A Model to Predict Life-Cycle-Cost of Reinforced Concrete Structures in Marine Environments

Mohamed R. Sakr*, Karim El-Dash, Osama El-Mahdy

Department of Civil Engineering, Faculty of Engineering at Shoubra, Benha University, 108 Shoubra St., Shoubra, Cairo, Egypt

* Corresponding author

Tel.: +20 114 440 72 81

E-mail address: mohamed.ramadan@feng.bu.edu.eg (M.R. Sakr)

Abstract

Chloride induced corrosion is the main cause of deterioration of reinforced concrete structures in marine environment. Corrosion damages require huge expenses to be repaired. Thus, it is required to design structures that need less repair in their life. A model to estimate Life-Cycle-Cost (LCC) of reinforced concrete structure in marine environment is proposed to be used as either an evaluation tool of a certain design, or as selection tool between suggested designs. It is concluded that life-cycle-cost analysis is an important tool to be used in assessing different specifications. Also, basing the decision upon the initial investment only can lead to an erroneous judgment. The total cost through the intended life has to be considered to choose the cost-effective solution as low initial investment can mean high maintenance and repair cost where the total cost may exceed high investments with low maintenance and repair costs.

1. Introduction

Reinforced Concrete (RC) is the most frequently used material in construction all over the world due to its versatility and availability of constituents. Structures can be exposed to severe exposure conditions during their service life resulting in deterioration of concrete. Deterioration forces owners to spend a significant part of their budget on rehabilitation (or replacement) of the existing structures when damage reaches a certain limit. Thus, there is a strong financial incentive to design RC structures which will require less maintenance and repair over their life time.

Marine environment is characterized by high temperature and relative humidity in addition to chloride attack. Under these conditions, chloride induced corrosion is the major deterioration mechanism of reinforced concrete structures [1]. Corrosion adversely affects the structural behavior of the corroded element. It causes reduction of flexural stiffness, shear capacity of the element, and loss of bond strength [2-4]. Moreover, rust produced as a result of corrosion expands to develop tensile stresses at the steel/concrete interface which ultimately results in cracking of concrete along the rebar length and eventual spalling of the concrete cover [5]. The repair of corrosion damage costs a huge expenses estimated by US$ 2.2 trillion in the world [6]. Hence, more attention was given in the last decades to develop effective corrosion control strategies regarding performance and cost.

In this paper, a model is proposed for estimating the life-cycle-cost of reinforced concrete (RC) structures in marine environment where chloride induced corrosion is the main deterioration mechanism for onshore and offshore structures. The life-cycle-cost model will be based on predicting the service life of structures subjected to chloride induced corrosion. The cost model serves as a tool for practice engineer to estimate the costs incurred in the life of structure, and to choose the cost effective solution from certain suggested alternatives.

2. Life Cycle Cost Model

The life cycle cost (LCC) of an item is the sum of fund required for a product from its conception and manufacture through its operation to the end of its useful life [7]. Regarding a structure, LCC is the sum of all expected costs associated with design, construction, operation and maintenance plus all the expenses related to possible failure modes or disposal of the structure over a period of time.

In the proposed mathematical cost model, the following assumptions are made:

- Costs related to the structure occur at different times. So, in order to obtain consistent results, the present value method is used in which all the future costs are discounted to their equivalent present value (base year value) [8].
- The initial investment and erection costs occur through the planning/construction period but the model considers it to occur on the base year.
- The various operations, maintenance, and repair costs are spread throughout the year. In this model, all the costs are assumed to be occurring at one point that is at the end of the year.

Based on these assumptions, the discounted LCC of a structure in a time period T is stated as:

$$LCC(T) = C_{IN} + C_O(T) + C_M(T) + C_{LS}(T) + C_D \quad (1)$$

where C_{IN} is the investment cost including design, construction (materials and labor), and quality assurance in the base year, $C_O(T)$ is the expected operation cost in a time period T. $C_M(T)$ is the expected cost throughout a time period T of regular structure maintenance which is carried out periodically from the as new state, even in the absence of any sign of deterioration, $C_{LS}(T)$ is the expected cost state in a time period T associated with exceeding a certain damage limit "*limit state*", and it can be regarded as the cost of corrective/essential maintenance that is required to restore functioning of the structure when the performance is unduly affected, and C_D is the cost of disposing the structure when it becomes unable to fulfill its requirement or disposal is more economic than keeping the structure in service.

In case of chloride induced corrosion, the service life of structure can be divided into two phases: initiation and propagation [9]. In the initiation phase, chlorides pass through concrete until reaching steel surface with concentration enough to initiate corrosion. Corrosion then propagates until cracking and spalling appear on the external surface of concrete. The sum of initiation and propagation phases can be considered as the service life.

It is important to mention here that, the length of the corrosion propagation stage in concrete is found to be relatively short, typically a few years. As a result, much of the emphasis on achieving long service life is put on achieving a long corrosion initiation stage [10]. Therefore, this study will focus on the initiation phase as the allowed limit state to take into consideration any uncertainties that might be involved. In other words, service life will be limited to the length of corrosion initiation period.

Serviceability failure caused by cracking is considered to be the most influential mode of failure for the estimation of life cycle cost for RC structures in marine environment as the associated loss of structural capacity for RC structure will not exceed 20% at the first sign of cracking [11]. As the corrosion propagation rate until cracking can be rapid, the serviceability limit state in this model is conservatively assumed to be reached only when corrosion initiates. C_{LS} is the expected cost of *major repair* required to restore the structure performance when corrosion initiates. Noting that $C_O(T)$, $C_M(T)$ and $C_{LS}(T)$ are discounted to the base year.

Discounting can be done by using the simple formula of interest as follows [12]:

- The one-time costs which do not occur annually (e.g. the major repair cost at some intermediate time and the replacement cost at the end of service life) are discounted by multiplying the future cost by the single present value (SPV) factor which is:

$$SPV = \frac{1}{(1+r)^t} \tag{2}$$

  where r is the discount rate which depends on economic and political factors, and t is the period of time elapsed from the base year until cost occurs.

- The costs which recur periodically are discounted by the uniform present value (UPV) factor. The periodically recurring costs include the operation and regular maintenance costs. These costs may recur in uniform or non-uniform manner. But in this model, all these costs have been assumed to be uniform recurring costs. For annually recurring costs, UPV is calculated with the help of following relation:

$$UPV = \sum_{x=1}^{T} \frac{1}{(1+r)^x} \tag{3}$$

  where x is the number of the year.

Therefore Eq. (1) for LCC in a period T can be rewritten as:

$$LCC(T) = C_{IN} + \sum_{x=1}^{T} \frac{O_x}{(1+r)^x} + \sum_{n=1}^{T/\Delta t} \frac{M_j}{(1+r)^j} + \sum_{m=1}^{l} \frac{R_k}{(1+r)^k} + \frac{C_D}{(1+r)^T} \tag{4}$$

where O_x is the operation cost in year x, n and m are the number of regular maintenance and major repair incidents respectively, Δt is the period between regular maintenance incidents, l is the total number of major repair incidents, M_j is the regular maintenance cost in year j where ($j = n * \Delta t; j \leq T$), and R_k is the major repair cost in year k according to the number of major repair incident.

It is worth noting that after repairing a structure, it does not restore its original performance. The degree to which the repair is efficient depends on the technique of repair applied, quality of materials used, and the skill of repair team. Thus, if the first repair performed a corrosion initiation time, T_i is reached, the second repair will be needed after a period less than $2T_i$ as long as repair is performed using the same concrete as that of original structure. So, a repair efficiency factor α is introduced to be multiplied with the subsequent initiation times. The application of such a factor will be indicated in Section 4.

3. Calculation of Corrosion Initiation Time

Although chloride ions may transport through concrete by different mechanisms, diffusion due to concentration gradient is considered to be the dominant mechanism [13]. Therefore, the selected model is based on Fick's second law of diffusion which can be written in the form of the following partial differential equation for one-dimensional diffusion problem [14]:

$$\frac{\partial C(x,t)}{\partial t} = D\,\frac{\partial^2 C(x,t)}{\partial x^2} \tag{5}$$

where $C(x,t)$ is the chloride concentration at distance x from the chloride-exposed surface at time t, and D is the chloride diffusion coefficient of concrete. Noting that in order to solve Fick's law, a boundary condition (chloride concentration at surface), and initial condition (chloride content of the concrete mix) are required.

3.1 Expression for diffusion coefficient (D)

It is obvious from Eq. (5) that D is the key element which controls the penetration of chloride ions into concrete. In this model, three main points are considered regarding the diffusion coefficient: i) It decreases with time as the hydration process proceeds due to the refinement of pore structure [15], ii) Increasing temperature results in increasing the diffusion coefficient without changing the trend of chloride profiles [16], iii) Relative humidity significantly influences the diffusion coefficient and considered along with the temperature, the main environmental parameters that affect the chloride ingress into concrete [17,18]. So, the next equation will be adopted for diffusion coefficient to consider the previously mentioned points:

$$D = D_{ref} * F(t) * F(T) * F(RH) \tag{6}$$

where D_{ref} is the reference diffusion coefficient at reference time t_{ref}, $F(t)$, $F(T)$,and $F(RH)$ are factors multiplied D_{ref} to account for the age effect, temperature, and relative humidity respectively. Thomas and Bentz [19] propose D_{ref} based on a large database of bulk diffusion tests to be:

$$D_{ref} = 10^{(-12.06+2.40*w/c)}\ (\mathrm{m^2/s}) \tag{7}$$

where w/c is the water to cement ratio.

The age factor $F(t)$ represents the reduction of D with time, and is generally accepted to be expressed as [19-21]:

$$F(t) = (\frac{t_{ref}}{t})^m \tag{8}$$

where t_{ref} is the reference age (usually 28 days), t is the age of exposure and m is an age factor to account for the refinement of pore structure, and is proposed to be [19]:

$$m = 0.2 + 0.4(\%FA/50 + \%SG/70) \leq 0.6 \tag{9}$$

where $\% FA$ and $\% SG$ are the % amount of fly ash (≤50%) and slag (≤70%), respectively. The equation for age factor m considers fly ash and slag only. Silica fume is assumed to have no effect on the age factor, but affects the reference diffusion by a multiplied factor$\exp(-0.165 * \%SF)$, where $\% SF$ is the % amount of silica fume. Value of $F(t)$ continues to decrease until the hydration process is complete and no further pore refinement takes place for a period that is assumed to be 25 years. Beyond this point $F(t)$ remains constant at the 25 years value [19].

The factor *F(T)* represent the effect of temperature and is obtained from Arrhenius law as [16]:

$$F(T) = \exp\left[\frac{U_c}{R}\left(\frac{1}{T_{ref}} - \frac{1}{T}\right)\right] \tag{10}$$

where U_c is the activation energy of chloride diffusion in concrete, reported as 23, 39.9 kJ/mol for water to cement ratio 0.35 and 0.6, respectively [16] (assumed to be 35 kJ/mol in this study), R is the gas constant (8.314 J/mol K), T_{ref} is the reference temperature (293.15 K), and T is the ambient temperature.

The effect of relative humidity was taken into account by multiplying the reference diffusion coefficient by a factor *F(RH)* [17]:

$$F(RH) = \left[1 + \frac{(1-RH)^4}{(1-RHc)^4}\right]^{-1} \tag{11}$$

Where RH is the ambient relative humidity and RH_c is the reference relative humidity (assumed 75% [22])

3.2 Expression for surface chloride concentration (C_s)

Surface chloride concentration C_s represents the severity of the surrounding environment to the building, and the boundary condition of the diffusion problem. Adopting appropriate values for C_s is important to predict adequately the future chloride penetration [23]. It is found that C_s builds up at concrete surface and increases with time [24-26]. Ann et al. [27] proposed a realistic relation for surface chloride build-up ($Cs = C_0 + k\sqrt{t}$; where C_0 is the initial build-up of surface chloride at initial exposure, k is a constant under a linear build-up condition). In this model, this proposed relation is adopted, but the constant k is used as a constant under a square root build-up condition.

4. Numerical Example: Comparison Between Durability Design Alternatives

By way of illustration, the life time cost for various durability design alternatives of a 5-m wide by 5-m long slab are calculated and compared to select the cost-effective solution to be applied.The structural design requirements are assumed to be: 125 mm effective slab thickness and 5 steel bars of diameter 10 mm in each meter for each direction.

Durability is achieved by special design requirement which may include the use of good quality concrete with low water to cement ratio, corrosion resistant steel, and/or use increased concrete cover. For the purpose of comparison, four alternatives are considered using concrete of the same water to cement ratio, summarized in Table 1.

Table 1: Selected durability design alternatives.

	Alternative 1	Alternative 2	Alternative 3	Alternative 4
Steel type	Carbon	Carbon	Carbon	Stainless
Clear cover (mm)	35	50	65	20
Water to cement ratio	0.50	0.50	0.50	0.50

4.1 Corrosion initiation time

Based on the criteria for each alternative, and using the predictive model described in Section 3, the corrosion initiation time is estimated. According to the value of water to cement ratio, the reference diffusion coefficient D_{28} is 1.38E-11 m^2/s. It is assumed that no mineral admixtures are used, thus, the age factor m is 0.20. The surface chloride concentration C_s is assumed to be $0.2+0.4\sqrt{t}$ (% of weight of cement). The annual average temperature and relative humidity are assumed to be 30°C and 65%, respectively. Based on these conditions, the model is analyzed numerically using COMSOL Multiphysics (version 5) [28].The chloride concentration variation with time at a distance equal to the cover from the surface for each alternative is shown in Fig.1.

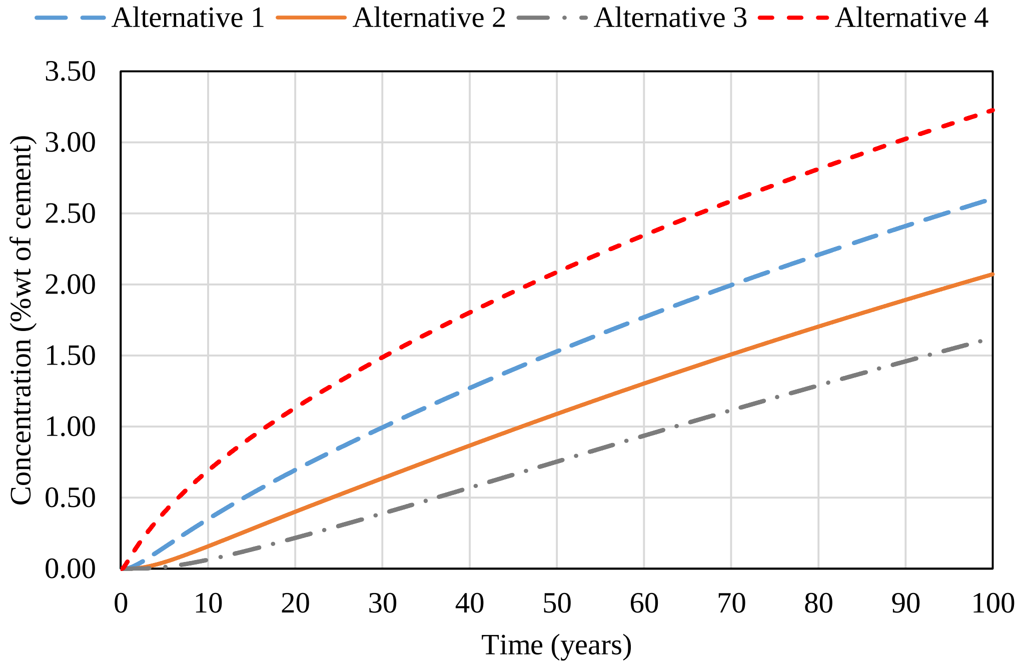


Fig.1: Variation of chloride concentration with time at cover depth for each alternative

The chloride threshold value at which corrosion initiates is assumed to be 0.20% when using carbon steel reinforcement [29]. For stainless steel, the threshold value is assumed to be 10 times that of carbon steel [19], i.e., 2% of weight of cement. Thus from the figure, it is illustrated that the predicted times to corrosion initiation for alternatives 1, 2, 3, and 4 are 6.2, 11.8, 19 and 46.8 years respectively.

4.2 Cost analysis

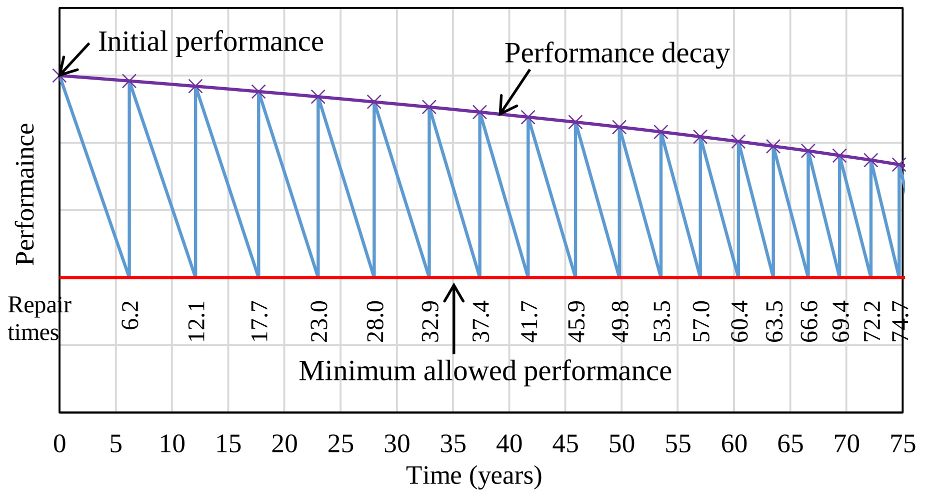
The cost of construction is the sum of material and labor costs. Material costs are the sum of concrete and steel reinforcement costs. In Egypt, labor costs are approximately estimated to be 30% of material costs for reinforced concrete members. Information about the proportions assumed to be used in mixing 1m^3 of concrete is given in Table 2. As indicted, once the corrosion initiated, repair is required to restore the structure functioning. The repair strategy scenario followed in this comparison is based on the next assumptions:

- Repair is carried out immediately once the corrosion initiated by removing concrete cover, cleaning steel surface, and placing another cover.
- Repair is performed using the same grade and water to cement ratio concrete by replacing the old concrete cover, cleaning steel, and placing a new cover of the same thickness, i.e. there is no improvement in durability performance.
- After repair, the element does not restore its original performance and the efficiency factor α is assumed to be 95%.The factor α is applied such as the corrosion initiation time required for the next repair is obtained by multiplying α with the initiation time of the current repair.

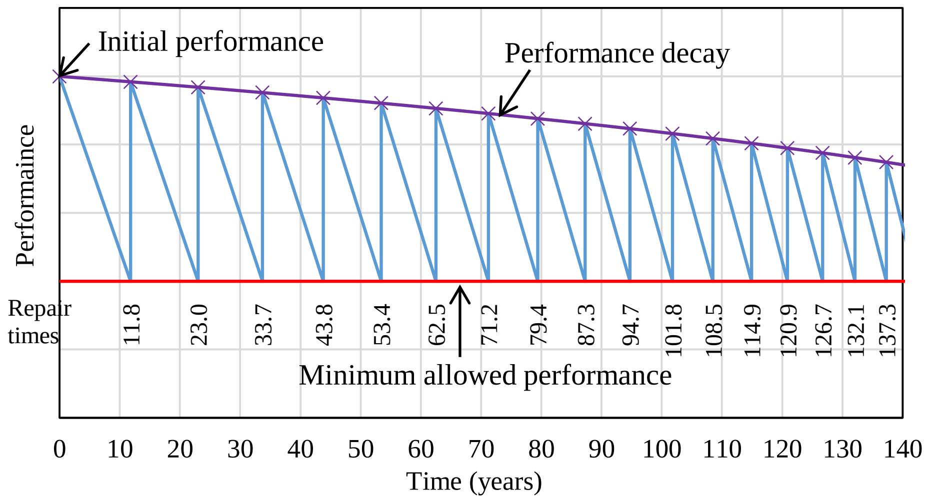
Figure 2 illustrates the performance degradation with time along with the required times for repair for each design specification. Repair times are shown at the drops of performance.

Table 2: Concrete mix design for 1m^3.

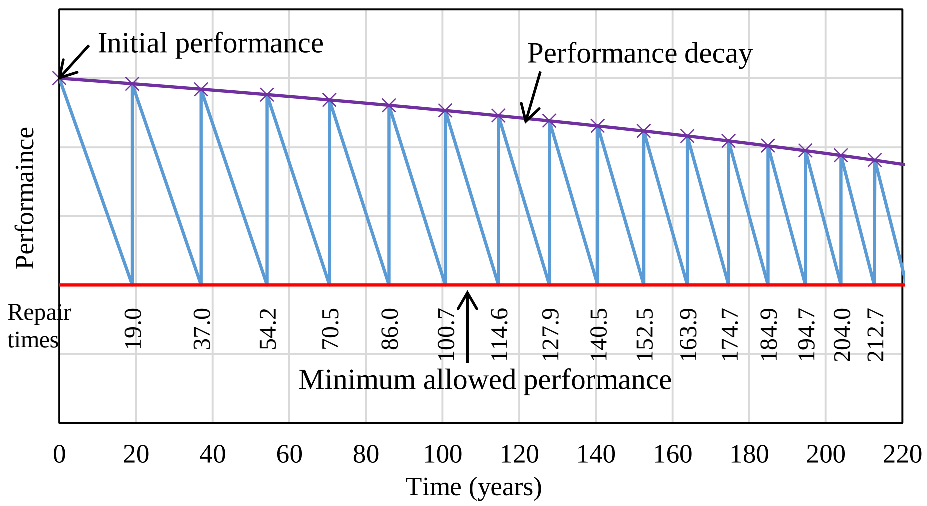
Item	Value	Item	value
Characteristic strength	32 MPa	Required cement content	400 kg/m^3
Water to cement ratio	0.50	Required water	200 kg/m^3
Specific gravity for cement	3.15	Required sand	546 kg/m^3
Specific gravity for sand	2.6	Sand volume	0.21 m^3
Fineness modulus for sand	2.6	Required gravel	1152 kg/m^3
Specific gravity for gravel	2.6	Gravel volume	0.44 m^3
Bulk density of gravel	1800 kg/m^3		


Initial performance
Performance decay
Performaince
Repair times
6.2 12.1 17.7 23.0 28.0 32.9 37.4 41.7 45.9 49.8 53.5 57.0 60.4 63.5 66.6 69.4 72.2 74.7
Minimum allowed performance
0 5 10 15 20 25 30 35 40 45 50 55 60 65 70 75
Time (years)

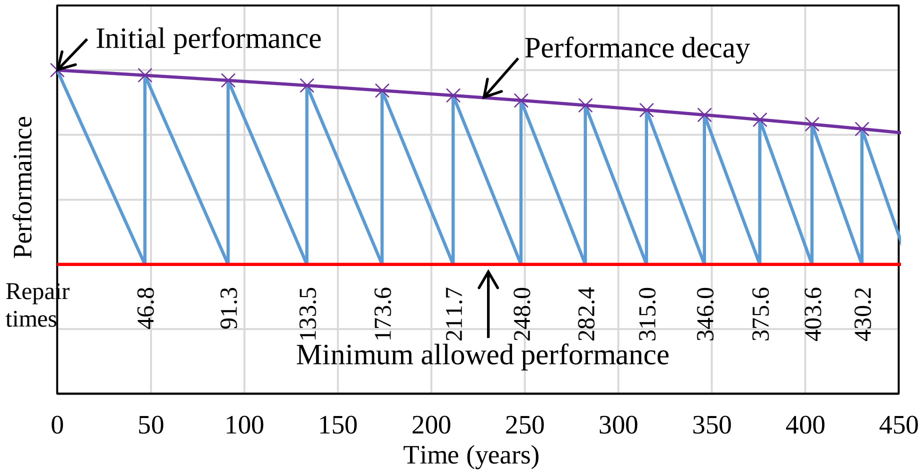
(a)


Initial performance
Performance decay
Performaince
Repair times
11.8 23.0 33.7 43.8 53.4 62.5 71.2 79.4 87.3 94.7 101.8 108.5 114.9 120.9 126.7 132.1 137.3
Minimum allowed performance
0 10 20 30 40 50 60 70 80 90 100 110 120 130 140
Time (years)

(b)


Initial performance
Performance decay
Performaince
Repair times
19.0 37.0 54.2 70.5 86.0 100.7 114.6 127.9 140.5 152.5 163.9 174.7 184.9 194.7 204.0 212.7
Minimum allowed performance
0 20 40 60 80 100 120 140 160 180 200 220
Time (years)

(c)



(d)

Fig.2: Performance degradation through time along with the required times for repair for alternative (a) 1, (b) 2, (c) 3, and (d) 4, respectively.

The costs associated with repair or replacement are structure and site specific. Hence, it is difficult to make generalization about these costs [11]. It is assumed that the cost associated with selected repair strategy equals double of the construction cost associated with the concrete cover. Also, it is assumed that design, operation, and regular maintenance costs are the same for different alternatives and so are not needed for this comparatives analysis.

In the Egyptian market, the cost of 1 ton of Portland cement is about 600 EGP, and the cost of 1m^3 of sand and 1m^3 of gravel are 35 and 100 EGP, respectively. Thus the cost of material used in one cubic meter of concrete neglecting the water cost can be calculated as:

$$Cost\ of\ 1\ m^3\ concrete\ = W_C * C_C + V_S * C_S + V_G * C_G = 0.4 * 600 + 0.21 * 35 + 0.44 * 100 = \ 291.4\ EGP/m^3 \qquad (12)$$

where W_C, V_S, and V_G are weight of cement (ton), volume of sand (m^3), and volume of gravel (m^3) respectively, and C_C, C_S, and C_G are costs of 1 ton, 1m^3, and 1m^3 of cement, sand and gravel respectively. The concrete cost of a certain thickness can be obtained by multiplying the volume of concrete (slab area [25 m^2] * thickness) by the cost of 1 m^3 concrete.

The cost of 1 ton of carbon steel reinforcement is 5500 EGP. The structural design of the slab specifies 5 bars of diameter 10 mm in each meter. Thus, the cost of steel bars is:

$$Cost\ of\ carbon\ steel = \gamma_S * V_S * C_{St} = 7.86 * 0.25\pi * (0.01)^2 * 250 * 5500 = 848\ EGP \tag{13}$$

Where γ_S is the density of steel material, V_S is the volume of all bars in the slab, and C_{St} is the cost of 1 ton of carbon steel. Noting that 250 is the sum of all bar lengths in the slab. Knudsen *et al.* [11] estimated the cost of stainless steel bars to be 6-9 times that of the carbon steel bars. Assuming that the cost of stainless steel is 7.5 times the cost of carbon steel, the cost of stainless steel is 7.5*848=6360 EGP.The material, labor and total construction costs at the base year for each alternative are shown in Table 3.

Table 3: Construction costs.

Alternative	Material cost (EGP)			Labor cost (EGP) (30% material cost)			Total construction cost (EGP) (material + labor)
	concrete		steel	concrete		steel	
	Effective depth (125 mm)	Cover	Steel	Effective depth (125 mm)	Cover	Steel	
1	911	255	848	273.5	76.5	254.4	2618.4
2	911	365	848	273.5	109.5	254.4	2761.4
3	911	474	848	273.5	142.5	254.4	2903.4
4	911	146	6360	273.5	44	1908	9642.5

The repair cost due to corrosion initiation can be estimated as shown in Eq. (4) where *R* is the future cost of repair and *r* is the discount rate that discounts the future cost value to the equivalent present value. Practically, each construction company should has a record of the past repair costs. Using this record, the future value of repair cost can be forecasted at any date. Also, the discount rate can be forecasted using the past data. So that, the future repair costs can be estimated as:

$$R = \sum_{m=1}^{l} \frac{\text{current repair cost} * (1+s)^{k}}{(1+r)^{k}} \qquad (14)$$

where s is the increase rate of repair cost, m is the number of major repair incident occurring at year k, and l is the total number of major repair incidents. The current repair cost in this study is assumed to be (2* cover construction cost) where the construction costs are the sum of columns 3 and 6 in Table 3.

It is important to mention that *s* and *r* are not constant values for all times. They may change according to the input data to the technique used in forecasting. For simplicity, it is assumed that *s* and *r* are constant values where s equals 5%, and *r* is the average of discount rates from 2005 to 2014, that is 10% [30].

To evaluate each alternative with respect to cost effectiveness, two cases are considered as follows.

4.2.1 Case I

In this case, costs are calculated assuming the structure is required to perform for a certain life time, 75 years. Table 4 and Fig. 3 show the construction and repair costs encountered in each alternative calculated to present value.

Table 4: Required construction and repair costs to achieve 75 years' service life

Alternative 1		Alternative 2		Alternative 3		Alternative 4	
Construction costs							
Base year	2618.4	Base year	2761.4	Base year	2903.4	Base year	9642.5
Repair costs							
Repair times	Cost	Repair times	Cost	Repair times	Cost	Repair times	Cost
6.2	496.1	11.8	548.1	19.0	509.4	46.8	43.1
12.1	377.0	23.0	325.5	37.0	220.5		
17.7	290.6	33.7	197.9	54.2	99.1		
23.0	227.1	43.8	123.7	70.5	46.4		
28.0	180.0	53.4	79.1				
32.9	143.3	62.5	51.8				
37.4	116.2	71.2	34.6				
41.7	95.1						
45.9	78.3						
49.8	65.3						
53.5	55.0						
57.0	46.7						
60.4	39.9						
63.5	34.5						
66.6	29.9						
69.4	26.2						
72.2	23.0						
74.7	20.5						
Sum	4962.976		4122.163		3778.842		9685.579

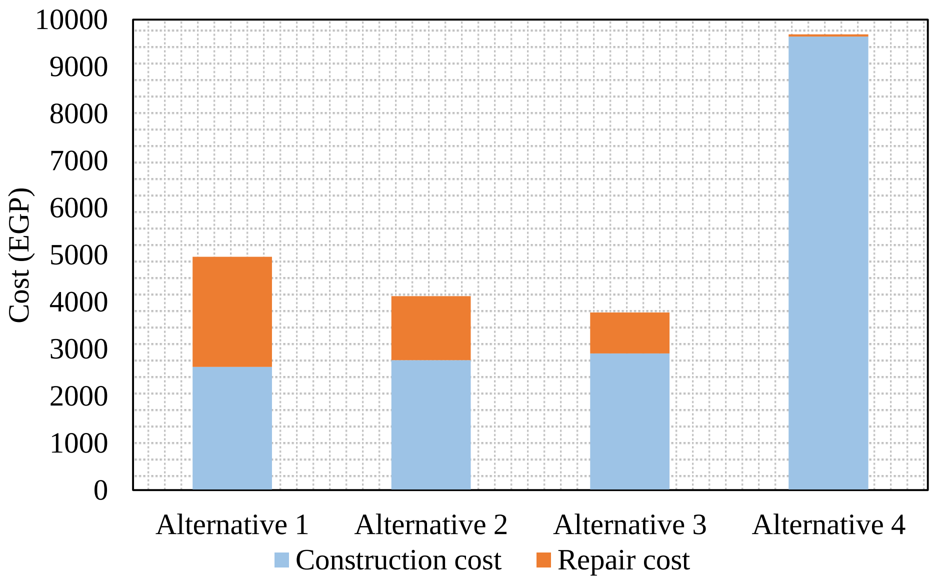


Fig. 3: Present construction and repair costs for each alternative.

All these alternatives can be used to achieve the intended service life of 75 years, but it can be seen from Fig. 3 that although Alternative 1 has the least initial cost, the life-cycle cost is not the minimum. It comes in the third rank as it required many repair works during the life. Alternative 3 is the best economic choice to achieve the durability requirements. It has the minimum total cost although its initial cost is 12% higher than Alternative 1. Alternative 2 comes in the second rank. Finally, alternative 4 it requires the maximum life cost. It has the maximum initial cost due to the use of stainless steel but it requires almost no repair cost, and it may be appropriate to use in other situations e.g. longer service life.

4.2.2 Case II

The second case assumes that the structure will perform until the economic service life is reached, i.e. the replacement of the structure is more economic than continuing the repairing process. As the economic service lifefor the alternatives may be different, the comparison cannot be performed by summing the costs occurring during each period.Thus, the comparison will be done by calculating for each alternative, the equivalent annual cost (*EAC*) which is the cost per year of the structure over its economic life where *EAC* [12]:

$$EAC = \frac{CPV * r(1+r)^t}{(1+r)^t - 1} \tag{15}$$

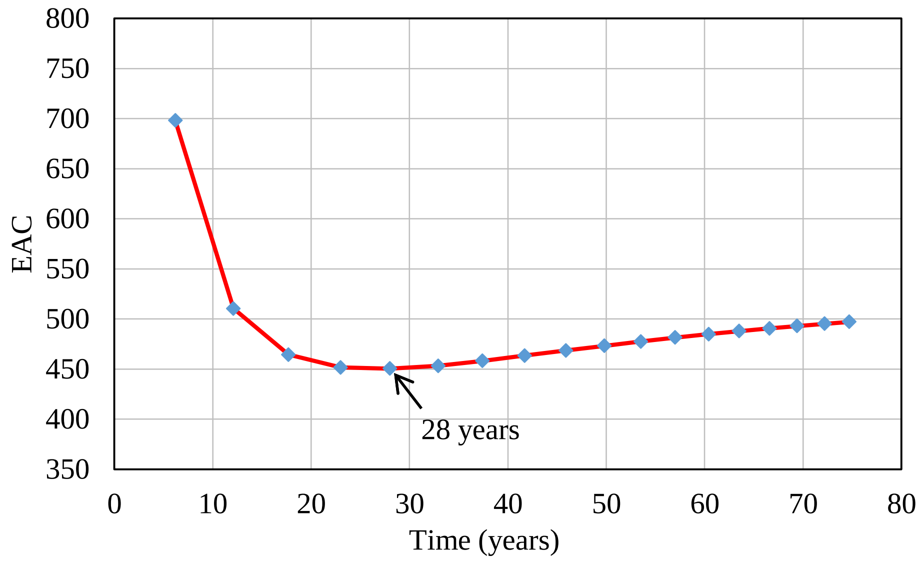
where *CPV* is the cumulative cost of the structure up to time *t*.

To determine the economic service life, *EAC* will be calculated at each repair time for all previous incurred cost. The point of reversal of *EAC* is considered to occur at the economic service life. Then, *EAC* values correspond to service lives of each alternative are compared to determine the most economic one. Table 5 shows the present cumulative cost for each alternative at each repair time.

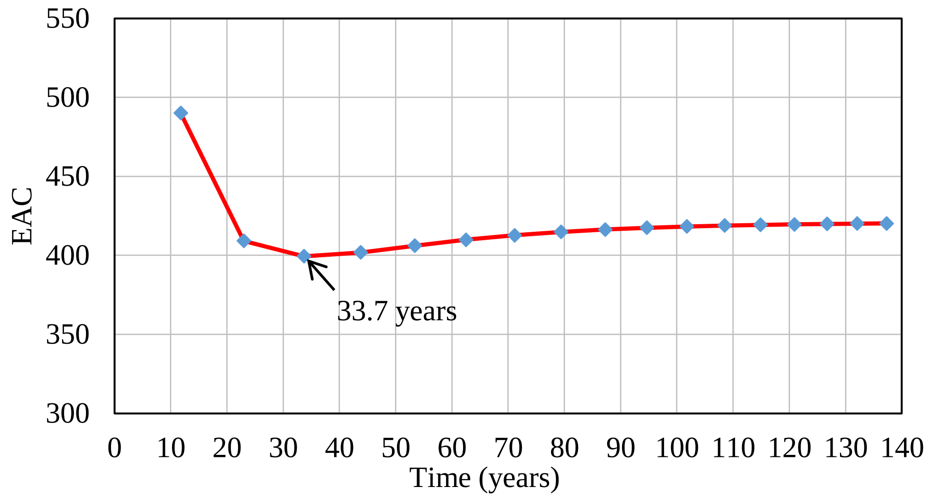
Table 5: Present cumulative cost for each alternative at each repair time

Alternative 1		Alternative 2		Alternative 3		Alternative 4	
Time	Cumulative cost	Time	Cumulative cost	Time	Cumulative cost	Time	Cumulative cost
Base year	2618.4	Base year	2761.4	Base year	2903.4	Base year	9642.5
6.20	3115.28	11.80	3309.51	19.00	3412.85	46.80	9685.58
12.10	3493.07	23.00	3634.88	37.00	3632.85	91.30	9691.02
17.70	3784.28	33.70	3833.14	54.20	3731.93	133.50	9691.79
23.00	4011.70	43.80	3956.97	70.50	3778.37	173.60	9691.91
28.00	4191.49	53.40	4036.16	86.00	3800.97	211.70	9691.93
32.90	4335.33	62.50	4087.94	100.70	3812.38	248.00	9691.93
37.40	4451.68	71.20	4122.53	114.60	3818.34	282.40	9691.93
41.70	4546.81	79.40	4146.11	127.90	3821.55	315.00	9691.93
45.90	4625.37	87.30	4162.49	140.50	3823.34	346.00	9691.93
49.80	4690.87	94.70	4174.08	152.50	3824.36	375.60	9691.93
53.50	4745.99	101.80	4182.42	163.90	3824.97	403.60	9691.93

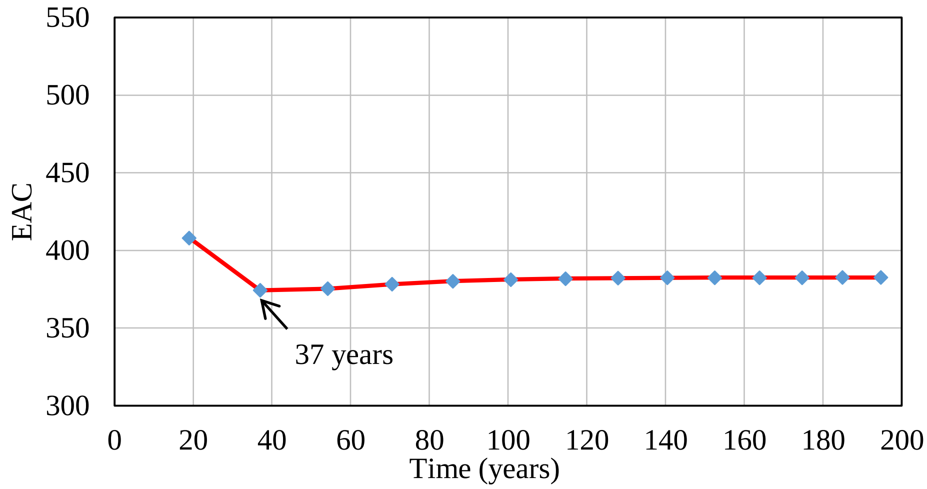
Fig. 4 shows the *EAC* values at each repair time for each alterative to determine the cost effective one.



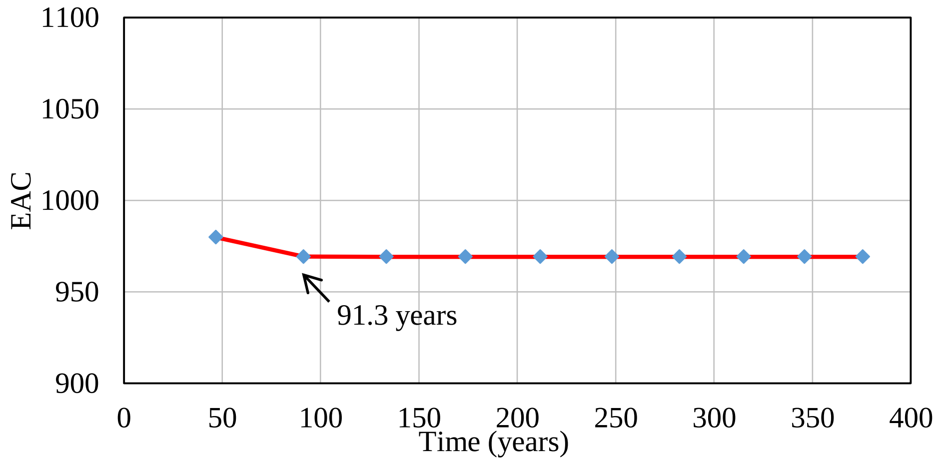
(a)



(b)



(c)



(d)

Fig. 4: Reversal point of *EAC* for (a) alternative 1, (b) alternative 2, (c) alternative 3, and (a) alternative 4 respectively

From the figure, the economic service life for each alternative (point of inflation of *EAC* curve is 28, 33.7, 37, and 91.3 years for alternatives 1, 2, 3, and 4 respectively. The corresponding values of *EAC* are 450.4, 399.4, 374.3, and 969.3 EGP respectively. It is indicated that alternative 3 is the best solution as it requires the minimum annual cost during its life. Alternatives 2, 1, and 4 come in the second, third, and fourth places. Values of *EAC* tend to become constant at certain time. This is because the present value of repair costs becomes lower such that the cumulative costs tend to constant with time.

5. Conclusions

In this paper, a model to estimate life cycle costs of reinforced concrete structure in marine environment is proposed. It is concluded that life cycle cost analysis is an important tool to be used in assessing different specifications. Also, basing the decision on the initial investment only can lead to an erroneous judgment, and the total cost through the intended life has to be considered to choose the cost-effective solution as low investments can mean high maintenance and repair costs where the total cost can exceed high investments with low maintenance and repair costs.

References

[1] Broomfield J. *Corrosion of steel in concrete, understanding, investigating & repair*. E&FN Spon, London, 1997.

[2] Vidal T, Castel A, François R. *Corrosion process and structural performance of a 17 year old reinforced concrete beam stored in chloride environment*. Cem. and Concr. Res. 2007; 37: 1551–61.

[3] Xia J, Jin W-L, Li L-Y. *Shear performance of reinforced concrete beams with corroded stirrups in chloride environment*. Corr. Sci. 2011; 53:1794-1805.

[4] Fang C, Lundgren K, Chen L, Zhu C. *Corrosion influence on bond in reinforced concrete*. Cem. Concr. Res. 2004; 34: 2159–67.

[5] Glanville J, Neville A. *Prediction of Concrete Durability*. E&FN Spon, London, 1997.

[6] Lim H. *Assessing Level and Effectiveness of Corrosion Education in the UAE*. International Journal of Corrosion 2012.

[7] White G, Ostwal P. *Life cycle costing*. Manag. Account. 1976; 57: 7-40.

[8] Fuller S, Petersen S. *Life cycle costing manual*. Washington, 1996.

[9] Tuutti k. *Corrosion of steel in concrete*. Swedish Cement and Concrete Research Institute (CBI), Repot Fo 4.82, Stockholm, 1982.

[10] Weyers R. *Corrosion service life model*. American Society of Civil Engineers, 1998.

[11] Val D, Stewart M. *Life-cycle cost analysis of reinforced concrete structures in marine environments*. Structural Safety 2003; 25: 343-362.

[12] Brown T. *Engineering economics and economic design for process engineers*. E&FN Spon, London, 2006.

[13] Poulsen E, Mejlbro L. *Diffusion of Chloride in Concrete: Theory and Application*. E&FN Spon, London, 2005.

[14] Crank, J.: *The mathematics of diffusion. Second edition*. Clarendon Press. Oxford, UK, 1986.

[15] Shim S. *Corner effect on chloride ion diffusion in rectangular concrete media*. KSCE J. Civil Eng. 2002; 6:19–24.

[16] Yuan Q, Shi C, De Schutter G, Audenaert K. *Effect of temperature on transport of chloride ions in concrete*. In M. G. Alexander et al. (Eds.), Concrete repair, rehabilitation and retrofitting II, 2009: 345-51.

[17] Saetta A, ScottaR, Vitaliani R. *Analysis of chloride diffusion into partially saturated concrete*. ACI Mater J 1993; 90(5): 441–51.

[18] Andrade C, & Castillo A. *Evolution of reinforcement corrosion due to climatic variations*. Mater.and Corr. 2003; 54(6): 379–86.

[19] Thomas M, Bentz E. *Computer program for predicting the service life and life-cycle costs of reinforced concrete exposed to chlorides*. Life365 Manual, 2001.

[20] Magnat P, Molly B. *Predicting of long term chloride concentration in concrete*. Mater. Struct. 1994; 27:338–46.

[21] Maage M, Helland S, Carlsen J. *Chloride penetration in high performance concrete exposed to marine environment*. Proc. of RILEM international Workshop on Durability of High Performance Concrete, 1994; 194-207.

[22] Bazant Z, Najjar L. *Nonlinear water diffusion in non-saturated concrete*. Mater. Struct. 1972; 5(25): 3-20.

[23] Meira G, Andrade C, Padaratz I, Alonso C, BorbaJr J. *Chloride penetration into concrete structures in the marine atmospheric zone –relationship between deposition of chlorides on the wet candle and chlorides accumulated into concrete*. Cem. Conc. Compos. 2007; 29(9): 667–76.

[24] Costa A, Appleton J. *Chloride penetration into concrete in marine environment – Part II: Prediction of long term chloride penetration*. Mater. and Struct. 1999; 32; 354-59.

[25] Thomas M, Bamforth P. *Modeling chloride diffusion in concrete effect of fly ash and slag*. Cem. Concr. Res. 1999; 29: 487–95.

[26] Song H, Lee C-H, Ann K. *Factors influencing chloride transport in concrete structures exposed to marine environments*. Cem. Concr. Compos. 2008; 30: 113–21.

[27] Ann K, Ahn J, Ryou J. *The importance of chloride content at the concrete surface in assessing the time to corrosion of steel in concrete structures*. Constr. Build. Mater. 2009; 23: 239–45.

[28] COMSOL Multiphysics, *user guide* (www.comsol.com).

[29] ACI 222R-01. *Protection of Metals in concrete against corrosion*.

[30] World Bank. http://data.worldbank.org/country/egypt-arab-republic. 2015.