Calibration and Application of Deterioration Models for Highway Bridges

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A correspondence between deterioration processes and the observed performance of bridges is established by creating new quantitative condition ratings. Condition ratings defined in terms of damage indexes are used to form Markov chain models that embody the performance of bridges in service. Markov chains in turn are used to generate mean transition times that can be used to compute parameters of deterioration process models. Variations in exposure of bridge beams and in regions of bridge beams are handled by a multiplicity of Markov chains. To calibrate multiple models, condition ratings must be tied to known locations in a bridge. For this purpose the practice of segment-based reporting is introduced. Quantitative condition ratings are linked to normalized remaining flexural and shear strengths of bridge beams. Normalized remaining strength, along with segment-based reporting, supports the estimation of live load rating of bridges. Four cases of an example steel beam bridge are considered under differing rates of corrosion and patterns of corrosion in cross sections. It is seen that the controlling segment for live load rating of a bridge may evolve under some corrosion environments and that estimated live load capacities based on quantitative condition ratings are reasonable approximations of the actual load ratings.

The performance of highway bridges may be measured as the ability of the bridges to remain strong and to provide adequate service over time. Bridges must withstand the actions of (a) a service environment defined by traffic loads, maintenance activities (or their lack), and operating practices such as application of deicing salts; and (b) a natural environment defined by temperature variations, freeze-thaw cycles, and air- and spray-borne chemical agents and salts. Collectively, these actions drive processes of deterioration that impair the strength of bridges. These deterioration processes can be modeled. With models, it is possible to examine the rates of deterioration processes and the importance of deterioration processes in the life cycle of bridges.

Deterioration rates for bridges in service, if known quantitatively, can provide direct evidence of the performance (success) of particular design details and structural materials. Most often, models of deterioration processes are taken as models obtained in exposure tests of material coupons and not from structures in service. Models of deterioration processes have been used to evaluate the strength and safety of bridges. Separately, there are models of time-domain changes in condition ratings. Condition ratings are, in present practice, qualitative indicators of the state of deterioration of a bridge, and models of condition ratings are used in bridge management systems (BMSs) to assist in the selection of repair strategies. The newest BMSs use

Markov chains to model condition ratings since these chains are easily calibrated to the observed changes in condition ratings for a network of bridges. Separately, then, there are models of deterioration process, methods for evaluating bridge strength that use deterioration models, models for condition ratings, and methods for calibrating models of condition ratings against the observed performance of bridges. What is lacking is a unified approach to modeling the observed performance of bridges, interpreting these models in terms of quantitative measures deterioration processes, and using the information obtained on processes to yield quantitative assessment of bridges.

This paper outlines a unified approach to modeling and calibration of models of deterioration processes for highway bridges and illustrates the use of modeling in the assessment of load ratings. Methods for modeling, calibration, and assessment are needed for the further development of BMS. Management systems are assuming a larger role in the planning of maintenance activities, but it is important both that BMS-derived planning be responsive to the strength and safety of bridges and that safety-related repairs, which now constrain BMS planning as an external factor, be included directly in BMS to achieve the best use of resources. The modeling and evaluation capabilities provided here require a modified practice of field inspection of bridges both to obtain the necessary condition ratings for elements and to link element ratings to specific locations within a bridge. The new inspection practice, which is the subject of an ongoing demonstration project being conducted with the Colorado Department of Transportation, is outlined.

Deterioration Modeling

Mathematical representations of the performance of bridges, called "deterioration models," exist today in two distinct categories. First, there are models of deterioration processes such as fatigue, corrosion of steel, and spalling of concrete. Models of processes of deterioration often describe deterioration as a function of one or a few parameters related to environment, exposure, or use. Deterioration process models yield a measure of loss of material, of strength, or of endurance, and these measures are useful in the evaluation of strength and safety of structures. Models of processes of deterioration have been used to investigate the effect of deterioration on strength of bridges (*1*) and can be used in the optimization of inspection intervals and maintenance programs (*2,3*). The studies are similar in that the models allow a deterioration process to be recognized as a change (loss) in section properties of structural members and thereby permit a reevaluation of bridge strength by familiar design-level computations.

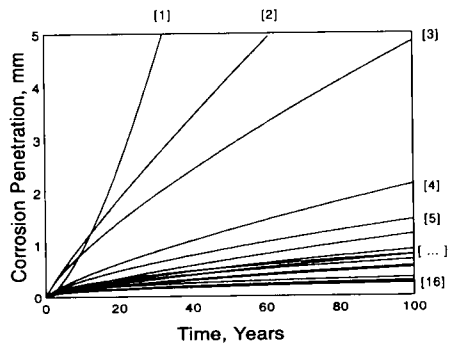
Models of deterioration processes require calibration. Rates of deterioration can respond strongly to environment and to exposure. This is evident in the models that have been developed for atmospheric corrosion of steel. Many studies of corrosion of steel coupons have used a power law relation to describe the progress of corrosion (*4–6*).

$$C = At^B \tag{1}$$

where

C = average penetration of corrosion, often in microns,
t = exposure time (years), and
A,B = constants obtained from linear regression in log space.

Many studies of steel corrosion have employed a power law model, and it is observed that the rates of corrosion can differ significantly among studies. Figure 1 shows 16 power law models that have been reported for the corrosion of steel. It is seen that corrosion losses diverge sharply. Differences in corrosion rates are due to environment, exposure, and the method of modeling. Three severities of environment may be identified: benign, moderate, and severe (*7*). Exposure may be open or sheltered and is influenced by thermal lag if steel is part of a larger structure (*8*). Methods of modeling include linear regression log space, power law fitting in linear space (*9*), and composite modeling using a power law



Corrosion Penetration - Models & Sources

[1] Marine, Albrecht 1984	[9] Marine, Albrecht 1984
[2] Industrial, Albrecht 1984	[10] Marine, Albrecht 1984
[3] Industrial, Albrecht 1984	[11] Rural, Albrecht 1984
[4] Industrial, Albrecht 1984	[12] Rural, Komp 1987
[5] Industrial, Albrecht 1984	[13] Industrial, Komp 1987
[6] Marine, Komp 1987	[14] Industrial, Komp 1987
[7] Marine, Albrecht 1984	[15] Industrial, Townsend 1982
[8] Rural, Albrecht 1984	[16] Industrial, Townsend 1982

FIGURE 1 Reported models of corrosion penetration for carbon steel.

for early-life corrosion and a linear rate for continuing corrosion (*10*). The models, different as they are, all represent the results of controlled tests of coupons.

For application to bridges, there may be a larger uncertainty in corrosion rates. Bridges are subject to all of the variables of environment and exposure and can be subject to service-related events, such water runoff, that may encourage rapid local corrosion. Available models for corrosion of steel are almost all models of coupon studies, and nearly all are models developed using standard test methods for exposure of coupons. Some studies have placed coupons on (and underneath) bridges to simulate the exposure and environment of bridges (*11*, p. 717), and some work is available for corrosion rates measured on bridges in service (*12*, p. 16). But the overall indication of studies is that corrosion rates vary strongly, and the use of any model must be supported by frequent checking and calibration.

The second category of deterioration models is models of condition ratings. In U.S. practice, condition ratings have been assigned to bridges according to the condition rating scale of the National Bridge Inventory (NBI) (*13*) since the early 1970s. The condition ratings are broad, qualitative indicators of the health of a bridge. Condition ratings can be correlated with repair needs for bridges, and therefore both the ratings and models for changes in ratings over time are useful to BMSs that seek to assess repair needs and to formulate repair programs. Models for NBI condition ratings have been developed (*14–16*); these typically are obtained by regression analysis and can involve several factors such as bridge age, average daily traffic, and type of construction to express the changes in NBI ratings. More recently, Markov chains have been employed as models of condition ratings, and new BMSs have proposed new condition rating scales in combination with Markov chains to obtain superior models for the assessment of present and future repair needs for networks of bridges. Cesare et al. applied Markov chains to condition ratings obtained in the inspection of highway bridges in New York State (*17*).

Markov chains used for condition ratings are discrete-state/discrete-time chains. The condition rating for a population of bridges or of bridge elements is represented by a vector $\{C\}$, and the change in the population of condition ratings in a single inspection cycle is computed by operating on $\{C\}$ with a matrix of transition probabilities $[P]$ as

$$\begin{Bmatrix} C_1 \\ C_2 \\ C_3 \\ C_4 \\ C_5 \end{Bmatrix}_{n+1} = \begin{bmatrix} P_{1,1} & & & & \\ P_{2,1} & P_{2,2} & & & \\ & P_{3,2} & P_{3,3} & & \\ & & P_{4,3} & P_{4,4} & \\ & & & P_{5,4} & P_{5,5} \end{bmatrix} \begin{Bmatrix} C_1 \\ C_2 \\ C_3 \\ C_4 \\ C_5 \end{Bmatrix}_{n} \qquad (2)$$

where

$\{C\}_n$ = vector of current condition ratings
$\{C\}_{n+1}$ = predicted vector of condition ratings after next inspection cycle and
$P_{j,i}$ = probability that a bridge with condition rating i in current state will have condition rating j at next inspection.

Equation 2 shows a Markov chain that is typical of those for element-level BMSs. Condition State 1 is perfect, and higher rating numbers indicate increasing deterioration.

Discrete-state/discrete-time Markov chains are a natural choice for modeling discrete valued ratings that are collected at regular intervals. Discrete-state/discrete-time Markov chains require few parameters. Typically, Markov chains for condition ratings require a number of independent transition probabilities equal to the number of condition rating values. This is because it is observed that condition ratings change by not more than a single rating value in one inspection cycle (*17*). The transition probabilities are computed directly from inspection data as the number of bridges or elements that change from their present condition rating to the next higher rating in one inspection cycle. The transition probabilities and the model are tied directly to the observed performance of the network of bridges.

In comparison with functional forms of models for condition ratings, Markov chains do not explicitly indicate the dependence of condition ratings on factors such as bridge age or level of traffic. Instead, differences in traffic or type of structure or exposure or any other parameter that may influence the rate of change in condition ratings are handled by a multiplicity of Markov chains. For each unique element or environment, a separate Markov chain is formed. The overall model for condition ratings then is a collection of Markov chains, each independent and each dedicated to a particular combination of parameters. Using Markov chains BMSs directly address the problem of variable deterioration rates by employing separate models (chains) where required and by calibrating all models against the observed performance of the network of bridges. Variability in rates and condition is simply accepted and absorbed.

Quantitative Reporting and Modeling of Deterioration

Discrete Markov chains of the form used for condition ratings can be used in quantitative modeling of deterioration processes. Discrete-time chains naturally fit the regular intervals of data gathering that are standard practice for bridge inspections. The use of discrete-state

chains requires that observed deterioration be recorded as ratings that define ranges of deterioration severity—that is, a discrete-state Markov chain can indicate the probability that a bridge or bridge element is within a particular range of a quantitative measure of deterioration. For steel beams subject to corrosion, a rating scale defined in terms of a damage index based on the remaining thickness of a web plate has been proposed (*18*). Quantitative condition ratings for steel bridge beams are defined in terms of a damage index that is computed as a normalized thickness loss in the web plate as

$$\mathrm{DI} = \frac{t_w - t_{wo}}{t_{wo}} \tag{3}$$

where

DI = damage index
t_w = current thickness of web plate of beam (as observed/estimated in most recent inspection of the bridge), and
t_{wo} = original thickness of web plate

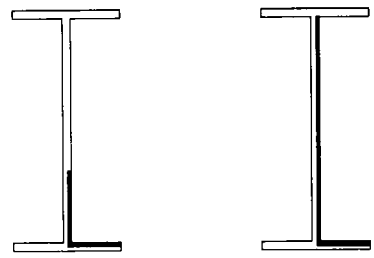
The condition rating scale is given in Table 1 and illustrated in Figure 2.

Quantitative condition ratings are used to form Markov chains in the same manner as present-day qualitative condition ratings. These Markov chains, in turn, allow the computation of parameters of models of deterioration process. This is illustrated for the *A* and *B* parameters of the power law model for corrosion of steel. A Markov chain for quantitative condition ratings defined for corrosion penetration is used to compute mean time to transitions to each of the four condition states 2 to 5. The mean time for transition from Condition Rating 1 to a deteriorated condition in rating *k* is

$$t_k = \sum_{1}^{\infty} P_{k,k-1}\{C_{k-1}\}_n \times n \tag{4}$$

TABLE 1 Normalized Remaining Strengths in Condition States

Condition State	Pattern *A*		Pattern *C*	
	M/Mo	*V/Vo*	*M/Mo*	*V/Vo*
1	1.0	1.0	1.0	1.0
2	0.98	0.97	0.98	0.92
3	0.95	0.92	0.95	0.77
4	0.92	0.87	0.91	0.62
5	0.89	0.82	0.87	0.34



$$DI_w = \frac{t_{wo} - t_w}{t_{wo}}$$

Condition State	DI_w
1	0 - .05
2	.05 - .15
3	.15 - .30
4	.30 - .45
5	.45 - .60

FIGURE 2 Corrosion patterns and quantitative condition ratings for steel beams.

$$\begin{Bmatrix} C_1 \\ C_2 \\ C_3 \\ C_4 \\ C_5 \end{Bmatrix}_n = \begin{bmatrix} P_{1,1} & & & & \\ P_{2,1} & P_{2,2} & & & \\ & P_{3,2} & P_{3,3} & & \\ & & P_{4,3} & P_{4,4} & \\ & & & P_{5,4} & P_{5,5} \end{bmatrix}^n \begin{Bmatrix} 1 \\ 0 \\ 0 \\ 0 \\ 0 \end{Bmatrix}_n$$

where t_k is the mean time for transition to condition rating *k*, $[P]^n$ indicates the repeated application of the transition probabilities, and the initial vector of condition ratings has all of the bridges, or elements, in Condition Rating 1. The mean times to transition for the condition states each correspond to the values of damage index that are the limits of the condition ratings. The computation of mean transition times yields pairs of (time, damage index) data points that are used in a normal log-space regression to form a power law model for the corrosion process that underlies the Markov chain (*19*).

Segment-Based Condition Reporting

Rates of deterioration respond to the exposure of the beam, and there is a need to form several Markov chains to handle various severities of environment and exposure and several rates of corrosion. To recognize differences in exposure for beams on the same bridge and to achieve independent calibration of models for different exposures, it is necessary to link condition rat-

ings to specific portions of bridge beams. To achieve this, a process of segment-based condition reporting has been developed. In this process each bridge element is represented as a set of segments.

Segments are bounded by physical features of a bridge. Bridge beams are represented as segments between diaphragms. In Figure 3, a schematic plan of a single-span beam bridge is shown. The seven beams and four lines of intermediate diaphragms yield 35 segments for beam elements. The segments collectively form a model of the bridge that is useful both for focused modeling of deterioration processes and for estimation of load capacity. The bridge shown in Figure 3 is used in an example of load capacity estimation later in this paper.

In field inspections, two condition ratings are assigned to each beam segment (each end of the segments rated). The field report is accomplished on a graphic reporting form that indicates the segment boundaries and the locations of ratings along with a terse form of the condition rating scale for reference. The assignment of ratings for individual segments entails many ratings for a bridge, but each rating is individually simple to assign. Inspectors can focus on a limited length of a single bridge beam and can report the ratings that correspond to their observations. This procedure is in contrast to NBI rating, in which the inspector must execute a form of mental averaging to assign a single rating for the superstructure (the item that the bridge beams would affect).

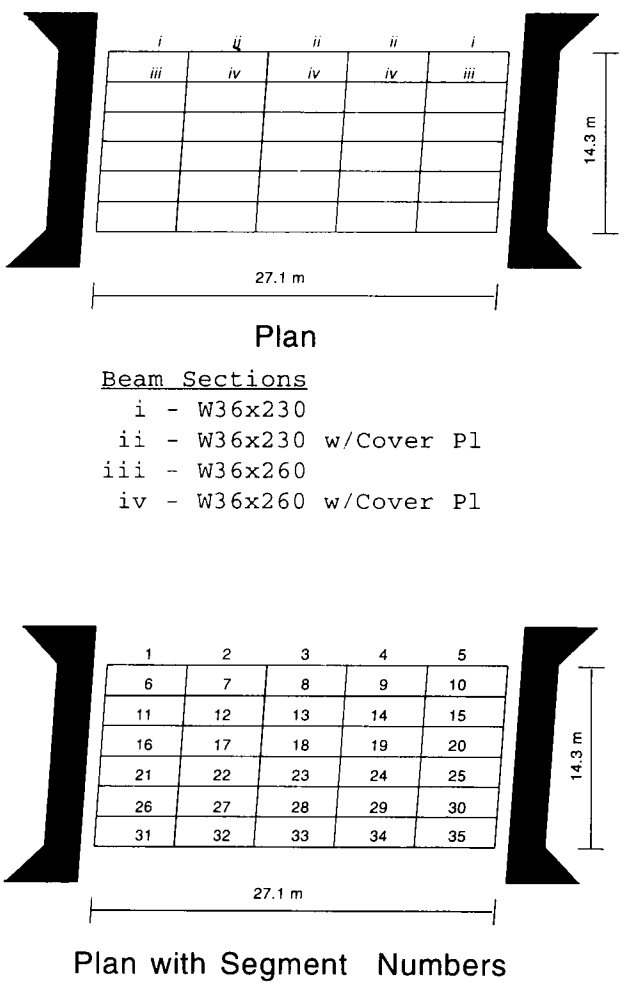


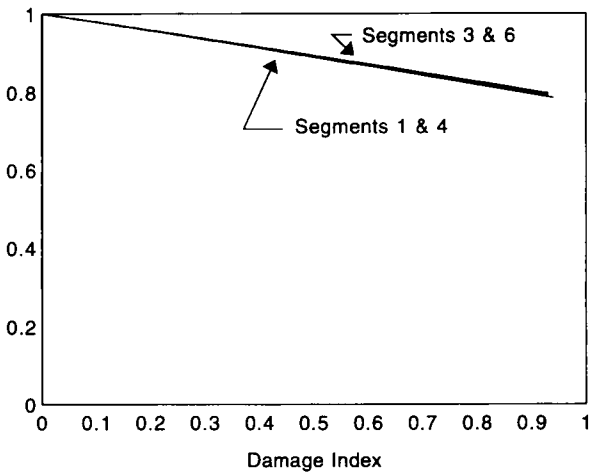
FIGURE 3 Bridge plan and segment model.

Segment-based reporting is also different from element quantity reporting required by new BMSs. The element quantities needed by BMS might require the measurement of quantities in the field, but in segment-based reporting the known quantities of individual segments may be summed in the various condition states as a part of the data processing that occurs in the office, after an inspection. Segment-based reporting for field inspections is the subject of a demonstration project being conducted by the University of Colorado and the Colorado Department of Transportation. Eight bridges, including steel, reinforced concrete, prestressed concrete, and timber bridges, are being inspected using a segment-based approach. This work will be reported separately.

Estimating Load Capacity

Quantitative condition ratings permit an estimation of the remaining strength of beams and of the load capacity of the bridge. To estimate remaining strength, both the severity of corrosion and the pattern of corrosion in the cross section must be known. The quantitative condition rating scale does not indicate the pattern of corrosion. The pattern must be reported separately as one of several characteristic patterns, or it may be assumed to be a characteristic pattern according to its position on the beam (*1*). Two common corrosion patterns for steel beams are shown in Figure 2. In Figure 4, normalized remaining flexural and shear strengths as a function of damage index are shown for four cross sections of bridge beams. The flexural strength plots are created for corrosion in Pattern *A*. Remaining shear strength has been created for corrosion in Pattern *B*. The close agreement of normalized remaining strength among different cross sections is a common finding for steel bridge beams.

Segment-based condition reporting links remaining strength with load demand, since load demand is a function of position within a bridge. The information provided by quantitative condition states and by segment-based reporting is enough to allow the estimation of load capacity of bridges by familiar means. Clearly, several choices are available here. The standard form of inventory rating for beams, with load demand com-



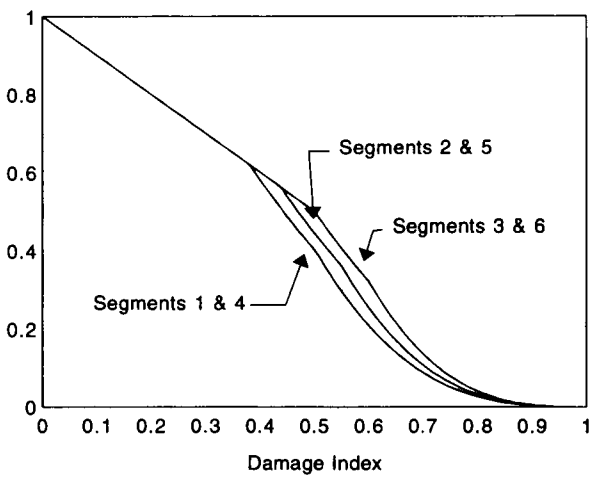


FIGURE 4 Beam strength versus damage index: *top*, flexure strength, M/M_o, Pattern *A*; *bottom*, shear strength, V/V_o, Pattern *C*.

puted on the basis of distribution factors, may be executed, and this is the method used in the example that follows. But other forms of load rating may be employed. Distribution factors may be eliminated in favor of a full three-dimensional analysis of load effects in the structure. Deterministic load effects may be replaced by an estimate of mean maximum load effects during a specified time interval (*20*, p. 1538), or load effects may be obtained from models that are either identified from field test data or calibrated against field test (*21*). In any of these approaches, the live load capacity of a bridge may be computed as

$$M_{LL+I} \leq M_n - M_{DL}$$
$$V_{LL+I} \leq V_n - V_{DL} \quad (5)$$

where

M_{LL+I}, V_{LL+I} = available flexure and shear strength, respectively, for live load and impact
M_n, V_n = current (present-day) strength of bridge beams, and
M_{DL}, V_{DL} = flexure and shear dead load demand, respectively.

Using data on strength and load demand for individual beam segments, the quantities for live load capacity, segment strength, and dead load demand exist as a finite set of values $M_{LL+I,i}$, $M_{n,i}$, and so on, where i indicates the segment number. Live load capacity is then computed for each segment individually as

$$M_{LL+I,i} \leq M_{n,i} - M_{DL,i}$$
$$V_{LL+I,i} \leq V_{n,i} - V_{DL,i} \quad (6)$$

where all quantities refer to individual segments. The lowest live load capacity among all segments is taken as the live load rating of the bridge.

The estimates of live load capacity shown here are estimates for known conditions of segments. They are point-in-time estimates. Estimates at other times, future times specifically, are computed with new estimates of remaining strength in segments. These new estimates are obtained from either the Markov chains of condition ratings or the models of processes of deterioration derived from Markov chains. The use of Markov chains and discrete-valued condition ratings of course leads to discrete values of segment strength and a stepwise alteration (decrease) in load rating with continuing deterioration in members.

EXAMPLE

The use of quantitative condition ratings, segment-based reporting, and deterioration models calibrated from condition ratings is illustrated in an example. A simple single-span steel beam bridge is shown in Figure 3. The span of the bridge is 26.8 m (88 ft), and the bridge has seven beams spaced at 2.4 m (7 ft 10 in.) on center. The bridge carries two lanes of traffic and a breakdown lane. Exterior beams are W36×230 with partial-length cover plates. Interior beams are W36×260, also with partial-length cover plates. The beams are composite with the 229-mm (9-in.) reinforced concrete deck. For this bridge without deterioration the inventory rating is controlled by the flexural strength of the exterior beams. In the segment model of this bridge (Figure 3), Segment 3 initially controls the live load rating.

Deterioration of the steel beams by corrosion will lessen the live load rating and could change the con-

trolling segment. For investigating this, three corrosion severities are assumed and their effects on the load rating of the bridge are examined. The three severities are termed benign, moderate, and severe. The power law models for these severities are given in Table 2, as are the transition probabilities that would be obtained from populations of bridge beams that experience corrosion damage according to these power laws. The transition probabilities have been determined through a simulation of aging in a population of bridges.

Benign, moderate, and severe corrosion severities are used with Corrosion Pattern *A* to examine the effect of overall environment on the load capacity of this bridge. In addition the severe corrosion model is used with Pattern C to simulate a special adverse local condition such as the failure of a deck joint and the resulting rapid corrosion of the ends of beams. Four cases are considered. They differ by rate of corrosion, corrosion pattern in the cross section of beams, and relative rates of corrosion for different regions of the bridge. The cases are

1. Moderate corrosion in Pattern *A* at all segments,
2. Severe corrosion in Pattern *A* at all segments,
3. Benign corrosion in Pattern *A* at the middle portions of beams combined with moderate corrosion at the ends of beams, and
4. Severe corrosion in Pattern C at beam ends combined with moderate corrosion in Pattern *A* on the middle portions of beams.

The cases are defined in Figure 5. In the figure, four cases are listed, the severity and pattern of corrosion are noted, and the application of corrosion models to three zones of the superstructure is identified. The zones recognize end, middle, and intermediate regions of beams. Additional zones might be identified. For example, interior beams and exterior beams could be considered as separate zones.

The influence of corrosion is examined by using the power law models to assess an exact corrosion penetration, by evaluating the remaining strength of segments for this corrosion, and by computing a live load rating for the controlling segment. In addition, the Markov models are used to compute condition ratings, and the normalized remaining strengths of segments in condition states are used to estimate segment strength and to estimate live load rating. The results of live load rating using an exact value of corrosion penetration are compared with the results obtained as estimates from condition ratings.

TABLE 2 Assumed Power Law Models and Derived Markov Chains for Corrosion of Steel Bridge Beams

Corrosion Severity	Power Laws		Transition Probabilities				
	A (mm)	B	$P_{1,1}$	$P_{2,2}$	$P_{3,3}$	$P_{4,4}$	$P_{5,5}$
Benign	0.075	0.75	0.884	0.962	0.970	0.974	0.976
Moderate	0.171	0.75	0.718	0.895	0.915	0.925	0.931
Severe	0.390	0.75	0.502	0.727	0.782	0.804	0.819

Case 1

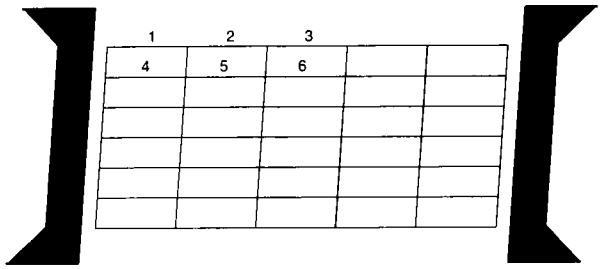
Corrosion at one rate in all beams leads to nearly equal losses in normalized strength in all beams—losses of 8 percent of the beams' original flexural strength and 13 percent of the beams' original shear strength over the life of the bridge. Overall, the loss in inventory rating for the bridge is about 4 HS rating numbers, or approximately one-sixth of the initial rating of the bridge. The inventory rating is controlled by the flexural capacity of the exterior beam throughout this simulated service life. The controlling segment is Segment 3. Since this uniform corrosion process yields the same normalized losses of strength in all members, there is no evolution of the critical segment.

The inventory ratings for Case 1 are shown in Figure 6. The upper plot shows the inventory ratings for each segment as a function of time. The upper bound on the plot is the original inventory rating of the bridge. As the corrosion advances it is seen that there are decreases in the inventory rating associated with Segments 2 and 3. Both of these are segments in the exterior beam, and both are suffering losses in flexural strength that limit the inventory rating of the bridge. Segments 1, 4, 5, and 6 suffer losses of flexural and shear strength, and indeed are subject to the same corrosion process, but their losses relative to their load demand do not cause any reduction in inventory rating over a 100-year service life.

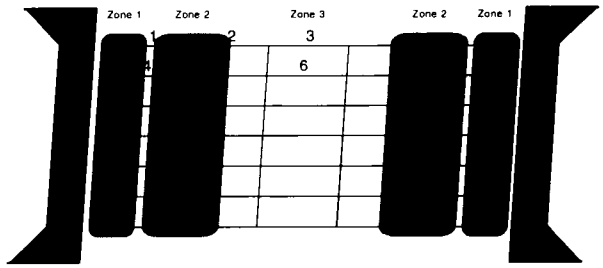
The application of segment condition ratings is shown in the lower plot. Each condition state corresponds to a range of corrosion losses, and therefore each condition state spans a range of values of flexural and shear strength. However, each condition provides a single value of normalized remaining strength. Here it is seen that the inventory ratings from condition state data decrease in a stepwise pattern as the controlling Segment 3 passes through States 1, 2, 3, and 4. The inventory ratings can be either over- or underestimates since the normalized remaining strength values are taken as the average strength in condition states. For this case, load ratings from condition states are within 1 HS number of the correct ratings.

Case 2

Case 2 considers a faster rate of corrosion for all bridge beams. The results are similar to those of Case 1, but



Schematic Plan with Segment Numbers



Zones for Corrosion Environment & Pattern

Case	Zone 1	Zone 2	Zone 3
1	Moderate-A	Moderate-A	Moderate-A
2	Severe-A	Severe-A	Severe-A
3	Severe-A	Moderate-A	Benign-A
4	Severe-C	Severe-A	Severe-A

FIGURE 5 Cases for corrosion example.

losses of strength are more severe. The decrease in inventory rating is about 10 HS numbers, or 40 percent of the original strength of the bridge. The controlling segment is again Segment 3, and it is the flexural strength of this segment that limits the inventory rating of the bridge. The change in inventory rating over time for Case 2 is shown in Figure 7. Again, the upper plot shows the inventory ratings associated with various segments as a function of time. And again, only Segments 2 and 3 suffer losses of strength sufficient to alter the inventory rating. The loss of strength over time is approximately linear. The errors in stepwise inventory ratings obtained with condition ratings for elements differ from the correct results by 1 HS number. Notice that the inventory ratings derived from condition states stop at about 50 years. Here, corrosion losses are already severe, and the condition of the bridge is such that a more thorough review of the structure is warranted. Automated ratings, presented here, are no longer adequate as the sole measure of strength once damage has become this severe.

Case 3

Case 3 considers two rates of corrosion: slow corrosion in the middle of beams, and moderate corrosion at other locations. For these rates, there is an evolution in the controlling segment for inventory rating. For the first 70 years of service, the live load rating of the bridge is controlled by Segment 3. Beyond this time, corrosion losses in Segment 2 are so great that this segment comes to control the live load rating despite the lower load demand in this segment. The plots for live load ratings are shown in Figure 8. Live load ratings derived from condition states again yield reasonable estimates, and it is observed that the condition rating model (the Markov

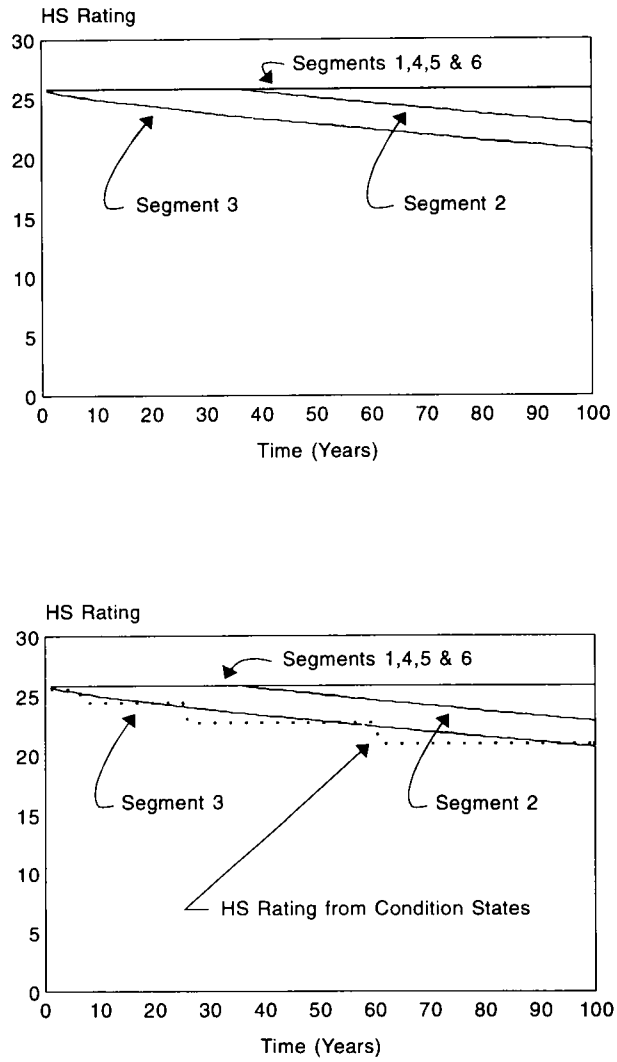


FIGURE 6 Case 1: moderate corrosion, inventory ratings.

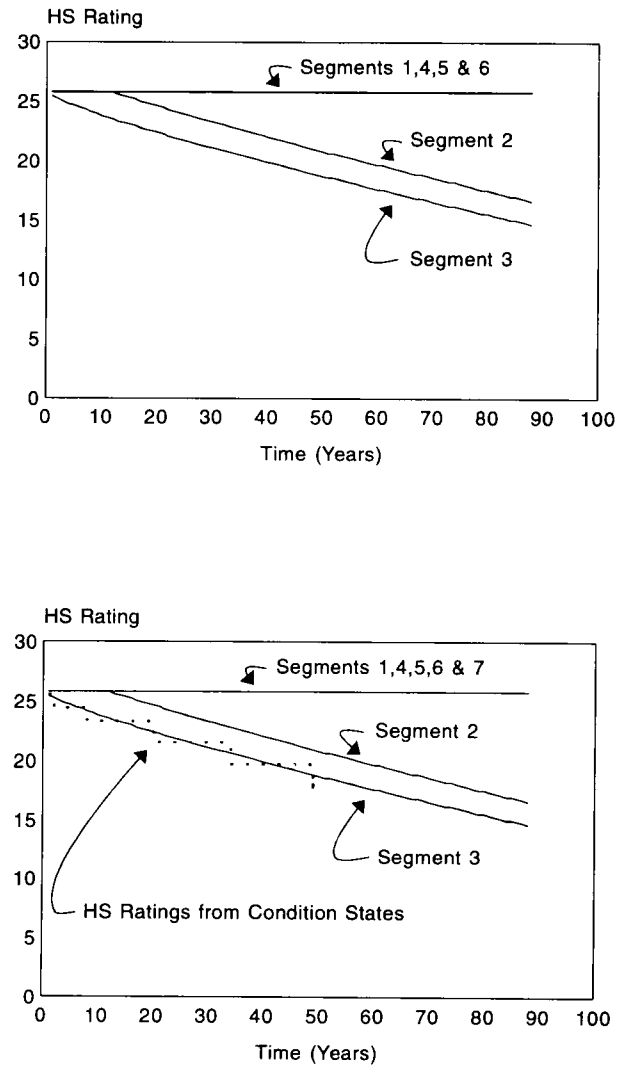


FIGURE 7 Case 2: severe corrosion, inventory ratings.

chain) recognizes the evolution of the controlling segment due to unequal corrosion rates.

Case 4

Case 4 considers a different, and more damaging, pattern of corrosion at the ends of beams. This case simulates a special, adverse condition such as the failure of a deck joint that produces a locally severe environment. The early part of the service life of the bridge is again controlled by the flexural strength of Segment 3. But accumulating damage in Segments 1 and 4 leads to severe and rapid losses in live load rating. This is a loss of shear strength in the ends of the beams due to corrosion attack on the entire height of the web. Figure 9 shows the plots of live load rating versus time; the rapid loss of shear strength is seen as the dashed lines for Segments 1 and 4. The lower plot shows the live load ratings obtained from condition ratings. Again, these estimates follow the correct results. Errors in estimate ratings can be high since the bridge is undergoing large and comparatively rapid change in load capacity. Estimates from condition ratings are generally conservative, however. The only large overestimate of live load capacity is encountered after the bridge is severely impaired. This is in a range of conditions where automated estimates should not be relied on as the sole measure of bridge strength.

Conclusions

Deterioration rates vary, and calibration is a necessary task in the application and use of deterioration models.

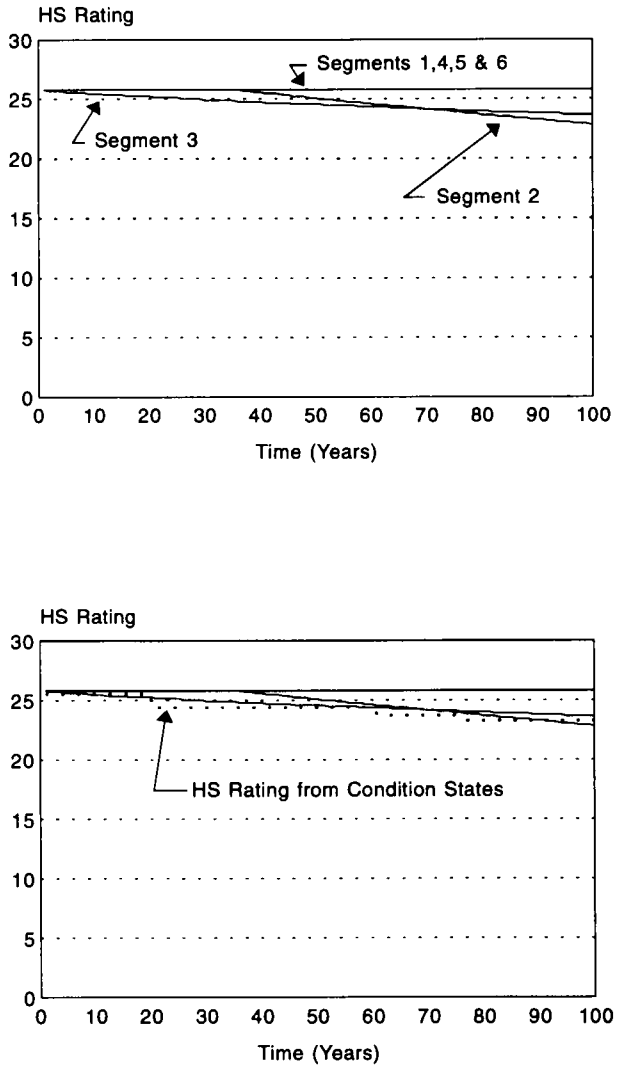


FIGURE 8 Case 3: moderate and benign corrosion, inventory ratings.

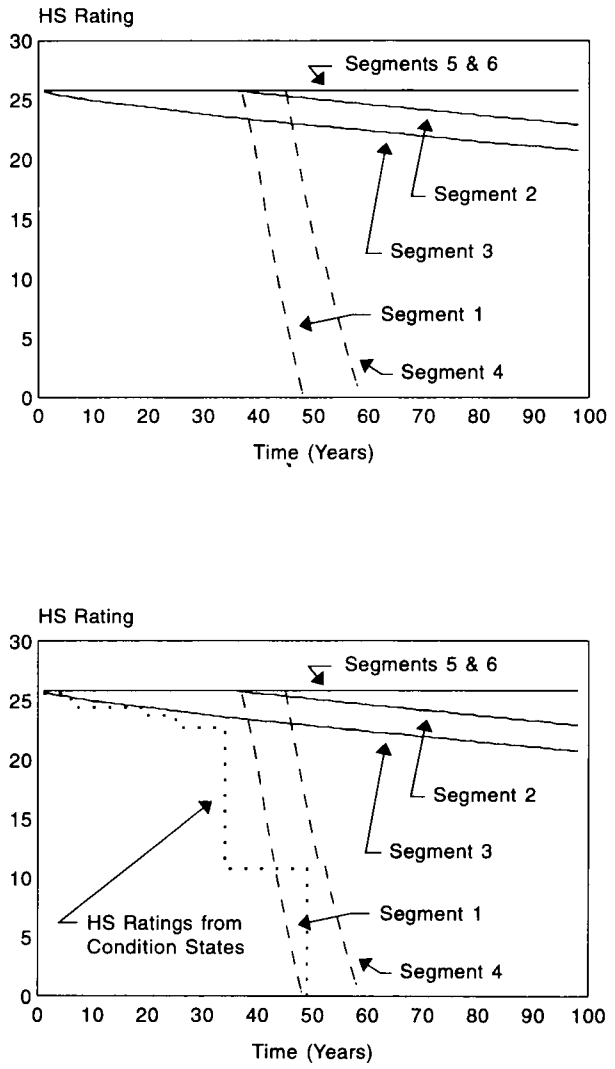


FIGURE 9 Case 4: severe and moderate corrosion, inventory ratings.

Calibrating deterioration models against the performance of bridges in service is desired but not easily achieved for standard forms of deterioration process models. The basic data on bridges and their conditions in service are provided by inspections, but such data often are not adequate to support calibration of deterioration models. In the United States, three condition ratings are the only formal data on the condition of a bridge that are routinely collected. Condition ratings are not quantitative data; condition ratings do not indicate the type or severity of deterioration observed on a bridge.

Condition ratings themselves can be modeled, and many BMSs employ models of ratings in planning and optimization procedures for maintenance programs. Management systems rely on correlations of condition ratings with repair actions to assess total costs of repairs in a network of bridges. Management systems exploit forms of models that are calibrated easily against the historical record of condition ratings and updated easily with new data in each inspection cycle. Modeling and updating procedures available in BMSs offer much of the capability that is desired in the formation and maintenance of models of deterioration processes for highway bridges. The limitation is in the condition ratings themselves.

New quantitative condition ratings may be defined in terms of damage indexes for bridge members. New condition ratings employ the familiar integer-scale form of condition ratings currently in use. The quantitative definitions of new condition ratings make them useful in calibrating deterioration models and in evaluating bridge strength and safety. In addition, quantitative condition ratings allow BMSs to consider quantitative

safety-related impacts of decisions in maintenance programs. This is a new and important extension of existing functions of management systems.

In this paper, examples of quantitative condition rating are presented along with a protocol for condition recording that links ratings to specific elements in a bridge. Procedures for the calibration of deterioration models are presented, and the use of quantitative condition rating in load capacity evaluations is illustrated. It is found that live load rating estimates from quantitative condition ratings are accurate and that the evolution of the controlling segment, when it occurs, is recognized by models for condition ratings.

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