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Effect of corrosion of reinforcement on the fatigue life of the high performance concrete beams

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1.Abstract
When the placement and compaction of the concrete are not of sufficiently good quality, the concrete becomes pervious thus allowing the ingress of harmful agencies like oxygen and moisture leading to corrosion of reinforcement. Corrosionofrebars is a major problem in the reinforced cement concrete. In the offshore and bridgereinforced concrete structures due the synergy between the corrosion of reinforcement and cyclic load they fail. If the fatigue life of the corroded beam is estimated then it will be safe to repair or rehabilitate in a proper manner. An experimental investigation was carried out on High Performance Concrete beams of M60 grade with corrodedreinforcement to assess the residual life of beams under cyclic loading. An accelerated corrosion regime of different degrees of corrosion damage of 10% and 25% were induced in the steel reinforcement of HPC concrete beams. The beams were then tested under cyclic loading until failure and noted the fatigue life of each beam. The results showed a marked reduction of fatigue life with the increase in percentage of corrosion. The failure occurred by fracturing of steel with its reduction in the cross sectional area and the surrounding surface was observedbrownish which indicates the corrosion effect and it demonstrated that the bond strength of concrete beams inversely variedwith theintensity of the corrosion.

Keywords- HPC concrete beams, corrosion, fatigue/cyclic performance, residual fatigue/cyclic life, Load-Deflection curve

2.Introduction
The electrochemical and non-destructive techniques from the point of view of corrosion assessment and their applications to bridges, buildings and other civil engineering structures were studied[1]. Corrosion of reinforcing steel is widely accepted as the primary cause of premature deterioration in reinforced concrete (RC) structures. Predominantly, with the extensive use of de-icing salt in cold weather regions, bridge decks and bridge piers are vulnerable to corrosion of steel reinforcement[2]. The high alkaline environment of good quality concrete forms a passive film on the surface of the embedded steel that normally prevents the steel from further corroding. However, under the influence of chloride and carbonation, the passive film is disrupted or destroyed and the steel corrodes. The corrosion products occupy a larger volume and these induce stresses in the cover concrete resulting in cracking, delamination and spalling. In addition to loss of cover concrete, a RC member may undergo structural damage due to loss of bond between steel and concrete, and loss of rebar cross-sectional area. To plan repair strategy for damaged structures, the strength of the existing structures needs to be estimated. The past research addressed on the flexural behavior of corrosion damaged concrete members. They indicated that load carrying capacity and ductility decreased as the reinforcing steel bars were corroded.[3,4] The influence of corrosion on bond characteristics between steel and concrete. They demonstrated that loss of bond increased with sectional loss. Corrosion causes a reduction of the sectional area, ductility and strength of rebars, of the compressive strength of concrete caused by cracking and consequent concrete spalling, of the bond strength between steel and concrete. The subsequent redistribution of internal stresses induces a reduction in ductility at ultimate limit state and a variation of the deformational behavior in serviceability conditions[5,6,7,8].Migrating Corrosion inhibitors (MCIs) showed versatility as admixtures, surface treatments, and rehabilitation programs. The effectiveness of two commercial inhibitors applied to the reinforced concrete surface was evaluated. The corrosion behaviour of the steel rebar was monitored using AC electrochemical impedance spectroscopy (EIS). Corrosion potentials and polarization resistance values corroborated the inhibiting effects of the amine carboxylate and amino alcohol chemistry in an aggressive environment. The MCI products have successfully inhibited corrosion of the rebar in a 3.5% NaCl solution for duration of testing. MCI protected samples showed an average corrosion rate of 0.34 $\mu A/cm^2$ compared to untreated samples that were 1.50 $\mu A/cm^2$.This will increase the life expectancy by more than 15 years. XPS analysis demonstrated the presence of inhibitor on the steel rebar surface indicating MCI migration through the concrete.[9] . The corrosion of reinforced concrete structures, factors influencing corrosion of reinforcement, and its remedial measures like producing high strength/ performance concretes, less water cement ratio, pore free concrete and more cover to the concrete to some

extent.[10].The effects of corrosion damage were studiedon the load bearing capacity of reinforced concrete beams and columns in marine environment and reported that the bearing capacities of corroded columnsand beams were not simply caused by reductions in strength due toeffective areas of the reinforcing bars but also by cracks formed during the corrosion process[11].The bond effects using two methods for different influences were observed. The first technique looked at the effect of spalling concrete. The second looked at the effect of corrosion products. This was accomplished by casting reinforced concrete slabs with the ends of the reinforcing bars anchored in the concrete for a known length; the centre unbonded. The ends were corroded to various corrosion levels and then tested in flexure. Also included is a test of the predictive power of this work.[12] The performance of RC beams with corroded reinforcement under cyclic loading were investigated. In addition, the mechanical properties of corroded steel reinforcement after fatigue loading were investigated up to failure. It was found that an increase in the corrosion degree of steel reinforcement decreased the fatigue life of the beams and caused them to collapse in a brittle failure mode. For the same fatigue loading history, the ratio of maximum elongation at rupture to yield strength of the corroded steel decreased with an increase in the fatigue stress magnitude. [13].As a part of a broad research program, reinforced concrete cylinders of different qualities were exposed to the marine atmosphere. The cylinders were electrochemically monitored over a period of 56 months. The time for the onset of active corrosion was shorter for rebars in concretes with a high water-to-cement (w/c) ratio compared to that for rebars in low w/c ratio concrete. Results also indicated, as expected, that for equal periods of exposure, nominal corrosion current density (I corr) values were generally higher for rebar in concrete with higher w/c ratio than those for rebar in low w/c ratio concrete. Analysis of the observed impedance spectra in terms of a modified Randles circuit (in which the ideal capacitor is replaced by a constant phase element (CPE)) appeared to be a reasonable approximation.[14]. Investigations were conducted on the fatigue bond behaviour of corroded steel reinforced concrete beams. Bond failure occurred in all the beams. The variables in this test series were,the type of load applied (monotonic or repeated loading), the repeated load range, whether the reinforcementinside the beam was corroded or not, and whether a carbon fiber reinforced polymer(CFRP) repair method was used or not, and the fatigue life of the beams. . Varied linearly with the range of applied load with a very shallow slope. Corroding time to cover cracking were discussed, beams to a low corrosion level decreased the fatigue bond strength by about 30%. Corrosion caused the concrete in between the lugs of the reinforcing bars to be partially crushed due to the formation of the rust products from the corrosion process. This reduced the strength of the concrete keys and increased the rate of slip in the bar under repeated loading.[15]Corroding was applied to the reinforced concretes to varying degrees by exposing to cyclic NaCl spray and 40^{o}C drying. The amount of corrosion products and induced damaged were measured using image analysis. It was found that the corrosion products can accumulate at steel-concrete interface as well as penetrate cement paste and deposit within the hydration products, relicts of reacted slag, and air voids.Only a small amount of corrosion is required to induce visible cover cracking. Implications on modeling time to cover cracking were discussed [16]. Experimental investigations were conducted on the beamswhich were corroded to the extent of 10% and 25% under static loading.Some of the beams were strengthened with GFRP sheet of Unidirectional type. The parameters studied were first crack load, first crack deflection, yield load, yield deflection, service load, service deflection, ultimate load. The ultimate deflections were compared with corroded beams and strengthened beams[17]. The problem of the chloride-induced corrosion of the rebar in reinforced concrete was investigated particularly in the case of slabs and decks of bridges. Two models, corresponding to two corrosion stages initiation and propagation, were incorporated in the same finite element program.First model, a comprehensive one for the chloride ingress into concrete, was presented with special attention to non-linear diffusion coefficients, chloride binding isotherms and convection phenomena. The model tried to use a priori parameter estimations, according to material characteristics and external environment conditions, instead of a posteriori best fit. Complex geometries, like random cracks, could also be incorporated. Based on the results of chloride diffusion, subsequent active corrosion was assumed and the radial expansion of the corroded reinforcement reproduced. For cracking simulation, the Strong Discontinuity Approach was applied. Comparisons with experimental results were carried out, with reasonably good agreements being obtained, especially for cracking patterns. Major limitations referred to difficulties in establishing precise levels of basic data such as the chloride ion content at concrete surface, the chloride threshold concentration that triggers active corrosion, the rate of oxide production or the rust mechanical properties.[18]. The deteriorated member is subjectedto bending, concrete in compression reaches suddenly its limit deformation hindering to develop the full rotation capacity. In the present paper, the effect of a uniform corrosion on reinforcement was analyzed. A numerical model, able to take into account corrosion effects and to describe the structural behaviour of concrete structures, was developed for this purpose[19]. The effects of chloride-induced corrosion on the Cracking, bond strength, flexural strength, shear strength,column behaviour have been investigated in the past. Models have been developed to link material and structural aspects of deterioration. Aspects such as time to cracking and residual load-carrying capacity were found to be sensitive to small variations in key parameters such as the cover and the surface chloride level. Predictions from a spreadsheet model indicated that structures designed and built toBS 8110 ;1997achieved their design life without the need for significant repair. The predictions also indicated that the UK Highways Agency was justified in making BD 57 :1995 more onerous than BS 5400 :1990. With the validation against further test data the procedures developed could form the basis for codes of practice for the assessment of corrosion-damaged concrete structures and the durability design of new concrete structures[20].ANSYS modelling was made using the available results ofa corroded reinforced concrete beam. The results are then compared with experimental results. It was observed that with an increase of the reinforcement corrosion rate, the load-carrying capacity of the concrete beam and the bond strength decreases. In addition, the area under the load-displacement curve of the concrete beam decreases with the increase of the reinforcement corrosion[21]. The available data on static and cyclic loading of HSC beams strengthened were compared with the modelledbeams using ANSYS and discussed the reliability of the software.To evaluate the static and fatigue response of HSC beams with externally bonded CFRP laminates using ANSYS software. The behavior of the beams under static and fatigue loading were discussed[22].

3. Research significance:

For assessing the condition of corrosion-damaged structures, the residual cyclic life of structures was to be estimated. For this purpose, the effects of maintenance and repair options on their service life were to be determined. To meet this objective, the present study is focused on the residual strength and ductility of high performance reinforced concrete beams that were subjected to different degrees of corrosion damage.

4. Materials:

The mix proportion of concrete was arrived at by trial and error method and also as per the IS code10262with the following proportions per cubic meter of it. The quantities of materials of the concrete mix used, i.e., cement: fine aggregate: coarse aggregate: silica fume= 1:1.73:2.51:0.055 as shown in Table 1.

Table 1 Quantities of materials used

Details Content	Quantities
Targeted 28 day concrete strength	60 MPa
Cement	450 kg
Max. size of coarse aggregate	20 mm
Weight of 20 mm aggregate	680 kg
Min. size of aggregate	10 mm
Weight of 10 mm aggregate	450 kg
Fine aggregate	780 kg
Silica Fume (S.F.)	25 kg
Hyperplastisizer-Gilinum233(BASF)	380 gr
= 0.8%(cement + SF)	0.35
Water/cement ratio	

Four beams of size 150 mm × 250 mm × 3000 mm were cast. Table 2 shows the reinforcing details.Two specimens were considered as control beams, one for static loading and the other for fatigue loading. Out of the two remaining beams, one beam was allowed to corrode for 10% and other beam for 25%. These two beams are tested under cyclic loading.Figure1shows the schematic diagram of reinforcement in the beams.

Table 2 Beam details

Compression Reinforcement SizeNumber	Tensile ReinforcementSizeNumber	Shear Reinforcement - Stirrups SizeSpacing
10 mm diameter2	12 mm diameter2	8 mm diameter150 mm

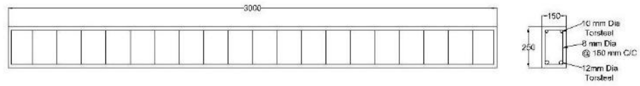

Fig. 1.Schematic representation ofReinforcement details of the beams

5. Experimental set up:

There were two experimental set ups. One was induction of corrosion and evaluation of corrosion process. The second one was testing of beams under four point bending. Figure 2 shows the induction of corrosion and assessment of corrosion.

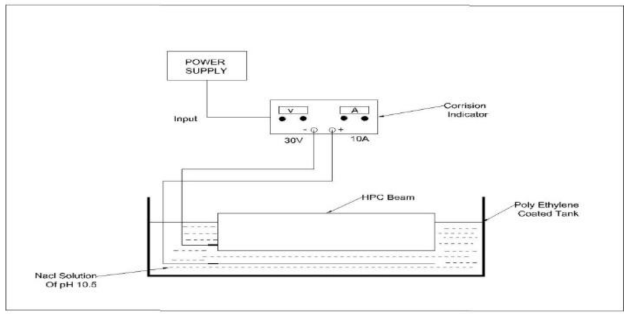

Fig.2. Schematic diagram of corrosion activity

5.1. Accelerated corrosion testing:

Beamsto be corroded (except control beams R0s and R0c) were subjected to accelerated corrosion condition. The beams were kept immersed in 3.5% NaCl solution in a high-density polyethylene tank. The beams were immersed for a day to ensure full saturation condition. The direction of the current was arranged so that the reinforcement cage served as the anode while a stainless steel plate acted as counter electrode. The accelerated corrosion process was achieved by applying a power supply with an output of 32 V and 11 amps. High voltage was used to accelerate the corrosion and shorten the test period. A schematic drawing of the corrosion testing is shown in Fig. 2 Two different degrees of corrosion damage of 10 and 25% were induced in this research study. The time for corrosion could be estimated by the Faraday's equation:

$$\Delta w = \frac{A_m . I . t}{Z.F} \qquad (1)$$

where:
Δw = Mass loss due to corrosion
A m = Atomic mass of steel (55.85 g)
I = Corrosion current in amps
t = Time since corrosion initiation (sec)
Z = Valency=2 (assuming that most of rust product is due to $Fe(OH)_2$)
F = Faraday's constant [96487 coulombs(g/equivalent)]
The corrosion activity was monitored for the beams by measuring the corrosion potential in

accordance with the ASTM[10] procedure. The probability of corrosion is based on specific ranges of potential of steel reinforcement with respect to standard reference electrode.
Total Weight of the reinforcement = 9577 gr.
10% of loss of weight = 957.7gr
25% of loss of weight =2394.25gr
Time of corrosion for 10% loss of weight = 84.4 hrs
Time of corrosion for 25% loss of weight = 208.84 hrs

Thus, by knowing the original mass of the rebar and the total current of the mass loss, the duration of time of corrosion activity can be determined.
Corrosion was measured by using Half cell potentiometer. As per the manufacturer's specifications the following standard chart was given for assessment of corrosion.
Ifthe voltage is measured as < 500mv-------Severe corrosion
If -500 mv to -350 mv ------Probability of corrosion > 90%
If -350 mv to -200 mv --------Probability of uncertain corrosion
If > -200 mv --------Probability of corrosion < 10%

5.2. Static loading:
At first a control beam R0 was tested under static loading. The ultimate load carrying capacity or load at failure was noted. It was 56.41 kn.Figure 3 shows the load-deflection curve of the beam tested under static loading. The trend of the curve is normal and the area below the curve indicates the energy absorption by the beam.

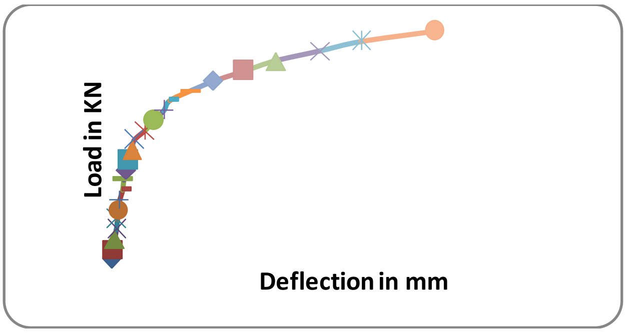

Fig. 3 Load-Deflectioncurve for static loading

5.3. Cyclic loading:
One control beam was tested under cyclic loading. Figure 4 shows the experimental set up for cyclic loading. The time series of the cyclic loading is given inTable 3.

Table 3 Time series of cyclic loading

Time in sec	0.1	0.1	0.1	0.5	0.1	0.1
Load in kN	0	5.00	10.00	19.74 (R0)	10.00	5.00

Fig. 4 Experimental set up for cyclic loading

The load imposed on each beam and the experimental results (deflections at failure) are given in the Table 4

Table 4 Beam details

Beam designation	Degree of corrosion (%)	Load in percentage of USL	Load (kN)
R0c	0	35	19.74
A0	10	60	33.84
B0	25	70	39.48

Figure 5 shows the behaviour of beam under cyclic loading.

Fig. 5 Cyclic behavior of the beams

Fig. 6 The failure pattern and the effect of corrosion

6. Results

The cyclic behaviour of the beams is shown in Fig. 4. Due to the corrosion damage the ductility of the beam notably decreased with increasing levels of corrosion damage The deflection ductility is defined by ultimate deflection to deflection at kink point (abrupt change in slope of the curves represents the onset of unstable crack propagation).

7. Discussion

The load imposed on the corroded beams was increased over the control beam by 25-35%, the cyclic life is decreased by 92.5% and 98%. It shows that an increase in load on the corroded beams reduces the number of cycles of the beam and it is not relative change. It can also be explained further that the slope of the stress-strain curve decreased with increasing degrees of corrosion damage level thereby it indicated the gradual reduction in the stiffness of the corroded beams. This indicated that the failure of beams was brittle in nature at higher degrees of corrosion. The ultimate strain is also reduced with increase in percentage of corrosion. But all the beams show same pattern of strain curve, i.e., step type. The ultimate deflection of the beams, decreased with increasing reinforcement corrosion level, leading to a reduction in ductility of the beams.The decrease in ductility was found to be 23.65 and 63.49%, respectively for beams subjected to 10 and 25% corrosion damage level. The crack propagation was observed along the reinforcement before loading butthe failure was not observed along the reinforcement.

8. Conclusions

Based on the results presented, the following conclusions are drawn:

1. The reduction in cyclic age was attributed to the loss of cross-sectional area of steel reinforcement due to corrosion
2. The deflection at failure was decreased with increasing level of corrosion damage, leading to a reduction in the ductility of the beams.
3. The increase in corrosion intensity decreased the absorbed energy and hence the ductility of the beams.
4. The corrosion damaged concrete beams failed in brittle manner at higher levels of corrosion.
5. The cyclic age is dependent on the load intensity and the corrosion damage without any particular proportion or ratio.

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