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ON IMPROVING THE INFRASTRUCTURE SERVICE LIFE USING ECC TO MITIGATE REBAR CORROSION

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Abstract

This paper presents the results of service life estimation and life cycle cost analysis of an infrastructure application – bridge decks. Two types of materials are investigated in the comparative assessment viz. Reinforced Concrete (R/C) and Reinforced Engineered Cementitious Composite (R/ECC). ECC is a micromechanically designed high performance fiber reinforced cementitious composite (HPFRCC) with high ductility and improved durability due to tight crack width. Life-365, an industry standard software, is used as a simple computation tool to predict service life and life cycle costs. Deterioration of reinforced concrete structures caused by corrosion of reinforcing steel bars due to chlorides is the main durability concern in this study. A framework has been developed to systematically predict the service life and life cycle costs based on the transport properties of the materials used. Results indicate that the use of ECC with or without inhibitors significantly prolongs the service life and reduces the life cycle costs. Self-healing of ECC further enhances the service life by sealing of cracks, thereby preventing the ingress of chloride ions. Upfront unit material cost of ECC is higher than concrete but its advantages in terms of life cycle costs to the agency (DOTs) can be quantified through such life cycle assessment.

1. INTRODUCTION

The vast amount of civil infrastructure in the US and other developed countries has been deteriorating, and requires billions of dollars for maintenance and repair every year. About 28% of bridges in the US were classified by the Federal Highway Administration as "deficient" in 2006.[1] One of the main causes of deterioration in reinforced concrete (R/C) bridges is the corrosion of reinforcing steel bars resulting in reduction of service life.[2] Life cycle analysis of bridge decks shows significant contribution of the use phase of a bridge deck towards the life cycle material resource consumption, primary energy usage, and CO_2

emissions due to repeated maintenance activities. Reducing corrosion-induced damage, therefore, is expected to contribute to the development of sustainable infrastructure systems.

Rebar steel in reinforced concrete forms a tightly adhering passive layer in the presence of highly alkaline environment, which is the result of cement hydration in concrete.[3,4] Thus, sound concrete itself acts an excellent inhibitor for steel corrosion, provided its alkalinity is maintained and any corrosive agent is prevented from reaching the steel rebar and dissolving the protective passive layer. However, steel reinforced concrete used in bridges is exposed to large amounts of deicing salts containing chloride ions and other corrosive agents, especially in the northern US. These corrosive agents permeate through the concrete cover and their concentration builds around the rebars gradually. Finally, a threshold concentration is reached which depassivates the protective layer on steel rebars and facilitates the corrosion of the core of the rebar by allowing access to pore water, oxygen, and other impurities. This generates flaky forms of iron oxide, i.e. rust, and hence the rebar starts to corrode.[5] Another phenomenon, which corrodes the rebar, is the loss of alkalinity due to carbonation of hydroxides in concrete. The later is lesser of the two problems, and in this paper, our main focus is on the deterioration caused due to chloride ion penetration.

Formation of rust not only decreases the cross section of the rebar but its volume is also 3-4 times larger than the reactants and as a result of this volumetric expansion, tensile hoop stresses build up around the rebar causing cracking in concrete.[3] Since concrete is a brittle material with very low tensile stress capacity, it starts forming radial cracks around the rebar which propagate up to the bridge-deck surface rapidly causing spalling of the concrete cover. This provides an easier path for the corrosive agents to reach the rebar, resulting in a self-feeding mechanism and accelerates the deterioration of the structure. The first stage of building up of chloride ions around the rebar to threshold levels is called the initiation stage followed by the propagation stage involving active corrosion of the rebar.[4]

Corrosion protection methods used in the field to prevent early deterioration of R/C structures mainly focus on delaying the initiation period because the propagation of corrosion is difficult to control in reinforced concrete. Commonly used techniques include improving the concrete microstructure by using lower w/c ratio, tighter packing particles and pozzolans such as slag, fly ash, and silica fume, increasing concrete cover thickness, and adopting epoxy coated bars or corrosion inhibitors. These methods can be effective if the concrete is not cracked. However, in practice, due to restrained shrinkage, thermal deformations, chemical reactions, poor construction practices, and mechanical loads, concrete unavoidably cracks creating an easy entry path for the corrosive agents to quickly reach the rebar depth, thereby limiting the effectiveness of the above corrosion initiation prevention methods.

Engineered cementitious composite (ECC) is a micromechanically designed high performance fiber reinforced cementitious composite (HPFRCC) with ultra high ductility (300 to 500 times more than concrete) and tight crack widths (less than 100 μm) even at large imposed deformations (Figure 1), which make it highly damage tolerant and durable under normal service conditions.[6] Previous researches on the transport properties of ECC suggest that microcracked ECC strained in tension up to 3% exhibits water permeability and effective chloride ion diffusivity comparable to uncracked concrete, by virtue of its intrinsically tight crack width.[7,8] The difference in performance between ECC and concrete is even more significant during the corrosion propagation stage as ECC can sustain tensile hoop stresses around the rebars without spalling by virtue of its tensile ductility. Thus ECC presents a good potential for enhanced corrosion resistance and service life of steel reinforced concrete infrastructure subjected to aggressive environments.

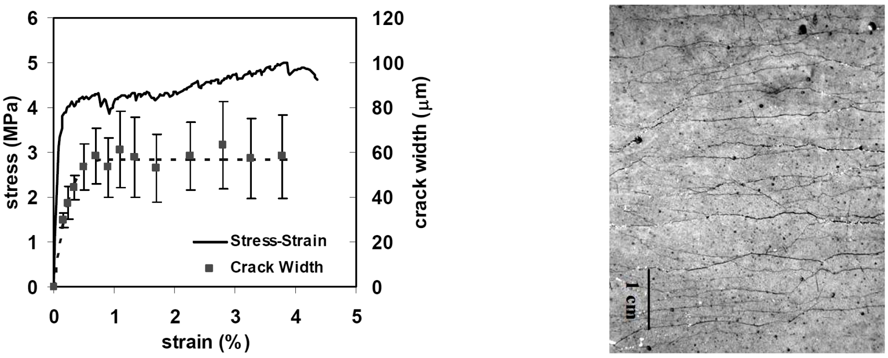

Figure 1: Typical direct tension stress-strain behavior of ECC and microcrack pattern

In this paper, service life and life cycle costs of a reinforced ECC (R/ECC) bridge deck and a reinforced concrete (R/C) bridge deck are quantitatively determined and compared, based on experimental data from previous studies on ECC and R/C cracking and rehealing behavior, and on chloride transport properties of cracked samples. Additionally, Life-365,[9] which is an industry standard software used by design consultants in North America to estimate the service life and life cycle costs of steel-reinforced concrete structures exposed to chlorides, is adopted in this analysis. This life-cycle model is used because of its simplicity and familiarity to service life engineers in the US. A more comprehensive (but more complex) model[2] accounting for sustainability indices beyond cost is available.

2. METHODOLOGY

Figure 2 shows a framework used to predict the service life and life cycle costs of any structural application made with R/C or R/ECC. This framework is general and can also be used effectively with other life cycle models. The first step is to choose a structural application, i.e. bridge deck in this paper, its geometry and reinforcement configuration along with the geographical location, i.e. Detroit urban regions. After this, a material is chosen, i.e. R/C or R/ECC. Based on these information and previous observations, crack patterns (i.e. average crack width and crack number) are determined for the material at the service strain level for the chosen structural application. The crack pattern in turn is employed to determine the appropriate value of chloride diffusion coefficient for use in the model in conjunction with experimentally determined data on effective diffusion coefficient of chloride ions in virgin and preloaded (cracked) ECC and concrete. The influence of crack self-healing in ECC is incorporated by adjusting the values of the diffusion coefficient due to changes in crack width and number resulting from self-healing. The diffusion coefficient along with the structural configuration specified above is then fed into Fick's law as input to estimate the service life for corrosion initiation. The bridge deck repair schedule is then determined based on structural service life, and the life cycle costs can be calculated. In the Life-365 software, the time to first repair is taken as the sum of time to initiation (t_i) and time for propagation (t_p) of reinforcement corrosion. After that, repairs are assumed to be performed every 10 years up to the end of analysis period (100 years in this study) for both R/C and R/ECC. It is further assumed that every repair event fixes 10% of the bridge deck's surface area, which is the default value used in Life-365. Table 1 lists the values for material costs and economic parameters used in the life cycle cost analysis. The total life cycle costs include construction costs and the subsequent repair costs. In the present study, a major departure from previous analysis is the explicit accounting of the effects of cracking in R/C and R/ECC structures on service life and life cycle cost.

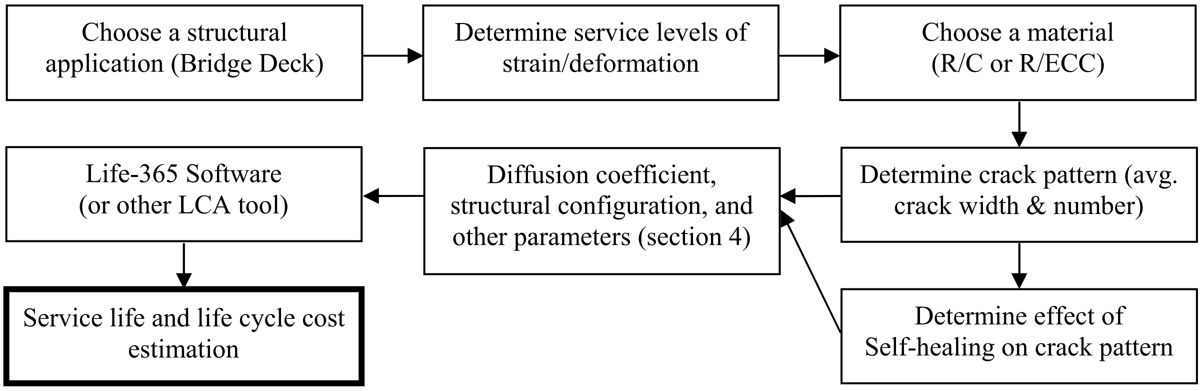

Figure 2: Service life and life cycle cost estimation framework

Table 1: Values of economic parameters used in Life-365 life cycle cost analysis

Construction (Material) Costs		Repair Costs	
Concrete	$100/cu.yd.	Repair	$37.16/sq.ft.
ECC	$300/cu.yd.	Area to repair	10% of surface area
Steel	$0.45/lb	Fixed repair interval	10 years
Inhibitor	$5.68/gal		
Other Parameters			
Base Year	2010	Analysis Period	100 years
Real Discount Rate	3.00%	Inflation Rate	1.6%

3. SYSTEM DEFINITION

The bridge design analyzed in this study is based on an overpass with a steel reinforced concrete deck located in an urban area in Detroit, Michigan. The bridge deck is 230 mm deep and rests on steel girders supported by a steel reinforced concrete substructure. The clear cover of the steel reinforcement is 60 mm. The analysis period, over which cost is accumulated and life cycle costs are calculated, is specified as 100 years. The type of chloride exposure depends on geographic location and structure type. It strongly influences the rate of chloride ingress and the corrosion initiation time. For an urban highway bridge deck in Detroit, Michigan, the temperature history, maximum level of chloride buildup over the bridge's lifetime, and the time for the buildup to reach its maximum level are shown in Figure 3, which are adopted from the database of Life-365.

Two types of materials, R/C and R/ECC, are included in this study. Three scenarios for R/C are investigated, considering three levels of cracking in concrete. In reality, crack width in concrete structures is highly variable. In ACI 318 Codes, prior to the 1999 edition, provisions were given for reinforcement distribution based on empirical equations using a 400 μm maximum crack width. In the current edition, provisions for reinforcement spacing are intended to limit surface cracks to a width that is "generally acceptable in practice but may vary widely in a given structure".[10] In this study, the crack width of concrete is assumed to be: (i) 0 (never cracked, which is rarely achieved in practice); (ii) 200 μm; and (iii) 400 μm (the maximum crack width specified by ACI codes prior to the 1999 edition).

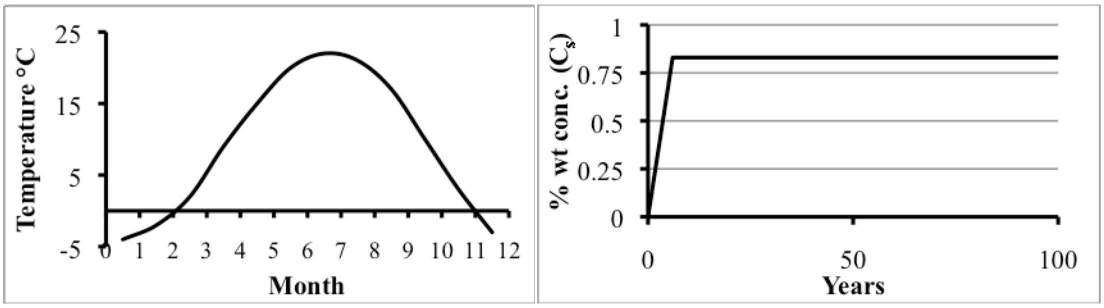

Figure 3: Temperature history and chloride exposure, urban highway bridges, Detroit, MI

Five scenarios for R/ECC are investigated in this study, considering three levels of cracking and the potential effects of self-healing in ECC. In contrast to R/C, crack width in R/ECC is an inherent material property and remains unchanged (Figure 1) with increasing deformation or load until the final failure, defined as the onset of localized fracture. Although the crack width of ECC remains constant, the crack number increases with increasing imposed deformation. The levels of tensile strain imposed on ECC specified in this analysis are: (i) 0 (no tensile strain imposed and no cracking, which is difficult to achieve in reality); (ii) 0.3%, approximately twice the shrinkage strain in ECC[11]; and (iii) 0.5%. Furthermore, the 0.3% and 0.5% tensile strain scenarios consider: (i) no self-healing and (ii) self-healing in ECC. In total, eight scenarios for R/C and R/ECC are investigated in this study. It should be noted that the 0.3% and 0.5% tensile strain levels are considered to be high in bridge deck structures. They are deliberately chosen in order to explore the ductility potential of ECC. At these two tensile strain levels, it is highly possible that crack width in R/C is much larger than 400 μm, considering the brittleness and 0.01% elastic strain capacity of concrete in tension. Therefore, this comparison between R/C and R/ECC in this study is unfavorable to R/ECC and provides a conservative evaluation of R/ECC durability performance.

4. SERVICE LIFE PREDICTION

It is commonly accepted from the Tutti deterioration model[2] that the corrosion of reinforcement can be divided into two phases, viz. initiation phase, in which chloride ions penetrate the concrete cover and build around the rebar to a threshold value in time t_i, and the propagation phase, where the reinforcement actively corrodes in time t_p. The initiation period (t_i) is a function of the concrete quality, depth of cover, the exposure conditions, including the concentration of chloride at the structural surface and the ambient temperature, and the threshold chloride concentration, C_t, required to initiate corrosion.

Fick's second law of diffusion (Equation 1) is used to predict the corrosion initiation period (t_i), which assumes that ionic diffusion is the governing mechanism of chloride transport through the concrete (or ECC) cover in the bridge deck.

$$\frac{dC(x,t)}{dt} = D_e(t,T)\frac{d^2C(x,t)}{dx^2} = \left[D_{ref} f(t,T)\right]\frac{d^2C(x,t)}{dx^2} \tag{1}$$

where, C(x,t) is the chloride ion concentration as percentage weight of cement at "x" cm from the concrete surface after "t" seconds, and De is the effective chloride diffusion coefficient in cm2/sec, which, in Life-365, is computed as a product of reference diffusion coefficient (Dref) at reference (room) temperature (20°C) at 28 days age, and a function f(t,T) to account for time (reflecting concrete maturity) and temperature.

The diffusion coefficient of ECC was found to vary linearly with the number of cracks (with crack width intrinsically constant even as imposed deformation increases), whereas the diffusion coefficient of R/C is proportional to the square of the crack width.7 According to Figure 4, the reference diffusion coefficients of the materials in this analysis (Dref) were determined based on the crack patterns for the two materials and tabulated in Table 2.

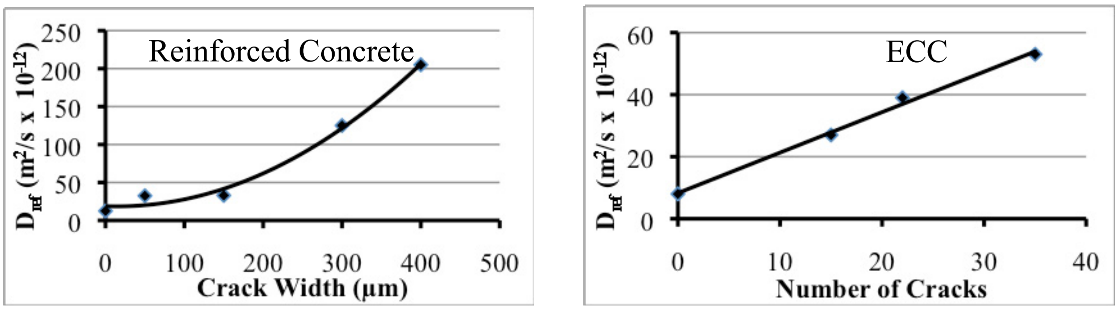

Figure 4: Chloride ion diffusion coefficient variation with crack pattern in R/C and ECC[7]

When the self-healing effect in ECC is considered, it is assumed that cracks with width less than 30 μm can be fully healed, leading to a reduction in crack number in ECC. This assumption is based on the experimental studies[12] carried out on ECC specimens pre-loaded up to 0.3% and 0.5% respectively under uniaxial tension, which regained their mechanical and transport properties after 10 wet/dry cycles. By studying the crack width distribution in ECC loaded to 0.3% and 0.5% tensile strain levels respectively, the crack number after self-healing effect is re-calculated for each case and the reference diffusion coefficient is then determined based on Figure 4.

The threshold chloride concentration, C_t, is strongly influenced by whether or not chemical corrosion inhibitor is used, and varies with the inhibitor dosage. In this study, 0, 15, and 30 liters/m^3 of 30% solution calcium nitrite inhibitor (CNI) are considered, corresponding to C_t (by % weight of concrete) equaling 0.05, 0.24, and 0.40.[8]

The corrosion propagation period, t_p, is assumed to be 60 years for R/ECC, according to a mechanistic corrosion model that calculated the time for the hoop strain, induced by the expanding reinforcing steel within ECC, to exceed strain capacity of ECC. t_p = 6 yrs is assumed for R/C, based on the studies of Weyers and others[13] who determined that time between corrosion initiation and cracking varied in the range from 3 to 7 years for R/C bridge decks in the U.S. The t_p = 60 yrs for R/ECC is based on the damage model analysis result by

Lepech[2] who accounted for the large tensile strain capacity of ECC to resist the radial fracture caused by rust expansion of the rebar. The substantially larger propagation time for R/ECC compared with that for R/C is further supported by experimental study on R/ECC and R/C cylinders subjected to accelerated corrosion by electrochemical method.[14]

5. RESULTS AND DISCUSSION

The results of the comparative service life prediction and cost analysis for a conventional steel reinforced bridge deck and a steel reinforced ECC bridge deck over a 100-year time horizon are presented in Table 2 below.

Table 2: Results of the study - Service life and life cycle costs of various materials using varying amount of inhibitors with and without self-healing (SH)

Material [D_{ref} in m^2/s]	No Inhibitor C_t: 0.05% of concrete wt		Inhibitor: 15 liter/m^3 C_t: 0.24% of concrete wt		Inhibitor: 30 liter/m^3 C_t: 0.40% of concrete wt	
	*[3]Service Life (yrs)	*[4]Life Cycle Cost (\$/m^2)	Service Life	Life Cycle Cost (\$/m^2)	Service Life	Life Cycle Cost (\$/m^2)
RC - uncracked [6.73E-12]	8.9 + 6 = 14.9	44 +183 = 227	25.2 + 6 = 31.2	44 +126 = 170	56.5 + 6 = 62.5	44 +57 = 101
RC - CW*[1] = 200 μm [6.54E-11]	1.9 + 6 = 7.9	44 + 212 = 256	5.2 + 6 = 11.2	44 + 191 = 235	7.7 + 6 = 13.7	44 + 186 = 230
RC - CW = 400 μm [2.06E-10]	1.1 + 6 = 7.1	44 + 213 = 257	3.5 + 6 = 9.5	44 + 206 = 250	5.3 + 6 = 11.3	44 + 191 = 235
RECC uncracked [6.73E-12]	8.9 + 60 = 68.9	90 + 52 = 142	25.2 + 60 = 85.2	90 + 24 = 114	56.3 + 60 = 116.3	90 + 0 = 90
RECC ε_t*[2]= 0.3% [2.12E-11]	3.9 + 60 = 63.9	90 + 56 = 146	9.3 + 60 = 69.3	90 + 52 = 142	17.9 + 60 = 77.9	90 + 37 = 127
RECC ε_t = 0.3%, SH [6.75E-12]	8.9 + 60 = 68.9	90 + 52 = 142	25.2 + 60 = 85.2	90 + 24 = 114	56.3 + 60 = 116.3	90 + 0 = 90
RECC ε_t = 0.5% [3.48E-11]	2.8 + 60 = 62.8	90 + 57 = 147	6.9 + 60 = 66.9	90 + 54 = 144	11.8 + 60 = 71.8	90 + 40 = 130
RECC ε_t = 0.5%, SH [1.30E-11]	5.5 + 60 = 65.5	90 + 54 = 144	13.6 + 60 = 73.6	90 + 39 = 129	29.1 + 60 = 89.1	90 + 23 = 113

*[1] CW: Crack Width   *[2] ε_t: Tensile strain   SH: With Self-Healing
*[3] Service Life = $t_i + t_p$ and   *[4] Life Cycle Cost = Construction (Material) Cost + Repair Cost

In the following discussion, we only highlight the effects on the corrosion initiation time and resulting gains in life cycle costs. The extraordinary gains in propagation time by using ECC instead of concrete can be attributed to the high tensile strain capacity of concrete and tight crack widths as explained above and has been well documented in previous researches.

In absence of inhibitor, uncracked R/C and R/ECC have the same t_i = 8.9 yrs. Cracking in concrete significantly reduces the initiation time from 8.9 yrs to 1.9 yrs (CW = 200 μm) and 1.1 yrs (CW = 400 μm). In contrast, cracking in ECC has a more moderate influence on the reduction of t_i. Under 0.3% tensile strain, t_i of R/ECC is reduced from 8.9 yrs to 3.9 yrs, and the self-healing effect brings t_i back to 8.9 yrs. In terms of life cycle cost (LCC), even when subjected to 0.5% tensile strain level, the life cycle cost (LCC) of the R/ECC bridge deck is 64% of the R/C bridge deck if concrete is uncracked.

Corrosion inhibitor (dosage: 15 liters/m^3 of mix volume) significantly extends t_i for R/C and R/ECC from 8.9 yrs to 25.2 yrs, when uncracked. However, t_i of cracked R/C is greatly reduced from 25.2 yrs to 5.2 yrs (CW = 200 μm) and 3.5 yrs (CW = 400 μm). For ECC subjected to 0.3% tensile strain, t_i is reduced from 25.2 yrs to 9.3 yrs, and the self-healing effect fully restores t_i to 25.2 yrs; for ECC subjected to 0.5% tensile strain, t_i is reduced from 25.2 yrs to 6.9 yrs, and the self-healing partially restores it to 13.6 yrs.

Increasing the inhibitor dosage to 30 liters/m^3 further improves t_i for both R/C and R/ECC. Comparing R/C and R/ECC with service level cracks, the LCC of R/C (CW = 200μm) having very high dosage of inhibitor (30 liter/m^3) is about 150% that of 0.3% strained R/ECC with no inhibitor. Therefore, ECC itself acts as a cost effective "inhibitor".

6. CONCLUSIONS

For sound concrete, inhibitor is effective in prolonging the corrosion initiation stage. However, cracking in concrete drastically reduces the corrosion initiation time despite using a large amount of inhibitor (up to 30 liters/m^3). In contrast, inhibitor retains its effectiveness in R/ECC due to its "smeared" multiple micro-cracking with tight crack width below 100 μm compared to the "localized" macro-cracks in concrete, whose crack width is difficult to control and widely varies with imposed deformation and structural properties. The corrosion inhibitor and tight crack width of ECC contribute synergistically to prolong corrosion initiation time. Without corrosion inhibitor, the R/ECC bridge deck has a longer service life than the R/C bridge deck - solely by prolonging the corrosion propagation time through the large tensile strain capacity of ECC. Results from this study show that 100-year service life is not difficult to obtain in an R/ECC bridge deck, even for 0.3% and 0.5% imposed strain levels. Self-healing of R/ECC further prolongs the service life by 3-5 years when no inhibitor is used, and by up to 40 years when a large amount of inhibitor is used.

The LCC of R/ECC is competitive with R/C in all cases (with or without inhibitors) but particularly when cracks are taken into consideration. In the crack damaged cases considered in this paper (Table 2), the averaged LCC per square meter of R/ECC is \$140 while that for R/C is \$244, a 43% reduction. If self-healing is taken into account, the R/ECC averaged LCC per square meter is further reduced to \$122, a 50% reduction from that of R/C. However, it should be cautioned that these cost calculations are narrowly focused on materials and repair cost only (as is performed in Life-365) for simplicity. A more complete life cycle analysis should consider other agency cost, user cost and environmental costs. Because of the reduced impact on traffic flows due to lesser repair events, it may be expected that the advantages of R/ECC would be further amplified when a complete life cycle analysis is performed. Since the placement of ECC employs common construction equipment such as ready mix trucks, there are no additional upfront costs. However, the higher ECC material cost spent earlier in the life cycle would incur a penalty when future discount rates are considered.

This study supports the notion that ECC can contribute to extending service life and reducing life-cycle cost of steel reinforced infrastructure, through its unique cracking and re-healing characteristics.

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