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Extending service life of high performance concrete bridge decks with internal curing

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ABSTRACT: High performance concrete (HPC) bridge decks are prone to premature cracking if movement is restrained. Internal curing (IC) can reduce shrinkage cracking in concrete structures, thus improving their performance and service life. However, there is very limited information available in the literature on the possible extension of service life due to internal curing. This paper addresses this question by using predictive models to estimate the service lives of typical bridge decks made with concrete using internal curing as opposed to conventional curing. Four options are compared: (i) normal concrete deck; (ii) HPC deck; (iii) HPC deck with internal curing; and (iv) very high performance concrete deck with internal curing. It was found that the use of internal curing can increase the service life of HPC bridge decks by almost ten years, which is mainly due to due a slower penetration of chlorides as a result of reduced cracking.

1 INTRODUCTION

Proper curing of concrete structures is important to ensure that they meet their intended performance and durability requirements. In low permeability concrete, conventional external curing may not be effective in preventing self-desiccation at the centre of thick concrete elements. Internal curing (IC) is a technique that can be used to provide additional moisture in concrete for a more effective cement hydration and reduced self-desiccation (Rilem TC-196, 2007). For that purpose, saturated porous lightweight aggregate (LWA) can be mixed into concrete in order to supply an internal source of water, which can replace the mix water consumed by chemical shrinkage during cement hydration (Weber & Reinhardt 1997). The required mass of LWA to introduce in concrete for adequate internal curing can be determined following the simple design procedure suggested by Bentz *et al*. (2005).

Internal curing with LWA has been successfully used recently in large construction projects of normal-density concrete structures. For example, in January 2005, about 190 000 m^3 of internally cured concrete was used in a large paving project in Hutchins, Texas (Villarreal & Crocker 2007). Field observations reported marginal pavement cracking, and strength tests indicated that 7-day flexural strengths reached 90% to 100% of the required 28-day flexural strength due to an improved cement hydration. They also found that the compressive strengths of air-cured cylinders were similar to those of wet-cured cylinders at all ages, suggesting that concrete with internal curing is less sensitive to poor external curing practices or unfavorable ambient conditions.

Although the benefits of internal curing for high performance concrete structures have been evidenced in laboratory and field investigations (such as those previously mentioned), the literature offers no significant quantitative information on the extent of service life that internal curing can offer to concrete structures. Using available test data from the literature, along with predictive models and conservative engineering judgment, the objective of this paper is to provide estimates of service lives for concrete bridge decks made with concrete using either conventional curing or internal curing.

2 CASE STUDY

To demonstrate the benefits of internal curing for high performance concrete structures in terms of service life performance, a typical concrete bridge deck case study is used for the analysis. The bridge element under consideration is a 200-mm thick concrete slab reinforced with conventional steel with a concrete cover of 75 mm, as shown in Figure 1. For simplicity, the concrete slab surface area is set to a width of 10 m and length of 30 m, supported by single-span concrete girders separated by expansion joints. The bridge superstructure and substructure are not considered in the analysis.

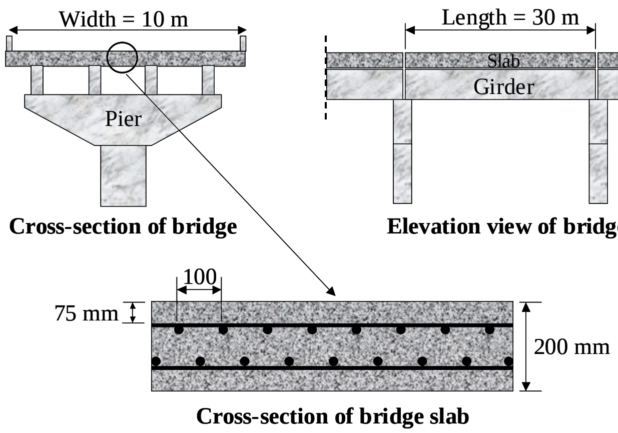

Figure 1. Geometry of case study bridge deck.

Table 1 presents the four concrete bridge deck options compared in this case study, including: (i) a bridge deck made of good quality normal concrete (NC) with a water-to-cementitious materials ratio (w/cm) of 0.4, and no supplementary cementing materials (SCM); (ii) deck made of high performance concrete (HPC), assuming early-age cracking due to autogenous shrinkage, with a w/cm of 0.35 and 25% SCM as partial cement replacement by mass; (iii) deck made of high performance concrete with internal curing (HPC-IC), also with a w/cm of 0.35 and 25% SCM; and (iv) deck made of very high performance concrete with internal curing (VHPC-IC), with a lower w/cm of 0.30, and 25% SCM. In this case, the SCM consist of 5% silica fume and 20% slag, and the LWA is assumed to provide 28 kg/m^3 of internal curing water in the concrete.

Table 1. Design of selected concrete bridge deck options.

Deck option	Initial cracking	Water (kg/m^3)	Cement (kg/m^3)	SCM (%)	LWA (kg/m^3)
NC	No	140	350	0	0
HPC	Yes	160	450	25	0
HPC-IC	No	160	450	25	185
VHPC-IC	No	160	525	25	185

Note: SCM included in cement content.

The concrete bridge decks are directly exposed to de-icing salts in winter over their respective service lives. For the four deck options, a surface chloride concentration of 9 kg/m^3 is assumed, which is typical of the severe conditions found in Northern USA or Canada (Weyers 1998). A chloride threshold of 0.7 kg/m^3 is assumed for the conventional steel reinforcement (ACI Committee 222, 2001), over which corrosion can initiate; and a moderate corrosion rate of 0.5 µA/cm^2 is assumed once corrosion has started (Rodriguez *et al.* 1994). It can be argued that these values may differ from actual values depending on the severity and variability of the local exposure conditions; however, the findings of this case study should be considered with respect to the normal concrete deck (base case), since all deck options are compared using the same surface chloride concentration, chloride threshold and corrosion rate.

3 SERVICE LIFE PERFORMANCE

In this study, most of the service life performance analysis was conducted using the analytical models developed by Lounis *et al.* (2006) and implemented in NRC's SLAB-D software (Daigle and Lounis 2006). The effects of early-age cracking and internal curing on chloride penetration were not included in the original SLAB-D models, but estimated using additional models presented in this paper.

Figure 2 presents the different stages of corrosion-induced damage developing in a typical reinforced concrete bridge deck. With time, each stage is the result of a higher damage level first caused by the initial cracking and then by the corrosion process. The different stages include: (i) early-age cracking due to restrained shrinkage (if any); (ii) initiation of reinforcement corrosion after a relatively long period of chloride diffusion through concrete; (iii) internal cracking around the reinforcing bars due to a build-up of corrosion products; (iv) surface cracking due to further progression of corrosion-induced cracks; (v) delamination or spalling of the concrete cover depending on the reinforcement spacing and diameter, and concrete cover thickness; and finally (vi) failure of the concrete deck, depending on the amount of delamination or spalling that is tolerated by the bridge owner before deck rehabilitation or replacement is considered.

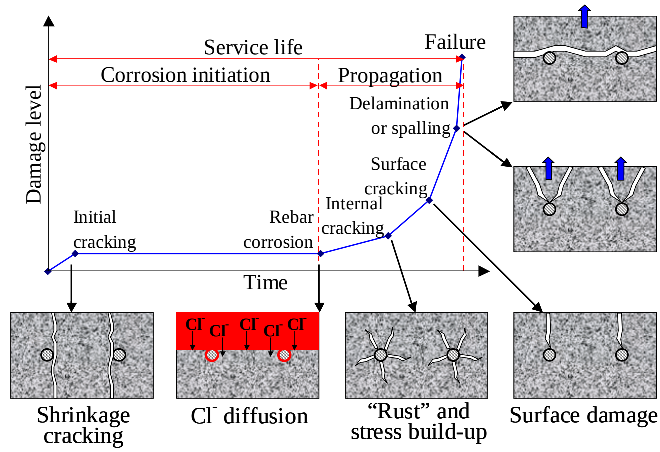

Figure 2. Concept used for service life performance analysis.

3.1 *Initial cracking due to restrained shrinkage*

Early-age cracking is a common problem on concrete bridge decks (TRB 1996) as more than 100,000 bridges in the United States are reported to have developed transverse cracking shortly after construction. This type of through cracking is of great concern to engineers and bridge owners since it may lead to premature reinforcement corrosion and concrete deterioration due to accelerated moisture and salt ingress through the cracks.

Rodriguez & Hooton (2003) studied the influence on chloride penetration of artificially created parallel cracks with widths ranging from 0.08 mm to 0.68 mm. They concluded that chloride diffusion in concrete was not affected by the width and wall roughness of individual cracks for the ranges studied. This finding allows the use of a simplified smeared approach proposed by Boulfiza *et al*. (2003) to estimate the effect of cracks on chloride ingress. It assumes that the chloride ingress into cracked concrete can be approximated using Fick's second law of diffusion, in which the following apparent diffusion coefficient (D_{app}) is used:

$$D_{app} = D_c + \frac{w_{cr}}{s_{cr}} D_{cr} \quad (1)$$

where D_c is the chloride diffusion coefficient in uncracked concrete; w_{cr} is the crack width; s_{cr} is the crack spacing; and D_{cr} is the diffusion coefficient inside the crack, which is assumed to be $5\text{x}10^{-10}$ m^2/s (Boulfiza *et al*. 2003). In the present case study, the value of w_{cr}/s_{cr} is set to 0.0003, which corresponds to the 7-day value of autogenous shrinkage strain measured for a concrete that was very similar to that selected for the HPC deck option (Cusson & Hoogeveen 2008). As a result, an average increase in the chloride diffusion coefficient of approximately $1.5\text{x}10^{-13}$ m^2/s is expected. For the decks made with NC, HPC-IC and VHPC-IC, the effect of cracking on chloride ingress is neglected (i.e. $D_{app} = D_c$), assuming that autogenous shrinkage in these concretes is not significant enough to result in concrete cracking.

3.2 *Corrosion initiation due to chlorides*

In uncracked concrete, the chloride ingress was determined by using Crank's solution of Fick's second law of diffusion (Crank 1975, Tuutti 1982):

$$C(x,t) = C_S \left[1 - erf\left(\frac{x}{2\sqrt{D\,t}} \right) \right] \quad (2)$$

where $C(x,t)$ is the chloride concentration at depth x after time of exposure t; C_s is the chloride concentration at the concrete surface; and *erf* is the error function. Equation 2 can also be rearranged to predict the time of corrosion initiation (t_i) by setting $C(x,t)$ equal to a chloride threshold value (C_{th}), at which the corrosion of the reinforcing steel is expected to initiate, and x equal to the effective cover depth of the reinforcing steel (d_c).

The chloride diffusion coefficients of the different concretes were estimated based on their water-cement ratios and the types of SCM selected for this case study. The empirical models of Boulfiza *et al.* (2003) developed from a large set of literature data were used:

$$Log\ D_C = -3.9(w/cm)^2 + 7.2(w/cm) - 14.0$$

(for concrete with no SCM) (3a)

$$Log\ D_C = -3.0(w/cm)^2 + 5.4(w/cm) - 13.7$$

(for concrete with silica fume and slag) (3b)

where D_c is expressed in units of m^2/s. Table 2 presents the resulting coefficients of diffusion calculated for the different concretes considered, including the effect of initial cracking on the apparent chloride diffusion coefficient (D_{app}).

Table 2. w/cm and diffusion coefficients.

Deck option	w/cm	D_c	D_{app}
		(10^{-13} m^2/s)	
NC	0.40	18	18
HPC	0.35	6.6	8.1
HPC-IC	0.35	6.6	6.6
VHPC-IC	0.30	4.4	4.4

It is important to note that the estimated values of D_c for the HPC-IC and VHPC-IC concretes are considered to be conservative values, since internal curing can also enhance cement hydration, thus reducing concrete permeability and diffusivity. Indirect observations of reduced permeability/porosity due to internal curing through improved early-age strengths can be found in the literature (Bentz 2007). Since there is not enough experimental data available in the literature to model the effect of internal curing on chloride diffusivity of concrete, this beneficial effect of internal curing was not accounted for.

Figure 3 presents the chloride profiles calculated for the four different concrete bridge decks after a 20-year exposure to de-icing salts. The critical zone corresponds to chloride concentrations at the reinforcement level (or deeper) that are higher than the chloride threshold of 0.7 kg/m^3, over which initiation of steel corrosion is likely to occur. It can be seen that after 20 years only, the chloride concentration in the normal concrete deck has reached the critical concentration at the 75-mm depth.

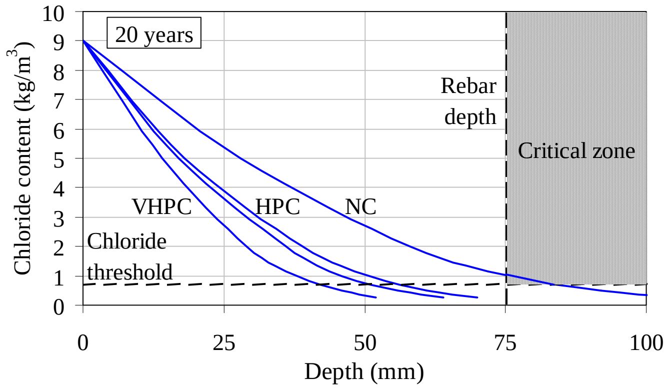

Figure 3. Chloride profiles in concrete after 20 years.

Figure 4 shows the chloride concentrations in the four concrete decks increasing over time at the reinforcement level (75 mm). It is found that the chloride concentrations would reach the threshold value after 16 years for the NC deck, 35 years for the HPC deck, 43 years for the HPC-IC deck, and 65 years for the VHPC-IC deck. The additional eight years provided by the use of internal curing can be attributed to reduced cracking (lower D_{app}).

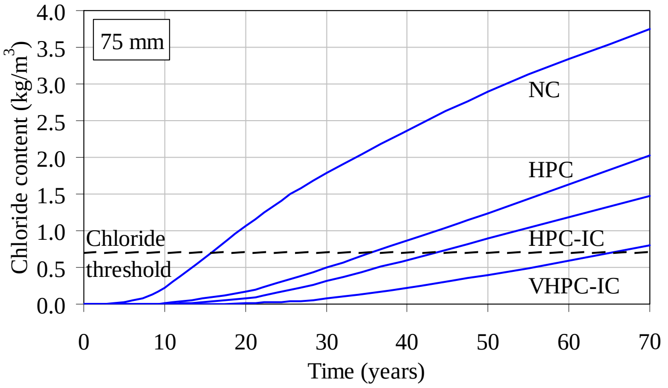

Figure 4. Chloride ingress over time at reinforcement level.

3.3 Corrosion-induced damage

Assuming that concrete is a homogeneous elastic material (in tension), with a tensile strength (f'_t), and that corrosion products are equally distributed around the perimeter of the reinforcing bars (Bažant 1979), stresses generated in the concrete deck by corrosion built-up products can be estimated using the thick-wall cylinder model (Timoshenko 1956). This model allows the calculation of the increase in the rebar diameter (Δd) related to different stages of corrosion-induced damage (Figure 5).

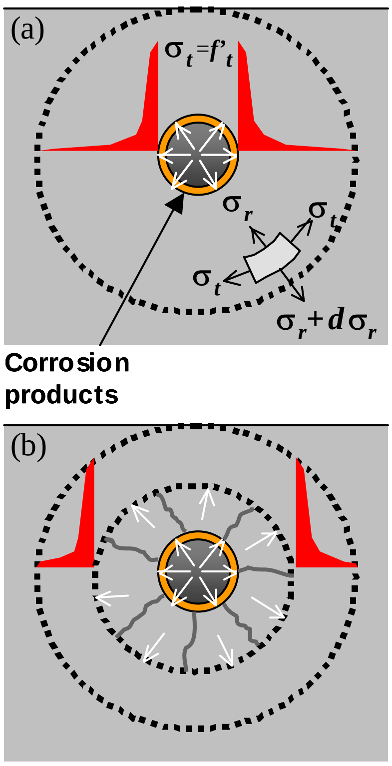

Figure 5. Thick-wall cylinder model of corroding reinforced concrete deck: (a) Tensile stresses developed at crack initiation; (b) Propagation of internal cracks in thick-wall concrete cylinder. Adapted from Lounis *et al.* 2006.

The corrosion propagation times (t_p), corresponding to the onset of internal cracking, surface cracking, and delamination/spalling are found as follows:

$$t_p = \frac{\pi\, d\,(\Delta d)}{2\, S\, j_r \left[\dfrac{1}{\rho_r} - \dfrac{\alpha}{\rho_s} \right]} \tag{4}$$

where d is the rebar diameter; S is the rebar spacing; j_r is the rust production per unit area (Bažant 1979); ρ_r is the density of corrosion products (assumed at 3600 kg/m^3 for $Fe(OH)_3$); ρ_s is the density of steel (7860 kg/m^3); and α is the molecular weight ratio of metal iron to the corrosion product (assumed at 0.52). The total time to reach a given corrosion-induced damage level is the sum of the corrosion initiation time (t_i) and the individual corrosion propagation times (t_p) up to that level.

Work by Allan & Cherry (1992) showed that there may be a porous zone around the steel reinforcement at the steel/concrete interface, in which a specific quantity of corrosion products can accumulate before tensile stresses can actually develop in concrete, thus delaying the damage initiation of the concrete cover. The thickness of the porous zone may be in the order of 12.5 µm (Liu & Weyers 1998); however, the thickness and porosity of the porous zone depend on many factors, such as: w/cm, presence and type of SCM and hydration, which in turn influence the overall concrete porosity.

Internal curing may affect the porous zone in two competing ways: (i) by decreasing the porosity of the cement paste due to an improved cement hydration; and (ii) by increasing the porosity of the zone due to the relatively large and empty pores left in the LWA after the internal curing water has migrated into the cement paste. Due to the lack of data in the literature on the effect of internal curing and w/cm ratio on the thickness and porosity of the porous zone, this possible beneficial effect has been neglected in the analysis. Preliminary calculations indicate that neglecting the effect of the porous zone would provide more conservative estimates of service lives by at most 2 or 3 years only.

3.4 Estimated service life

In this case study, the service life of a bridge deck is defined as the time to initiate delamination or spalling (which ever comes first). Figure 6 illustrates the average service life estimated for each concrete deck option, in which the times to reach lower levels of concrete damage are also illustrated. Based on the above hypotheses, the service life increased due to the use of a lower water-cement ratio (reduced chloride diffusion), in general, and the reduction of early-age cracking due to internal curing. The service lives were 19 years for the 0.40 *w/cm* NC deck; 38 years for the 0.35 *w/cm* HPC deck; 46 years for the 0.35 *w/cm* HPC-IC deck; and 68 years for the 0.30 *w/cm* VHPC-IC deck. For simplicity, other factors such as live loads, dead loads and thermal effects were not considered in this analysis.

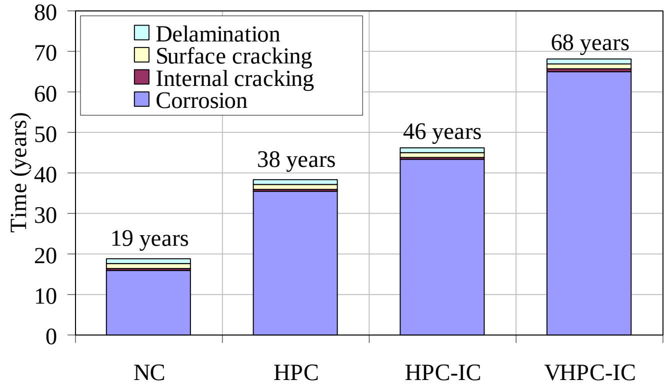

Figure 6. Service life predictions from deterministic analysis.

The service live predictions presented in Figure 6 were obtained through a deterministic analysis, i.e. an analysis that do not consider the variability and uncertainty associated with the main parameters governing the different mechanisms of deterioration of reinforced concrete decks. Deterministic analyses are based on mean or characteristic values of the variables and can only predict the times to reach the different stages of corrosion initiation and corrosion-induced damage caused by an "average" condition.

The variability and uncertainty of the main parameters (e.g. material properties, structure dimensions, reinforcement, initial conditions, and environment) that govern service life predictions are the sources of the field-observed variable performance of concrete decks, which is better described with a reliability-based approach. The same four concrete deck options were analyzed with the reliability-based option of SLAB-D, considering the variability of six parameters (Table 3) expressed in terms of their coefficients of variation (COV).

Table 3. Average and COV values of main parameters.

Parameter	Average	COV (%)
Surface chlorides (kg/m^3)	9	30
Diffusion coef. (10^{-13} m^2/s)		
- NC	18	30
- HPC	8.1	30
- HPC-IC	6.6	30
- VHPC-IC	4.4	30
Concrete cover depth (mm)	75	30
Chloride threshold (kg/m^3)	0.7	30
Corrosion rate (μA/cm^2)	0.5	30
Bar spacing (mm)	100	5

Figure 7 presents the results from the reliability analysis for the four deck options, in which the probability of delamination increased over time. Considering that the bridge deck can be divided into a very large number of small surface areas with the same probability (P%) of delamination after a number of years (t), it can be approximated that a certain percentage (P%) of them are delaminated. The 10%, 25% and 50% probabilities of delamination are highlighted in Figure 7 (short vertical lines) for the four options as they set the limits between the five condition states for concrete deck condition assessment (Table 4) defined in the AASHTO (1998) Guide, which is used in the bridge management system of many states in USA.

Table 4. AASHTO description of deck condition states.

Condition state	Description
1	The surface of the deck has no patched area and no spalls in the deck surface.
2	The combined distress area (i.e. existing patches, delamination and spalling) of the deck is less than 10%.
3	The combined distress area of the deck is between 10% and 25%.
4	The combined distress area of the deck is between 25% and 50%.
5	The combined distress area of the deck is more than 50%.

For the normal concrete deck, condition states 3, 4 and 5 are reached after approximately 8, 13, and 19 years. For the HPC, HPC-IC and VHPC-IC decks, these same condition states are reached after 15, 24 and 38 years; 18, 29 and 47 years; and 27, 44, and 70 years, respectively. As the apparent coefficients of diffusion of the four investigated deck options decreased, the times to reach condition state 3 (and subsequent states) increased. Depending on bridge owner policy and the estimated costs and benefits of repairs, the optimum recommended action to take for each deck condition state can be different, ranging from the "do-nothing" approach, to delamination and spalling repairs, and ultimately to deck replacement. In addition to the extended service life provided by reduced initial cracking and reduced chloride diffusion coefficients, the reliability-based analysis also shows that the use of lower w/c concretes with internal curing results in longer periods of time before major repairs or rehabilitation become necessary, as indicated by the times needed to reach the condition states 3, 4 and 5.

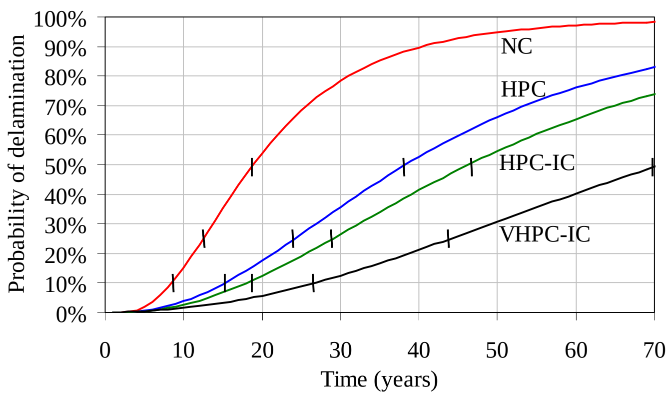

Figure 7. Time-dependent probability of delamination.

4 SUMMARY AND CONCLUSIONS

The service life performances of four bridge decks made with different concretes were estimated using predictive models for chloride diffusion, corrosion initiation, and corrosion propagation leading to cracking, delamination and/or spalling. The concretes selected for the bridge decks were: (i) normal concrete deck (w/cm=0.40); (ii) HPC deck (w/cm=0.35); (iii) HPC deck with internal curing (w/cm=0.35); and (iv) very high performance concrete deck with internal curing (w/cm=0.30). From this study, the following conclusions can be drawn:

1. The use of internal curing can increase the service lives of high performance concrete bridge decks by almost ten years, due to the absence of initial cracking.
2. This estimated life extension due to internal curing alone is considered to be on the conservative side, since the beneficial effect of internal curing on the reduction of concrete permeability to water and chloride ions was not accounted for in this analysis.
3. The service life of concrete bridge decks can be extended by 50 years when considering the use of internally cured very high performance concrete (w/cm=0.30) over normal concrete (w/cm=0.40).
4. The use of HPC and VHPC with internal curing in bridge decks can bring an additional economical benefit by delaying the times at which repairs and rehabilitation of the reinforced concrete deck are required.

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