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NON-BRITTLE CONCRETE FOR DURABLE INFRASTRUCTURE IN COASTAL REGIONS

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Abstract

Corrosion related deterioration is a great concern for reinforced or prestressed concrete structures in coastal regions. It leads to a reduction of structural service life and increased life cycle costs, social and environmental burdens. Current concrete materials, especially high strength concrete, are susceptible to cracking due to their brittle nature, which greatly compromises concrete's resistance to chloride penetration and cover spalling. Such limitations call for a new generation of non-brittle concrete materials, Engineered Cementitious Composites (ECC), which intrinsically controls cracking under combined mechanical and chloride exposure and exhibits large tensile ductility to suppress cover spalling. This paper discusses the unique microcracking, self-healing and tensile ductility behavior of ECC and their contribution to prolonging corrosion initiation and propagation time. A case study of a marine structure revealed that reinforced ECC can achieve a service life approximately 10 times that of reinforced concrete under the same mechanical loading and chloride environmental exposure levels, and reduce life cycle cost by 43-50%. This paper proposes that ECC, as a non-brittle concrete, can be used to significantly improve the durability of concrete infrastructure in coastal regions.

1. INTRODUCTION

Deterioration of civil infrastructure is a serious concern faced by many countries. The US infrastructure was assigned an average grade of D in 2009 by the American Society of Civil Engineers, with an estimated need of US$2.2 trillion over five years for repairs and retrofits[i]. Infrastructure deterioration has important ramifications in social, environmental and economic costs.

Worldwide, corrosion of reinforcing steel has been identified as the most prevalent and damaging form of deterioration in reinforced concrete (R/C) structures[ii]. Steel corrosion reduces service life of structures and can also lead to safety concerns, as witnessed by the 2006 de la Concorde overpass collapse near Montreal, Canada that was linked to reinforcing steel corrosion, amongst other contributing factors[iii]. Corrosion related deterioration is particularly serious in coastal regions where the climatic conditions and saltwater exposure promote rapid corrosion of embedded reinforcing steel once chloride penetrates through the concrete cover[iv]. For example, in the Persian Gulf, where R/C structures are exposed to one of the most aggressive environments in the world, premature structural deterioration is a major economic concern[v,vi,vii].

The mechanisms leading to corrosion of embedded steel in concrete structure are well known. The penetration of chloride through the concrete cover depassivates the embedded steel, and allows the setting up of an electro-chemical cell in the presence of oxygen and water. These conditions are easily met in marine environments. It is a common notion that a dense microstructure in the form of high performance concrete or high strength concrete (HSC) can provide greater resistance to water permeation and chloride ion diffusion than normal concrete (NC). However, this corrosion protection approach has not been effective in extending the service life of R/C bridge decks in the US[viii], as the increasing adoption of HSC for R/C bridge decks did not slow down their deterioration over the last thirty years.

Mehta[8] presented a holistic approach to understanding infrastructure deterioration (Figure 1). This approach applies to various types of concrete deterioration but is particularly suitable to describing the stages leading to embedded steel corrosion. Stage 1 involves the gradual loss of water tightness of the concrete cover due to cracking resulting in penetration of water, oxygen, carbon dioxide and chloride ions through the cracks. Stage 2 involves the initiation and propagation of corrosion damage. As pointed out by Mehta high strength is not equivalent to high durability in concrete structures, as HSC is more susceptible to cracking[ix]. With more finely ground particles and lower water/cement ratio, HSC has higher early-age autogenously shrinkage than NC. Restrained shrinkage induces larger tensile stress in the HSC due to its higher elastic modulus and smaller creep deformation, and results in cracking. This is supported by the full-depth crack development observed in over 100,000 bridge decks within their first month of service[x]. Thus, a faster flow of salt-laden water channels through the cracks, rather than a slow diffusion of chloride through uncracked concrete, shortening the corrosion initiation time.

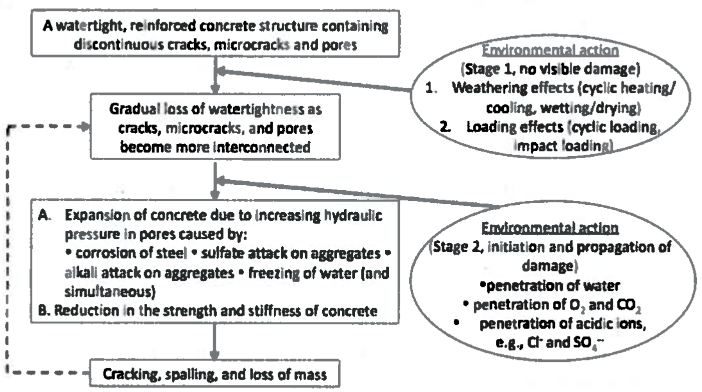

Figure 1: A Holistic Model of Concrete Deterioration from Environmental Effects[8]

These ideas were captured in a cracking potential parameter p[xi] defined as

$$p=\varepsilon_{sh}-(\varepsilon_e+\varepsilon_i+\varepsilon_{cp}) \qquad (1)$$

where ε_{sh} is the material's shrinkage strain that drives it to crack. ε_c, ε_i and ε_{cp} are elastic tensile strain capacity, inelastic tensile strain capacity (0 for HSC and NC) and tensile creep strain, the sum of which accounts for the material's ability to reduce the tensile strain build-up. The tensile stress build up is then equal to p multiplied by the material elastic stiffness. Table 1 shows that the p values of HSC are higher than that of NC.

Table 1: Shrinkage cracking potential values

Properties	ε_{sh} (%)	ε_e (%)	ε_i (%)	ε_{cp} (%)	p (%)
Concrete	0.07	0.01	0	0.02 - 0.06	0 - 0.04
HSC	0.10	0.01	0	0.037	0.053
ECC	0.177	0.015	2.5 - 5	0.07	(-4.908) - (-2.408)

Cracks in concrete renders corrosion control methods such as corrosion-inhibiting admixtures less effective[xii]. Epoxy-coated steel bars need to be applied in conjunction with crack-free concrete not constantly wet and other exposure conditions are not as severe[xiii]. These considerations call for a new concrete material with tight crack width control in order to resist chloride penetration and steel corrosion.

Engineered cementitious composite (ECC) is a micromechanically designed high performance fiber reinforced cementitious composite (HPFRCC) with high tensile ductility (300 to 500 ***times more than concrete) and tight crack widths (less than 100 m) even at large imposed*** deformations (Fig. 1). Therefore it is highly damage-tolerant and durable under normal service conditions[xiv,xv,xvi]. This paper discusses and quantifies ECC's potential in prolonging the corrosion initiation and propagation times by virtue of its unique damage tolerant and crack width control properties. The contribution of ECC self-healing to further enhance structural service life is also examined.

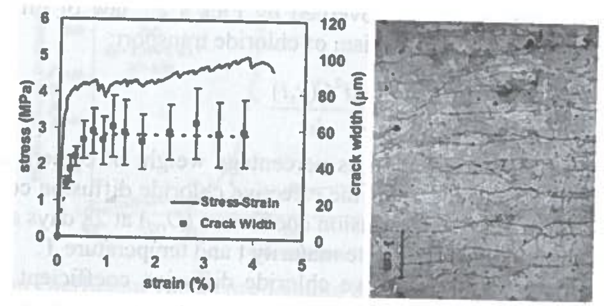

Fig. 1: Typical uniaxial tensile stress-strain behavior of ECC and microcrack pattern

2. ENGINEERED CEMENTIOUS COMPOSITES

The tensile strain-hardening behavior (Figure 1), i.e. increasing load capacity with increasing straining without localizing into a fracture plane, distinguishes ECC from normal fiber reinforced concrete (FRC) that exhibits tension-softening after first cracking. The tight crack width of ECC during its strain-hardening stage is independent of the applied deformation level, reinforcement ratio and structural member dimensions[xvii] – it is an intrinsic material property. It has been experimentally demonstrated that R/ECC structural members, such as beams[xviii], columns[xix], walls[xx], and connections[xxi] surpass normal R/C structural members in structural load carrying capacity, deformability, and energy absorption capacity under monotonic and reverse cyclic loading[xxii,xxiii,xxiv].

Recent studies suggest that the tensile ductility and tight crack width improve the durability of R/C structures by slowing chloride penetration and corrosion–induced cover cracking and spalling[xxv,xxvi,xxvii]. ECC overlays inhibit interfacial delamination caused by restrained volume change[xxviii], and eliminate reflective cracking under stress concentration and fatigue[xxix]. ECC material itself remains durable under various environmental conditions, including freezing and thawing, strong sulphate and chloride, and elevated temperature environments[xxvii].

The combination of high performance and moderate fiber content of ECC is achieved by the micromechanics-based composites optimization[xiv,xxx]. Micromechanics provides guidance in the selection and tailoring of the type, size, amount, and proprieties of ingredients at micrometer and nanometer scales. The ECC micromechanics-based design framework elevates the concrete materials design from conventional trial-and-error empirical combination of individual constituents to systematic material "engineering". ECC is emerging in full-scale applications in Asia, Europe, and the US[xxxi,xxxii,xxxiii,xxxiv].

3. ECC CORROSION RESISTANCE AND ITS IMPACT ON SERVICE LIFE

The corrosion of reinforcement can be divided into two phases[xxxv], namely the initiation phase, in which chloride ions penetrate through the concrete cover and build up around the rebar to a threshold value in time t_i, and the propagation phase, where the reinforcement actively corrodes until concrete cracking or spalling in time t_p. 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, and is governed by Fick's 2nd law of diffusion that assumes ionic diffusion as the governing mechanism of chloride transport:

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

$C(x,t)$ is the chloride ion concentration as percentage weight of cement at "x" cm from the concrete surface after "t" seconds. D_e is the effective chloride diffusion coefficient in cm²/sec computed as a product of reference diffusion coefficient (D_{ref}) at 28 days age and at 20℃, and a function $f(t,T)$ that accounts for concrete maturity t and temperature T.

Concrete cracking increases the effective chloride diffusion coefficient D_e. In contrast, the formation of dispersed microcracks in ECC can be regarded as inelastic straining $_i$ (3-5%) on the macroscopic structural scale, thus resulting in a highly negative cracking potential p (Table 1). Due to such unique cracking behavior, D_{ref} of ECC was found to vary linearly with the number of cracks (with crack width intrinsically constant as imposed deformation increases), whereas D_{ref} of R/C is proportional to the square of the crack width[28] (Fig. 2). Under imposed deformation, ECC multiple microcracking behavior leads to a D_{ref} significantly lower than that for R/C (Fig. 3). This lower D_{ref} of ECC increases the corrosion initiation time t_i.

The high tensile ductility of ECC leads to high corrosion-induced-spalling resistance and further prolonged service life of R/ECC structures by increasing the corrosion propagation period, t_p. According to a mechanistic corrosion model that calculated the time for the hoop strain, induced by rust expansion of the rebar within ECC, to exceed the tensile strain capacity of ECC, t_p is estimated to be 60 yrs for R/ECC[xxxvi]. This is ten times the 6 yrs corrosion propagation time for R/C based on the studies of Weyers et al[xxxvii]. The substantially larger t_p for R/ECC compared with that for R/C is supported by the accelerated corrosion studies[29] on R/ECC and R/C. Corrosion-induced crack width of R/C specimens increased with time as

corrosion activity progressed. Larger crack widths, up to 2 mm wide, were obtained at higher levels of corrosion. On the other hand, crack widths of R/ECC remained nearly constant (~ 60 µm) with time as corrosion activity progressed, while the number of cracks on the surface of the specimen increased. This study also showed that if the 0.3 mm maximum crack width limit for outdoor exposures as specified by AASHTO (2004)[xxxviii] were used to represent the serviceability limit of R/C structures, the service life of R/ECC would be at least 15 times that of R/C, indicating t_p = 60 yrs for R/ECC is a reasonable and conservative estimation.

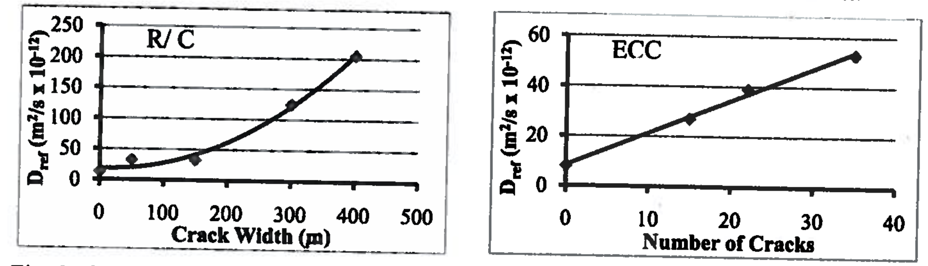

Fig. 2: Chloride ion diffusion coefficient variation with crack pattern in R/C and ECC[xxv]

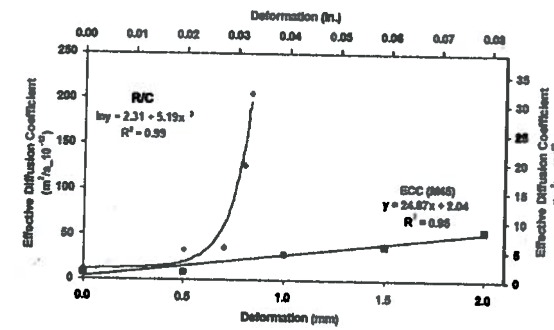

Fig. 3: Diffusion coefficient versus pre-loading deformation level for R/C and ECC[xxv]

After steel corrosion starts, the mass loss of steel results in reduction in stiffness and load carrying capacity of R/C members. According to Sahmaran et al[xxvi], after 25 hours of accelerated corrosion exposure, the flexural strength of a R/C beam was reduced to 34% of its original flexural capacity. In contrast, the R/ECC beam after 50 hours of accelerated corrosion exposure retained almost 100% of its original flexural capacity. Beyond 50 hours, the flexural capacity decreased, but retained over 45% that of the original capacity even after 300 hours of accelerated corrosion exposure. Longitudinal cracks due to expansion of the corrosion products also affected the failure mode of the R/C beam under four-point bend load. In contrast, steel corrosion within R/ECC did not modify the ductile failure mode in the R/ECC beam. This study suggests that the propagation period of corrosion could be safely included in estimating the service life of a structure when concrete is replaced by ECC.

Under combined mechanical loading and chloride exposure, ECC maintains its tensile ductility and multiple microcracking behavior with tight crack width. This is supported by experimental studies on pre-loaded ECC specimens (uniaxial tension up to 2% strain) that were exposed to two different chloride environment exposure conditions: (1) fully immersion in 3% chloride solution[xxxix], and (2) freezing and thawing cycles in the presence of de-icing salts for 25 and 50 cycles[xl]. The specimens were subsequently reloaded until failure to measure residual tensile properties. These results confirm that ECC, both virgin and micro-cracked, retains tensile ductility larger than 3% and microcrack width less than 100 μm to protect steel at both corrosion initiation and propagation stages, under combined mechanical loading and chloride exposure.

Self-healing[43] ***has been observed in concrete with cracks limited to less than 300 m and*** often substantially less. Although such tight crack width is difficult to achieve consistently in concrete, it is an intrinsic property of ECC, suggesting reliable self-healing in R/ECC structures. Reduction of chloride diffusion coefficient and recovery of mechanical properties in pre-loaded ECC specimens have been observed under concentrated chloride environment, indicating autogenous self-healing[xxv,xxxix]. Microcracks below 30 μm were fully healed, supporting the notion that self-healing reduces the crack number in ECC and leads to a further reduction in D_{ref} and increase in t_i.

4. R/ECC SERVICE LIFE AND LIFE-CYCLE COST PREDICTION

To illustrate quantitatively the enhancement of service life and reduction of life cycle cost of a structure in a marine environment, we choose a structure for analysis in this study located in a marine tidal zone in Key West, Florida. The structural member depth is 230 mm, and the clear cover of the steel reinforcement is 60 mm. Reinforcement ratio is 1.2%. The local temperature history, maximum level of chloride buildup, and the time for the buildup to reach its maximum level are shown in Fig. 4. The values of economic parameters used in life cycle cost analysis are summarized in Table 2.

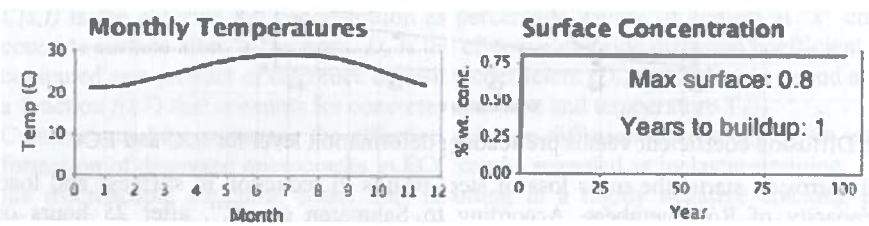

Fig. 4: Temperature history and chloride exposure, marine tidal zone, Key West, Florida

Table 2: Values of economic parameters used in life cycle cost analysis

Construction (Material) Costs		Repair Costs	
Concrete	$100/m³	Repair	$400/m²
ECC	$300/m³	Area to Repair	10% of surface area
Steel	$1.33/kg	Fixed Repair Interval After 1st Repair	10 years
Inhibitor	$1.5/L		
Other Parameters			
Base Year	2010	Analysis Period	100 years
Real Discount Rate	3.00%	Inflation Rate	1.6%

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[xli], prior to the 1999 edition,

provisions were given for distribution of reinforcements based on empirical equations using a calculated maximum crack width of 400 μm. In the current ACI 318-08 code[xlii], such limit on maximum crack width is no longer explicitly specified. Instead, 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". In this study, the crack width of concrete is assumed to be: (i) 0 (never cracked throughout the structure's lifetime, which is rarely achieved in practice); (ii) 200 μm (half of the maximum crack width specified by ACI codes prior to the 1999 edition); and (iii) 400 μm (the maximum crack width specified by ACI codes prior to the 1999 edition).

Five scenarios for R/ECC are investigated in this study, considering three levels of microcracking and the potential effects of self-healing in ECC. Although the crack number increases with increasing deformation or load, the crack width of ECC remains constant. Taking into account such microcracking behavior of ECC, the levels of tensile strain imposed on ECC, instead of crack width, are specified in this analysis as: (i) 0 (no tensile strain is imposed and cracking never takes place throughout the bridge deck lifetime, which is difficult to achieve in reality); (ii) 0.3% tensile strain, which is approximately twice the shrinkage strain in ECC[xliii]; and (iii) 0.5% tensile strain, which is more than three times the shrinkage strain in ECC[xliii]). Furthermore, the 0.3% and 0.5% tensile strain scenarios consider: (i) no self-healing in ECC and (ii) self-healing in ECC. The large 0.3% and 0.5% tensile strain levels are deliberately chosen to explore the ductility potential of ECC. It is greatly possible at such strain levels that crack width in R/C is much larger than 400 μm, considering the brittleness and 0.01% elastic strain capacity of concrete. Therefore, this comparison between R/C and R/ECC in this study is highly unfavorable to R/ECC.

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.

D_{ref} for R/C and R/ECC (before and after self-healing) are determined based on Fig. 2. 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 to determine the new reduced D_{ref} when self-healing is assumed.

The total service life, which is also the time to first repair, is taken as the sum of t_i and t_p. 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. As listed in Table 2, the total life cycle costs include construction costs, which account for the concrete, ECC, steel and corrosion inhibitor material costs, and the repair costs, all of which are discounted to their present values using inflation rate and real discount rate and summed up.

The service life prediction and life cycle costs (LCC) of R/ECC compared with R/C over a 100-year time horizon under different scenarios are summarized in Table 3. In the absence of inhibitor, uncracked R/C and R/ECC have the same t_i = 5.3 yrs. Cracking in concrete reduces the initiation time from 5.3 yrs to 0.8 yrs (CW = 200 μm) and 0.3 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 5.3 yrs to 1.8 yrs, and the self-healing effect brings t_i back to 5.3 yrs. Even when subjected to 0.5% tensile strain level, the LCC of the R/ECC is 62% of the R/C 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 5.3yrs to 20.6 yrs, when uncracked. However, t_i of cracked R/C is greatly reduced from 20.6 yrs to 1.8 yrs (CW = 200 μm) and 0.9 yrs (CW = 400 μm). For ECC subjected to 0.3% tensile strain, t_i is reduced from 20.5 yrs to 5.5 yrs, and the self-healing effect fully restores t_i to 20.5 yrs; for ECC subjected to 0.5% tensile strain, t_i is reduced from 20.5 yrs to 3.3 yrs, and the self-healing partially restores it to 9.5 yrs. The LCC of the R/ECC subject d to 0.5% tensile strain is only 78% of uncracked R/C.

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 cracks under service conditions, the LCC of R/ECC (0.5% ***strain level) with no inhibitor is 60% that of R/C (CW = m) with a high dosage of*** inhibitor (30 liter/m^3). Therefore, ECC is more effective than corrosion inhibitor for prolonging structural service life and reducing life cycle costs.

Table 3: Service life and life cycle costs of R/C and R/ECC

Material [D_{eff} 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]	5.3 + 6 = 11.3	44 + 191 = 235	20.6 + 6 = 26.6	50 + 146 = 196	55.2 + 6 = 61.2	55 + 57 = 112
RC - CW*^{1} = 200 um [6.54E-11]	0.8 + 6 = 6.8	44 + 215 = 259	1.8 + 6 = 7.8	50 + 212 = 262	4.2 + 6 = 10.2	55 + 193 = 248
RC - CW = 400 um [2.06E-10]	0.3 + 6 = 6.3	44 + 215 = 259	0.9 + 6 = 6.9	50 + 215 = 265	1.5 + 6 = 7.5	55 + 212 = 267
R/ECC uncracked [6.73E-12]	5.3 + 60 = 65.3	90 + 54 = 145	20.5 + 60 = 80.5	96 + 25 = 122	55.0 + 60 = 115.0	101 + 0 = 101
R/ECC ε_t*^{2} = 0.3% [2.12E-11]	1.8 + 60 = 61.8	90 + 57 = 148	5.5 + 60 = 65.5	96 + 54 = 150	14.7 + 60 = 74.7	101 + 38 = 139
R/ECC ε_t = 0.3%, SH [6.75E-12]	5.3 + 60 = 65.3	90 + 54 = 144	20.5 + 60 = 80.5	96 + 25 = 121	55.0 + 60 = 115.0	101 + 0 = 101
R/ECC ε_t = 0.5% [3.48E-11]	1.2 + 60 = 61.2	90 + 57 = 148	3.3 + 60 = 63.3	96 + 56 = 152	8.2 + 60 = 68.2	101 + 52 = 153
R/ECC ε_t = 0.5%, SH [1.30E-11]	2.8 + 60 = 62.8	90 + 56 = 147	9.5 + 60 = 69.5	96 + 51 = 147	26.2 + 60 = 86.2	101 + 23 = 124

*^{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

5. CONCLUSIONS

ECC is a non-brittle concrete material that can significantly improve durability of structures subjected to the aggressive chloride environment in coastal regions. This is achieved by ECC's multiple microcracking behavior with self-controlled crack width under 100 μm and high tensile ductility more than 3%. Due to its intrinsic tight crack width, even under large applied deformation or load, ECC has a low chloride diffusion coefficient. Self-healing of

microcracks recovers ECC transport and mechanical properties, and further prolongs corrosion initiation time in R/ECC. The large tensile ductility of ECC contributes to a 10 times (at least) longer corrosion propagation time in R/ECC than R/C, as it allows corrosion products to expand without fracturing the ECC cover. By prolonging both the corrosion initiation and propagation stages, R/ECC can achieve a service life approximately 10 times that of R/C at under the same mechanical loading level and chloride environmental exposure. This conclusion is supported by the acceleration corrosion studies and LCC analysis.

For sound concrete, inhibitor is effective in prolonging the corrosion initiation stage. However, cracking in brittle concrete drastically reduces the corrosion initiation time despite using a high 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 distinct from the "localized" macro-cracks in concrete, whose crack width varies widely and is difficult to control. The corrosion inhibitor and tight crack width of ECC contribute synergistically to prolonging corrosion initiation time. Without corrosion inhibitor, the R/ECC structure has a longer service life than the R/C structure - solely by prolonging the corrosion propagation time through ECC's large tensile strain capacity. Self-healing of R/ECC further prolongs the service life by 2-6 years when no inhibitor is used, and by up to 40 years when a high amount of inhibitor is used.

When subjected to an aggressive chloride environment in coastal regions, i.e. a marine tidal zone in Florida, 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. When cracks are accounted for, the averaged LCC of R/ECC is $148/m^2 while that for R/C is $260/m^2, a 43% reduction. If self-healing is taken into account, the R/ECC averaged LCC is further reduced to $131/m^2, a 50% reduction from that of R/C. It should be noted that such comparison is conducted under assumed conditions that are unfavorable to ECC (i.e. larger applied strain level on ECC). It should also be cautioned that these cost calculations are narrowly focused on materials and repair cost only 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 is expected that the advantages of R/ECC would be further amplified when a complete life cycle analysis is performed.

This study supports the notion that ECC can be a "future concrete" to be used for R/C structures in aggressive chloride environment, e.g. coastal region, where embedded steel corrosion is a severe concern.

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