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PREDICTION OF SERVICE LIFE OF STRUCTURES CONTAIN METAKAOLIN CONCRETE

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ABSTRACT

This paper presents a specific case study for estimating the service life of structures contains Metakaolin concrete using the SA.Du2020 model. Reckoning the service life helps reinforce the discernibility of these buildings and controls some economic aspects. The case in consideration was applied to the data collected for a mix contains 15% Metakaolin. The physical and chemical features of metakaolin Concrete will be examined. Durability tests of metakaolin Concrete encompass chloride penetration, water permeability, abrasion, water absorption, bond resistance, corrosion resistance and concrete resistance tests. The results of the service life prediction provide a perspective for the expected life span of buildings of similar conditions. The total cost through the intended life has to be considered.

KEYWORDS: Metakaolin Concrete; Life Service; Prediction; SA.DU2020; Cost.

1. INTRODUCTION

The service life of reinforced concrete structures is dependent on the integrity of the steel within the concrete. By time, due to exposure, chloride and other minerals can penetrate the concrete. Once these materials reach the reinforcing steel, the chloride and oxygen begin to corrode and deteriorate the steel. This deterioration will reduce the service life of a concrete structure drastically.Structures durability is an essential property that can elongate the service life of these structures. Service life is known as the period of time to unacceptable damage [1]. Deterioration of structures includes but not limited to cracking, spalling and delamination. The reason for these damages is ascribed to corrosion of reinforcement steel due to chloride ions penetrating the concrete surface and piling of a sufficient quantity at the embedded steel level to initiate corrosion. Dicing salts are the main sources for chloride ions and used to melt snow during winter times. Corrosion of embedded reinforcement is the most common cause of concrete deterioration in structures [2]. Therefore, structures components require repair and rehabilitation to enhance their serviceability. Billions of dollars are spent to repair and replace defected concrete infrastructure due to corrosion damage yearly [3].Many trials have been implemented to forecast the life span of buildings. Most of these trials concentrated on corrosion and carbonization as the main reasons for the deterioration of concrete buildings [4-5-6]. The Factor Method considers many parameters in forecasting the service life of a building or its elements. The method highlights that a building's durability is a function of its components and the environment affecting these components. The International Organization for Standardization adopted the factor method in its ISO 15686 publications as a comprehensive tool for service life forecast. This method is based upon collecting data related to the behavior of the constituting materials, construction process, the environment inside and outside, and the facility's use and maintenance processes. This study uses the factor method to forecast the service life of public buildings in severe weather according to an extensive material and structural investigation of those buildings [7-8-9-10-11].SA.Du2020 software was used as an analysis tool to conduct this study on a reinforced concrete structure to predict the service life and life cycle cost when it is exposed to chlorides. SA.Du2020 software emerged as the need for a model that is used to evaluate service life and life cycle cost in concrete structures had been raised [3]. The service life for concrete structures is a measure of their durability. Deterioration in reinforced concrete structures exposed to chloride from dicing salts, groundwater and seawater is the main reason for reducing their service life due to embedded reinforcement corrosion. The period of time between construction and first repair or any unacceptable damage is called service life [1].

2. EXPERMINTAL WARK AND TEST

Material

Based on the principal of high strength concrete, the materials used in this experiment are shown in Table (1).

Table (1): Material used

Material	**Cement**	**Fine Aggregate**	**Coarse Aggregate**	**Chemical Admixture**	**Additive**
Type	Sinai Portland Cement 52.5 N	Sand	Basalt	Super plasticizer	Metakaolin
Characterizes	Specific Surface Area = 3590 cm2/gm Comply with the Egyptian standard specifications	Specific Gravity = 2.6 gm/cm^3 Fineness Modulus = 2.75	Specific Gravity = 2.9 gm/cm^3 Water Absorption Ratio = 1.85	Increasing the workability of concrete without additional amount of water. Sika Viscocrete 3425	Specific Gravity = 2.65 gm/cm^3

The cement type is ordinary Portland cement produced by Sinai Company (52.5 N) was used in all mixes with a content $400kg/m^3$ as cement content. Testing was carried out according to the Egyptian standard specification 4756- 1-2009 [12]. Metakaolin is produced by heat-treating kaolin at 750° C. Kaolin is one of the most abundant natural minerals which is a fine, white clay that has traditionally been used in the manufacture of porcelain and as a coating for paper. The used Metakaolin was produced by Nour-metec for building and Refractories Company. The aggregate used is brought from Suez Area, and its properties meet with the Egyptian specifications. Sand was used to optimize the grading curve of the granular mixture between coarse and fine aggregate. Tap water that was clean, drinkable, fresh and free from impurities was used for mixing and curing the tested samples according to the Egyptian code of practice. The high-range water-reducing (HRWR) admixtures often referred to as super-plasticizers. It helps in increasing the workability of concrete without additional amount of water. Sika Viscocrete 3425 Super-plasticizer is poly carboxylate based super plasticizer supplied by Sika Egypt for construction chemicals company is a third-generation super-plasticizer for concrete and mortar, which meets the requirements of super –plasticizer according to ASTM-C-494 [13], types G and F (ASTM C494, 2003).

Results

Table (2) shows the different tests results conducted on Metakaolin concrete.

Compressive Strength

Mixes contain ordinary Portland cement and 15% replacement from metakaolin, compressive strength increased approximately 9.0 ~ 17.6% related to the control mix.

Tensile Strength

Mixes contain ordinary Portland cement and 15% replacement from metakaolin, tensile strength increased approximately 12.1 ~ 16.7% related to the control mix.

Accelerated Corrosion Test

The accelerated corrosion test was performed as shown in Figure (1). The samples were connected to a DC power supply acting as an anode (+), while a steel mesh was positioned under the samples as a cathode (−). The samples were connected as parallel connections to the circuit board to maintain a constant voltage of 12 volts throughout the whole experiment. The samples were totally submerged in a 5% NaCl solution. The results obtained related to corrosion behavior studies showed that as the replacement ratio increases the corrosion does not exists. Corrosion rate was found to be lesser at 15% replacement from metakaolin, it showed slight higher values. It is concluded that 15% addition with metakaolin replacement with cement showed to be good corrosion resistance property values with control concrete mix.

Chloride Penetration Test

Metakaolin concrete mixes show very low chloride permeability. Thus, mains that high reactivity metakaolin substantially reduced chloride ion penetration in metakaolin concrete and such reductions can be expected to have a substantial impact on the service life of reinforced concrete in chloride environments.

Table (2): Concrete Tests results

Mix	Stresses(MPa)		Corrosion (Day)	Chloride Penetration Test	
	Compressive Strength	Tensile Strength		Charge Passed (Coulombs)	Chloride Ion Penetrability
M0	**82.5**	**3.983**	**17**	**1100**	**Low**
M15	**93.767**	**4.633**	**21**	**370**	**Very Low**

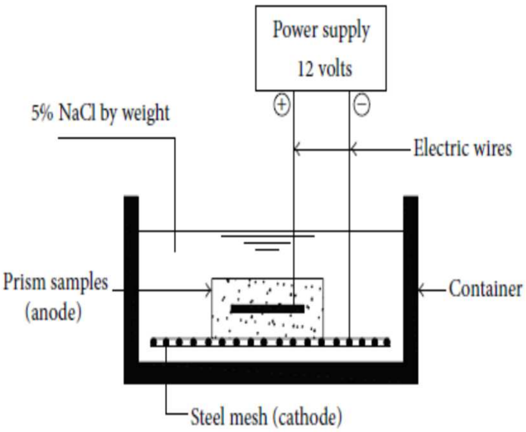

Figure (1): Accelerated corrosion test

3. LIFE SERVICE OF REINFORCED CONCRETE STRUCTURES

Service life of a RC structure can be defined as time period during which structure will fulfill the required performance under defined repair and maintenance. Performance and service life of a RC structure are governed by several parameters such as strength, quality of concrete, concrete cover, age, and most significantly by exposure conditions. Behaviour or performance of concrete structures depends on various physical and chemical. The factors affecting service life can vary, not only from building to building, but even within a given building.The service life of structures depends on a variety of factors, such as (i) their purpose; (ii) socio-economic considerations; (iii) materials of construction; (iv) surrounding environment; and (v) degree of maintenance. SA.Du2020 depends on stresses, corrosion and chloride penetration to calculate life service of concrete structures. The model accounts for parameters such as stress level, water/cement ratio, age, humidity, and chloride diffusion coefficient.

- **Stress**

Chloride concentration and diffusion coefficient decreases with the increase of compressive stress and increases of tensile stress. The results of reinforced concrete stresses give a clear impression of the concrete's behavior and durability. Concrete has relatively high compressive strength, but significantly lower tensile strength. As a result, without compensating, concrete would almost always fail from tensile stresses even when loaded in compression.

- **Corrosion**

Corrosion is initiated when materials that are harmful to steel, such as CO2 and chloride from de-icing salt, start to penetrate concrete and reach the steel reinforcement. As an electrochemical reaction, electrons migrate from the anodic zone to the cathodic zone, releasing ferrous ions at the anode and hydroxide ions at the cathode. This will eventually lead to a potential difference between the anodic and cathodic areas at the surface of the steel reinforcement. This results in the creation of rust as a byproduct. Since rust occupies a larger volume than steel, it exerts internal pressure which causes the surrounding concrete to crack and become damaged. These cracks make their way to the surface of the concrete which causes even more CO2 and chloride to penetrate the concrete and speed up the process of corrosion. In this study, accelerated corrosion test was performed and taken into account in calculating life service for reinforced concrete.

- **Chloride Penetration**

Reinforced concrete structures are exposed to harsh environments yet is often expected to last with little or no repair or maintenance for long periods of time. To do this, a durable structure needs to be produced. For reinforced concrete, one of the major forms of environmental attack is chloride ingress, which leads to corrosion of the reinforcing steel and a subsequent reduction in the strength, serviceability, and aesthetics of the structure. This may lead to early repair or premature replacement of the structure. A common method of preventing such deterioration is to prevent chlorides from penetrating the structure to the level of the reinforcing steel bar by using relatively impenetrable concrete. The ability of chloride ions to penetrate the concrete must then be known for design as well as quality control purposes. The penetration of the concrete by chloride ions, however, is a slow process. It cannot be determined directly in a time frame that would be useful as a quality control measure. Therefore, in order to assess chloride penetration, a test method that accelerates the process is needed, to allow the determination of diffusion values in a reasonable time.

Analysis of Experimental Results

The time to corrosion-induced cracking was estimated using a two-part damage model first proposed [14-15]. The design of the life span is based on durability of used concrete and rate of corrosion of reinforcement. Figure (2) shows Prediction model of service life of concrete structures for concrete contains chlorides, subjected to externals chloride attaches.

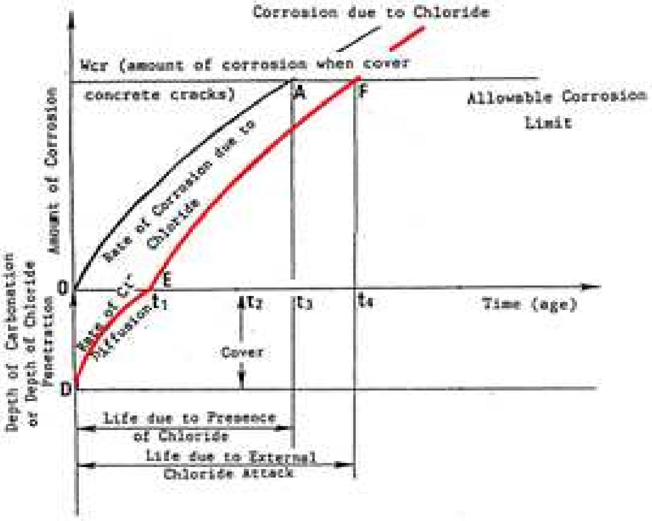

Figure (2): Prediction model of service life of concrete structures for concrete contains chlorides, subjected to externals chloride attach

The life span for structure subjected to external chloride attack can be predicted as follows;

$T_s = T_i + T_{cr}$ Eq. 1

Where

($T_i = t_1$) is the time required for chloride to penetrate into the concrete cover.

($T_{cr} = t_4 - t_1$) is the time required for a critical amount of corrosion to propagate cracks in concrete cover

Time-to-corrosion-initiation

Time-to-corrosion-initiation is a function of the transport properties of the concrete, geometry, the confinement conditions that exist for a specific environment and application, and the required concentration of chlorides to begin the corrosion of the reinforcing steel. Corrosion initiation is known as the time that a chloride takes from the surrounding environment to penetrate the concrete cover and accumulate to a sufficient concentration at the reinforcement surface to initiate corrosion. Chloride concentrations above the chloride threshold locally decrease the pH close to the reinforcement, which causes discouragement of the shield oxide layer and subsequent corrosion of the steel reinforcement. Estimates of initiation period to Fick's second law error function solution;

$$c(x,t) = c_s \left[1 - erf\left(\frac{x}{\sqrt{4Dt}}\right)\right] \qquad \text{Eq. 2}$$

Where c(x,t) is the concentration at depth x and time t, c_s is the surface concentration, erf is the error function, and D is the diffusion coefficient).

$$T_i = c_s \, \mathrm{x} \left[1 - erf\left(\frac{C_t}{2\,\mathrm{x}\sqrt{D_c \mathrm{x}\, t_{ini}}}\right)\right] \qquad \text{Eq. 3}$$

$$T_i = \frac{{c_t}^2\left[erf^{-1}\left(1-\frac{c_{th}}{c_s}\right)\right]^{-2}}{4\,D_c} \qquad \text{Eq. 4}$$

$$T_i = \frac{1}{12\,D_{app}}\left(\frac{C}{1-\sqrt{\frac{c_{th}}{c_s}}}\right)^2 \qquad Eq.\ 5$$

Where

C- Cover thickness (mm).

C_{th}- The threshold value of the chloride concentration (kg/m³ of concrete).

C_s- The concentration of chloride ions in pores of concrete at the surface (kg/m³ of concrete).

D_{app} - Apparent chloride diffusion coefficient (cm²/s).

Effect of w/c ratio

The w/c ratio is renowned of its impacts on the chloride diffusion coefficients. The effect of w/c ratio on the chloride diffusion coefficient was described by;

$$D_{28} = 2.17 \,\mathrm{x}\, 10^{-12} e^{(w/c)/0.279} \qquad \text{Eq. 6}$$

Where D_{28} is the 28-day diffusion coefficient (m²/s) and w/c is in decimal form.

Effect of supplementary cementitious materials

A supplementary cementitious material in concrete modifies many constituent properties of concrete, encompassing chloride resistance. Supplementary cementitious materials also affect fresh-state workability, set time, and early strength gain of concrete when exploited in abundant amounts Other materials, such as metakaolin, may be speculated to have a beneficial impact on either the initial value of the diffusion coefficient or the degree to which the diffusivity decreases by time. However, there are insufficient pieces of information to develop a general relationship within the model and the user is referred to the published literature and encouraged to input these materials as user-defined scenarios. As proposed in [16-17] three SCMs, namely silica fume (SF), fly ash (FA), and metakaolin (MK), were considered to affect D_{28} according to;

$$D_{SF} = D_{28} \,.\left(0.206 + 0.794e^{(-SF/2.5\mathrm{D}c)}\right) \qquad \text{Eq. 7}$$

$$D_{UFFA} = D_{28} \,.\left(0.170 + 0.829e^{(-UFFA/6.07)}\right) \qquad \text{Eq. 8}$$

$$D_{Mk} = D_{28}.\left(0.191 + 0.809e^{(-Mk/6.12)}\right) \qquad \text{Eq. 9}$$

Where D_{SF}, D_{FA}, and D_{MK} are the modified 28-day diffusion coefficients due to the addition of SF, FA, and MK, respectively. SF, FA, and MK are the percent replacement (in whole-number percent) of ordinary Portland cement.

Time impact

The chloride diffusion coefficient is a time dependent parameter that is famous for reduction by time. This reduction is ascribed to continued cement hydration and densification of the concrete beyond the first 28 days, among other mechanisms. At each time step, the bulk diffusivity was recalculated according to the following relationship;

$$D_t = D_{28}\left(\frac{t_{28}}{t}\right)^m + D_{ult}\left(1-\left(\frac{t_{28}}{t}\right)^m\right) \qquad \text{Eq. 10}$$

Where D_t is the time-dependent diffusion coefficient, D_{28} is either the 28-day diffusion coefficient calculated by Eq. 6 (without SCM addition) or Eq. 7-8-9 (with SCM addition), and the reference time, t_{28}, is typically taken as 28 days. The chloride diffusion coefficient eventually plateaus to a final value. The second is the 100-year ultimate diffusion coefficient.

Temperature impact

Temperature is an important consideration for a non-steady-state diffusion model as it can change the rate of concrete densification as well as the rate of chloride ion diffusion.

Chloride boundary condition modeling

Time-dependent chloride boundary conditions were modelled identically in SA.Du2020.

Time to cracking

Time-to-cracking is known as the time from corrosion initiation to stress induced cracking of the concrete cover based on the time-dependent formation of oxidation products. Two mathematical models that was utilized to calculate the time to crack the reinforced concrete t_{cr} was developed by Liu & Weyers [18] (in years) and Sharobim. K.G and Tazawa. E [22] (in years).

Basic assumptions in modeling the cracking behavior of concrete cover

Corrosion cracking models are entirely based on basic assumptions, but yet they are experimentally tested and compared. Despite these basic assumptions, the evaluations may have significant differences compared to natural occurrence of corrosion. Following basic assumptions are generally made for those mathematical models to be implemented in corrosion cracking;
(i) Corrosion development is spatially uniform around the interface of the steel reinforcement and the concrete contact surface which results in a uniform radial expansive pressure. In some cases, steel corrosion in concrete is not uniform and rust foci takes place. Nevertheless, as pitting corrosion being subjected to occur continuously, the limitations of using such mathematical models reduce quite drastically. Hence assumptions are made that the corrosion is uniform along the reinforcement.
(ii) The concrete area outside the steel reinforcement is treated as a thick-walled cylindrical section where the wall thickness is usually taken as the thinnest concrete cover.
(iii) The only cause of stress exertion is due to the expansion of corrosion products, and there are no other loading effects that may influence the concrete cover cracking.
(iv) The concrete cover is an isotropic linear elastic material. The mathematical models only consider the effects within elastic region because the linear strain compatibilities are used in the elastic limit at corrosion cracking of the concrete material.
(v) The steel-concrete interface has a certain thickness which consists of porous material. This porous zone around the steel reinforcing bar caused by the transition from cement paste to steel, entrapped/entrained air voids, and capillary voids in the cement paste into which corrosion products diffuse.
(vi) During the corrosion initiation, a fraction of the corrosion products shall be accommodated within the micro cracks and the porous zone. At the time when corrosion builds up, very thin hairline micro cracks might be present in the steel-concrete interface due to the material transition. These micro cracks are directly faced into the corrosion phases and as the rust outputs accumulate, they will be pushed into the micro cracks. The process of reinforcement corrosion in concrete takes place at sequential steps as it is schematically depicted in Figure (3). As the corrosion products are being built up in the concrete systems, a portion of its quantity will be pushed into the porous zone and the micro cracks that are already developed near the interface of the steel and concrete. Once the

voids are filled with the corrosion products, an expansive pressure will be exerted on the concrete walls. And eventually, the concrete cracking is observed when expansive pressure exceeds the tensile strength capacity of the concrete system.

Theoretical aspect of the prediction models for reinforced concrete corrosion

A conceptual model for service life prediction of corroded reinforced concrete structures was developed by Tuutti [19].

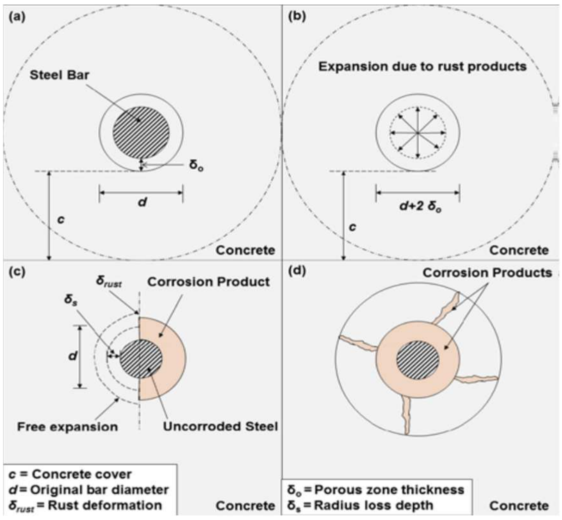

Figure (3): Idealization of concrete cover as a thick-walled cylinder; (a) undamaged rebar embedded in concrete, (b) deformation of concrete due to corrosion expansion, (c) deformation of corrosion products and (d) accumulation of rust into micro cracks and porous region

According to this conceptual model illustrated in Figure (4), the deterioration due to corrosion is occurred over two distinct time periods. There, the corrosion initiation period is administered by the transport properties of the concrete (i.e. diffusion and permeability of chloride ions or carbon dioxide). Once the onset corrosion is taken place, the crack initiation period is mainly governed by the chemical, electrochemical and geometric properties of the concrete. However, this conceptualized model by Tuutti concluded that it underestimated the time to crack concrete in comparison with times obtained from field and laboratory observations. This is one of the reasons why the service life model was further modified by adding a third period called the "free expansion" after the corrosion initiation period. This is the time it takes to fill up the porous zone completely with corrosion products

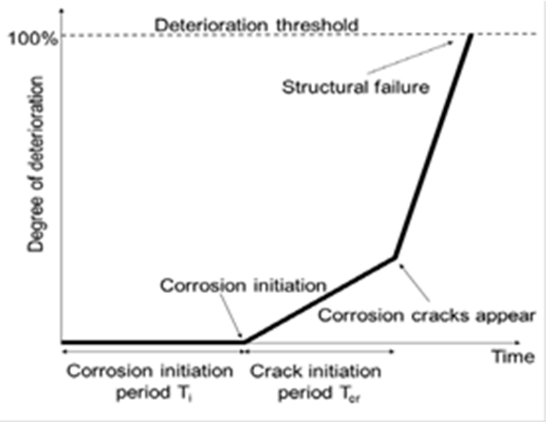

Figure (4): Schematic of a conceptual service life model for corroded reinforced concrete structures [19]

The amount of rust products that are pushed into the voids upon corrosion initiation is very crucial in determining the time to crack the concrete cover. It is reported that in order to obtain a complete filling of rust products into

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the porous zone, it takes approximately 15-50 µm of radius loss given that the specimens were subjected to a uniform corrosion.

In a practical point of view, all the micro cracks including porous zones cannot be entirely filled with rust products before expansive stresses are exerted on the concrete walls. One reason is that, such radial micro cracks are different in lengths extending outward from the steel-concrete interface to the cover depth whereas, the existing crack paths and the porous zones might be highly tortuous. On the other hand, the crack development and penetration of corrosion products will be influenced by the corrosion intensity. Therefore, the filling process of rust products inside voids might be higher at higher corrosion intensity regions. This has been taken into consideration.

Evaluation of time for reinforcement corrosion

Mass loss of steel reinforcements embedded in concrete due to corrosion is proportional to the amount of current that flows through the electrochemical cell connected through anode (Fe atoms) and cathode (OH– ions). In order to establish the relationship with time have used Faraday's law as shown in Equation 1.

$$M_{loss} = \frac{MI_{corr}t}{ZF} \qquad \text{Eq. 11}$$

Where Mloss being the weight loss of consumed steel (g); M being the atomic weight of iron (56 g/mol); Fbeing the Faraday constant (96,500 C/mol); z being the number of electrons freed upon oxidizing and t being the corrosion time (sec). The corrosion current density, icorr (µA/cm²) is defined as the corrosion current per unit steel surface area along the reinforcement. The corrosion current Icorr is calculated based on the following relation given in Eq 12;

$$I_{corr} = l(cm) \text{ x } \pi d(cm) \text{ x } i_{corr}\left(\frac{A}{cm^2}\right) \text{x} 10^{-6} \qquad \text{Eq. 12}$$

Where l is the reinforcement bar length that the corrosion current had flowed and d is the reinforcement bar diameter.

Mathematical models

In this study, two mathematical models were used to observe the statistical behaviour of the time to crack the concrete. The time to crack the concrete is considered as the time from the corrosion initiation to the time it creates corrosion cracks on the concrete. The first mathematical model that was utilized to calculate the time to crack the reinforced concrete tc developed by Liu & Weyers [20] (in years) is shown in Eq 13. Although many factors have been considered in the model given in the equation, some of the factors are common for the entire mathematical model provided in the equation. Hence, these common factors were identified and eventually, they were used in the statistical analysis to evaluate the factor significance on the time to crack the concrete. Among those, some of these factors were kept as constants.

$$T_{cr} = \frac{\left(\left(\frac{1}{\rho_r} - \frac{\alpha_m}{\rho_s}\right)^{-1} \pi d\left(\frac{cf_{ct}}{E_C}\left(\frac{(R_O+C)^2+R_O^2}{(R_O+C)^2-R_O^2} + \upsilon_c\right) + \delta_0\right) \text{x } 1000\right)^2}{2 \text{ x } 0.098 \frac{1}{\alpha_m} \pi \, d \, i_{corr} \text{ x } 1.07} \qquad \text{Eq. 13}$$

Where;
d = Diameter of the rebar (mm)
Tcr = Time to crack the concrete
C = Cover (mm).
fct =Tensile strength of concrete (MPa)
E =Elastic modulus of concrete, E =4500$\sqrt{fc'}$ (MPa) according to Canadian standard
Ec = Eff. elastic modulus (MPa),E/(1+ ϕcreep)
ϕcreep = Creep coefficient
δ0 = Thickness of the porous zone (mm)
$R_0 = \frac{d}{2} + \delta_0$(mm)
vc = Poisson's ratio of concrete
$i$corr =corrosion current density (µA/cm2)
ρr =Density of rust (g/mm3)

αm =Molar mass of corrosion products to that of steel
ρs =Steel density (g/mm3)

The constant variables were used for v_c, α_m, I_{corr}, ρ_r, ρ_s as 0.2, 0.629, 120 µA/cm^2, 0.00378 g/mm^3, 0.00785 g/mm^3 respectively [21].

The second mathematical model that was utilized to calculate the time to crack the reinforced concrete tc developed by Sharobim. K.G and Tazawa. E [22] (in years) is shown in Eq 16. The time from starting corrosion up to cracking of concrete cover (T_{cr}) depends on rate of corrosion (η) and the critical amount of corrosion when cover cracks (Wcr). The rate of corrosion (η) can be calculated from the following Eq. 14;
$\eta = a\,[100\,N\,(W/C)^2 + 10\,(W/C)^2 - 12\,N - b]\,d/C^2$ Eq. 14

Where;
W/C= water-cement ratio
N = chloride ions content as % of cement weight
d = steel bar diameter
C = cover thickness
a&b = factors dependent upon cover-bar diameter and concrete quality
The critical amount of corrosion (Wcr) when cover cracks can be calculated from the following Eq.15;
$Wcr = 0.33\,B.d\,(C/d + 0.50)^{0.85}$ Eq. 15
Where;
B = factor dependent upon tensile strength of concrete
d = steel bar diameter
C = cover thickness
Then, the time required for a critical amount of corrosion to propagate cracks in concrete cover (Tcr) can be calculated from the Eq. 16;
$(Tcr) = Wcr / \eta$ Eq. 16

4. LIFE CYCLE COST CALCULATE

The cost of construction is the sum of material and labour costs. Material costs are the sum of concrete and steel reinforcement costs. In Egypt, labour costs are approximately estimated to be 30% of material costs for reinforced concrete members. The life cycle cost 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. Regarding a structure, life cycle cost is the sum of all expected costs associated with design, construction, operation and maintenance.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. In this paper, the life cost cycle of structure has been divided into three types of cost; construction cost, CO_2 treatment cost and repair cost each type will be shown separately.

Construction Cost

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 1m3 of concrete [23].

CO_2 Treatment Cost

The environmental impact of concrete, its manufacture and applications, are complex. The cement industry is one of the main producers of carbon dioxide, a potent greenhouse gas. Concrete causes damage to the most fertile layer of the earth, the topsoil.The cement industry is one of the two largest producers of carbon dioxide (CO_2), creating up to 8% of worldwide man-made emissions of this gas, of which 50% is from the chemical process and 40% from burning fuel. The CO_2 emission from the concrete production is directly proportional to the cement content used in the concrete mix; 900 kg of CO_2 are emitted for the fabrication of every ton of cement, accounting for 88% of the emissions associated with the average concrete mix. Cement manufacture contributes greenhouse gases both directly through the production of carbon dioxide when calcium carbonate is thermally decomposed, producing lime and carbon dioxide, and also through the use of energy, particularly from the combustion of fossil fuels.One area of the concrete life cycle worth noting is the fact that concrete has a very low embodied energy relative to the quantity that is used. This is primarily the result of the fact that the materials used in concrete construction, such as aggregates, pozzolans, and water, are relatively plentiful and can often be drawn from local sources. This means that transportation only accounts for 7% of the embodied energy of concrete, while the cement production accounts for 70%. With a total embodied energy of 1.69 GJ/tonne concrete is lower than any other building material besides wood. It is worth noting that this value is based on mix proportions for concrete of no more than 20% fly ash. It is estimated that one percent replacement of cement with fly ash represents a 7% reduction in energy consumption. With some proposed mixes containing as much as 80% fly ash, this would represent a considerable energy savings.It was found that the manufacture of a ton of metakaolin produces 250 kg of carbon dioxide gas, which is a small percentage compared to the cement industry. It is worth noting that the cost of treating carbon dioxide is 10000 Egyptian pounds per ton.

Repair Cost

Corrosion is caused within concrete from carbonation and chloride contamination, either individually or through a combination of both. Carbonation is caused by the alkaline elements reacting with acidic gases, usually the carbon dioxide that is present in the atmosphere. This reduces the alkalinity of the concrete so the protective layer on the steel breaks down. Chlorination is a process where there is ingress of chloride ions from a variety of sources, including de-icing salts, marine environments and contaminated water resulting in more localized corrosion. The repair cost is predicted based on the accurate determination of the time of the first repair of structure, which is determined according to the appearance of corrosion.

5. LIFE SERVICE AND COST PROGRAM

The service life prediction methodology proposed in this paper depends on stresses. The proposed process which has been followed for determining the estimate of residual service life is schematically explained in Figure (5).

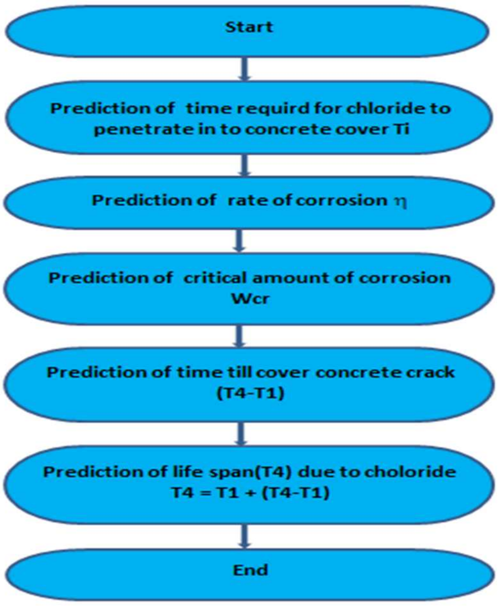
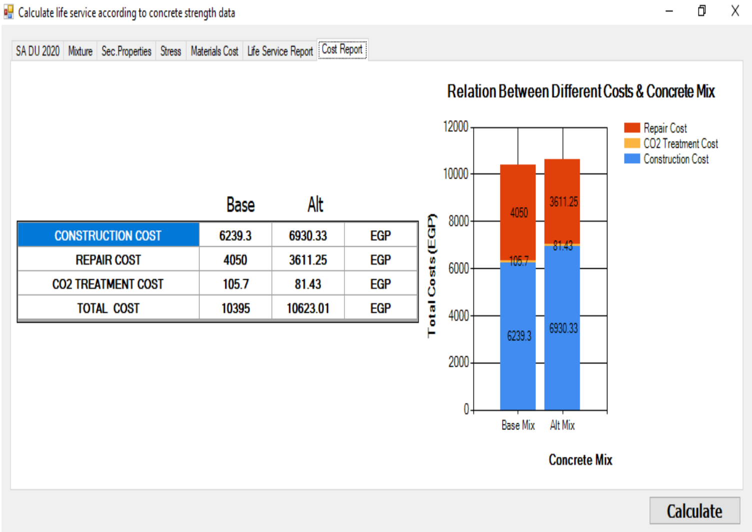


Figure (5): flow chart showed the determining the estimate of residual service life

LIFE SERVICE BASE MIX	49.24	Year
LIFE SERVICE ALT.MIX	54.85	Year

Figure (6): life service graph



	Base	Alt	
CONSTRUCTION COST	6239.3	6930.33	EGP
REPAIR COST	4050	3611.25	EGP
CO2 TREATMENT COST	105.7	81.43	EGP
TOTAL COST	10395	10623.01	EGP

Figure (7): cost graph

6. CONCLUSION

The following reflections are based on the empirical work tackled in this paper.

- This study highlights the replacement of 15 % of the Portland cement with metakaolin. Using metakaolin escalates the construction cost for the concrete mixtures because of the high price of metakaolin vis-a-vis Portland cement. However, the reduction in life cycle costs can be pertained to the ability of metakaolin to reinforce the service life. Metakaolin was particularly successful at elongating the service life by reducing the concrete permeability by lowering the diffusion coefficient. The average increase percentage for service life due to metakaolin usage was 13.7%.
- All the metakaolin concrete mixes demonstrate very low chloride permeability. Thus, means that high reactivity metakaolin sustainably reduced chloride ion penetration in metakaolin concrete and such reductions can be expected to have a vital effect on the service life of reinforced concrete in chloride environments.

The numerical method in this study could be extended to be used as a guide for the service life concrete element.

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