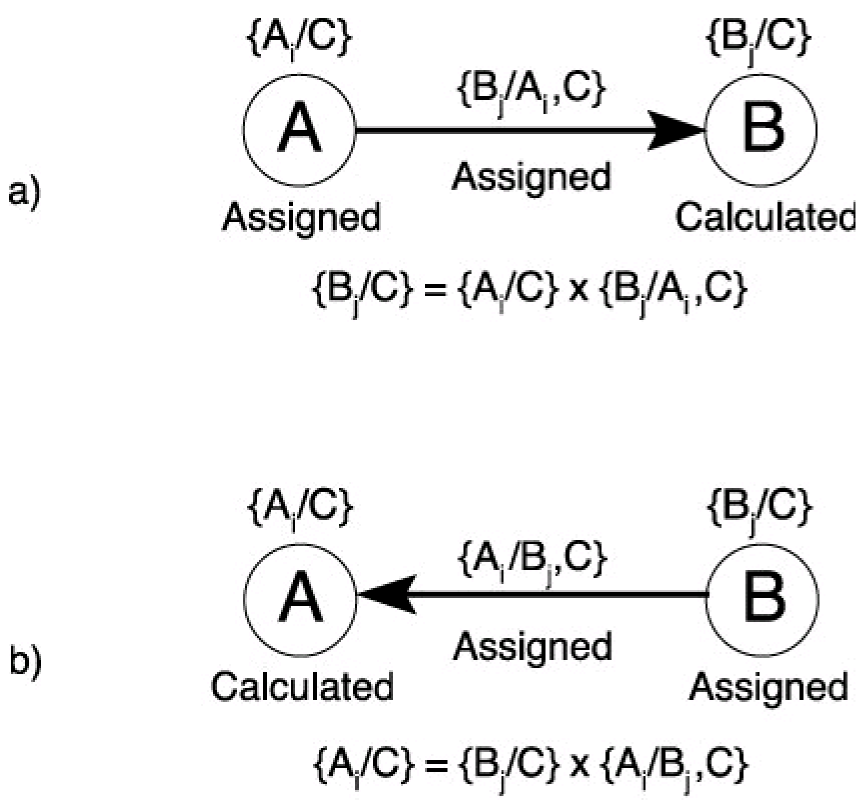

FIGURE 3 Assigned probabilities.

- The indirect predecessor set of a node is the set formed by removing its direct predecessor set from its predecessor set.
- The successor set of a node is the set of nodes with a path leading from the given node.
- The direct successor of a node is the set of nodes with an arrow connected directly from the given node.
- The indirect successor set of a node is the set formed by removing all elements of its direct successor set from its successor.

The nodes of the influence diagram can be numbered, say $X_1, X_2, \ldots, X_m$, so $i < j$ whenever there is an arrow (information arrow or relevance arrow) from X_i on X_j. (This means that $i < j$ whenever X_i is a predecessor of X_j). Such numbering is called influence numbering. Most influence diagrams have more than one numbering. The existence of influence numbering makes it clear to a probabilist why choosing a decision node determines a joint probability distribution for all variables in an influence diagram.

These decision rules, together with conditional probabilities for the other nodes (probabilistic and deterministic), amount to a specification of a conditional probability for each variable given a subset of preceding variables in numbering. Multiplying these conditional probabilities produces a joint probability distribution.

Figures 4 and 5 show a simple influence diagram with the corresponding decision tree. For simple problems, a decision tree is appealing, but for large and complex problems, representation in a decision tree can become more challenging.

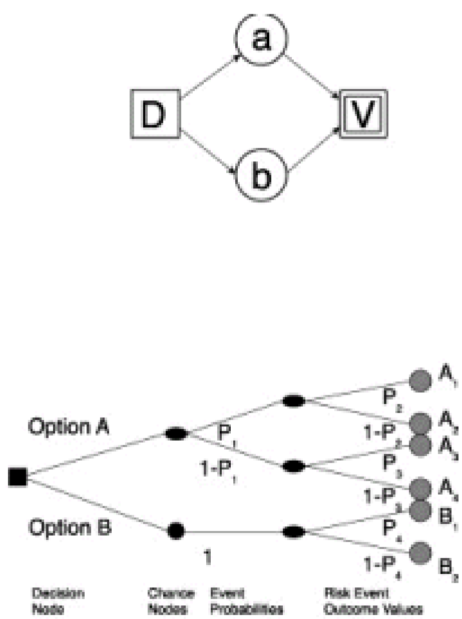

FIGURE 4 Simple influence diagram (above) and associated decision tree (below).

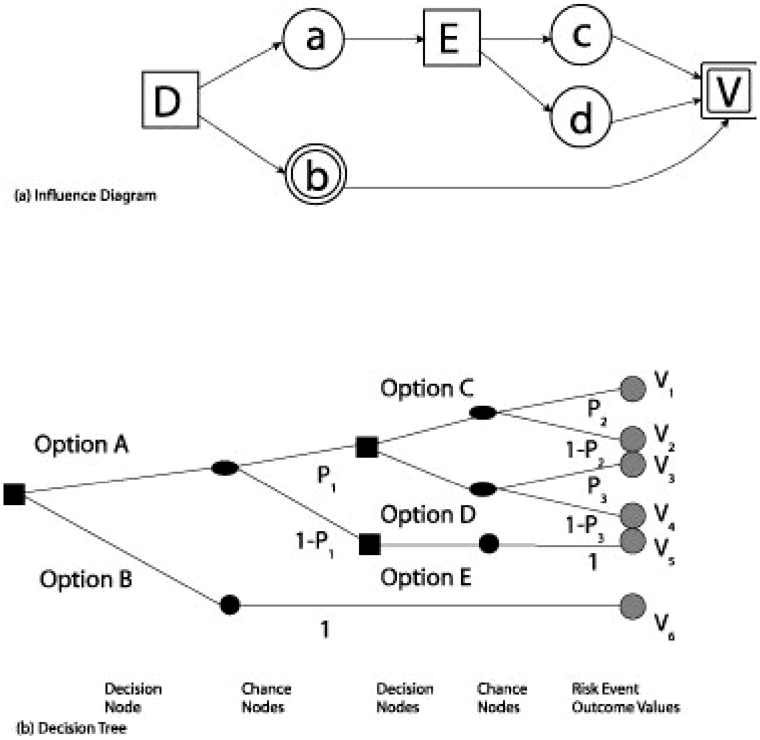

FIGURE 5 Three-decision influence diagram (above) and associated decision tree (below).

BRIDGE MANAGEMENT SYSTEM MODEL

A BMS influence diagram can be utilized at four levels in a BMS setting, as shown in Figure 6:

Level I—communication level, which helps identify variables that affect the probability of a successful BMS;
Level II—probability level, which is assignment of probabilities;
Level III—objective function in terms of utility;
Level IV—computation and determination of optimum policy and factors that drive the model.

Figure 6 provides intuitive steps for the construction of a generic BMS model.

Figure 7 is a prototype of the Bayesian influence diagram for BMS decision making. For the flow of information, an influence diagram model was formulated of ten nodes connected by directed arcs. There are six probabilities nodes, one deterministic node, two decision nodes, and one payoff node.

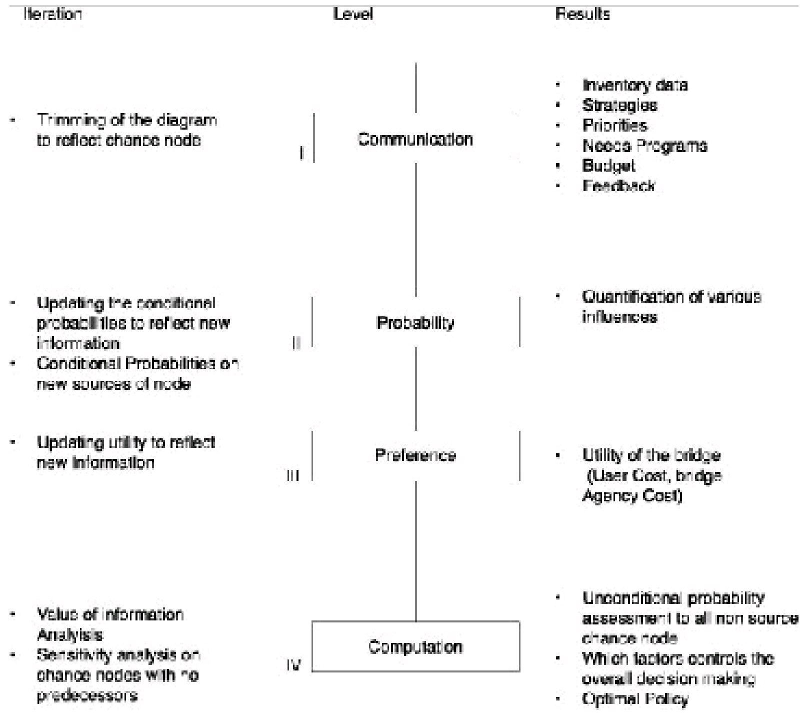

FIGURE 6 Levels in influence diagrams of a bridge and management system corresponding iteration and results.

FIGURE 7 Prototype example of Bayesian Influence Diagram for BMS decision making.

Decisions

Bridge engineers and managers might face a large variety of decisions in responding to bridge maintenance and rehabilitation decisions that generally fall into two categories:

- Data collection strategy. The bridge engineer must decide when, how, and what type of data should be collected depending on equipment availability and budget level. In this paper, the data collected is assumed to be related to roughness and cracks. The data collection strategies vary by agency.
- Maintenance strategies. Three alternatives considered are routine maintenance, overlay, and reconstruction.

Outcome and Uncertainties

The various outcomes and certainties include

- Deterioration model—multivariate equation based on previous data records, which can be assigned probabilities according to the prediction accuracy from previous analysis and experience (In this paper, two types of outcomes were assigned: high and low.);
- Prior deterioration node—probabilistic node based on current condition data falling into two classes with equal probability of occurrence;
- Posterior probability—probabilistic node based on current data conditions and improvement caused by the new maintenance strategies, used to determine the level of the bridge after maintenance;
- Cost of strategies—monetary value of the implemented strategies;
- Cost of data collection—dependent on type of equipment used and frequency of data collection;
- Implementation cost—cost of feedback information in the data collection procedure and new innovation in the data collection process; and
- Future potential cost—cost associated with future data collection procedures.

Expected Value

A payoff node in terms of the user is used to identify the optimal policy. The payoff data was partly based on the author’s experience and field data.

The influence diagram relies on specific types to represent the sequence of events. For example, the arc from the deterioration model to the maintenance strategies decision node indicates that the bridge management engineer knows the outcome of the deterioration results before making decisions on developmental strategies. The node labeled Payoff is a value node, which contains the information that is usually shown at the ends of the branches of a decision tree. The cost-of-strategies node and the posterior deterioration model are events that affect the utility directly.

The following general principles are useful when building an influence diagram (*14*):

- Start at the value node and work back to the decision nodes. The value node reflects the outcome of interest to the bridge engineer. Furthermore, the parents of the value node reflect the attribute of the utility function to be used in the analysis.

- Draw the arcs in the direction that makes the probabilities easiest to assess.
- Use informational arcs to specify which events have been observed at the time each decision is made.
- Ensure that missing arcs reflect intentional assertions about conditional independence and timing of observations.
- Ensure the influence diagram has no cycles.

INFERENCES IN INFLUENCE DIAGRAMS

Relatively few methods exist for making inferences in influence diagrams. The most common method is the arc reversal and node reduction method developed by Schachter (*15*). The operation of the method is analogous to the use of Bayes' theorem. Three basic components of the method are (1) arc reversal (Bayes rule), (2) barren node removal (marginalization), and (3) merging with value node (expectation and maximization). In the analysis, chance nodes are removed by averaging (or by integration if the data is continuous). At the decision node, the bridge engineer selects the alternative with the highest expected utility and then removes that node. This process is referred to as decision node removal by policy determination. Another promising method is the potential influence diagram approach developed by Ndilikilikesha (*16*), which finds the solution differently. First, each chance node in the diagram is associated with an arbitrary nonnegative function, called a potential, instead of a conditional probability table. This generalization allows the bridge engineer or the analyst to remove chance nodes without reversing arcs.

Generally, an influence diagram has more than one solution. Assignment of a decision rule to each decision node is called the solution of the influence diagram if it results in a joint distribution that maximizes the expected value. In this paper the Shachter approach (*15*) will be used. The steps for solving are as follows (*17*):

1. Eliminate all nodes (except the value node) that do not point to another node (barren nodes).
2. As long as there are one or more nodes that point into the value node, do the following:
    a. If there is a decision node that points into the value node, and if all other nodes that point into the value node also point into that decision node, remove the decision node by policy determination. Remove any nodes (other than the value) that no longer point to any other node. Go back to Step 2.
    b. If there is a chance node that points into only the value node, remove it by averaging. Go back to Step 2.
    c. Find a chance node that points into the value node and not into any decision. Reverse all arcs that point from that chance node into other chance nodes without creating a cycle.

The influence diagram can be used to perform sensitivity analysis. The decision analytic concept of expected value of perfect information is defined as the difference between expected values of a decision problem with and without the perfect information. If the expected value of information is greater than the cost of obtaining the information, the procedure shows a positive effect. Otherwise, the information is not worth obtaining.

Sensitivity analysis helps in discovering which variables drive the model, and it answers the question "what matters in this decision?" This concept is important, especially for uncertain variables, because to be performed properly, probabilistic assessment requires a significant expenditure of resources.

COMPUTATIONAL EXAMPLES

Figure 7 is an example of applying an influence diagram to bridge management decision making. To illustrate the analysis process, Figure 8 is a simple three-node influence diagram of a network-level bridge management decision-making problem. *C1* is a chance node, an outcome of engineering-economic analyses of the network. The three possible outcomes are

1. The budget limit is too high.
2. The budget limit is about right.
3. The budget limit is very low.

D1 is a decision node, which is based on what should be the general maintenance and rehabilitation of the network. There are three decisions:

1. Do nothing.
2. Minor overlay.
3. Major rehabilitation.

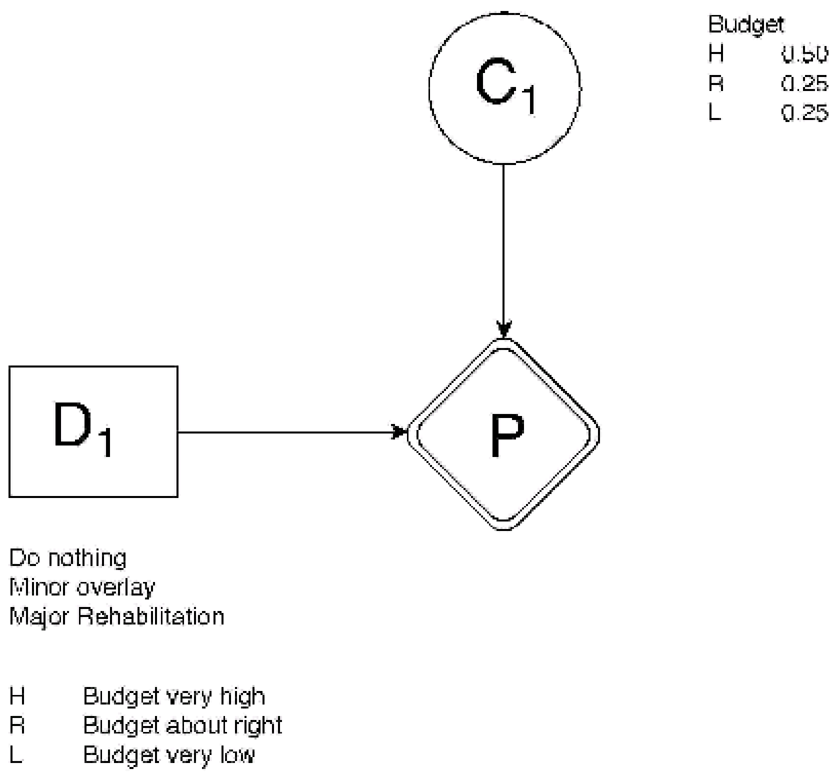

FIGURE 8 Network-level bridge management decision.

The corresponding decision tree to Figure 8 is shown in Figure 9. Figure 10 shows the chance-node removal and decision node by policy determination. For example, the expected utility for do-nothing is

$$EU\ (\text{Do Nothing}) = P(H)U(20) + P(R)U(35) + P(L)U(30)$$

for minor overlay

$$EU\ (\text{Minor Overlay}) = P(H)U(35) + P(R)U(25) + P(L)U(20)$$

for major rehabilitation

$$EU\ (\text{Major Rehabilitation}) = P(H)U(25) + P(R)U(15) + P(L)U(10)$$

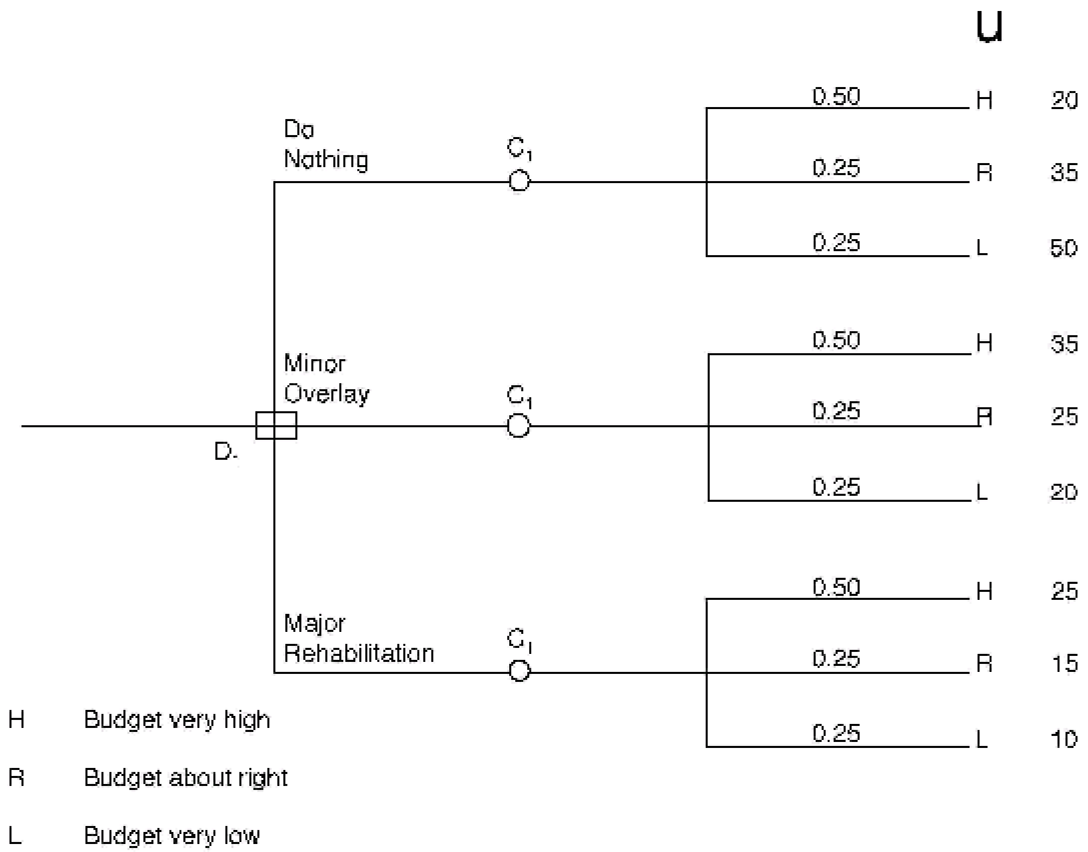

FIGURE 9 Decision tree.

The maximum of all the expected utilities represents the payoff (see Figure 10). The example illustrates the solution procedure. The utilities are provided by engineers on the basis of past records and experience.

The probability associated with an outcome expresses the decision maker's (bridge engineer's) belief in the likelihood of that outcome relative to other possible outcomes for uncertain events. Subjective probabilities are used in application of influence diagrams. This is a probability that expresses a person's degree of belief in the proposition on the basis of the person's current information (*18*). Subjective probabilities are used to encode expert knowledge in domains in which few or no direct empirical data are available. A subjectivist might start with a prior belief about the fairness of the event, perhaps based on experience with the events, and then update the belief using the Bayes theorem as information becomes available from field data and experience. The probabilities presented in the paper are viewed as a reflection of the author's belief about the relative likelihood of the possible outcomes. Table 1 shows the outcomes and alternatives for nodes. Conditional probability tables can then be developed on the basis of the possible outcomes and informational arcs. The possible values have any number of states. In this example, only a few states have been shown, for demonstration purposes only. For conditional probabilities various matrices and vectors can be obtained, as illustrated in Figure 3*a* and 3*b*.

Once all conditional probabilities and payoff values have been included, commercial software can be used for policy determination, as in Figure 10.

Although no formal analysis of Figure 7 has been presented, Figure 10 illustrates the

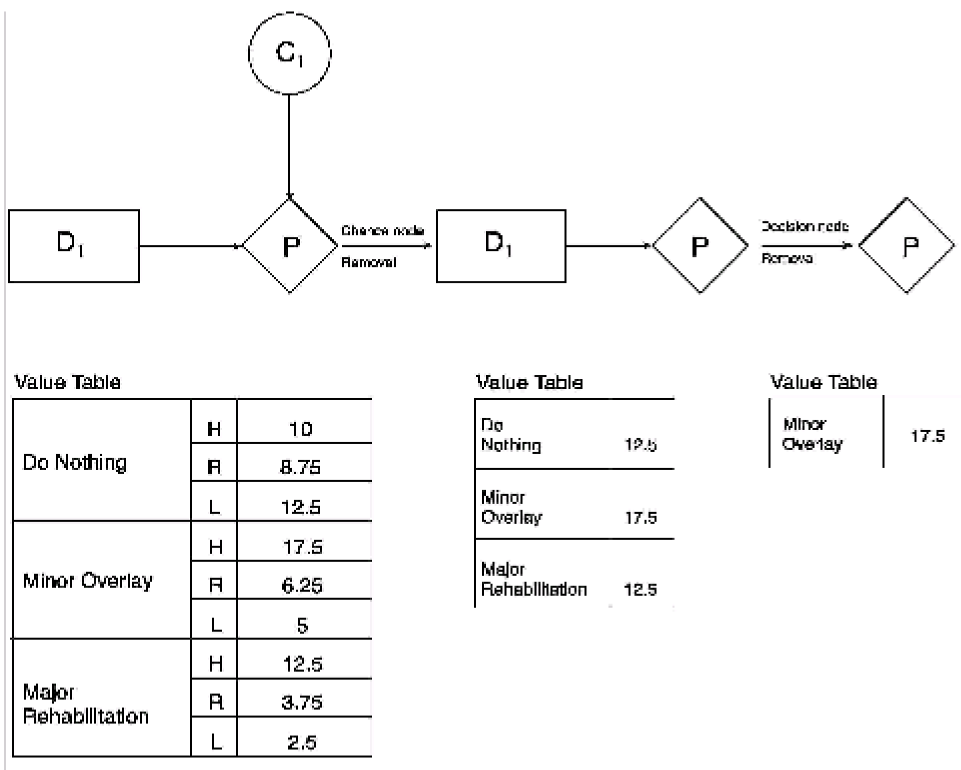

FIGURE 10 Solution procedure.

TABLE 1 Outcomes and Alternatives

Node Name	Possible Values
Deterioration Model	High or Low
Prior Deterioration Model	Acceptable or Not Acceptable
Posterior Deterioration Model	Acceptable or Not Acceptable
Cost of Strategies	High or Low
Cost of Data Collection	High or Low
Implementation Cost	High or Low
Future Cost	High or Low
Data Collection Strategies	Roughness or Cracks
Maintenance Strategies	Do Nothing, Minor Overlay, or Major Rehabilitation

steps involved and the corresponding decision tree. More complicated influence diagrams can be analyzed effectively only by software (*19*). The results (expected values) depend on the initial probabilities supplied by the engineer.

CONCLUDING REMARKS

This paper presents the formulation and analysis of problems in bridge management decision making. The approach relies on the application of subjective probabilities as input for policy determination (expected values). The representation is more appropriate than decision trees, especially in compactness and information flow. The arcs and other dependence properties clearly exhibit the physical and cognitive nature of the decision-making process. Graphical outcomes of the influence diagram include cumulative risk profiles and “tornado” diagrams. The cumulative risk profiles can be used for dominance (to show which variables are dominating the decision-making process), and “tornado” diagrams can be used to identify the relative sensitivity of various probabilistic nodes. Another advantage of an influence diagram is that decision makers can initially focus on a problem’s formulation and structure without major concern as to how to assess the probabilities. Consequently, there is no preprocessing of the probabilities, as with decision trees.

Although influence diagrams are appropriate for decision-making problems, they may not be versatile enough for asymmetric problems and for situations in which the outcome is relatively large. This would create huge conditional probability tables that are not tractable. In general, the influence diagram approach can be applied to any infrastructure management decision making.

REFERENCES

1. *Guidelines for Bridge Management Systems*. AASHTO, Washington, D.C., 1993.
2. Lee, H., and W. R. Hudson. Reorganizing the PMS Concept. In *Proc., Pavement Managing Systems: North American Conference on Managing Highways*, Ontario, Canada, 1985, pp. 59–80.
3. Sanford, K. L. Data Modeling for Improved Condition Assessment. *Proc., ASCE Conference, Infrastructure Condition Assessment: Art, Science, and Practice* (M. Saito, ed..), ASCE, Reston, Va., 1997.

4. Adams, T. M., and P. R. Sianipar. Tools for Sensitivity Analysis of Bridge MR&R Policies. In *Infrastructure Condition Assessment: Art, Science and Practice*, ASCE, Reston, Va., 1997, pp. 474–483.
5. Sobanjo, J. O. Cost Estimating Under Uncertainty Issues in Bridge Management. *Transportation Research Circular 498: Eighth Conference on Bridge Management,* Vol. I., TRB, National Research Council, Washington, D.C., 1999.
6. Thomas, P., and T. Merlo. The New Ontario Bridge Management System. In *Transportation Research Circular 498: Eighth Conference on Bridge Management*, Vol. I., TRB, National Research Council, Washington, D.C., 1999.
7. Bien, J. Expert Functions in Bridge Management Systems." In *Transportation Research Circular 498: Eighth Conference on Bridge Management,* Vol. II, TRB, National Research Council, Washington, D.C., 1999.
8. Attoh-Okine, N. O., and Martinelli. Belief-Functions Framework for Handling Uncertainties in Pavement Management Systems Decision Making. In *Transportation Research Record No. 1455,* TRB, National Research Council, Washington, D.C., 1994, pp. 153–158.
9. Attoh-Okine, N. O. Potential Application of Bayesian Influence Diagram in Pavement Management Systems. In *Proc., Second International Symposium (IEEE) on Uncertainty Modeling and Analysis,* College Park, Md., 1993, pp. 379–386.
10. Attoh-Okine, N. O. Addressing Uncertainties in Flexible Pavement Maintenance Decisions at Project Level Using Bayesian Influence Diagrams. In *Proc., Conference on Infrastructure Management: New Challenges, New Methods,* Denver, Colo., 1993, pp. 362–366.
11. Attoh-Okine, N. O. and K. Roddis. Uncertainties of Asphalt Layer Thickness Determination in Flexible Pavements-Influence Diagram Approach. *Civil Engineering and Environmental Systems,* Vol. 15, 1998, pp. 107–124.
12. Howard, R. A., and J. E. Matheson. Influence Diagrams. In *Principles and Application of Decision Analysis,* Vol. II (R. A. Howard and J. E. Matheson, eds.), Strategic Decisions Group, Menlo Park, Calif., 1984, pp. 721–762.
13. Hong, Y., and G. Apostolakis. Conditional Influence Diagrams in Risk Management. *Risk Analysis,* Vol. 13, No. 6, 1993, pp. 625–636.
14. Nease, R. F., and D. K. Owens. Use of Influence Diagrams to Structure Medical Decisions. *Medical Decision Making,* Vol. 17, 1997, pp. 263–275.
15. Shachter, R. D. Evaluating Influence Diagrams. *Operations Research,* Vol. 36, No. 6, 1986, pp. 871–882.
16. Ndilikilikesha, P. Potential Influence Diagrams. *International Journal of Approximate Reasoning,* Vol. 10, 1994, pp. 251–285.
17. Owens, D. K. Representation and Analysis of Medical Decision Problems with Influence Diagrams. *Medical Decision Making,* Vol. 17, 1997, pp. 241–262.
18. Henrion, M., J. S. Breese, and E. J. Horvitz. Decision Analysis and Expert Systems. *AI Magazine,* Vol. 12, No. 4, 1991, pp. 64–91.
19. Analytica Software. Lumina Decision Systems, Los Gatos, Calif., 1999.

Concrete Deterioration

Prediction of Deterioration

Start Application of Deicing Agent Taken into Account

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Predicting future deterioration is not as easy as extrapolating observed deterioration from the past. During the last century many aspects of bridge construction changed in ways that will influence tomorrow's condition. Three of the most influential changes are application of deicing agents, cover thickness, and the diffusion coefficient. These changes influence the rate of deterioration. The structure's age when the deicing agent was first applied has a significant influence on the chloride ingress, as a result of the decline of the coefficient of diffusion in time. An equation with a time-dependent coefficient of diffusion that includes the duration of exposure to the deicing agent and the age when exposure started was developed. The well-known phenomena that cause deterioration of concrete are alkali-silicate reaction, sulfate attack, frost–thaw, acid attack, carbonation, and chloride ingress. Taking all these phenomena into consideration would lead to a complex deterioration model. For the time being, it is more useful to take the decisive phenomenon into consideration—chloride-initiated corrosion. The probability of corrosion from chloride ingress is determined with Fick's second law of diffusion. Only by using a first-order reliability method can the results be compared with observed deterioration. The observed deterioration of more than 50 bridges during a period of 10 years was compared with the predicted deterioration from the newly proposed model that includes the point in time that application of deicing agents began.

In the Netherlands deterioration of concrete structures became an important issue over the last decade. Most bridges were constructed during the early 1970s, when the highway network was rolled out in the Netherlands (Figure 1). Although the bridges are still relatively young, the most frequently observed form of deterioration is chloride-initiated corrosion.

Current bridge deterioration models in bridge management systems are not successful in capturing the effects of bridges' construction history on their future condition. This paper, taking from the experience gained in Dutch bridge construction, gives a method to predict the deterioration of concrete bridges.

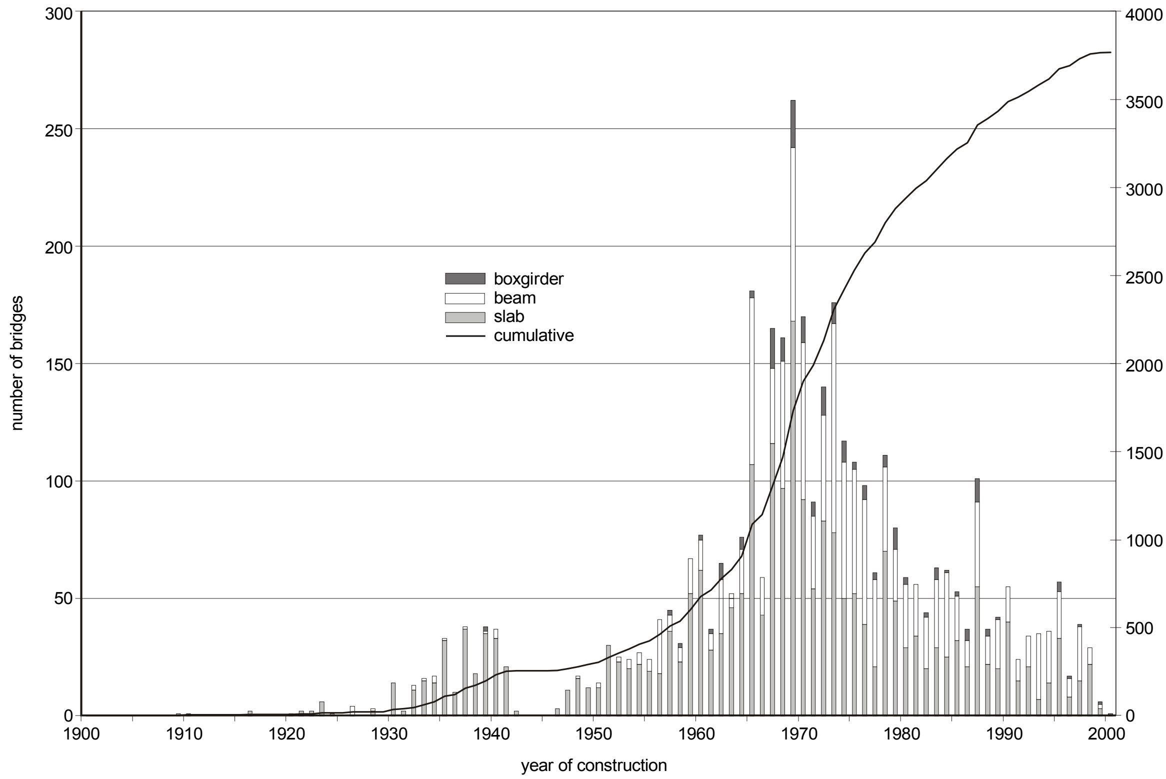

FIGURE 1 History of bridge construction on Dutch highways.

HISTORY OF DUTCH BRIDGE CONSTRUCTION

Predicting future deterioration is not as easy as extrapolating from observed deterioration. During the last century many aspects of bridge construction changed in ways that will influence tomorrow's condition. Three of the most influential changes over the last century are application of deicing agents, cover thickness, and the diffusion coefficient, as a consequence of the use of improved concrete mixes. These changes have a significant influence on the rate of deterioration.

Chloride Diffusion Coefficient

The resistance of concrete to ingress of chlorides is characterized by the chloride diffusion coefficient. This coefficient is strongly related to the compressive strength. An increase in compressive strength generally leads to a reduced coefficient of diffusion. Over the last century compressive strength has gradually increased. After World War II concrete with a characteristic compressive strength of 15 MPa (2,177 psi) was considered to be of high quality. A compressive strength of 65 MPa (9,433 psi) is now commonly used in today's bridge construction in the Netherlands.

In additon to the above-mentioned reduction of the design coefficient of diffusion, the coefficient declines when the structure ages. The coefficient will decrease in time, because pores in the hardened cement paste will gradually be blocked or become reduced in size because of the increased degree of hydration. Blended cements with a high replacement of fly ash or ground granulated blast furnace slag (GGBS) show a larger declinae of the diffusion coefficient than ordinary portland cement concrete PCC). Since World War II the Dutch Ministry of Transport has required the use of cement with a replacement of 50% to 70% GGBS.

On the basis of 8 years of research on concrete exposed to an outdoor climate in Scotland, a time-dependent coefficient of diffusion was established by Bamforth (*1*) (Figure 2). The decline in the coefficient of diffusion is especially important when deicing agents are applied for the first time several years after the structure has been completed.

Application of Deicing Agent
Until the early and middle 1960s, only roughness-increasing materials were used to keep roads accessible in the Netherlands during the winter. Examples of these roughness-increasing materials are sand and gravel. These materials were no longer sufficient, because of an increased public demand for ice- and snow-free roads and bridges. At first, this problem was overcome by adding common salt (sodium chloride) to the sand. However, removing the sand off the roads after the winter was too costly. From the early 1970s, only common salt was used, because the results of using sodium chloride as a deicing agent were satisfactory.

Cover Thickness
After material properties, the cover thickness is the largest influence on the quality of structures. Once the chloride content at the reinforcement exceeds the critical chloride content, corrosion of the reinforcement begins.

Over the years, the depth of the cover has changed. At the beginning of the 20th century the common idea was that the reinforcement would not corrode because the concrete protected it. Now engineers know better. Figure 3 shows the development of the cover thickness that was required by Dutch standards. Until 1974, the cover thickness of slabs was small by today's standard. Given that a huge number of bridges were built during the early 1970s, the development of the cover thickness over the years should be considered in predicting future deterioration.

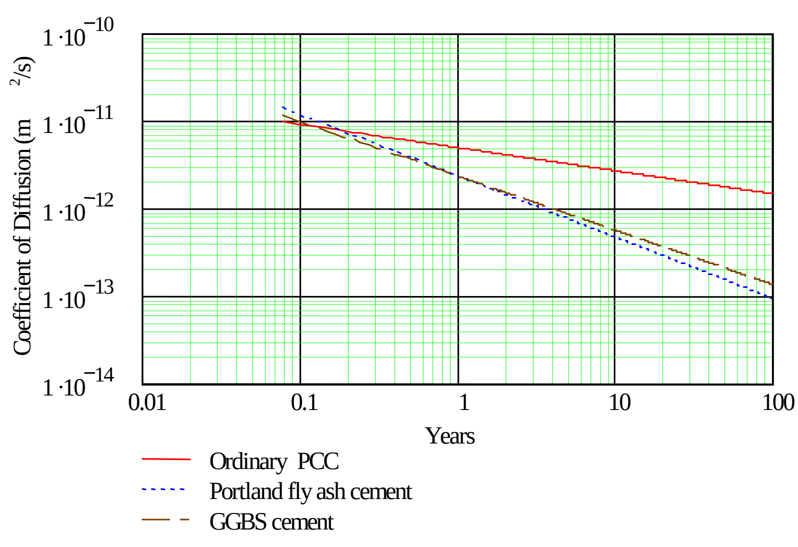

FIGURE 2 Time-dependent coefficient of diffusion.

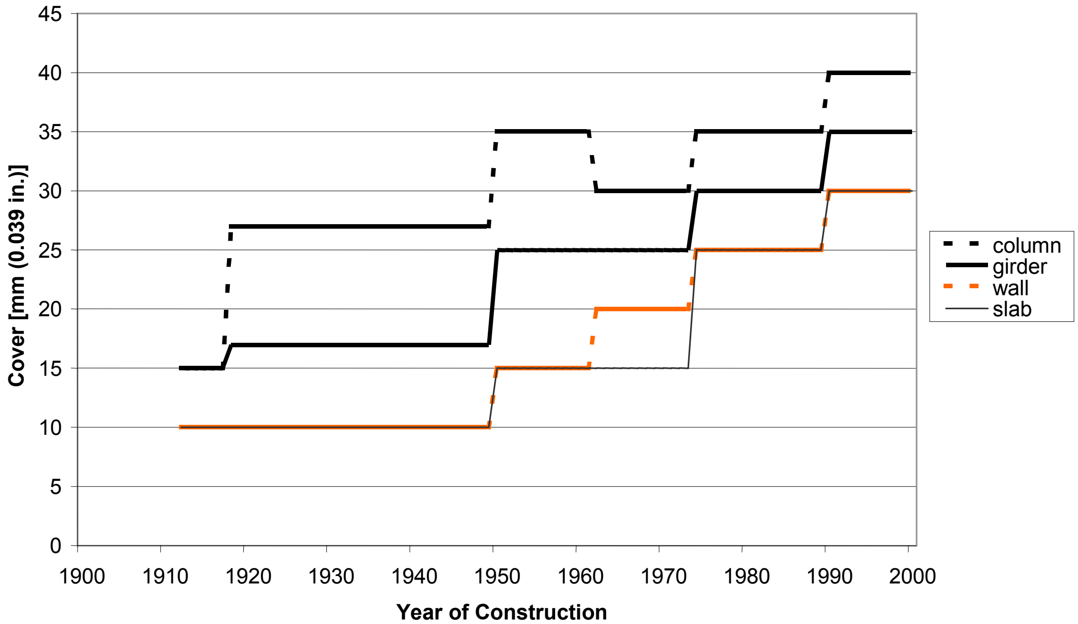

FIGURE 3 Cover thickness according to Dutch standards (GBV 1912–VBC 1995).

CONCEPT OF DETERIORATION MODEL

Several models have been developed since the early 1980s to determine the (future) condition of concrete structures. These models can be grouped into two categories: empirical versus physical models and deterministic versus stochastic approaches.

Choice of Model Type

Empirical models are obtained from experience rather than from a scientific theory. These types of models overcome the problem of unobserved variables that influence deterioration. Physical models, on the other hand, relate the factors affecting deterioration and condition. The variables that influence deterioration (e.g., cover thickness) changed considerably over the last century and may change in the future. In this article future deterioration is predicted with a physical model, because only then can the changes in bridge construction over the last century be taken into account.

A deterministic approach to the problem of deterioration is a powerful way to assess the whole bridge stock of the Dutch Ministry of Transport. However, uncertainty caused by the inherent stochastic behavior of the infrastructure's deterioration makes a stochastic approach to the problem more logical. A stochastic approach to predicting the deterioration of bridges is most commonly carried out by a Markov model. The drawback of this technique is that it assumes discrete transition intervals. When the probability of corrosion is calculated with physical models of each bridge in the bridge stock, with today's computers it is possible to use the first-order reliability method instead of the Markov method.

In this paper, the probability of deterioration is calculated with a first-order reliability method in which the reliability function is based on a physical model of deterioration.

Choice of Model

To reliably predict deterioration, the phenomena that leads to it should be known. The well-known phenomena that cause deterioration of concrete are alkali–silicate reaction, sulfate attack, frost–thaw, acid attack, carbonation, and chloride ingress. Taking all these phenomena into consideration would lead to a complex deterioration model. For the time being, it is more useful to take the decisive phenomena into consideration. From an analysis of the decay processes, it emerged that chloride-initiated corrosion is the governing deterioration process (Figure 4). It becomes clear that almost 70% of spalling is caused by corrosion from chloride ingress.

Numerous models are available to model chloride ingress over time. The biggest challenge in choosing a model is to find the balance between an equation with a small number of variables and a model that describes the phenomena accurately.

One model for calculating the ingress of chlorides is Fick's second law of diffusion. Fick's second law is the physical model used in this article to predict initiation of corrosion. It is assumed that corrosion of the reinforcement is initiated if the chloride content at the reinforcement exceeds a defined critical chloride content. Fick's law is a physical model that describes the chloride ingress on the basis of four variables: cover thickness, diffusion coefficient, surface chloride content, and initial chloride content.

Composition of Maintenance System

Figure 5 gives an overview of a system that estimates the future demand of maintenance of concrete bridges. The main program, in the center of Figure 5, determines the probability of failure during a certain period. Two databases supply the main program with the necessary information. The main database is filled with data from Dutch bridges obtained from the Dutch Ministry of Transport. The data consist of 1 year of construction, width and length of the bridge deck, and type of span. Over the last decade many highways in the Netherlands were widened from two to three lanes. To grasp the full scale of deterioration it is necessary to take the width and the length of each bridge into account. The second database contains the material properties of the concrete and design details of the bridge (e.g., cover thickness) and other important data, such as the start of application of deicing agents.

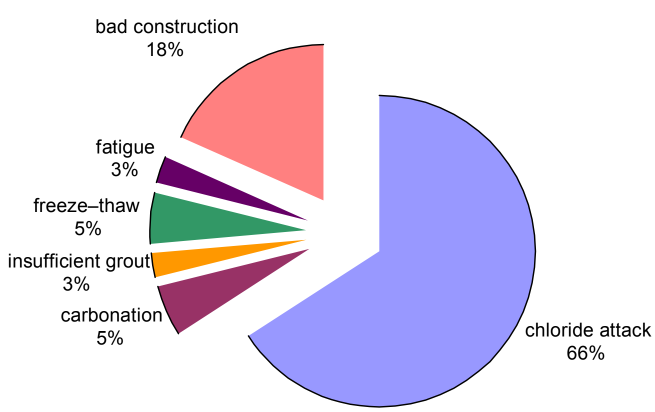

FIGURE 4 Cause of spalling (*2*).

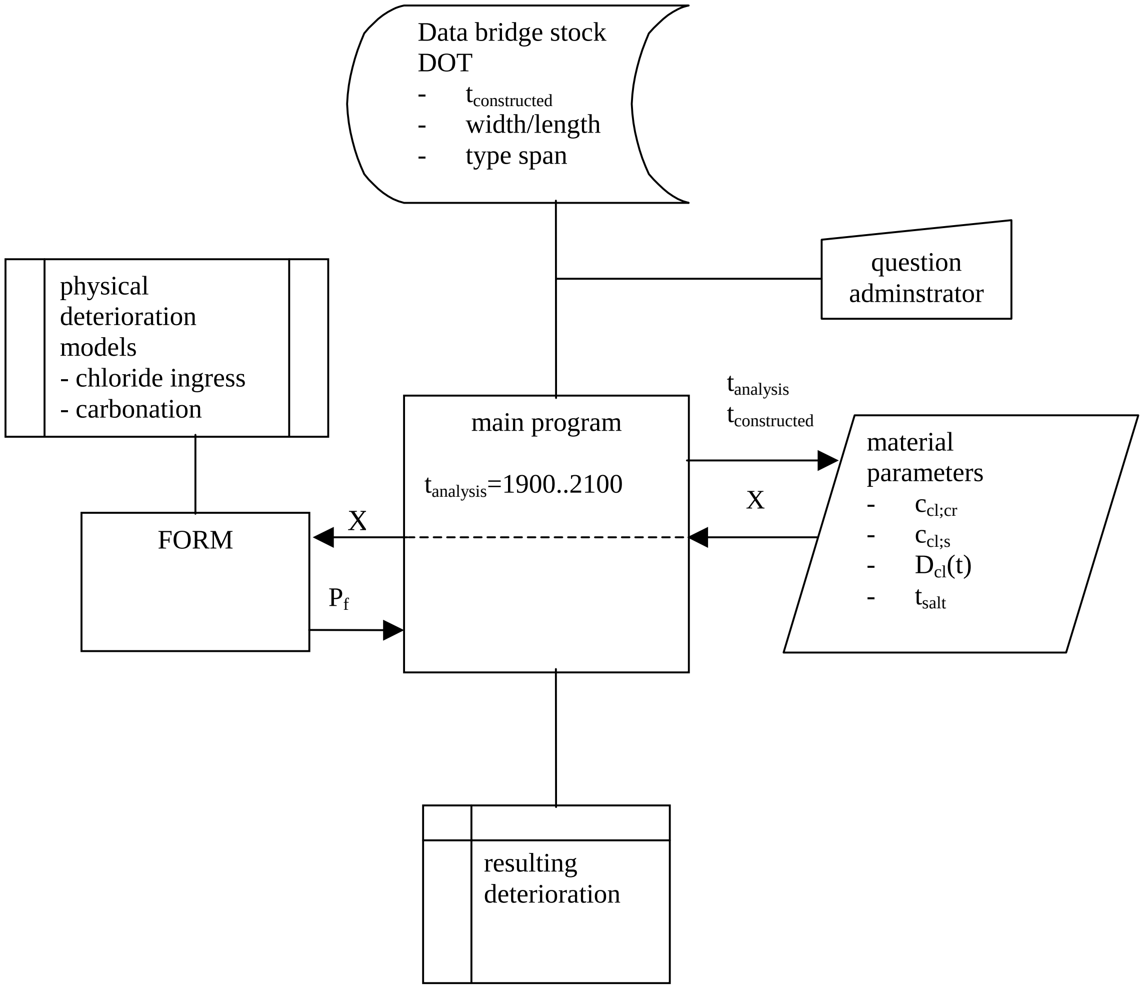

FIGURE 5 Structure of maintenance system.

An important condition for predicting future deterioration is choosing the right input, because the results are as good as the input. The development of the cover thickness is based on 1,100 measurements of cover thicknesses of more than 80 bridges constructed since World War II. The critical chloride content has been assumed after an extensive literature review at 0.5% of the binder weight. The coefficient of diffusion was determined on the basis of more than 150 cores taken from 16 bridges constructed during the last 5 decades.

CHLORIDE INGRESS

Many equations have been published to determine chloride ingress into concrete. The equation that is based on the physical process of diffusion was first published by Collepardi (*3*).

Constant Coefficient of Diffusion

Collepardi's equation (*3*) is the solution for Fick's second law of diffusion (*1*). It is assumed that the coefficient of diffusion and the surface chloride content are constant.

$$\frac{\partial C}{\partial t} = D\frac{\partial^2 C}{\partial x^2} \tag{1}$$

$$c_{cl}(x,t) = c_{cl,i} + (c_{cl,s} - c_{cl,i})\, erfc\left(\frac{x}{\sqrt{4D_{cl}t}}\right) \tag{2}$$

where

$c_{cl}(x,t)$ = concentration of chloride ions at time, t, and depth, x,
$c_{cl,i}$ = initial uniform chloride concentration in concrete,
$c_{cl,s}$ = concentration of chlorides at concrete surface,
x = depth coordinate from concrete surface into concrete,
t = lifetime of structures, and
D_{cl} = achieved chloride diffusion coefficient.

The error function and complementary error function are special cases of the incomplete gamma function. Their definitions are

$$erf(x) = \frac{2}{\sqrt{\pi}} \cdot \int_0^x e^{-t^2} \cdot dt \tag{3}$$

and

$$erfc(x) = 1 - erf(x)$$

Time-Dependent Coefficient of Diffusion

In contradiction with the earlier assumption, the coefficient of diffusion will decrease in time, because the pores are blocked or reduced in size as a result of an increased degree of hydration. This process is called aging and is expressed by the variable n. A reference diffusion coefficient ($D_{cl,0}$) is introduced that is transformed to a time-dependent coefficient of diffusion [$D_{cl}(t)$] by

$$D_{cl}(t) = D_{cl,0}\left(\frac{t_0}{t}\right)^n \tag{4}$$

The reference coefficient of diffusion is the diffusion at 28 days (t_0) after casting.

The total history of the changing diffusion coefficient is only included when solving Fick's second law of diffusion again. It results in a time history of the diffusion coefficient given by integrating Equation 4:

$$\int_0^t D_{cl}(t)dt = \frac{D_{cl,0}}{1-n}\left(\frac{t_0}{t}\right)^n t \tag{5}$$

The factor 1 / (1 – n) is missing in most analyses, as noted earlier by Bentz en Feng (*4*). Because Equation 4 is integrated, it is now valid again to substitute Equation 5 in Equation 2, as has been done by Visser et al. (*5*), resulting in

$$c_{cl}(x,t) = c_{cl,i} + (c_{cl,s} - c_{cl,i})\, erfc\left(\frac{x}{\sqrt{4\dfrac{D_{cl,0}}{1-n}\left(\dfrac{t_0}{t}\right)^n t}}\right) \quad (6)$$

Taking Start of Application of Deicing Agents into Account

The age of the structure when application of the deicing agent began has a significant influence on the chloride ingress, because of the decline in the coefficient of diffusion. The time-dependent coefficient of diffusion is presented as Equation 4. The total history of the changing diffusion coefficient and the age at which the structure was exposed to chlorides are only included when Fick's second law of diffusion is solved again. A graphical representation of the history is represented in Figure 6. It results in a time history of the diffusion coefficient given by

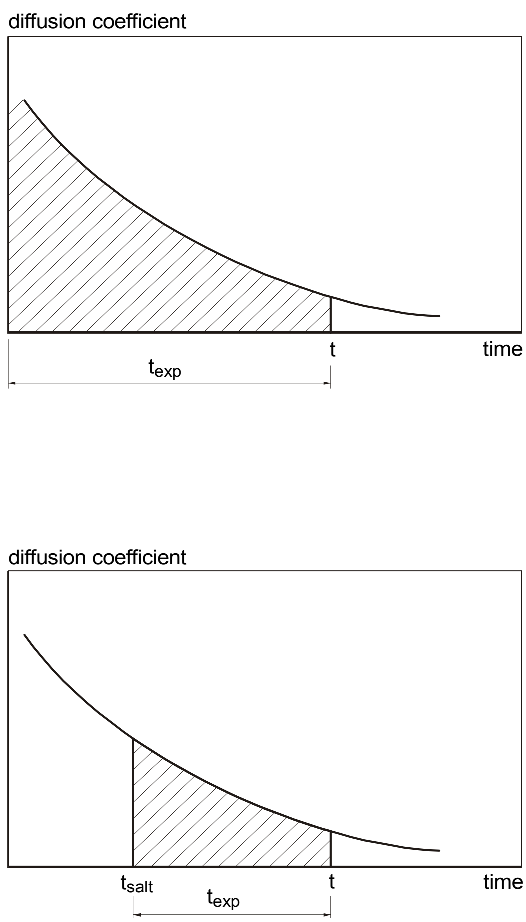

FIGURE 6 Duration exposure of deicing agent.

integrating Equation 4. This equation included the duration of exposure ($t_{\exp}$) to the deicing agent and the age at which exposure started (t_s):

$$\int_{t_s}^{t_s+t_{\exp}} D_{cl}(t)dt = \frac{D_{cl,0}}{1-n} t_0^{\ n}\left[(t_{\exp}+t_s)^{1-n} - t_s^{\ 1-n}\right] \tag{7}$$

As a result of the integration, it is now possible again to substitute Equation 7 in Equation 2, resulting in

$$c_{cl}(x,t) = c_{cl,i} + (c_{cl,s} - c_{cl,i})\ erfc \frac{x}{\sqrt{4\frac{D_{cl,0}}{1\text{-}n} t_0^{\ n}[(t_{exp}+t_s)^{1\text{-}n} - t_s^{\ 1\text{-}n}]}} \tag{8}$$

Equation 8 will be used to determine the deterioration of the Dutch bridge stock.

If the structure is exposed to deicing agents several years after completion, the concrete shows a larger density. In Figure 7 the ingress of chloride is determined for different exposure scenarios. This figure shows the chloride ingress if the structure is exposed at 0, 5, 15, and 25 years after completion. The reduced slope of the chloride ingress in Figure 7 expresses the aging of the concrete. All the chloride profiles start at an initial chloride content of 0.03% of the binder weight. The chlorides that are present in the mixing water cause this initial chloride content.

INSPECTION DATA

One of the biggest problems in predicting deterioration is how to obtain high-quality inspection data. Most inspections only give a qualitative judgment on the structure. However, a predictive model can only be verified with quantitative data of spalling (or any other form of deterioration

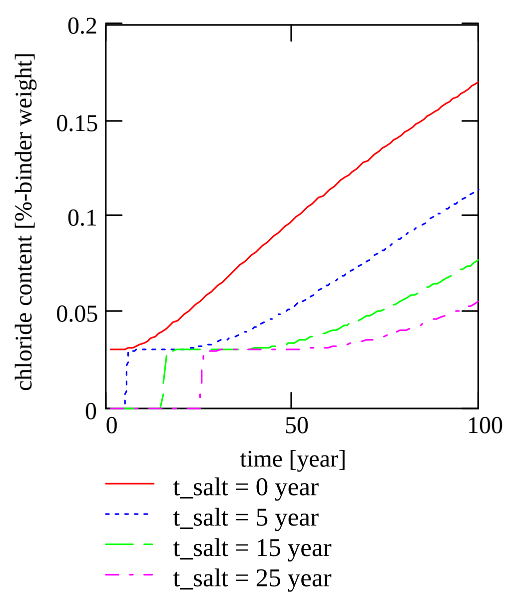

FIGURE 7 Influence of start of application of deicing agent.

The department of transportation in the Dutch province of Drenthe is one of the few administrations that carried out quantitative inspections of spalling. The data from that province was used check if the model is correct.

More than 50 bridges in Dutch highways in Drenthe have been inspected during a period of 10 years. The results of the quantitative inspections of chlorides are given in Figure 8. This figure shows the ratio of the area of spalling to total area (width by length) of the bridge deck. The straight line in Figure 8 is a linear fit of the observed spalling.

The model that predicts future deterioration gives the probability of initiation of corrosion. The propagation phase should also take into account model spalling. According to Cady and Weyers (*6*), spalling will occur 2 to 5 years after initiation of corrosion. In this article the probability of spalling is modeled by shifting the probability of initiation of corrosion by 5 years.

The question is: can the probability of spalling be compared with the observed spalling? In this article it is assumed that the probability of spalling derived from the model indicates the spalling that will occur at the bridges of the assessed bridge stock. It is assumed that the variables that influence deterioration are not correlated. As a consequence, this assumption is not valid for a single bridge. In case of a single bridge, both the cover thickness and the diffusion coefficient can be below average as a result of bad workmanship during construction. The quality of workmanship and quality of materials of a series of bridges are independent of each other, and in such a situation the predicted probability can be compared with the observed deterioration. A single bridge might deviate from the predicted deterioration, but on average the model will meet the observed deterioration.

In Figure 9, the deterioration (spalling) of the earlier mentioned 50 bridges in Drenthe is predicted. Spalling was predicted by taking into account the start of application of deicing agents, changes in cover thickness, and aging of the concrete. The observed spalling is also given in Figure 9. From Figure 9 it can be concluded that the model, between the years 1980 and 2000, closely matches the fitted line in Figure 8.

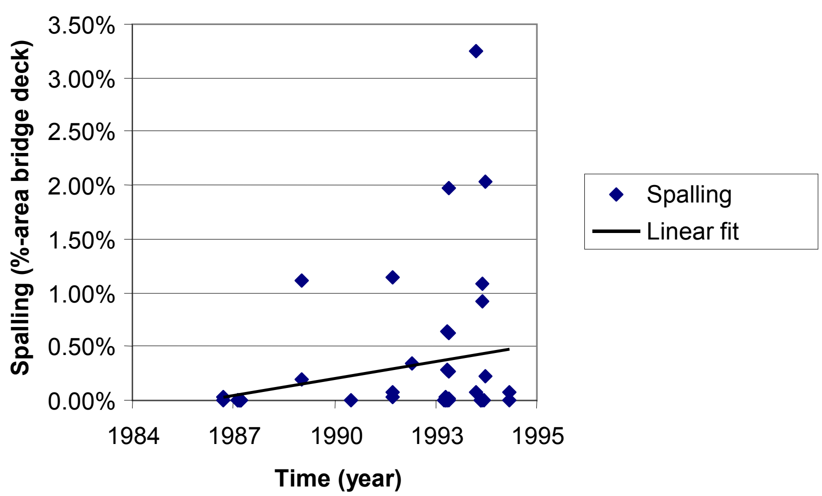

FIGURE 8 Observed spalling at highway bridges in the Dutch province of Drenthe.

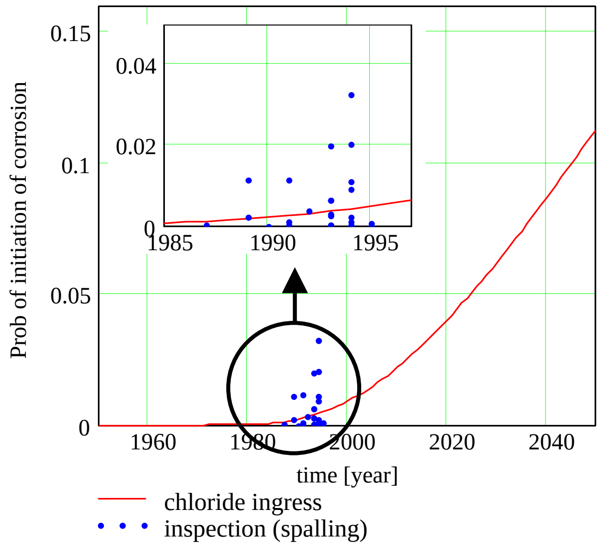

FIGURE 9 Deterioration: inspection versus prediction.

CONCLUSIONS

The results of the prediction of deterioration closely match the observed deterioration of a series of bridges in the Dutch province of Drenthe. Taking the time of first application of deicing agents into account, together with the development of cover thickness and diffusion coefficient in time, leads to a prediction that closely matches the observed deterioration.

ACKNOWLEDGMENT

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REFERENCES

1. Bamforth, P. B. The Derivation of Input Data for Modeling Chloride Ingress from Eight Years of U.K. Coastal Exposure Trials. *Magazine of Concrete Research*, April 1999, pp. 87–96.
2. Mutsuyoshi, H. Present Situation of Durability of Post-Tensioned PC Bridges in Japan. In *Durability of Post-Tensioning Tendons* (L. Taerwe, ed.), Fédération Internationale du Béton, Lausanne, Switzerland, 2001.
3. Collepardi, M., A. Marcialis, and R. Turriziani. Penetration of Chloride Ions into Cement Pastes and Concretes. *Journal of American Ceramic Society*, Oct. 1972, pp. 534–535.
4. Bentz, D. P., and X. Feng. Time-Dependent Diffusivities: Possible Misinterpretation Due to Spatial Dependency. In *Proc. 19: Testing and Modeling the Chloride Ingress into Concrete* (C. Andrade, and J. Kropp, eds.), International Union of Laboratories and Experts in Construction Materials, Systems, and Structures, RILEM Publishing, Bagneux, France, 2000.
5. Visser, J. H. M., G. C. M. Gaal, and M. R. de Rooij. Time Dependency of Chloride Diffusion Coefficient in Concrete. In *Proc. 21: Testing and Modeling the Chloride Ingress into Concrete* (C. Andrade, and J. Kropp, eds.), International Union of Laboratories and Experts in Construction Materials, Systems, and Structures, RILEM Publishing, Bagneux, France, 2002.
6. Cady, P. D., and R. E. Weyers. Deterioration Rate of Concrete Bridge Decks. *Journal of Transport Engineering*, Vol. 110, No.1, pp. 34–44.

CONCRETE DETERIORATION

Methodology for Prediction of Condition of Concrete Bridge Decks at Network Level

KHOSSROW BABAEI
Wilbur Smith Associates

Bridge management systems (BMSs) are useful tools for highway agencies' projection of the future condition of bridges to identify repair and maintenance needs at the network level. However, when a BMS is first implemented, no historical data on the condition of bridges are available for developing prediction models. A study was conducted by Wilbur Smith Associates and sponsored by FHWA to develop a methodology for highway agencies to predict the condition of concrete bridge decks at the network level for use in the Pontis BMS (or similar systems) when historical condition data are unavailable. The methodology was developed in a step-by-step format so that matrices can be constructed to determine the probability of transitioning bridge decks from one condition state to another in 1 year. These matrices can readily be used by highway agencies to predict the percentage of network bridge decks in each Pontis condition state for any future given year. In developing the methodology, it was recognized that corrosion of the reinforcing steel is the main cause of deterioration of concrete bridge decks in the United States. Prediction of the condition of bridge decks was possible by taking into account the severity of the corrosive environment, the permeability of concrete, and the concrete cover thickness. To construct the transition probability matrices for highway agencies, bridge decks were divided into groups with three levels of deicing salt exposure, three levels of specified water–cement ratio, and three levels of bar cover depth. By combining these bridge deck groups, 27 categories of decks were identified for the majority of U.S. highway bridge decks. Bridge decks in the same category share the same deicer exposure, specified water–cement ratio, and specified cover depth. Accordingly, 27 transition probability matrices were developed to predict the performance of decks in each category.

Symptoms of bar corrosion, such as spalls and delaminations, play a major role in defining the amount of deterioration and condition state of bridge decks. Five condition states are defined in Pontis for bridge decks on the basis of the condition of concrete as shown below:

Deterioration (% of area)	Pontis Condition State
0	I
> 0, but ≤ 2	II
> 2, but ≤ 10	III
> 10, but ≤ 25	IV
> 25	V

The aim of this project was to develop a methodology to predict the condition of bridge decks at the network level. In pursuing this goal, it was decided to develop Age Versus Condition tables for groups of bridge decks built at the same time and to the same specifications and served in the same environment. These tables would give the percentage of bridge decks in each condition state for a given bridge deck age. Table 1 shows the format of the Age Versus Condition table. Those tables can then be used to predict the percentage of network bridge decks in each Pontis condition state for any given future year. This is typically done by converting the Age Versus Condition table to the Pontis transition probability matrix through an established mathematical procedure. The research is documented in a final report under FHWA Contract DTFH61-96-C-00028 (*1*).

METHODOLOGY

In developing the methodology, it was recognized that corrosion of the reinforcing steel is the main cause of deterioration of concrete bridge decks. Prediction of condition of bridge decks at the network level was possible by taking into account three major factors affecting deterioration:

- Severity of corrosive environment,
- Permeability of concrete, and
- Concrete cover thickness.

These three factors vary significantly from one area to another in the United States, which causes significant differences in bridge deck performance. To predict deterioration accurately, it was decided to categorize bridge decks so those with the same corrosive environment, permeability of concrete, and bar cover thickness would be in the same category. Thus, the three factors influencing the performance had to be quantified.

Environment

Corrosive environment is primarily linked to exposure to deicing salt. This exposure was quantified by tons of salt applied per lane-mile (or lane-kilometer) per year. Three ranges of deicing salt exposure—low, medium, and high—were identified, as shown in Table 2, from information on usage of deicing salt usage in the United States (*2*).

TABLE 1 Format of Age Versus Condition Table for Use in Pontis

Deck Age (years)	Decks in Each Pontis Condition State (%)				
	I	II	III	IV	V
0	100	0	0	0	0
1	xx	xx	xx	xx	xx
2	xx	xx	xx	xx	xx
—	—	—	—	—	—
—	—	—	—	—	—
49	xx	xx	xx	xx	xx
50	0	0	0	0	100

NOTE: Deterioration as % of deck area: (I = none) (II = <2%) (III = 2–10%) (IV = 10–25%) (V = >25%).

TABLE 2 Quantifying Deicer Exposure

Deicer Exposure (Salt Application)	Low	Medium	High
Tons/Lane-Mile/Year			
Range	<2.5	2.5–5.0	>5.0
Average	1.5	3.7	7.5
Tons/Lane-Kilometer/Year			
Range	<1.4	1.4–2.8	>2.8
Average	0.8	2.1	4.2

Permeability

Bridge decks typically incorporate conventional concrete. Permeability of conventional concrete is usually presented as the concrete's water–cement ratio. In the past three decades, the specified water–cement ratio of bridge deck concrete has gradually been decreased from a maximum of 0.50 to a maximum of 0.45 to comply with AASHTO specifications (*3*). Three ranges of specified water–cement ratio—representing low-, medium-, and high-permeability concrete—were identified, as shown in Table 3.

Cover Thickness

For bridge deck slabs, current AASHTO bridge design code requires a minimum of 2 in. (51 mm) bar cover depth in mild climates and 2.5 in. (64 mm) in severe climates. The current code was implemented by states in the early 1980s. Before then, it was not unusual to design bridge decks with cover depths as low as 1.5 in. (38 mm). In accordance with the past and current design practices, three design bar cover depths were identified that represent the depths of almost all U.S. bridge decks designed since the 1950s, as given in Table 4.

Decks Categories

Bridge decks were divided into groups with three levels of corrosive environment (represented by annual salt applications), three levels of concrete permeability (represented by specified water–cement ratio), and three levels of bar cover depth (represented by minimum required bar cover thickness, or design cover). By combining the bridge deck groups, 27 categories of decks were identified representing the majority of highway bridge decks in the United States. Bridge decks in the same category shared the same salt exposure, specified water–cement ratio, and design bar cover thickness.

The aim of the project was to devise a methodology to develop an Age Versus Condition table for each bridge deck category, so that performance of decks in each category could be predicted independently and with greater accuracy.

Deck Performance in Same Category

Theoretically, performance of bridge decks in each category should not differ, because decks in the same category share the same chloride exposure and are designed to the same water–cement ratio and bar cover thickness. However, in reality, bar cover depth and water–cement ratio are subject to deviation from design values during construction. This deviation, in turn, can cause variation in deck performance within each category.

TABLE 3 Quantifying Permeability

Permeability (water–cement ratio)	Low	Medium	High
Range	≤0.43	0.44–0.46	≥0.47
Average	0.42	0.45	0.48

TABLE 4 Quantifying Design Bar Cover Depth

Cover Protection (cover thickness)	Low	Medium	High
Inches	1.5	2.0	2.5
Millimeters	38	51	64

Normal deviations of water–cement ratio from the specified values are generally low (0.01 to 0.02) and do not significantly affect performance. However, depending on the quality control procedure practiced by the highway agency during construction, a maximum of 0.03 may be added to the specified water–cement ratio to represent the permeability of the in-place concrete (*3*).

On the other hand, deviation of bar cover depth from its design value significantly affects performance of bridge decks, given that bar cover depth is the single most important factor influencing corrosion-induced deterioration in concrete. Surveys of bar cover depth of bridge decks have shown that the minimum in-place cover can be 1.2 in. (30 mm) less than the minimum specified value (*4*). Research shows that for a typical bridge deck concrete, a reduction of 1.0 in. (25 mm) in concrete cover has the same effect on chloride protection as an increase of 0.10 in water–cement ratio (*5*). By extrapolation, a reduction of 1.2 in. (30 mm) in specified concrete cover, which can occur in the field, has the same effect on chloride protection as an increase of 0.12 in water–cement ratio.

Average Cover Depth

The results of FHWA investigations into bridge deck bar cover depth show that cover depth is generally equally distributed around the average (*5*). Analyzing bridge deck bar cover information from several sources indicated that the average in-place bar cover depth of bridge decks can be assumed to be about 10% higher than the minimum required bar cover depth (Figure 1*a*).

Standard Deviation of Cover Depth

Although the average cover depth for a bridge deck can be assumed to be 10% higher than the minimum required cover depth, the variance of in-place cover depth, or standard deviation of cover depth, varies among a bridge deck population, but is generally independent of the average cover depth (*6*). The variance of in-place cover depth is related to the quality of construction: the better the quality, the lower the variance of in-place cover depth (Figure 1*a*).

Previous research has not determined the distribution of the variance of cover depth for a bridge deck population. This research attempted to do this task. On the basis of the analysis of field data available and the researcher's experience, the distribution of the variance of bridge deck cover depth was assumed to be a normal distribution. This distribution was quantified as follows.

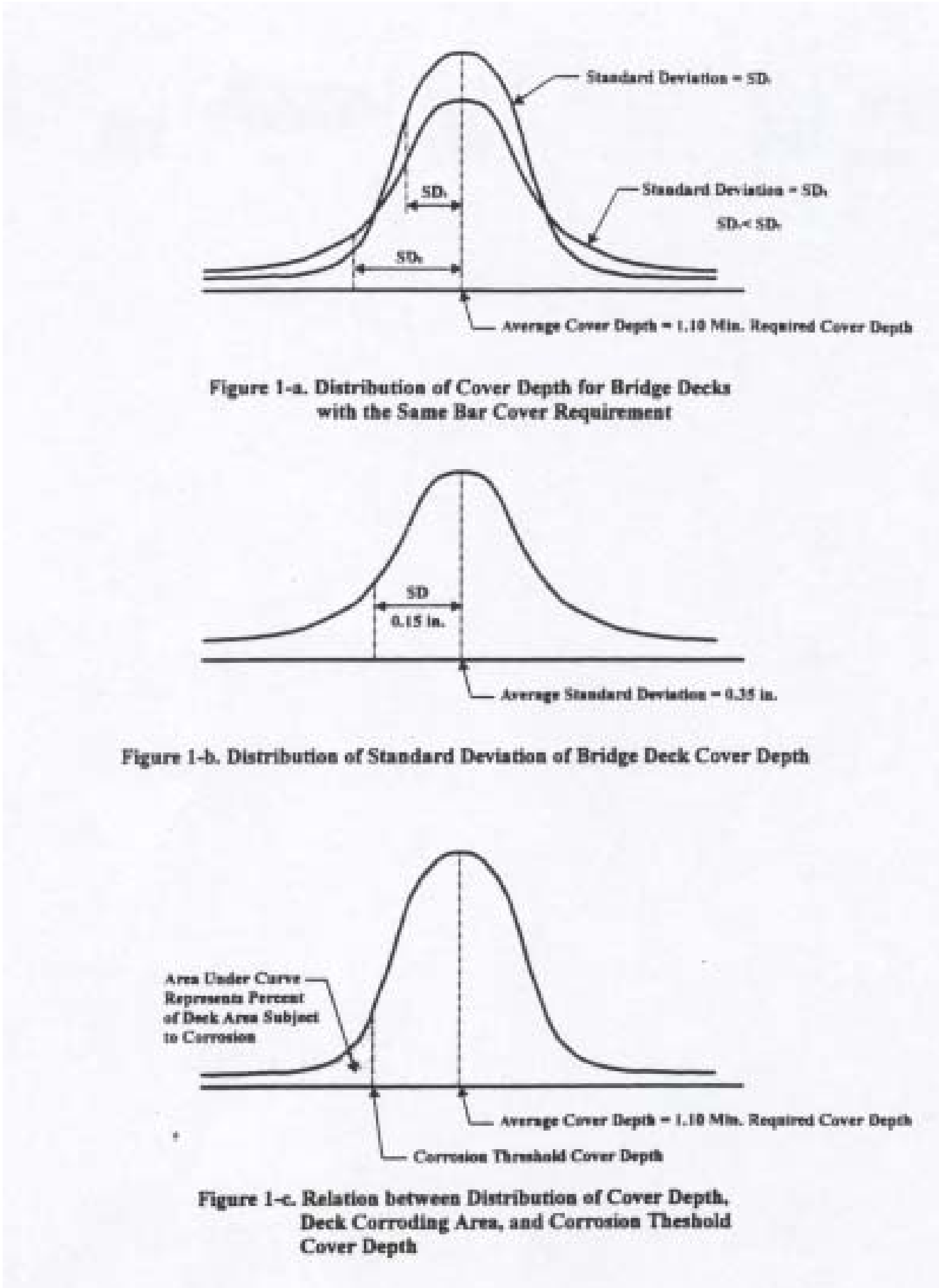

Figure 1-a. Distribution of Cover Depth for Bridge Decks with the Same Bar Cover Requirement

Figure 1-b. Distribution of Standard Deviation of Bridge Deck Cover Depth

Figure 1-c. Relation between Distribution of Cover Depth, Deck Corroding Area, and Corrosion Theshold Cover Depth

FIGURE 1 Distribution of bridge deck bar cover depth.

- Average standard deviation of cover depth for bridge deck population was assumed to be 0.35 in. (9 mm), and
- Variance of the standard deviation of cover depth within a bridge deck population (i.e., standard deviation of the standard deviation) was assumed 0.15 in. (4 mm) (Figure 1*b*).

PREDICTION OF DECK PERFORMANCE

NCHRP Report 297 (*3*) gives empirical relations between the cumulative number of salt applications (*S)* [tons per lane-mi (1 ton/lane-mi = 0.56 ton/lane-km)], water–cement ratio of concrete, and the depth of cover at which the reinforcing steel will be subject to corrosion in bridge decks, or the corrosion threshold cover depth (*CD*). The NCHRP equations are given in Table 5. Those equations are applied to concrete bridge decks to find the *CD* for any given bridge deck age.

The water–cement ratio in Table 5 is that specified for good construction practices. If the agency does not practice quality control for sound construction, up to 0.03 may be added to the specified water–cement ratio to account for construction tolerance.

The number of salt applications (*N*) in the NCHRP equations is based on 0.310 ton/lane-mi (0.175 ton per lane-km) of salt per application. Thus, *N* is determined from the following equation:

$$N = \frac{S}{0.310} \tag{1}$$

TABLE 5 NCHRP Empirical Relations Between *N*, Water–Content Ratio, and *CD*

Water–Cement Ratio	*CD* and *N*
0.60	$CD = (N / 6.83)^{0.2804}$
0.53	$CD = (N / 10.80)^{0.2741}$
0.52	$CD = (N / 11.60)^{0.2729}$
0.51	$CD = (N / 12.47)^{0.2720}$
0.50	$CD = (N / 13.40)^{0.2711}$
0.49	$CD = (N / 16.29)^{0.2731}$
0.48	$CD = (N / 20.73)^{0.2789}$
0.47	$CD = (N / 25.49)^{0.2824}$
0.46	$\mathrm{CD} = (N / 31.73)^{0.2861}$
0.45	$CD = (N / 39.47)^{0.2901}$
0.44	$CD = (N / 51.10)^{0.2979}$
0.43	$CD = (\mathrm{N} / 64.99)^{0.3038}$
0.42	$CD = (\mathrm{N} / 83.00)^{0.3097}$
0.40	$CD = (N / 141.85)^{0.3293}$

NOTE: 1 in. = 25.4 mm.

Assuming normal distribution for the bridge deck cover depth, the percentage of deck area with cover depth less than *CD* can be determined statistically, for a given water–cement ratio and *S* applied on the deck (Figure 1*c*). This area is subject to corrosion-induced deterioration. To find this area, the values of *CD* and *SD* (standard deviation of cover depth) are used in the following equation to determine Parameter α (standard normal deviate):

$$\alpha = \frac{(\text{average cover depth} - CD)}{SD} \tag{2}$$

By using Parameter α in the normal distribution table, the percentage of area with cover depth less than *CD* is obtained.

Assuming the average cover depth of a bridge deck is 10% higher than the minimum required cover depth, Equation 2 is reduced to Equation 3 as shown below:

$$\alpha = \frac{(1.10\ \text{minimum required cover depth} - CD)}{SD} \tag{3}$$

Given that *SD* in Equation 3 is variable among bridge decks, the value of α depends on the value of *SD*. In other words, for a given *S*, water–cement ratio, and minimum required cover depth, the percentage of deck area subject to corrosion will depend on the value of *SD* of bar cover depth. The higher the variance of a deck's cover depth, the higher the value of *SD*, and the lower the value of α. In accordance with the normal distribution table, a lower value of α corresponds to a higher percentage of deck area that is subject to corrosion.

Boundary Values of Bar Cover *SD*

The concept described above was used to determine the boundary values of α and bar cover *SD* corresponding to the boundaries of the five Pontis condition states for bridge decks (see Table 6). As shown in Table 6, first the boundaries of α relating to the boundaries of the five Pontis condition states were determined from the normal distribution table. Next, the boundaries of bar cover *SD* were calculated from the corresponding boundaries of α as per Equation 3.

It is noted in Table 6 that the boundary values of bar cover *SD* depend on Parameter A, and Parameter A is obtained from the minimum required cover depth and *CD*. *CD* in turn is a function of concrete water–cement ratio and *S* on the bridge deck, and it is found from Table 5 in conjunction with Equation 1.

Distribution of Decks Among Pontis Condition States

The boundary values of bar cover *SD* were used to distribute network bridge decks among the five Pontis condition states. This was possible by assuming a normal distribution for bar cover *SD* with an average of 0.35 in. (9 mm), and variance (*SD*) was assumed to be 0.15 in. (4 mm), as previously discussed. This is demonstrated in Table 7.

In constructing Table 7, first the maximum bar cover *SD* for each Pontis condition state was determined from Table 6. Considering a normal distribution for *SD* of cover depth among

TABLE 6 Boundary Values of Bar Cover *SD*

Pontis Condition State	**Deck Deterioration (% Area)**	**Boundary Values of α** [a]	**Boundary Values of *SD*** [b]
I	< 0.1 [c]	$\alpha > 3.09$	$SD < A / 3.09$
II	0.1 to 2	$3.09 > \alpha > 2.05$	$A / 3.09 < SD < A / 2.05$
III	2 to 10	$2.05 > \alpha > 1.28$	$A / 2.05 < SD < A / 1.28$
IV	10 to 25	$1.28 > \alpha > 0.67$	$A / 1.28 < SD < A / 0.67$
V	> 25	$\alpha < 0.67$	$SD > A / 0.67$

[a] Determined from normal distribution table; % area deteriorated represents % cumulative area under distribution curve.
[b] A = (1.10 minimum required cover depth – *CD*), per Equation 3.
[c] Arbitrarily selected.

the bridge deck population (Figure 1*b*), the percentage of bridge decks with cover depth *SD* less than this maximum is determined statistically for each Pontis condition state. This percentage represents the percentage of decks in the corresponding Pontis condition state plus the percentage in the preceding condition states. The difference between this percentage and its preceding value gives the percentage of decks in the corresponding Pontis condition state.

As noted from the previous discussion, the key to distribution of decks among Pontis condition states is to determine the percentage of bridge decks with cover depth *SD* less than the maximum value for a given Pontis condition state. To find this percentage, first the following relation is used to find Parameter α' (standard normal deviate):

$$\alpha' = \frac{(\text{average } SD - \text{maximum } SD \text{ for the condition state})}{(\text{standard deviation of } SD)}$$

In the above relation, *SD* is the *SD* of cover depth of individual bridge decks. For the bridge deck population, the average *SD* is 0.35 in. (9 mm), and standard deviation of *SD* is 0.15 in. (4 mm), as noted previously. By using the maximum *SD* for each Pontis condition state in the above relation, Parameter α' is obtained for each condition state. Parameter α' is then used in the normal distribution table to obtain percentage of decks with bar cover *SD* less than the maximum *SD*.

Construction of Age Versus Condition Tables

The previous section presented a methodology to distribute bridge decks of a given category and same age among the five Pontis condition states. This methodology is applied to each consecutive bridge deck age up to 50 years, and the results are aggregated to produce the Age Versus Condition table for a bridge deck category of interest. The format of the Age Versus Condition table is shown in Table 1.

Such Age Versus Condition tables, however, are applicable to corroding areas of the deck, whereas Pontis classification of condition applies to deteriorating areas of the deck. Given that corrosion-induced deterioration typically lags about 3 years behind the reinforcing steel corrosion (*3*), the Age Versus Condition table can be adjusted to represent deteriorating areas, instead of corroding areas. This is done by subtracting 3 years from the bridge deck age and using the adjusted age in the

TABLE 7 Distribution of Bridge Decks Among Pontis Condition States

Pontis Condition State	Maximum Bar [a] Cover *SD* (in.)	α' Parameter [b]	% Decks with Bar Cover [c] *SD* Less Than Maximum Standard Deviation	% Decks in Each Pontis State
I	A / 3.09	(0.35-A/3.09) / 0.15	$P_{(I)}$	$P_{(I)}$
II	A / 2.05	(0.35-A/2.05) / 0.15	$P_{(I–II)}$	$P_{(I–II)} – P_{(I)}$
III	A / 1.28	(0.35-A/1.28) / 0.15	$P_{(I–III)}$	$P_{(I–III)} – P_{(I–II)}$
IV	A / 0.67	(0.35-A/0.67)/ 0.15	$P_{(I–IV)}$	$P_{(I–IV)} – P_{(I–III)}$
V	—	—	—	$100 – P_{(I–IV)}$

[a] From Table 6 in which A = 1.10 minimum required cover depth – *CD* (in.).
[b] In accordance with normal distribution of *SD* of cover and assuming average cover *SD* = 0.35 in. (9 mm) and variance (standard deviation) of cover *SD* = 0.15 in. (4 mm).
[c] Determined from normal distribution table based on α' parameter.

methodology to determine the percentage of bridge decks in each Pontis condition state. Following this concept, bridge decks with ages up to 3 years will have 100% of the decks in Condition State I.

DEVELOPMENT OF SYSTEMATIC METHODOLOGY

Step-by-Step Procedure

This study developed and documented a systematic, step-by-step procedure to construct the Age Versus Condition table for any category of concrete bridge decks of the same age, having the same deicing salt exposure, specified water–cement ratio, and specified bar cover depth. Accordingly, 27 Age Versus Condition tables were generated for 27 categories of decks representing the majority of highway bridge decks in the United States.

Example

With this step-by-step methodology, the Age Versus Condition table was developed for a category of concrete bridge decks of the same age and with the following characteristics:

- Decks have medium design bar cover depth [2.0 in. (51 mm), Table 4].
- Specified water–cement ratio is medium (0.44 to 0.46, or average of 0.45, Table 3).
- Salt exposure is medium [2.5 to 5.0 ton/lane-mi/year (1.4 to 2.8 ton/lane-km/year)] or average of 3.7 ton/lane-mi/year (2.1 ton/lane-km/year, Table 2).

Table 8 shows the Age Versus Condition table generated with the methodology.

TABLE 8 Example of Age Versus Condition Table for Use in Pontis

Age (Years)	Pontis Condition State* (% Decks)				
	I	II	III	IV	V
0	100	0	0	0	0
1	100	0	0	0	0
2	100	0	0	0	0
3	100	0	0	0	0
4	81	18	1	0	0
5	70	28	2	0	0
6	63	32	5	0	0
7	55	37	8	0	0
8	50	37	13	0	0
9	45	37	18	0	0
10	39	40	21	0	0
11	34	39	26	1	0
12	32	36	30	2	0
13	30	33	34	3	0
14	25	33	38	4	0
15	23	32	40	5	0
16	21	29	41	9	0
17	19	26	43	12	0
18	18	24	44	14	0
19	16	21	44	19	0
20	14	20	43	23	0
……	…………	…………	…………	……………	…………
……	…………	…………	…………	……………	…………
35	4	3	11	45	37
……	…………	…………	…………	……………	…………
……	…………	…………	…………	……………	…………
40	3	1	5	23	68
41	3	1	3	18	75
42	2	1	3	13	81
43	2	1	2	13	82
44	2	1	1	9	87
45	2	0	2	6	90
46	2	0	2	5	91
47	2	0	1	3	94
48	1	1	0	3	95
49	1	0	1	1	97
50	1	0	1	1	97

* Deicer exposure: medium (2.5 to 5.0 ton/lane-mi/year) (1.4 to 2.8 ton/lane-km/year); water–cement ratio: medium (0.44 to 0.46); minimum required cover: medium (2 in.) (51 mm).

METHODOLOGY FOR DECKS WITH EPOXY-COATED BARS

Early research in the United States indicated that epoxy-coated reinforcing bars will perform well in salt-contaminated concrete, indefinitely. The current general opinion is that although bridge deck performance with epoxy-coated bars has been satisfactory, this corrosion control alternative may not provide the totally maintenance-free service life that was forecast earlier.

Information in the literature was insufficient to precisely project the extended corrosion-free life as a result of the use of epoxy-coated bars. Agencies must determine from their own experience the extended life with the performance of bridge decks with epoxy-coated bars. A default value of 10 years was suggested for the extended corrosion-free life in the absence of any information. Accordingly, a procedure was suggested for developing Age Versus Condition tables for bridge decks with epoxy-coated bars, as indicated below.

The Age Versus Condition tables developed for decks with uncoated reinforcing steel may be used for decks with epoxy-coated bars, provided the extended corrosion-free life is subtracted from the age of bridge decks. For example, if the age of bridge decks with epoxy-coated bars is 18 years, and if the default value for the extended corrosion-free life from epoxy coating is used (i.e., 10 years), the Pontis condition state information for that bridge deck age can be obtained from the corresponding table for uncoated bars assuming a bridge deck age of 8 years (18 – 10 = 8 years).

FIELD VALIDATION OF METHODOLOGY

Survey information was available on service life of large samples of bridge decks with uncoated reinforcement from three states: Virginia, Pennsylvania, and New York (*7*). That information included the percentage of bridge decks that had been repaired or overlaid. This project used the information to evaluate the developed methodology's accuracy. The results are shown in Table 9.

The level of deterioration that requires repair or overlay is typically more than 10% of the deck area, and it can be as high as 50%. Therefore, the decks at the time of repair or overlay were in Pontis Condition States IV and V.

As shown in Table 9, the information generated by the methodology on the percentage of bridge decks in Pontis Condition States IV and V is in close agreement with the field information. It is also noted in Table 9 that for New York and Pennsylvania, the percentage of decks in Condition States IV and V, as derived from the methodology, is slightly larger than the actual cumulative percentage of decks treated. This is normal, as in any given year not all decks that qualify for treatment are treated, and a construction backlog is usually expected due to limited funds.

FHWA investigations (*8*) has documented performance of samples of U.S. bridge deck with epoxy-coated bars in the United States. Performance of deck samples with 10 or more bridges and with ages 10 years or more was available from Michigan, Minnesota, New York, and West Virginia. Table 10 shows the actual condition of those decks and their condition as predicted by the methodology. In using the methodology, the extended corrosion-free life attributed to epoxy coating was assumed to be 10 years. As shown in Table 10, the information generated by the methodology is in close agreement with the field information.

TABLE 9 Comparison of Field Condition and Condition Predicted by Methodology

State	Deck Sample Age (years)	Field Information Decks Treated (%)	Methodology Information Decks in Conditions IV and V (%)
New York	18.5	74	82
Pennsylvania	24	35	42
Virginia	20	11	11

TABLE 10 Comparison of Field Condition and Condition Predicted by Methodology for Decks with Epoxy-Coated Bars

Location	Average Age of Deck Sample (Years)	Decks with <2% Deterioration (Condition State II)	
		Field Information	Methodology Information
Michigan	13	1 of 12 decks	0% (none of 12 decks)
Minnesota	15–20	None of 12 decks	2% (none of 12 decks)
New York	10	None of 14 decks	0% (none of 14 decks)
West Virginia	17	1 of 14 decks	7% (1 of 14 decks)

REFERENCES

1. Babaei, K., and A. Fouladgar. *Pontis Bridge Management System Enhancement Through Nondestructive Concrete Evaluation Technologies: Part 1. Prediction of Condition of Concrete Bridge Decks at Network Level.* FHWA, U.S. Department of Transportation, Sept. 1998.
2. *Survey of Salt, Calcium Chloride, and Abrasives Used in the United States and Canada for 1981–1982 and 1982–1983.* Salt Institute, Alexandria, Va.
3. Babaei, K., and N. M. Hawkins. *NCHRP Report 297: Evaluation of Bridge Deck Protective Strategies*. TRB, National Research Council, Washington, D.C., Sept. 1987.
4. Chamberlin, *W. P. Long-Term Evaluation of Unprotected Concrete Bridge Decks.* Research Report 128. New York State Department of Transportation, Albany, Nov. 1985.
5. Clear, K. C. *Time-to-Corrosion of Reinforcing Steel in Concrete Slabs, Volume 3: Performance after 830 Daily Salt Applications.* Report No. FHWA/RD-76-70. FHWA, U.S. Department of Transportation, April 1976.
6. O'Rourke, P. W., and J. M. Ritchie. Bridge Deck Concrete Cover Investigation in Michigan. In *Transportation Research Record No. 676,* TRB, National Research Council, Washington, D.C., 1978.
7. Weyers, R. E., I. L. Al-Qadi, M. G. Fitch, E. P. Larsen, J. E. Peterson, and R. O. Drumm. Technique Service Life Estimates. SHRP Contract C-103, Task 1. TRB, National Research Council, Washington, D.C., Nov. 1992.
8. Smith, J. L., and Y. P. Virmani. Performance of Epoxy-Coated Rebars in Bridge Decks. *Public Roads*, Autumn 1996.

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