Time-dependent safety performance of reinforced concrete structures

Utino Worabo Woju and A.S. Balu
Department of Civil Engineering, National Institute of Technology Karnataka, Surathkal, India

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Abstract

Purpose – Performance of the structure depends on design, construction, environment, utilization and reliability aspects. Other factors can be controlled by adopting proper design and construction techniques, but the environmental factors are difficult to control. Hence, mostly in practice, the environmental factors are not considered in the analysis and design appropriately; however, their impact on the performance of the structures is significant along with the design life. It is in this light that this paper aims to perform the time-dependent performance analysis of reinforced concrete structures majorly considering environmental factors.

Design/methodology/approach – To achieve the intended objective, a simply supported reinforced concrete beam was designed and detailed as per the Euro Code (EC2). The time-dependent design parameters, corrosion parameters, creep and shrinkage were identified through thorough literature review. The common empirical equations were modified to consider the identified parameters, and finally, the time-dependent performance of reinforced concrete beam was performed.

Findings – Findings indicate that attention has to be paid to appropriate consideration of the environmental effect on reinforced concrete structures. In that, the time-dependent performance of reinforced concrete beam significantly decreases with time due to corrosion of reinforcement steel, creep and shrinkage.

Originality/value – However, the Euro code, Ethiopian code and Indian code threat the exposure condition of reinforced concrete by providing corresponding concrete cover that retards the corrosion initiation time but does not eliminate environmental effects. The results of this study clearly indicate that the capacity of reinforced concrete structure degrades with time due to corrosion and creep, whereas the action on the structure due to shrinkage increases. Therefore, appropriate remedial measures have to be taken to control the defects of structures due to the environmental factors to overcome the early failure of the structure.

Keywords Environmental factors, Reinforced concrete, Shear, Flexure, Safety performance, Time-dependent

Paper type Research paper

Introduction

Nowadays in urban and suburban areas, reinforced concrete structures are very common. Private buildings like apartments, hotels, commercial centers and offices cover major areas. Even though it consumes more energy, material and economy its lifetime is limited to 50 years (BS 7543, 2003). The euro code, Ethiopian code and Indian code treat the exposure condition of reinforced concrete by only providing respective concrete cover. In case the supporting soil which contains aggressive chemical like chloride, the member of the building can be exposed to ingression of chloride that causes corrosion of reinforcing steel.

The performance of a structure is assessed by its safety, serviceability and economy. The information of input variables is never certain, precise and complete (Ranganathan, 1999). The sources of uncertainties may be physical uncertainty, statistical uncertainty, model uncertainty and gross errors. Civil engineering structures such as buildings, bridges and

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transmission towers are complex and usually large in nature: hence, there will be almost no chance to test the prototype rather than checking specific criteria on uncertain numerical models which are based on the knowledge level of the expert to solve a real problem. Due to the presence of these uncertainties of material strengths, loads on the structure during its life, structural idealization model, limitation of numerical methods and so on, the absolute safety of a structure is impossible (Biondini *et al.*, 2004).

However, there are unavoidable uncertainties; the structure is designed to achieve safe, serviceable and economy criteria without failure in its design life. For a structure to be reliable, it must perform a certain function or function satisfactorily for which it has been designed to (Stewart and Rosowsky, 1998), i.e. safety against flexure, shear and torsion; serviceable level with respect to crack; and deflection. The structural performance depends on material properties, cross-sectional dimensions, loading conditions and environmental conditions, which are completely time independent for the structure. For instance, structural loading in the service life, deterioration of material properties and cross-section dimensions due to corrosion, and the shrinkage. These time-variant properties of the input variables induce variation in the output performance (Fan *et al.*, 2019).

The performance of reinforced concrete structures subjected to sustained load and corrosion is well developed numerically and experimentally (Al-Majidi *et al.*, 2019; Sahmaran *et al.*, 2015; Chaitanya and Vamsi, 2014; Stanish, 1997). On the contrary, the time-dependent performance of concrete structures subjected to sustained load, corrosion and creep and shrinkage lack sufficient research. Therefore, this study is significant to evaluate the level of time-dependent performance to recommend the periodic maintenance of the built structures and remedial measures that has to be taken while designing, constructing and/or operating the structure to prevent corrosion and other important factors that can cause the premature failure of the structure. In this paper, time-dependent performance of safety criteria via flexure and shear of the reinforced concrete beam with respect to uncertain environmental factors was investigated.

Literature review

The failure of the structural member occurs when the action load exceeds the resistance of the section. Therefore, the failure mode of reinforced concrete beam is obtained by:

- ultimate limit state of collapse, i.e. flexure, shear, torsion and/or combination of flexure torsion or shear-torsion; and
- serviceability limit state, i.e. excessive deflection, crack and vibration.

Reinforced concrete

The main objective of reinforced concrete design is to achieve a structure or part structure that will result in a safe, serviceable and economical solution. To achieve safety, serviceability and economy criteria of structures, three design philosophies had been adopted to consider uncertainty in loading pattern and materials strength. These are working stress design method, ultimate strength design method and limit state design method. Out of these, limit state design method, in which both load and material uncertainties, and serviceability criteria are considered, is recommended to be adopted in design codes. From three possible sections of reinforced concrete member design codes (EN 1992-1-2, 2004; IS456, 2000), recommend adopting an under-reinforced section, in which the performance of the section is governed by the reinforcing steel.

Performance of structure

The engineering design aims at providing minimum levels of serviceability and safety during the structural lifetime. This is a difficult task because there are important sources of uncertainty that could lead to over- or under-design solutions. In many practical engineering applications, the distributions of some random variables (Balu and Rao, 2012) may not be precisely known or uncertainties may not be appropriately represented. Eventually, factitious assumptions for distribution of the uncertainties will lead to inaccuracy of results. For example, there are uncertainties related to environmental exposure, loading, material properties and engineering models.

Let R be the resistance (capacity or strength) of the structure and S the action (load or load effect, namely, bending moment, shear force, etc.) on the structure. For reliability analysis, the space D of variables may be divided into the *failure* and the *safety regions* as shown in Figure 1. The failure region D_f is defined by $D_f = \{X|G(X) \leq 0\}$ and the *safety region*, D_s (Bastidas-Arteaga and Soubra, 2014; Gomes and Awruch, 2002), $D_s = \{X|G(X) > 0\}$ where $G(X)$ represents the performance function.

Notice that $G(X) = 0$ is the boundary between failure and safety regions, and it is called the *limit state surface*. In the simplest case, the performance function $G(X)$ is expressed as the difference between the resistance $R(X)$ and the demand or action on the system $S(X)$:

$$G(X) = R(X) - S(X) \tag{1}$$

In reliability analysis, $G(X)$ is usually expressed in terms of displacement, strain, stress, etc. The performance functions can be related to the following structural conditions:

Serviceability limit state. Under this condition, "failure" is related to a serviceability loss that does not imply a significant decay of structural safety. For example, if the reliability analysis of a given structural component focuses on maximum crack width, $w_{k,\max}$ the performance function can be written as:

$$G(X) = w_{k,\max} - w_k(X) \tag{2}$$

where: $w_{k,\max}$ could be fixed by standards or particular serviceability constraints depending on exposure condition, and $w_k(X)$ is the crack width of the point of interest that depends on variables X (material strength, geometry, load, etc.). In the case of failure, $w_k(X) > w_{k,\max}$ but the structural component is still considered safe.

Ultimate limit state. This condition describes the state at which structural safety is highly affected and may lead to total failure or collapse. For instance, if the reliability analysis focuses on the shear force of a beam, the performance function is:

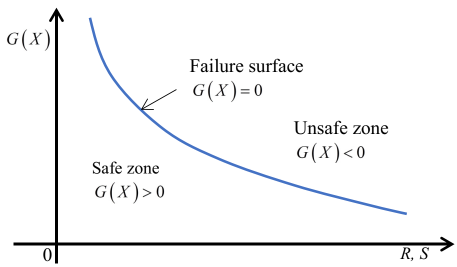

Figure 1.
Showing limit state, safe and unsafe regions

$$G(X) = V_r(X) - V_s(X) \quad (3)$$

where: $V_r(X)$ is the resistant shear force of the beam that depends on variables X (material strength, sectional geometry, etc.), and V_s is the demanding shear force.

Factors affecting the performance of concrete structures

Concrete is one of the composite materials normally used at every stage of construction and it may suffer damages or defects (EN 1992-1-1, 2004; Bakri and Mydin, 2014; MacGregor *et al.*, 1997) during its service life due to number of reasons i.e. poor workmanship, faulty design, structural overloading, moisture, chemical reaction, creep, shrinkage, permeability of concrete, corrosion of reinforcement, poor maintenance and foundation settlement. These factors are generally classified as:

- design aspects include the design criteria, accuracy of analytical and design equations and design errors;
- construction aspects include the natural variation of strength parameters and construction errors;
- environment aspects include aggressive chemicals which leads corrosion, and ambient humidity and temperature which cause creep and shrinkage;
- utilization aspects include the natural variations of service loads, utilization errors and the man-made hazards; and
- reliability aspects include the statistical and modeling uncertainties (Arafah, 2000).

However, it may not be possible to eliminate defects altogether (Kumar *et al.*, 2001; Wilmot, 2006; Ho and Rahman, 2004; Bremner *et al.*, 2001; Bakri and Mydin, 2014; Liubin *et al.*, 2011; Mohamed *et al.*, 2018; Qiao *et al.*, 2014); the remedial measures for minimizing the defects of concrete structures can be used. These are adopting appropriate design, construction, utilization and maintenance; improving quality of concrete by adopting appropriate constituents selection, mix design, compaction and curing; preventing corrosion of reinforcement by increasing depth of concrete cover, coating rebars, chloride extraction, waterproofing and patch repair and upgrading supporting ground to control foundation settlement.

Creep and shrinkage

Creep of concrete is defined as plastic deformation under sustained load or stress. Whereas, *shrinkage of concrete* is the property of diminishing in the volume of concrete during the process of hardening. Creep and shrinkage of the concrete depend on the ambient humidity, the dimensions of the element and the composition of the concrete. Creep is also influenced by the maturity of the concrete when the load is first applied and depends on the duration and magnitude of the loading (EN 1992-1-1, 2004).

Corrosion

Many researchers investigated the effects of aggressive environments on reinforced concrete structures. The aggressiveness of the environment is a very important factor to be considered when examining the concrete structure that shows signs of possible distress. Fan *et al.* (2019) suggested that reinforcing steel corrosion is one of the most serious deterioration mechanisms in reinforced concrete structures and is also an important issue that needs to be considered when evaluating and rehabilitating reinforced concrete structures. Generally, a corrosion attack is initiated either due to the carbonation of the concrete or due to the diffusion of the chloride ions to the reinforcing steel bar surface or both. As per (Almusallam, 2001) investigation, corrosion of reinforcing steel and the consequent cracking of concrete due to the ingress (diffusion) of chloride ions to the reinforcing steel bar surface is more predominant than that due to carbonation of concrete.

The effect of reinforcement steel corrosion has been investigated by researchers and found that reduce load carrying capacity; loss of diameter or effective cross-sectional area; significantly reduce bond strength; increase crack width; and strongly reduced elongation of reinforcement steel (Xia *et al.*, 2013; Almusallam *et al.*, 1996; Liu, 1996; ZandiHanjari *et al.*, 2008; Loreto *et al.*, 2011; Adukpo *et al.*, 2013; Chaitanya and Vamsi, 2014; Baskaran and Gopinath, 2016; Zhou *et al.*, 2014; François *et al.*, 2013; Stanish, 1997; Cabrera, 1996; Yalciner *et al.*, 2012; Shaikh, 2018). In addition, corrosion exerts pressure that easily exceeds the very limited tensile strength of the concrete and thus leads to cracking and spalling of the concrete cover (Allam *et al.*, 1994), hence reducing the service life of the structure. According to François *et al.* (2013), however, the degree of corrosion strongly affects the mechanical properties of the steel, particularly the ultimate stress and strain; the true yield strength and elastic yield stress of all the corroded steel bars remains almost constant while their true ultimate strength significantly increases. In general, (François *et al.*, 2013; Cabrera, 1996; Qing Li, 2004; Hagino *et al.*, 2013) in aggressive environment, reinforcing steel is sensitive to corrosion, which reduces the flexural strength, exerts a pressure that easily exceeds the very limited tensile strength of concrete, weakens the bond between steel and concrete which leads to cracking and spalling of concrete cover, and hence reduce the service life of the structure.

From the experimental and numerical investigation of reinforced concrete beam exposed to corrosion (Chaitanya and Vamsi, 2014; Zhao *et al.*, 2018), the load-carrying capacity and ductility of the beam are reduced, but the curvature increases consequently the deflection increases. From another experimental investigation (Ullah, 2017; Apostolopoulos and Kappatos, 2013; Ahmad, 2017), the ultimate capacities of all the beams are less than that of the control beam, and capacity loss is more in heavy corrosion. For higher flexure reinforcement ratio, the design failure occurs by shear failure instead of crushing of concrete for heavily corroded beams. Failure mode can change with stirrup corrosion, in that for beams with widely spaced stirrups the degree of corrosion does not affect the capacity loss but design requirement matters. Almusallam (2001), found that the reinforcing steel bars with more than 12 per cent corrosion indicate a brittle failure. The corrosion-induced concrete cracking would occur in reinforced concrete flexural members at about 18 per cent of its total service life, and that, once reinforced concrete flexural members become unserviceable due to corrosion induced excessive deflection, there is about 13 per cent of the service life remaining before the structures finally become unsafe (Qing Li, 2004). O'Flaherty *et al.* (2008) showed that under-reinforced beams with lower $M_{t(0)}/M_c$ ratios suffer lower flexural strength loss when subjected to tensile reinforcement corrosion, which $M_{t(0)}$ is the tensile moment of resistance, M_c is the compressive zone moment of resistance of the.

Before corrosion takes place, the structure would be subjected to the applied load, and creep and shrinkage. To prevent the effect of aggressive environment, design codes (EN 1992-1-1, 2004; IS456, 2000) provided respective concrete cover based on the exposure condition of the structural element. The concrete cover may delay the corrosion initiation time but does not fully control the corrosion. To consider the effect of corrosion on the serviceability of the structure, the corrosion initiation time is a very important factor.

Corrosion initiation period refers to the time during which the passivation of steel is destroyed and the reinforcement starts to corrode actively. The rate of chloride penetration into concrete, as a function of depth from the concrete surface and time, can be represented by Fick's law of diffusion (Thoft-Christensen *et al.*, 1996) as follows:

$$\frac{\delta c}{\delta t} = D_c \frac{\delta^2 c}{\delta x^2} \tag{4}$$

where c is the chloride ion concentration, as per cent of the weight of cement, at distance x cm from the concrete surface after t seconds of exposure to the chloride source. D_c is the chloride diffusion coefficient expressed in cm^2/s. The solution of the equation (4) is:

$$C(x,t) = C_0\left\{1 - erf\left(\frac{x}{2\sqrt{D_c.t}}\right)\right\} \tag{5}$$

where x is the distance from the concrete surface in (cm); t is the time in (sec); *erf* is the error function, and $C(x, t)$ is the chloride concentration at any position x at a time t. In a real structure, $C(x, t)$ is assumed to be the chloride corrosion threshold and x is the thickness of the concrete cover. The corrosion initiation period, T_i, can be calculated based on a knowledge of the parameters C_0 and D_c give in expression (Thoft-Christensen *et al.*, 1996) as:

$$T_i = \frac{C^2}{4D_c}\left[erf^{-1}\left(\frac{C_0 - C_{cr}}{C_0}\right)\right]^{-2} \tag{6}$$

where C(cm) is the concrete cover thickness, D_c(cm^2/year) is the chloride diffusion coefficient, C_0 (per cent *weight of concrete*) is the equilibrium chloride concentration at the concrete surface, and C_{cr} (per cent *weight of concrete*) is the critical chloride concentration.

The time-variant resistance of the concrete section is then calculated by considering the reduction of the steel section with reference to the corrosion initiation time. The reduction of the bar diameter is given by (Val *et al.*, 1998):

$$A_s(t) = \frac{\pi}{4}\sum_{i=1}^{n}\left(D_i(t)\right)^2 \tag{7}$$

$$D_i(t) = \begin{cases} D_i & for\ t \leq T_i \\ D_i - r_{corr}(t - T_i) & for\ T_i \leq t \leq T_i + \dfrac{D_i}{r_{corr}} \\ 0 & for\ t \geq T_i + \dfrac{D_i}{r_{corr}} \end{cases} \tag{8}$$

where $D_i(t)$ is the ith diameter of the reinforcing bars at a time, t; D_i is the initial diameter of the ith bar at the time; n is the number of the bars; and r_{corr} is the rate of corrosion in μm/year.

The rate of corrosion is obtained from corrosion density, i_{corr} = 0.1-0.5 μA/cm^2 for low to moderate corrosion; i_{corr} = 0.5-1 μA/cm^2 for moderate to high corrosion; and $i_{corr} > 1$ μA/cm^2 for high corrosion rate (Val *et al.*, 1998; Enright and Frangopol, 1998), in which i_{corr} = 1 μA/cm^2 is equal to i_{corr} = 11.6 μm/year.

Corrosion of embedded reinforcement reduces not only the bar diameter but also the compressive strength of concrete with time. The reduction of compressive strength of concrete is given by expression (Kliukas *et al.*, 2015) as:

$$f_{cc}(t) = \alpha_{cc} k_2(t) f_{ck} \tag{9}$$

in which,

$$\alpha_{cc}(t) = 1 - 0.1 N_G/N_E \quad \text{or} \quad \alpha_{cc}(t) = 1 - 0.1 M_G/M_E \tag{9a}$$

$$k_2(t) = 0.85 - 1.7\rho(t) \quad \text{and} \quad \rho(t) = \frac{A_s(t)}{A_c} \tag{9b}$$

where N_G is the permanent force; N_E is the transient force; M_G is the bending moment caused by permanent force; M_E is bending moment caused by permanent and transient loads $\rho(t)$ is time-dependent reinforcement ratio.

Methodology and materials

Ethiopia is an under-constructed country, but nowadays the construction industry is aggressively increasing. The reinforced concrete is being used extensively in the construction industry. In this study, the site that had been selected is Addis Ababa, the capital city of Ethiopia, where the average annual temperature is 15.9°C and the average minimum temperature for consecutive three months (November, December and January) is 7°C and relative humidity of 60.7 per cent (Internet 1). The minimum average temperature indicates the temperature even below 7°C exist in which the salt deposit may meltdown. The building had been intended for salt storage because Addis Ababa is also a commercial center and salt is stored from the "*Danakil Salt Pan*" Afar, Ethiopia, which is the 7th largest salt mine in the world (Internet 2). In case, when salt meltdown due to cold temperature the chloride ions diffuses into the reinforced concrete member that causes corrosion of the embedded reinforcement. Thus, a beam contact with soil at the salt deposit area or suspended beam in building that has been intended for salt storage in wet area the embedded reinforcement is susceptible to corrosion.

To achieve the desired objective, a simply supported reinforced concrete beam of effective span 6 m, which has direct contact with soil is subjected to the dead load of 12 kN/m including self-weight and imposed load of 18 kN/m for salt storage building in Addis Ababa has been deigned conventionally based on the real data.

From the input data

Concrete characteristic strength: f_{ck} = 25 MPa concrete cross-sectional area: A_c = 0.15 m^2, three parts of cross-section perimeter exposed to drying: u = 1.3 m (Figure 2), age of concrete at loading: t_0 = 7 days, ambient temperature: T = 15.9°C, relative humidity of ambient environment: RH = 60.7 per cent, RH_0 = 100 per cent, cement type: Class N, and age of concrete at end of curing: t_s = 7 days.

Based on the environmental data of the proposed site and designed reinforced concrete beam section (see Figure 2), the time-dependent creep and shrinkage are modeled as shown in Figure 3 using expressions provided in EC2 (EN 1992-1-1, 2004) in Annex B. The model is developed for the notional size h_0 of $h_0 = 2A_c/u = 2 \times 0.15$ m^2/1.30 m = 331 mm. k_h is a coefficient depending on the notional size h_0 which is 231 mm, which lies between 200 and

300 mm. Therefore with linear interpolation, the k_h value is obtained as 0.82. The adjusted value of concrete age at loading t_0 by taking into account both contributions of ambient temperature and cement type is $t_0 = 5.756$ days.

The time-development curve of the creep coefficient $\varphi(t, t_0)$ as a function of time t is calculated as the product $\varphi(t, t_0) = \varphi_0 . \beta_c(t, t_0)$. The final creep coefficient $\varphi(\infty, t_0)$ at the infinite time is obtained by setting $t = \infty$ which corresponds to setting $\beta(t, t_0) = 1$; therefore, the final creep coefficient at the infinite time is calculated as $\varphi(\infty, t_0) = 3.159$. The time-development curve of the drying shrinkage strain $\varepsilon_{cd}(t)$ as a function of time t is calculated

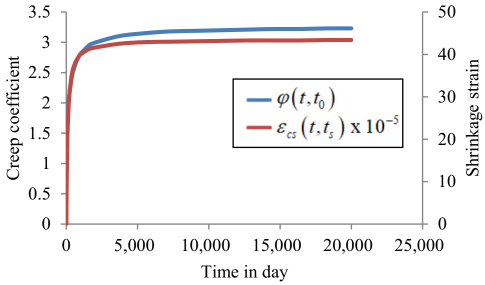

Figure 2.
The variation of creep coefficient and shrinkage strain with time

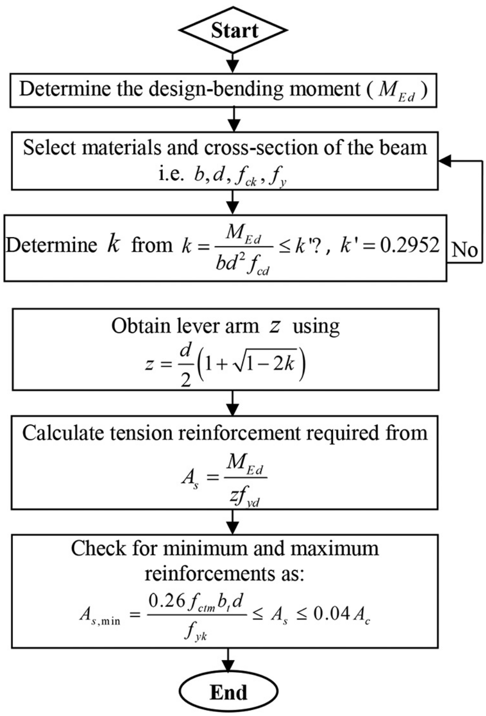

Figure 3.
Procedure for determining flexural reinforcement for singly reinforced section

as the product: $\varepsilon_{cd}(t) = \beta_{ds}(t, t_s)k_h.\varepsilon_{cd,0}$. The final total shrinkage strain $\varepsilon_{cs}(\infty, t_s)$ at the infinite time is obtained by setting $t = \infty$ which corresponds to setting $\beta_{ds}(\infty, t_s) = 1.0$ and $\beta_{as}(\infty, t_s) = 1.0$, and therefore the final total shrinkage strain at the infinite time is calculated as $\varepsilon_{cs}(\infty, t_s) = 40.97 \times 10^{-5}$.

From the analysis of the beam for the given loading based on (EN 1992-1-2, 2004), the design bending moment is 194.4kNm and shear force at the center of support is 129.6 kN. Design is carried out based on EC2 as shown in Figure 3 using materials of concrete grade C25/30 and steel grade 460 N/mm^2 of the longitudinal bar, 250 N/mm^2 for transverse or shear reinforcement and maximum size of aggregate, d_a = 25 mm. The design has been governed by the deflection of a beam by checking its long-term deflection under consideration creep and shrinkage.

Result and discussion

The effect of corrosion depends on chloride diffusion, chloride concentration, concrete cover, corrosion rate; in that its effect is directly proportional to chloride diffusion and rate of corrosion, and inversely proportional to concrete cover. The baseline values from the reference (Enright and Frangopol, 1998) taken as chloride diffusion coefficient D_c = 1.29cm^2/year, surface chloride concentration C_0 = 0.10 per cent weight of concrete, and critical chloride concentration C_{cr} = 0.04 per cent weight of concrete, the concrete cover of 5.8 and 5.0 cm for flexural and shear reinforcement, respectively, are used for parametric studies. In this study, of time-dependent safety analysis of simply supported reinforced concrete beam, using equation (6) the corrosion initiation time is 18.41 year and 13.368 year for flexural and shear reinforcement, respectively.

The current density of corrosion, $i_{corr} = 1\ \mu$A/cm^2, corresponds to $i_{corr} = 11.6\ \mu$m/year, in this paper, moderate corrosion rate is considered as the current density of $i_{corr} = 0.75$ $\mu A/cm^2$ is $i_{corr} = 8.7\ \mu$m/year. The rate of corrosion is determined as, $r_{corr} = 0.0232 i_{corr}$. Therefore, the time-variant diameter of the bar is obtained by using equation (8) (Figure 5) as:

$$D_i(t) = 8 - 0.20184(t - 13.68)\ [\mathrm{mm}] \quad \text{for shear reinforcement}$$

$$D_i(t) = 22 - 0.20184(t - 18.41)\ [\mathrm{mm}] \quad \text{for flexural reinforcement}$$

As shown in Figure 4, the reduction of reinforcing steel diameter increases with increased corrosion and vice versa. Table I shows the reduction of the reinforcing steel area for moderate corrosion rate $i_{corr} = 0.75\ \mu$A/cm^2 for which corrosion initiates in 18.41 years for corresponding corrosion parameters, the result obtained also satisfy (François *et al.*, 2013;

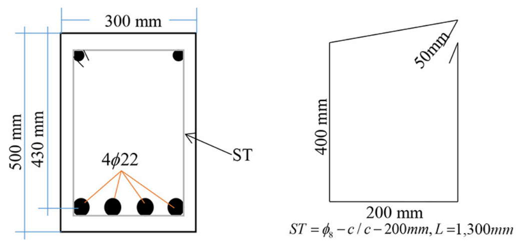

Figure 4.
Detail of beam section

Hajj Chehade *et al.*, 2018. The reduction in reinforcing steel area impairs the strength, safety and serviceability performance of the concrete cross-section.

Performance safety criteria

The reinforced concrete beam structure would be subjected to normal stress σ due to bending moments, M shear stresses τ results from shear forces V and in some cases also torsional stress results from equilibrium or compatibility moments or their combination in a lifetime. Bending is considered as the main stress in the design of a reinforced concrete member (Koteš *et al.*, 2015), and the size of the section and the arrangement of the longitudinal reinforcement are contemplated as first to provide the necessary moment resistance. Then, the member is designed for shear to ensure that the section is safe against the shear stress (EN 1992-1-2, 2004; Mosley *et al.*, 2007) in a whole lifetime because a shear failure is mostly sudden and brittle.

To satisfy the safety performance, a concrete structure must be safe and perform its intended purpose throughout its design life against all stresses. In this paper, two important safety criteria such as flexure and shear are investigated by considering time-variant parameters.

Flexural performance. Important factors that need to be considered in the calculation of time-dependent flexure are the section resistance, appropriate design loads, appropriate material properties, corrosion, creep and shrinkage. In concrete structures, the flexural capacity of the section decreases with time due reduction of reinforcing steel area through corrosion (Table I), because the under-reinforced section resistance depends mainly on reinforcement steel property and area.

Creep and shrinkage increase with time which induces additional flexure in the section as shown (Mosley *et al.*, 2007) in equation (10). The flexure induced by creep and shrinkage is time-dependent and can be obtained from the moment-curvature relationship as follows:

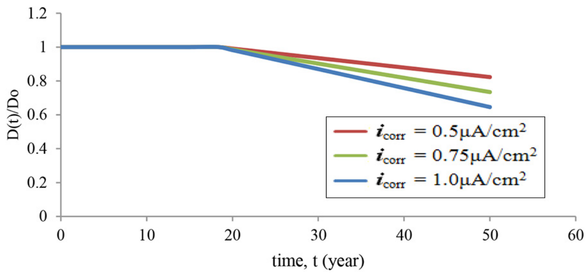

Figure 5. Reduction of reinforcement diameter vs time for various corrosion rates

Table I. Time-variant cross-sectional area of reinforcement due to corrosion with $i_{corr} = 0.75\,\mu A/cm^2$

Time (year)	$D_i(t)$ (mm)	$A_s(t)$ (mm^2)	Time,t (year)	$D_i(t)$ (mm)	$A_s(t)$ (mm^2)
0	22	1,520.531	25	21.661	1,473.988
5	22	1,520.531	30	20.670	1,342.226
10	22	1,520.531	35	19.661	1,214.358
15	22	1,520.531	40	17.642	977.820
18.41	22	1,520.531	45	16.633	869.150
20	21.679	1,476.493	50	15.624	766.880

$$\frac{1}{r_{t,sh}} = \frac{M_{sh}(t)}{E_{c,ef}(t)I_c(t)} \tag{10}$$

in which $1/r_{t,cs}$ is the curvature from shrinkage, and they can be expressed as:

$$\frac{1}{r_{t,sh}} = \xi\left(\frac{1}{r}(t)\right)_{sc} + (1-\xi)\left(\frac{1}{r}(t)\right)_{su} \tag{11}$$

where ξ is the coefficient given by $1-\beta(\sigma_{sr}/\sigma_s)$ allowing for tension stiffening; $\left(\frac{1}{r}(t)\right)_{su}$ is the curvature of uncracked section; $\left(\frac{1}{r}(t)\right)_{sc}$ is the curvature of cracked section; $\sigma_{sr}/\sigma_s = M_{cr}/M_{Qp}$; σ_{sr} is the steel stress at first cracking; σ_s is the steel stress of quasi-permanent service load; β is the coefficient takes in to account duration of loading and which is 0.5 for sustained loads; M_{cr} is the crack moment of concrete section and $M_{cr} = f_{ctm}.I_u/(h-x_u)$; M_{Qp} is the moment of the quasi-permanent moment at critical section.

The effect of creep is to increase flexure with time by increased curvature with time and thus should be allowed for in the calculations by using an effective modulus, $E_{c,ef}(t)$, using the equation:

$$E_{c,ef}(t) = E_{cm}(t)/(1+\varphi(t,t_0)) \tag{12}$$

where $E_{cm}(t)$ is the elastic modulus of concrete and can be obtained by (Mosley *et al.*, 2007) using $E_{cm}(t) = 22[(f_{ck}+8)/10]^{0.3}\,(t)$ in $[GPa]$.

The curvature of both uncracked and cracked condition for shrinkage is estimated from the equations (13) and (14)(Mosley *et al.*, 2007; EN 1992-1-2, 2004), respectively.

$$\left(\frac{1}{r}(t)\right)_{su} = \frac{\varepsilon_{cs}(t).\alpha_e(t).S_u}{I_u} \tag{13}$$

$$\left(\frac{1}{r}(t)\right)_{sc} = \frac{\varepsilon_{cs}(t).\alpha_e(t).S_c(t)}{I_c} \tag{14}$$

where $E_{c,eff}$ is the long-term elastic modulus of concrete; α_e is the effective modular ratio: $\alpha_e(t) = E_s/E_{c,eff}(t)$; E_s is the elastic modulus for reinforcement (200 GPa); $\varepsilon_{cs}(t)$ is the shrinkage strain; I_u is the second moment of area for uncracked condition; x_u is neutral axis for uncracked condition; I_c is the second moment of area for cracked condition; x_c is the neutral axis for cracked condition; Su and $Sc(t)$ are the first moments of area of the reinforcement about the centroid of the uncracked and fully cracked sections, respectively.

A sufficiently accurate calculation has been performed for neutral axis depth and the second moment of area of the section (Mosley *et al.*, 2007; MacGregor *et al.*, 1997), by considering the transformed section as shown in Figure 6. The neutral axis depth, the first moment of area and the second moment of area of the transformed section can be determined from equation (15) through equation (20).

Uncracked section properties

Depth of the neutral axis, x_u:

$$x_u = \text{Area Moment}/\text{Area} \tag{15}$$

where:

$$\text{Area Moment} = (\alpha_e - 1)A_{sc}d' + (\alpha_e - 1)A_s d + bh^2/2 \tag{15a}$$

$$\text{Area} = (\alpha_e - 1)A_{sc} + (\alpha_e - 1)A_s + bh \tag{15b}$$

The first moment of area of reinforcement about the centroid of the section, S_u:

$$S_u = A_s.(d - x_u) \tag{16}$$

The second moment of area, I_u:

$$I_u = I_{u,conc} + I_{u,A_{sc}} + I_{u,A_s} \tag{17}$$

where:

$$I_{u,conc} = bh^3/12 + bh(x_u - h/2)^2 \tag{17a}$$

$$I_{u,A_{sc}} = (\alpha_e - 1)A_{sc}(x_u - d')^2 \tag{17b}$$

$$I_{u,A_s} = (\alpha_e - 1)A_s(d - x_u)^2 \tag{17c}$$

Cracked section properties.
Depth to the neutral axis, x_c:

$$x_c = \left[-b_{eq} + \left(b_{eq} - 4ac\right)^{0.5}\right]/2a \tag{18}$$

where:

$$a = b/2 \tag{18a}$$

$$b_{eq}(t) = (\alpha_e(t) - 1)A_{sc} + \alpha_e(t)A_s \tag{18b}$$

$$c = -[(\alpha_e(t) - 1)A_{sc}d' + \alpha_e(t)A_s d] \tag{18c}$$

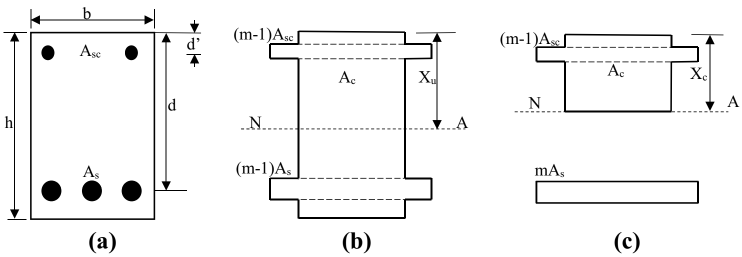

Figure 6.
The transformed sections

Notes: (a) Cross-section; (b) uncracked transformed section; (c) cracked transformed section

The first moment of area of reinforcement about the centroid of the section, $S_c(t)$:

$$S_c(t) = A_s(t).(d - x_c(t)) \tag{19}$$

The second moment of area, I_c:

$$I_c(t) = I_{c,conc}(t) + I_{c,A_{sc}}(t) + I_{c,A_s}(t) \tag{20}$$

where:

$$I_{c,conc}(t) = bx_c{}^3(t)/12 + bx_c{}^3(t)/4 = bx_c{}^3(t)/3 \tag{20a}$$

$$I_{c,A_{sc}}(t) = (\alpha_e(t) - 1)A_{sc}(t)(x_c(t) - d')^2 \tag{20b}$$

$$I_{c,A_s}(t) = \alpha_e(t)A_s(t)(d - x_c(t))^2 \tag{20c}$$

Time-dependent performance analysis for flexure is performed for the designed beam section is represented (Hajj Chehade *et al.*, 2018) in the following equation.

$$G(X_i(t)) = M_R(A_s, f_y, f_{ck}, d, b, \ldots)(t) - M_a(w_{DL}, w_{LL}, sh, \ldots)(t) \tag{21}$$

where $M_R(t)$ is the ultimate bending moment capacity of the beam, and $M_a(t)$ is the actual bending moment load at mid-span.

The time-variant flexural resistance of the section and applied bending moment are given in equations (16) and (17).

$$M_R(t) = A_s(t)f_y[d - 0.4x_c(t)] \tag{22}$$

$$M_a(t) = w_{d,DL}L^2/8 + w_{d,LL}L^2/8 + E_{c,ef}(t)I_c(t)1/r_{t,sh} \tag{23}$$

As expressed in equation (21), the flexure resistance of reinforced concrete structure depends on material properties, cross-sectional dimension, area of reinforcing steel, structural system (i.e. supporting system), loading condition, creep and shrinkage. The increment of applied bending moment from shrinkage grows rapidly and reaches the highest intensity at an early age (Figure 7), due to the maturity of concrete and the effect of creep, which grows rapidly in early age as shown in Figure 2 that reduces the elastic modulus of concrete. In addition, the performance of reinforced concrete beam significantly decreases with time due to corrosion of reinforcement steel which deteriorates the effective diameter of reinforcement steel and concrete strength and reduces the elastic modulus of concrete with creep increases.

As shown in Figure 7 at a time of Year 22 the flexural capacity and demand of the section coincides, i.e. the capacity to demand ratio is equal at that point. In other words, the point at which demand and capacity coincide that point is said to be a limit state, $G_M(X, t) = M_R(X, t) - M_a(X, t) = 0$ (Figure 7). Therefore, at the time after Year 22, the structure becomes unsafe due to its limited flexural capacity, i.e. demand is greater than the capacity of the section.

Shear performance. The transfer of shear in reinforced concrete members occurs by the combination of: shear resistance of the *uncracked* concrete in compression, *aggregate* *interlock* force that can be developed tangentially along the expected crack propagation, and similar to a frictional force due to irregular interlocking of aggregates along the rough concrete surface on each side of the crack, *dowel action* of the longitudinal reinforcement which is the resistance of the longitudinal reinforcement to transverse force, and *shear reinforcement* resistance from stirrups. From these, the sum of shear in the uncracked compression zone, aggregate interlock force and dowel action of the longitudinal reinforcement is taken as $V_{Rd,c}$, the shear resisted by concrete section (EN 1992-1-2, 2004; Mosley *et al.*, 2007); the remaining shear force $V_{Rd,s}$ is resisted by stirrups.

The performance of the reinforced concrete structure for shear can be estimated from the expression:

$$G(X_i(t)) = V_R(V_{Rd,c}(t), V_{Rd,s}(t)) - V_a(V_{DL}, V_{LL}, \ldots, V_{sh}(t)) \tag{24}$$

where $V_{Rd,c}(t)$ is the design shear resistance of the member without shear reinforcement; $V_{Rd,s}(t)$ is the shear resistance of the member by stirrups; V_{DL}, V_{LL} is the applied shear from the live load and dead load, respectively; and $V_{sh}(t)$ is the additional load-induced from creep and shrinkage.

The time-variant shear resistance and applied shear force are given in the following equations:

$$V_{Rd,c}(t) = \left[C_{Rd,c}k(t)(100\rho_l(t)f_{ck}(t))^{1/3}\right]b_w d \geq v_{\min}bd \tag{25}$$

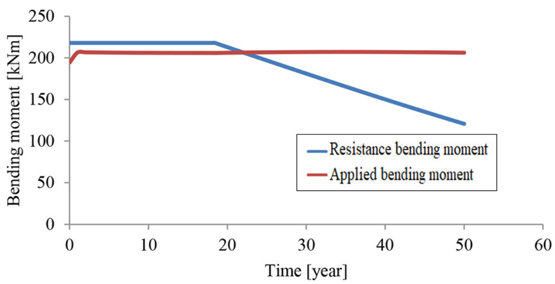

Figure 7.
Time-dependent flexural performance of the RC beam section

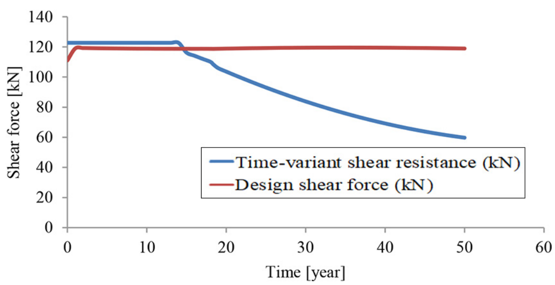

Figure 8.
Time-dependent shear performance of reinforced beam concrete section

$$V_{Rd,s}(t) = \frac{A_{sw}(t)}{s} z f_{ywd} \tag{26}$$

$$V_{DL}, V_{LL} = \frac{w_{DL}L}{2} + \frac{w_{LL}L}{2} \tag{27}$$

$$V_{sh}(t) = \frac{4E_{c,ef}(t)I_c(t)1/r_{t,sh}}{L} \tag{28}$$

As provided in equations (9), (25) and (28) corrosion affects the shear resistance of the reinforced concrete beam section mainly through the loss of effective reinforcement area and degrading the strength of the concrete [equation (8)], so that the shear resistance of the section decreases aggressively. In addition, the concrete cover of the stirrups is lower than the concrete cover of main longitudinal reinforcement, so, it means that the beam is more susceptible to shear failure due to corrosion than to moment failure in time (Figures 7 and 8).

At the time of Year 14.4, the shear capacity and demand of the section coincide, so the limit state of the section, $G_V(X(t)) = V_R(X, t) - V_a(X, t) = 0$, takes place (Figure 8). Therefore, at the time after Year 14.4, the structure becomes unsafe due to its limited shear capacity.

Conclusions

The performance of structure degrades with time due to corrosion from aggressive chemical attack, creep and shrinkage and utility. The results clearly indicate that the performance of the reinforced concrete beam significantly decreases with the increase of corrosion rate, creep and shrinkage in its design life, whereas the action on the structure increases due to shrinkage. The reinforced concrete beam section is more susceptible to the shear failure due to corrosion than flexural failure with time because the concrete cover of the links is less than the cover of longitudinal reinforcement in which corrosion of links initiates before longitudinal reinforcement, and both the effective area of links and concrete strength reduces due to corrosion eventually the shear capacity of the section significantly decreases with time. From the time-dependent performance evaluation, considering creep and shrinkage for specified ambient temperature and relative humidity, and moderate corrosion rate of the current density 0.75 $\mu A/cm^2$, the service life of the reinforced concrete beam is reduced to less than 50 per cent of its design life. Therefore, the time-dependent performance evaluation of the structure gives important inputs to know the level of damage and decide the maintenance strategy for preventing the premature failure of structures due to environmental effects.

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Corresponding author

Utino Worabo Woju can be contacted at: utinoworabo@gmail.com