DEVELOPMENT OF A DETERIORATION MODEL TO PROJECT FUTURE CONCRETE REINFORCEMENT CORROSION IN A DUAL MARINE BRIDGE

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ABSTRACT

Two 4.1-km-long (2.5-mi-long), 31-year-old parallel bridges in Northern Florida marine service were examined to assess and forecast the extent of concrete reinforcement corrosion. A preliminary inspection showed that the chloride concentration at the depth of the reinforcement in the cylindrical piling was approaching the level normally associated with the onset of corrosion. Future traffic projections required deciding between alternatives that included expanding the present structures or rebuilding. To select the most appropriate alternative, an investigation was conducted to develop an approximate forecast of future corrosion development. The investigation included assessing the present condition, and developing a quantitative corrosion deterioration model. The corrosion condition was assessed by visual observation, direct examination of reinforcement, and electrochemical corrosion measurements. Chloride-penetration profiles were obtained from extracted concrete cores. Reinforcement cover was measured by direct observation. The chloride profile data were analyzed to obtain apparent chloride ion diffusivities, surface concentrations and bulk concentrations. The deterioration model used the statistical distributions of concrete cover, diffusion coefficient and surface concentration to estimate the distribution of time for corrosion initiation and appearance of external damage over the bridge substructure. The output of the model was a damage function indicating the amount and location of repairs needed as a function of bridge age.

INTRODUCTION

The parallel twin Escambia Bay bridges were built in 1966 to span Escambia Bay near Pensacola, Florida. The water in contact with the bridges has a variable chloride content, with concentrations exceeding 10,000 ppm at times. In 1996 the Florida Department of Transportation (FDOT) began reviewing alternatives for upgrading these bridges, including widening or replacing the bridges. To assist in deciding between alternatives, an assessment of the corrosion condition of the bridges and the development of a quantitative model of future deterioration were commissioned. This paper focuses on the modeling approach used to forecast corrosion-induced deterioration in the bridges.

Each bridge is 4.1-km-long (2.5-mi-long), with 223 substructure bents. The bents in the higher elevations of the bridges are comprised of 268, 1.37-m (4.5-ft) diameter Raymond piles [1] which are connected by cast-in-place struts above the high tide elevation. Smaller diameter 0.91-m (3-ft) Raymond piles (1,218 in water), with no struts, support the lower elevations of the bridges. The bents at the channel spans consist of 4 crash walls with square columns.

Evaluation of the bridges included a comprehensive condition survey which consisted of visual observation, direct examination of reinforcement, electrochemical corrosion measurements, concrete cover measurements, and determination of chloride ion penetration profiles [2]. The cast-in-place struts showed evidence of ongoing corrosion deterioration and replacement of the struts is foreseen as part of any of the alternatives being considered. The crash walls showed no obvious signs of corrosion damage and no delaminations were detected, although extensive cracks had been epoxy-injected in the past. However, because of their relatively small number, the crash walls represented only a minor potential cost fraction in future maintenance/repair schemes. The following addresses only the durability projections for the piles.

No corrosion-induced damage or deterioration of the 31-year-old round piles themselves was identified. The clear concrete cover of the piles (average of 2.84 cm (1.12 in) for 47 test spots and 2.64 cm (1.04 in) for 14 test spots in the 0.91-m (3-ft) and 1.37-m (4.5-ft) piles, respectively) corresponded to the spiral stirrup wire wrapped around the longitudinal prestressed cables. The cover measurements were performed by direct observation in drilled holes.

For electrochemical and chloride penetration assessment, tests were performed at three elevations corresponding to the tidal zone (TZ), about 0.15 m (0.5 ft) below high tide elevation (-0.15 m (-0.5 ft) above high tide, AHT), the upper splash zone (US), about 0.75 m (2.5 ft) AHT for the 0.91-m (3-ft) piles and 1.2-m (4-ft) AHT for the 1.37-m (4.5-ft) piles, and the above-splash zone (AS), about 1.5 m (5 ft) AHT.

Nominal corrosion rate measurements were made with a Gecor 6 test device at 41 pile locations selected to minimize sampling bias and representing the three elevation ranges indicated above. Average nominal corrosion current densities for both types of pile in the TZ, US and AS zones were in the range normally associated with low or negligible corrosion rates of steel in concrete [3]. Chloride concentration profiles were obtained successfully at 17 unbiased sampling locations in the three elevation regimes indicated above. The results, discussed in detail below, indicated that at the time of the survey the chloride concentrations in the US and AS zones at the depth of the stirrup wire were below (but near), and in the TZ were above, the range of values normally associated with the onset of active corrosion of steel in concrete [3].

The results of the survey indicated that the corrosion condition of the piles was very good, especially considering the high Cl^- content of the water, the bridge age, and the low concrete cover thickness. However, the chloride profile results suggested that corrosion initiation had possibly already started in the TZ and was likely in the near future for the US and AS zones. To conduct a quantitative estimate of future corrosion development, a projection model was formulated using

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the initiation-propagation concept first established by Tuutti [4]. In the corrosion initiation stage, chloride concentration builds up at the rebar surface but has not yet reached the threshold value C_T needed to trigger the onset of active corrosion. The corrosion propagation stage starts when the concentration exceeds C_T at time tb and continues for an additional time period tp until external damage is observed and repair need ensues. The model also took into account the random variability in concrete cover [5] and in chloride transport properties existing across the structure, and systematic variability of transport properties with elevation. The output of the model consisted of an estimate of the amount of substructure surface requiring repair as a function of structure age.

DEFINITIONS AND ASSUMPTIONS

1. This model addresses only round piles. The bridge pile substructure is divided into three elevation ranges designated by i=1(tidal, T); i=2 (lower splash, LS) and i=3 (upper splash and above-splash, US-AS). The LS range was introduced as an artificial intermediate range of average properties between those of the T and the LS-US ranges, to address a region of possible early deterioration. The minority of piles with surrounding struts is treated conservatively as if the struts offered no resistance to chloride penetration.

2. Each elevation range has Ni surface elements (1,2,...,j,...Ni) of equal area Ae. Ni includes the elements of both twin bridges. The concrete properties, steel positioning and exposure parameters for all piles are, for simplicity, taken to be the same within each elevation range.

3. Each element j in range i has a concrete rebar cover $cc_{i,j}$ and a chloride surface concentration $Cs_{i,j}$ that are invariant within the element.

4. Chloride ions move by near-flat geometry Fickian diffusion. The chloride concentration threshold C_{Ti} is the same everywhere within each elevation range. Each element j in range i has an apparent chloride ion diffusion coefficient D_{ij} invariant within the element. The concrete surface chloride concentration $Cs_{i,j}$ at each element is invariant with time. The native chloride content of the bulk concrete is the same throughout the bridge and negligibly small for the purposes of this model. The distributions of surface concentration, apparent diffusivity and concrete cover thickness are uncorrelated.

5. The number of elements at elevation range i that have concrete cover $cc_{i,j} \leq x$ will be called $N'c_i(x)$. The corresponding cumulative distribution function is defined as $Nc_i (x) = N'c_i(x) / N_i$.

6. The number of elements at elevation range i that have surface concentration $Cs_{i,j} \leq Cs$ will be called $N'cs_i (Cs)$. The cumulative distribution is $Ncs_i (Cs) = N'cs_i(Cs) / N_i$. The corresponding probability distribution is $Pcs_i (Cs) = dNcs_i / dCs$ (a continuum approximation is used in this and similar subsequent definitions).

7. The number of elements at elevation range i that have $D_{i,j} \leq D$ will be called $N'd_i$ (D). The corresponding distribution functions are $Nd_i (D) = N'd_i(D) / N_i$ and $Pd_i (D) = dNd_i / dD$.

8. The chloride concentration C in the concrete is a function of the element location, distance $x_{i,j}$ from the surface, and time t since the structure was placed in the service environment. The concentration is thus designated as $C(x_{i,j}, t)$. Corrosion at element i,j begins at the moment $tb_{i,j}$ when:

$$C(cc_{i,j}, tb_{i,j}) = C_{Ti} \qquad [1]$$

9. Damage (cracking or spalling) of element i,j occurs at moment

$$ts_{i,j} = tb_{i,j} + tp_i \qquad [2]$$

where tp_i is the corrosion propagation time, same everywhere within each elevation range. Damage affects the entire surface element area Ae (any repairs must treat the entire element). This model addresses only first damage and not damage after repairs.

10. At moment t, all elements satisfying

$$ts_{i,j} < t \qquad [3]$$

will have experienced damage. At elevation range i, the number of those elements will be called $N's_i$ (t). The total number of damaged elements at moment t will be called

$$Ns(t) = \sum_i N's_i (t) \qquad [4]$$

PREDICTIVE MODEL

Per assumption in item 4 [6],

$$C(cc_{i,j}, t) = Cs_{i,j} (1\text{-erf}(cc_{i,j} / 2 (D_{ij}\, t)^{1/2})) \qquad [5]$$

Therefore:

$$tb_{i,j} = cc_{i,j}^2 [\text{erf}^{-1} (1\text{-}C_{Ti} / Cs_{i,j})]^{-2} / 4 D_{ij} \qquad [6]$$

Defining the function

$$T_i (Cs) = [\text{erf}^{-1} (1\text{-}C_{Ti} /Cs)]^{-2}$$

[7]

Then from Eq.[6]:

$$tb_{i,j} = T_i (Cs_{i,j}) cc_{i,j}^2 / 4 D_{i,j} \qquad [8]$$

And from Eq.[2]:

$$ts_{i,j} = T_i (Cs_{i,j}) cc_{i,j}^2 / 4 D_{i,j} + tp_i$$

[9]

Therefore, from Eq.[9], at time $t > tp_i$ any element satisfying

$$cc_{i,j} < | (4 D_{i,j} (t-tp_i)/T_i (Cs_{i,j}))^{1/2} | \qquad [10]$$

will have experienced damage.

Therefore, at elevation range i, of the elements satisfying both

$$Cs < Cs_{i,j} < Cs + dCs \qquad [11a]$$

and

$$D < D_{i,j} < D + dD \qquad [11b]$$

the following fraction (per items 5 and 8, and Eq.[10]) will have experienced damage by time $t > tp_i$

$$fcs_i = Nc_i (| [4 D (t-tp_i) / T_i (Cs)]^{1/2} |) \qquad [12]$$

Per Items 5 and 6, the number of elements satisfying both Eq.[11a] and Eq.[11b] is given by $N_i \; Pcs_i (Cs) \, Pd_i (D) \, dCs \, dD$. The number $dN's_i$ of damaged elements with surface concentration in the dCs interval around Cs *and* diffusivity in the dD interval around D is therefore:

$$dN's_i = N_i \quad Pcs_i (Cs) \, Pd_i (D) \, Nc_i (|[4 D (t-tp_i) / T_i (Cs_{i,j})]^{1/2}|) \, dCs \, dD \qquad [13]$$

Thus the total number of damaged elements $N's_i(t)$ in elevation range i at time $t > tp_i$ is given by:

$$N's_i(t) = N_i \int_{Dli}^{Dhi} \int_{Csli}^{Cshi} Pcs_i (Cs) \, Pd_i(D) \quad Nc_i (|[4 D (t-tp_i) / T_i (Cs)]^{1/2}|) \, dCs \, dD \qquad [14]$$

where Dli, Csli and Dhi, Cshi represent the lowest and highest values respectively of D and Cs in elevation range i. For times $t \le tp_i$, $N's_i(t) = 0$.

The number of damaged elements N's(t) for the entire bridge by time t is given by:

$$N's(t) = \sum_i N's_i\,(t) \qquad [15]$$

The total damaged surface area S(t) in the substructure at age t is then:

$$S(t) = N's(t)\,Ae \qquad [16]$$

IMPLEMENTATION

Model Input

The following input parameters are required for each elevation range i (except for Ae, which is global)

a) Ae — Chosen to represent the area covered in a typical repair patch.

b) N_i — From Ae, pile dimensions, number of piles of each type and elevation range limits.

c) C_{Ti} — Chosen to be representative of the concrete and rebar conditions.

d) $Pcs_i\,(Cs)$
$Pd_i\,(D)$
$Nc_i\,(x)$ — Distribution functions obtained by fitting the measured populations to ideal normal distributions, each with an average and a standard deviation.

e) tp_i — Chosen to be consistent with other marine substructure experience and corrosion rate measurement results.

Computation Method

Once the input parameters were available, the calculations were conducted with a MATHCAD worksheet. The distribution functions were calculated first. Equation [14] was implemented as a double summation with (typically) 20 terms per summation. Additional terms yielded only minor changes in output while extending computation time considerably. The program output was the extent of damaged area (m^2 added for the two bridges) at each of the three elevation ranges as a function of time since construction. Results were presented for intervals of (typ.) 5 years, up to a service life approaching 100 years.

Selection/Calculation of Input Variables

a) Ae: Chosen to be 0.1 m^2 (1.1 ft^2), representing typical expected required patch sizes.

b) N_i: Based on available information, the number of elements was computed as shown in Table 1. The T (i=1) elevation range extended from the high tide (HT) level to 0.45 m (1.5 ft) below, reflecting the typical tidal variation in Escambia Bay. The LS (i=2) elevation range extended from HT to 0.3 m (1.0 ft) above high tide (AHT). The US+AS (i=3) range was from 0.3 m (1.0 ft) AHT to 1.8 m (6.0 ft) AHT. Elevations higher than 1.8 m (6.0 ft) AHT were assumed to result in negligible corrosion development in the time frame of interest.

Table 1

Piles	Number in water	Perimeter (m)	Range Height (m)			Range Area, Both Bridges (m^2)		
			T i=1	LS i=2	US+AS i=3	T i=1	LS i=2	US+AS i=3
0.91-m	1218	2.87	0.45	0.3	1.5	1573	1049	5243
1.37-m	268	4.31	0.45	0.3	1.5	520	347	1733
			Both Piles (m^2) :			2093	1395	6976
			Number of elements for Ae = 0.1 m^2, Both Piles (N_i) :			20928	13952	69761

c) C_{Ti}: Assumed to be M • CF at all three elevation ranges. CF is the cement factor of the concrete used in the piles. M is a multiplier which is often assumed to be 0.004 for design purposes [7]. However, because of uncertainty in this parameter for marine substructures in Florida, three alternative cases A, B, and C were evaluated with M= 0.004, 0.008 and 0.012, respectively. Using CF = 400 kg/m^3 (674 pcy) resulted in C_{Ti} = 1.6 kg/m^3 (2.7 pcy), 3.2 kg/m^3 (5.4 pcy), and 5.8 kg/m^3 (9.8 pcy). Within each alternative projection, C_{Ti} is the same for i=1, 2, and 3.

d) Cs and D distribution functions: A concrete unit weight of 2,547 kg/m^3 (4,290 pcy) was used to convert chloride concentrations from percent by weight of concrete to kg/m^3 (pcy). Based on measurements at depths of ≈10 cm (4 in), the native chloride content was assumed to be ≈0.12 kg/m^3 (0.2 pcy). Figure 1 shows the values of D and Cs as a function of elevation obtained by analysis of 17 extracted cores from both types of piles. There was no significant evidence of different trends for the 0.91-m (3-ft) and 1.37-m (4.5-ft) piles. Both Cs and D tended to be higher in the T range than in the US and AS ranges. The results of the two latter ranges were not clearly differentiated and were consequently grouped together. Table 2 presents the average and standard deviation values of D and Cs for each of the two distinct groups thus identified.

Table 2

	TIDAL (i=1)	D(in^2/y)	Cs(%)	D(m^2/s)	Cs(kg/m^3)
T i=1	AVG:	1.04e-02	0.98	2.13e-13	25.02
	STDEV:	7.0e-03	0.47	1.4e-13	12.1
LS* i=2	AVG:	5.00e-03	0.60	1.0e-13	15.3
	STDEV:	2.5e-03	0.30	5.1e-14	7.6
US+AS i=3	AVG:	2.42e-03	0.385	4.95e-14	9.80
	STDEV:	1.3e-03	0.20	2.6e-14	5.2

*assigned values

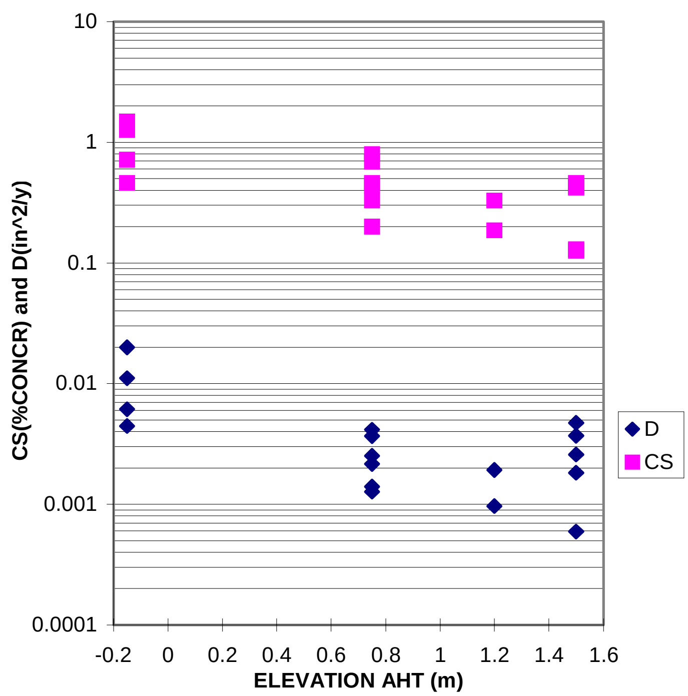

Figure 1. D and Cs as a function of elevation.

The populations of both groups (especially that for the Tidal regime) are small, so the standard deviation values can only be considered as nominal values. Nevertheless, at least for the US + AS regimes, there is reasonable approximation between an ideal normal distribution and the actual cumulative value counts, as shown in Figure 2. Nominal parameter values were assigned for the LS zone and listed in Table 2. These values were intermediate between those for i=1 and i=3 and

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chosen to follow, at an elevation of .15 m (0.5 ft) AHT, the general trends of Figure 1.

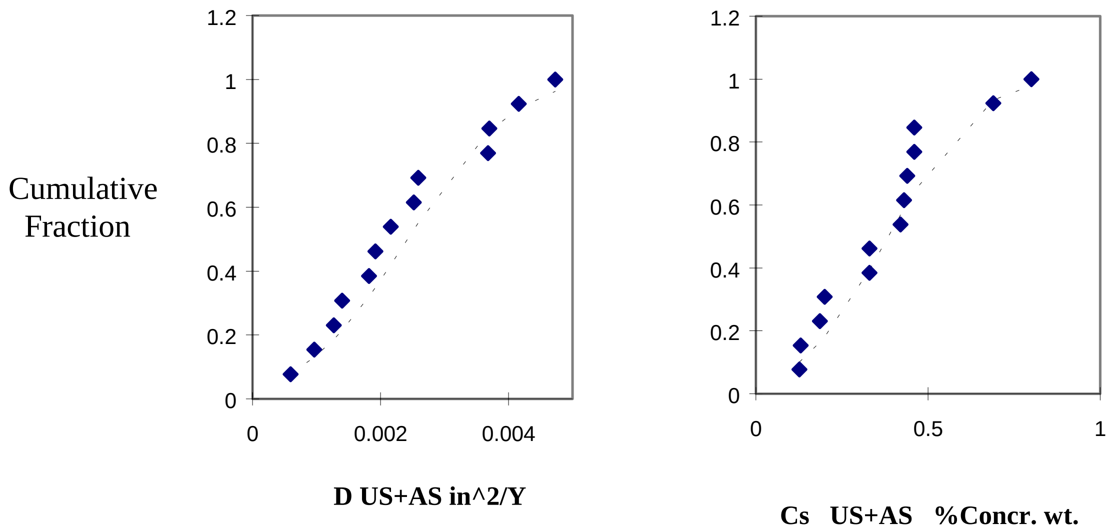

Figure 2. Cumulative normal distributions (dashed lines) based on the average and standard deviation values in Table 2 for D and Cs in elevation ranges 2 and 3, and actual distribution of values.

e) Concrete cover: Direct measurement of the concrete cover in the 0.91-m (3-ft) and 1.37-m (4.5-ft) piles yielded similar results, as shown in Table 3. The spiral pitch is only ≈7.5 cm (3 in) (design detail drawings), resulting in a large amount of stirrup steel. It is then expected that the first corrosion-related damage requiring extensive repair will be from the spiral hooping. Since the piles were precast, the same values (overall average = 2.79 cm (1.1 in); standard deviation = 0.63 cm (0.25 in)) were used for i = 1 to 3. No distinction was made between 0.91-m (3-ft) and 1.37-m (4.5-ft) piles.

Table 3

Pile	Number or test spots	Strands				Stirrups			
		Avg (cm)	St. Dev. (cm)	Highest (cm)	Lowest (cm)	Avg (cm)	St. Dev. (cm)	Highest (cm)	Lowest (cm)
0.91-m	47	4.04	1.12	5.71	2.54	2.84	0.66	5.08	1.90
1.37-m	14	3.51	1.24	5.08	1.27	2.64	0.51	3.17	1.90

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Figure 3 shows the cumulative distribution of stirrup cover values for the 0.91-m (3-ft) piles, compared with an ideal cumulative normal distribution having the average and standard deviations for those piles. The resolution of the field measurements was ≈6 mm (0.25 inch). No values lower than 1.9 cm (0.75 inch) were recorded for any of the stirrup measurements in either the 0.91-m (3-ft) or 1.37-m (4.5-ft) piles. In an ideal normal distribution with the parameters for Figure 3, 0.85% of the stirrup measurements (less than 1 in a field of 47 tests) would have been 1.27 cm (0.5 inch) or less. Thus, the absence of lower readings in the present sampling is not by itself statistically indicative that the concrete cover in the stirrups was limited by construction to 1.9 cm (0.75 in). However, some form of cover limitation (for example, by the use of saddles) is likely in the precast procedure. Moreover, corrosion damage was not conspicuous anywhere in the 1,486 piles on water after 31 years of service. For the purposes of the model, the distribution was truncated at 1.9 cm (0.75 in).

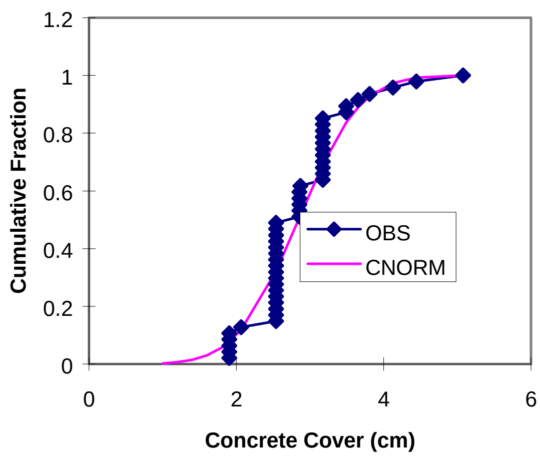

Figure 3. Cumulative normal distribution of stirrup concrete cover and observed values (OBS) for the 0.91-m (3-ft) piles.

f) tp_i: In the absence of corrosion measurements of confirmed active steel, the value of tp_i for elevation ranges 2 and 3 was assigned to be 7 years ($2.2\ 10^8$ sec). Because of the apparent high concrete quality, this value was twice the nominal value used in previous estimates of durability for FDOT bridges [8]. The value assigned to $tp_{i,}$ in the T elevation range was 30 years ($9.5\ 10^8$ sec), an estimation based on the assumption of much lower corrosion rates in the tidal region where very slow oxygen transport is expected.

<u>Model Output</u>

Figures 4, 5 and 6 show the model output for Cases A, B and C (C_T equal to 1.6, 3.2 and 4.8

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kg/m^3, respectively). The amount of damage (total for the two bridges) is given in m^2 by the value of the $TS1_k$, $TS2_k$ and $TS3_k$ for elevation regimes i=1, i=2 and i=3 respectively. The sum of the damage in the three regimes is given by TSA_k.

DISCUSSION

The model outputs show a period of no significant corrosion damage followed by the gradual development of deterioration afterwards. The shape of the curves for each elevation range reflects the assumed dispersion of model parameters (concrete cover, surface concentration, and diffusivity) around their respective average values. An assumption of no dispersion would have resulted in a sharp step damage function for each range, with damage starting at the time corresponding to that dictated by the average parameter values plus the assumed propagation time. The model outputs project the most damage taking place in the Tidal zone during the next few decades.

The projected time for observation of significant corrosion was about 20 years for the most conservative of the alternative cases (C_T = 1.6 kg/m^3 (2.7 pcy), Case A), and almost 40 years for the least conservative Case C. Thus, the alternatives bracket the present actual experience of no significant damage by age 31. In all 3 realizations the total projected damage reached 1000 m^2 (10,764 ft^2) some 20 years after the first appearances of significant damage. Detailed cost estimates for rehabilitation were prepared based on the repair/rehabilitation alternatives considered [2].

The model presented in this paper is not an absolute prediction tool. Instead, the model should be viewed as a means of providing quantitative projections to assist in comparing repair and future construction alternatives. As illustrated by the results from cases A though C, the output was highly sensitive to the assumed value of key parameters, exemplified by the value of the concentration threshold, which are subject to much uncertainty. The overall modeling assumptions involved also numerous simplifications that ignore important issues such as (to name a few) effective diffusivity and surface concentration variations with time [9], the effect of chloride ion binding on diffusion [10,11], alternative chloride transport mechanisms [12], effect of potential on C_T [13], non-flat surfaces [6,11], and the factors altering the length of the propagation stage [14]. Improvement is also needed to discern between actual variability and measurement uncertainty in the parameters (concrete cover, diffusivity, surface concentration) that were used as distributed values.

CONCLUSION

Quantitative projections of future deterioration can be performed by taking into account the compounded variability of concrete cover, chloride diffusivity, and chloride surface concentration in the substructure of marine bridges. The projected damage functions reflected the dispersion of the assumed controlling model variables. The model is not an absolute prediction tool, but should be viewed instead as a means to assist in comparing design alternatives.

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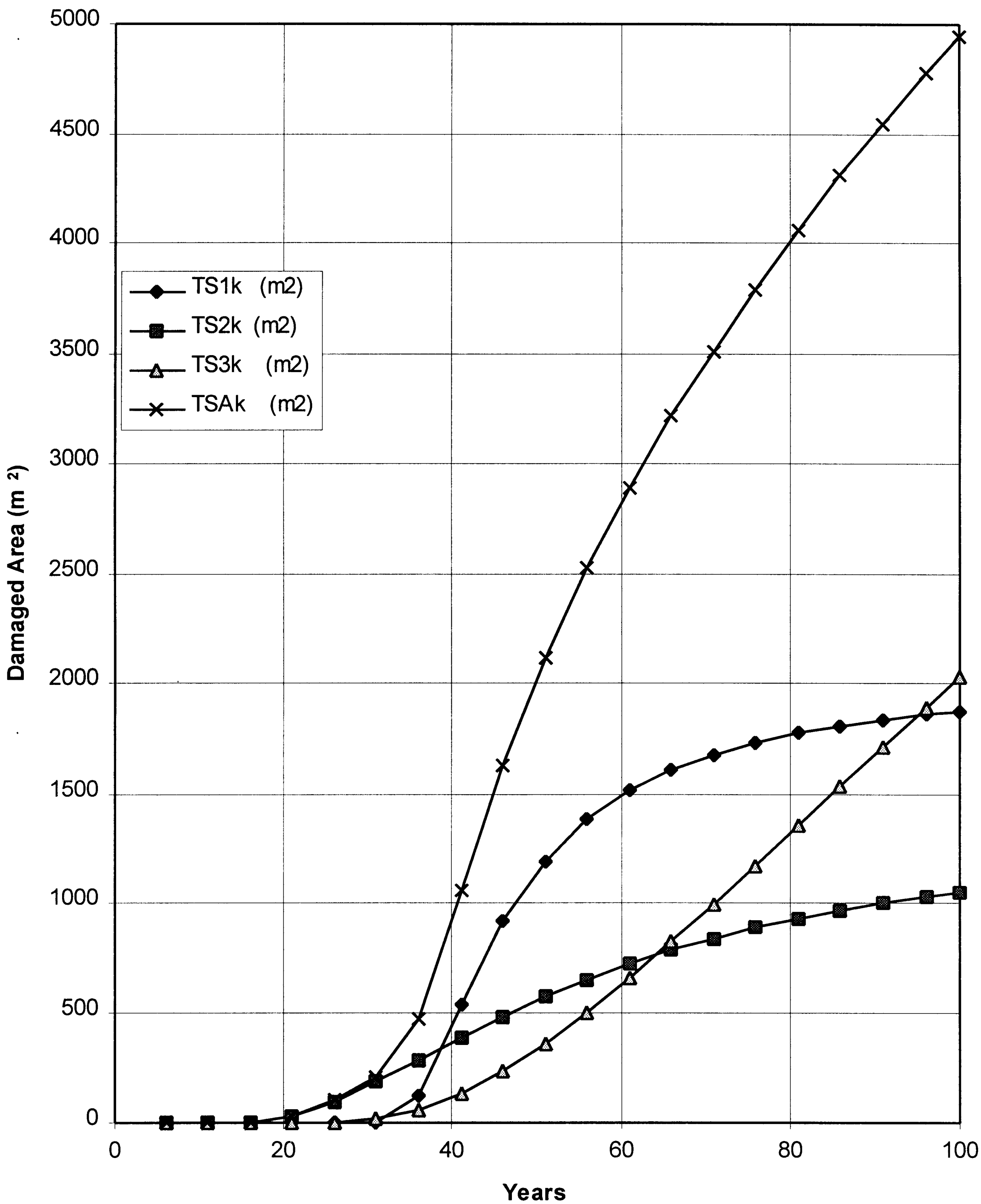

Figure 4. Deterioration model output for case A (C_T = 1.6 kg/m^3).

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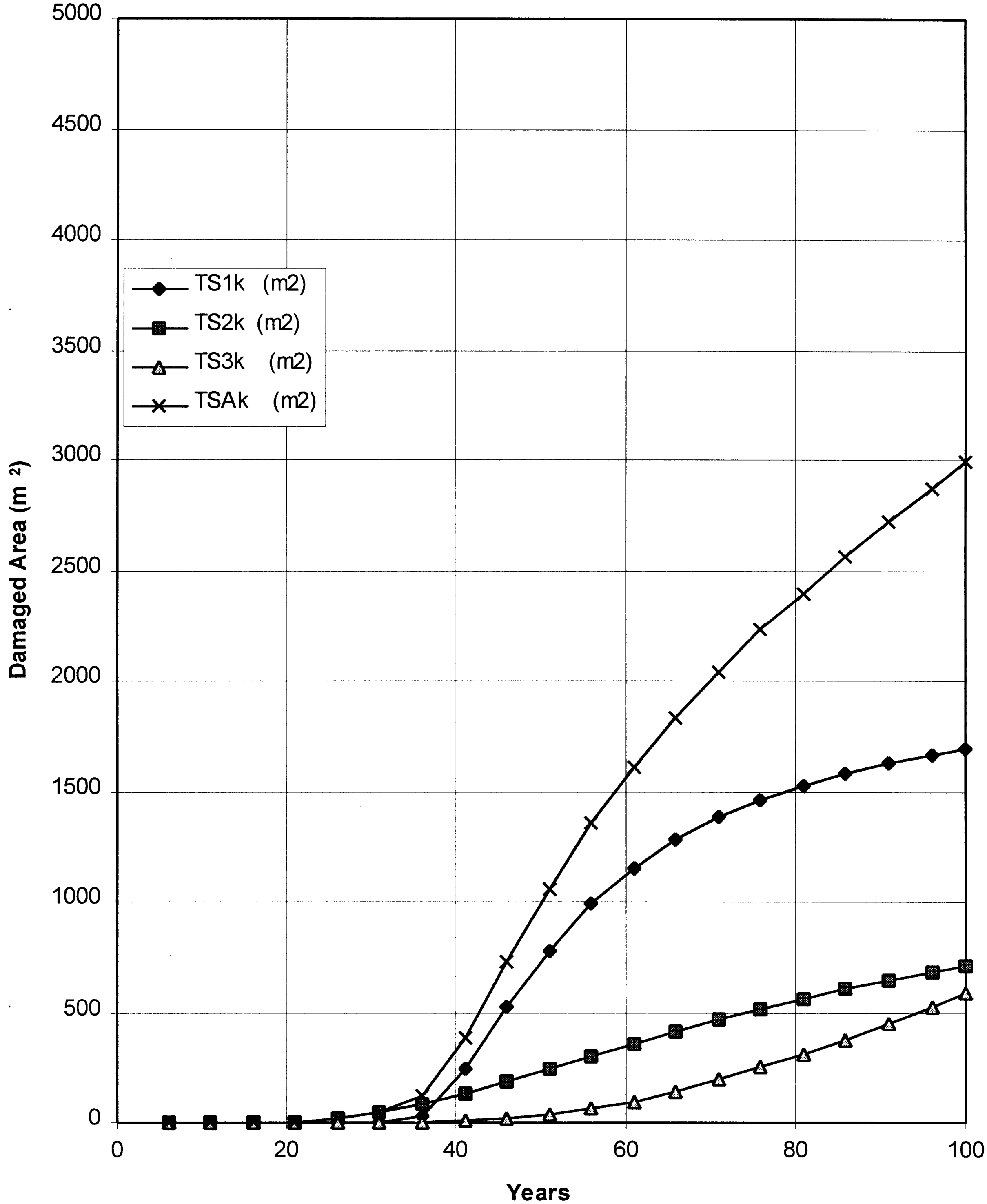

Figure 5. Deterioration model ouput for case B ($C_T = 3.2$ kg/m^3).

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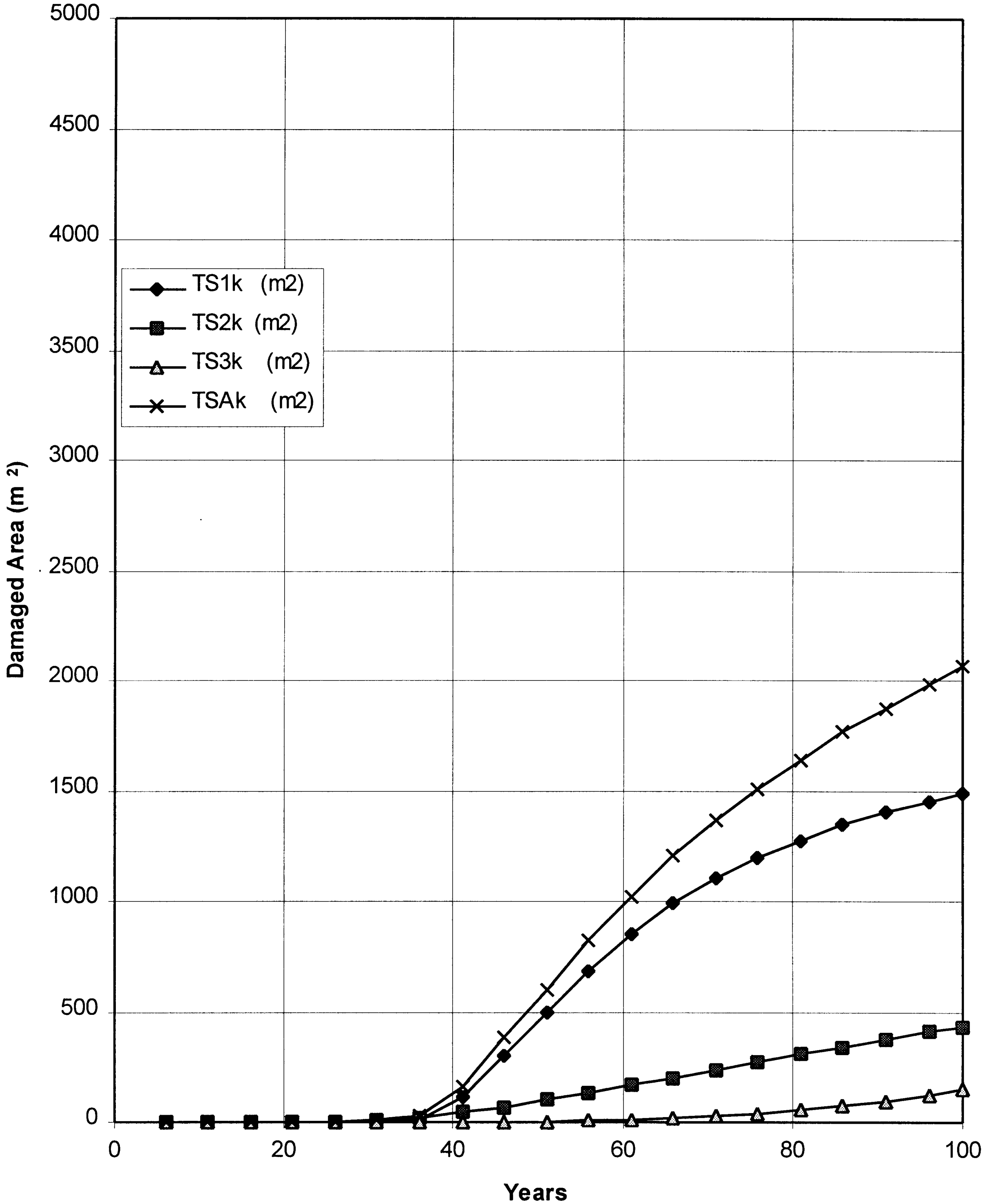

Figure 6. Deterioration model output for case C (C_T = 4.8 kg/m^3).

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BIBLIOGRAPHY

[1] Raymond International, Inc., “The Raymond Cylinder Pile and Cylinder Pile Structures”, Houston, Texas (1969).

[2] Scannel, W., Soh, F., Sohanghpurwala, A. A. and Sagüés, A., “Assessment of Rehabilitation Alternatives for Bridge Substructure Components” Final Report, CONCORR, Inc., Ashburn, VA, 1997.

[3] Andrade, C., and Alonso, C., "Values of Corrosion Rate of Steel in Concrete in Order to Predict Service Life of Concrete Structures", in "Application of Accelerated Corrosion Tests to Service Life Prediction of Materials", STP 1194, G. Cragnolino, Ed., ASTM, Philadelphia, 1992.

[4] K. Tuutti, "Corrosion of Steel in Concrete" (ISSN 0346-6906), Swedish Cement and Concrete Research Institute, Stockholm, 1982.

[5] Cady, P.D., and Weyers, R.E., “Deterioration Rules of Concrete Bridge Decks”, Journal of Transportation Engineering, American Society of Civil Engineers, Vol. 110, No. 1, January 1984, pp. 35 - 44.

[6] Sagüés, A., Kranc, S. and Presuel-Moreno, F., “Applied Modeling for Corrosion Protection Design for Marine Bridge Substructures”, Report No. 0510718, June, 1997, available from Florida Department of Transportation, Research Center, Tallahassee, Florida.

[7] Bamforth, P., “Improving the Durability of Concrete by the Use of Mineral Additives”, Paper presented at “Concrete Durability in the Arabian Gulf”, Bahrain Society of Engineers, 19-21 March, 1995.

[8] Sagüés, A., "Corrosion of Epoxy-Coated Rebar in Florida Bridges", Final Report to Florida D.O.T., WPI No. 0510603, May, 1994, available from Florida Department of Transportation, Research Center, Tallahassee, Florida.

[9] Mangat, P. and Molloy, B., “Prediction of long term chloride concentration in concrete”, Materials and Structures, Vol. 27, p.338-346, 1994.

[10] Nilsson, L.O., Massat, M., and Tang, L., "The Effect of Non-linear Chloride Binding on the Prediction of Chloride Penetration into Concrete Structures", p. 469 in Durability of Concrete, ACI Publication SP-145, V.M. Malhotra, Ed., American Concrete Institute, Detroit, 1994.

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[11] Sagüés, A and Kranc,S.C. "Effect of Structural Shape and Chloride Binding on Time to Corrosion of Steel in Concrete in Marine Service", p. 105-114 in Corrosion of Reinforcement in Concrete Construction, C.L. Page, P.B. Bamforth and J.W. Figg, Eds, The Royal Society of Chemistry, Cambridge, 1996.

[12] Dihr, R., Jones, M., Byars, E. and Shaaban, I., "Predicting concrete durability from its absorption", p. 1177, in "Durability of Concrete", V. Malhotra, Ed., SP 145, ACI, Detroit, 1994.

[13] Bertolini, L., Bolzoni, F., Pastore, T. and Pedeferri, P., "New Experiences on Cathodic Prevention of Reinforced Concrete Structures", pp. 390-398 in Corrosion of Reinforcement in Concrete Construction", C. Page, P. Bamforth and J. Figg, Eds. Society for Chemical Industry, Special Publication No. 183, London, 1996.

[14] Weyers, R., "Corrosion Service Life Model Concrete Structures", p.105, in Repair and Rehabilitation of Reinforced Concrete Structures: The State of the Art, W.P. Silva-Araya, O. T. de Rincon and L. Pumarada O'Neill, eds, ASCE, Reston, VA, 1998.

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