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Effect of corrosion products on bond strength and flexural behaviour of reinforced concrete slabs

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New performance-based design codes are currently being developed, where the design life of reinforced concrete structures will be estimated by taking into account not only the time to initiation of reinforcement corrosion, but also the time it would take for the extent of corrosion to reach a level where the structure is no longer fit for purpose. It is therefore important to establish what level of corrosion, if any, can be permitted before the structural behaviour of the reinforced concrete member is affected. In this paper the effect of corrosion products on the bond strength and flexural behaviour of reinforced concrete slabs was investigated. Pull-out tests confirmed that low levels of corrosion (less than 2% loss in steel cross-sectional area) can result in improved bond between the reinforcing bars and the concrete. At higher corrosion levels the empirical bond decay functions proposed by various researchers accurately predict the bond strength. The flexural behaviour of the slabs is affected by the reduced bond between the steel and the concrete, and this manifests during the load tests in a reduction in the number of cracks but an increase in crack width, with increased corrosion levels. At high corrosion levels (more than 8% loss in steel cross-sectional area) the flexural behaviour of the slabs is affected to such an extent that brittle failure occurs.

INTRODUCTION

The use of reinforcing steel to improve the tensile properties of concrete has been an accepted practice for many years. Reinforced concrete (RC) is known for its durability and its ability to withstand harsh environmental conditions. Nonetheless, corrosion of reinforcing steel is widely accepted as the primary cause of premature deterioration in reinforced concrete structures (Shetty *et al* 2011). Currently new performance-based design codes are being developed, where the design life of reinforced concrete structures will be estimated by taking into account not only the time to initiation of reinforcement corrosion, but also the time it would take for the extent of corrosion to reach a level where the structure is no longer fit for purpose. It is therefore important to establish what level of corrosion, if any, can be permitted before the structural behaviour of the reinforced concrete member is affected.

Two mechanisms that occur when reinforcing steel corrodes can result in a decrease in load-carrying capacity of reinforced concrete components. Firstly, the cross-sectional area of the steel bars decreases as the effect of corrosion increases, and the severity of reinforcement corrosion can have a significant effect on flexural strength, deformational behaviour, ductility, bond strength and mode of failure of RC slabs (Shetty *et al* 2011). A less studied mechanism is the effect of corrosion on the bond at the interface of the steel rebar and the concrete. The expansive nature of the corrosion products that build up at the interface exerts a radial pressure on the surrounding concrete, which leads to cracking and spalling. Spalling will reduce the bond by removing the concrete cover, which in turn will reduce the confinement of the steel rebar and expose the concrete to further corrosion activity. Corrosion also causes the ribs on the rebar to deteriorate, which changes the surface area of the bar and decreases bond strength. Some researchers have, however, found that low levels of corrosion produce a firmly adherent layer of rust that may contribute to an enhancement in bond strength (Kivell *et al* 2011).

The aim of this study is to determine the effect of corrosion on bond strength by considering different corrosion levels. Various concrete specimens containing the same size of steel bars and using a single concrete mix were corroded to low and high levels of corrosion. The effect of the corrosion products on the surrounding concrete, as a result of accelerated corrosion, was determined. The bond stress was evaluated for cylindrical specimens, subjected to a pull-out test, to determine if low corrosion levels (< 2% mass loss of reinforcing bars) can actually cause an increase in bond strength. The structural performance of various slab specimens was

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compared by considering the effect of corrosion on the moment capacity of the slabs. The effect of corrosion level on the distribution and size of cracks that form during flexural testing was examined.

BACKGROUND

One of the major consequences of reinforcement affected by corrosion is the reduction in cross-sectional area. The obvious effect of corrosion is the formation of corrosion products known as rust. This is a weak material that accumulates at the steel surface and has a larger volume than the initial volume of the un-corroded steel bar. This causes stresses in the concrete and leads to cracking, which will have some influence on the structural performance of the RC structure. This can also result in spalling of the concrete cover. RC that experiences a decrease in the alkalinity of the pore solution due to carbonation of the concrete may cause the reinforcing to corrode fairly uniformly. However, if chlorides are concentrated near the surface of the steel, or water and oxygen ingress is restricted to a single location where severe cracks are present, pitting corrosion will occur. Pitting corrosion could result in the rapid decrease in reinforcement cross-sectional area at critical sections, thus significantly affecting the load-carrying capacity of structural members.

Bond between concrete and reinforcing steel is a complicated phenomenon which RC structures rely on in order to withstand design loads. Due to the different characteristics of the two materials, the stress transfer from the concrete to the reinforcement is essential in the design of safe structures. Bond stress can be defined as the change in the force within the reinforcing bar divided by the surface area of the bar over which the change in force takes place (Hassan 2003). According to Shetty *et al* (2011) bond strength is attributed to four factors:

- Chemical adhesion of the concrete to the steel
- Friction at the bar–concrete interface from mill-scale, rust and other surface irregularities
- Bearing against the rib faces
- Shear acting along a cylindrical concrete surface between adjacent ribs.

The chemical adhesion is due to the weak bonds between the steel and the hardened hydrated cement paste of the concrete, which is lost when the applied load on the steel bar is increased. Once the embedded bar begins to slip, the friction contributes to the bond strength at the concrete–steel interface. However, bond is primarily derived from the bearing and mechanical interlock of the ribs on the surface of the steel bar with the concrete. Owing to the angle of the ribs, a horizontal force develops between the concrete and the rib face angle, which exerts bursting forces that tend to split the concrete. The thickness of the concrete cover and the confinement of the reinforcement now limit the magnitude of the failure load.

In general, the level of corrosion can be calculated as the percentage mass loss of the corroded specimen. This mass loss is caused by the loss in cross-sectional area of the reinforcing, and the percentage loss is calculated over the embedded length of the bar subjected to corrosion. The results of the experimental study conducted by Hassan (2003) indicated an increase in bond strength of 6% and 9% for 0.34 and 0.71 percentage of mass loss for regular steel bars and stainless steel bars respectively. At low levels of corrosion the corrosion products adhere firmly to the steel surface, which increases the friction component without exerting a bursting pressure on the surrounding concrete. More severe levels of corrosion will lead to a bursting pressure in the cover concrete, which leads to cracking and spalling of the concrete. Increased crack widths will cause the concrete to become more susceptible to moisture ingress and cause the friction component of bond to deteriorate, thus decreasing the bond strength. The experimental results of Al-Sulaimani *et al* (1990) indicated extensive cracking, referred to as the post-cracking stage, at approximately 7% to 8% mass loss. The bond strength decreased slightly during the cracking stage when visible cracks appeared on the concrete surface, but was reduced to approximately 75% of the original strength when severe cracking occurred. The authors did not define severe cracking or measure crack width.

Chung *et al* (2004) found that the level of corrosion had a significant impact on the flexural crack pattern of RC slabs subjected to four-point loading. The slabs that were tested at lower corrosion levels displayed more flexural cracks than the slabs subjected to severe corrosion. Although a greater number of cracks formed at low levels of corrosion, the cracks were distributed along the length of the slab with small crack widths and sufficient warning of impending failure. At higher corrosion levels a smaller amount of localised cracks was produced that had much larger crack widths and propagated at a rapid pace. Thus, it can be said that a loss in bond due to corrosion will result in wider, more localised cracks, which makes the concrete more susceptible to moisture ingress and increases the rate of deterioration of the structure.

Researchers have proposed various empirical equations to calculate bond strength as a function of corrosion level, including the following equations:

Table 1 Concrete mix design

Material	kg/m^3
9.5 mm dolomite stone	1 100
dolomite sand	910
water	200
cement (Cem IV 32.5R)	300
w/c ratio	0.67
slump	32 mm

- Stanish *et al* (1999):

$$\frac{u_b}{\sqrt{f'_c}} = 0.77 - 0.027C_0$$

- Cabrera (1996):

$$u_b = 23.478 - 1.313C_0$$

- Lee *et al* (2002):

$$u_b = 5.21e^{-0.0561C_0} \quad for\ C_0 \geq C_C$$
$$u_b = 0.34f_{cu} - 1.93 \quad for\ C_0 \geq C_C$$

where

u_b is bond strength in MPa
f_{cu} is the concrete compressive cube strength in MPa
f_c' is the concrete cylinder strength (assumed to be $0.8^* f_{cu}$) in MPa
C_0 is corrosion percentage
C_c is corrosion percentage at cracking.

The equation proposed by Stanish *et al* (1999) was derived by normalising the estimated bond strength by the square root of the 28-day compressive strength, f'_c, and performing a linear regression analysis with the available data points. The value of $u_b/\sqrt{f_c'}$ calculated when no corrosion is present (x = 0) corresponds to the estimated value of 0.66 given by ACI 408.1R-90. However, this equation seems to indicate that there is no increase in bond strength at low corrosion levels and any corrosion would immediately result in a decrease in bond strength. Cabrera (1996) also utilised linear regression to derive a numerical relationship to calculate the reduction in bond strength as a result of corrosion, but disregarded the values where corrosion resulted in an increase in bond strength. The empirical equation proposed by Cabrera (1996) was obtained for normal Portland cement concrete and only applies to corrosion levels greater than approximately 0.9%. Lee *et al* (2002) proposed two equations to express bond properties of reinforcement as a function of corrosion percentage. Neither of these equations makes provision for the fact that bond strength seems to increase at low corrosion levels (for less than 2% reduction in reinforcing bar mass).

The proposed empirical equations estimate the bond stress to decrease as the

Figure 1 Pull-out test

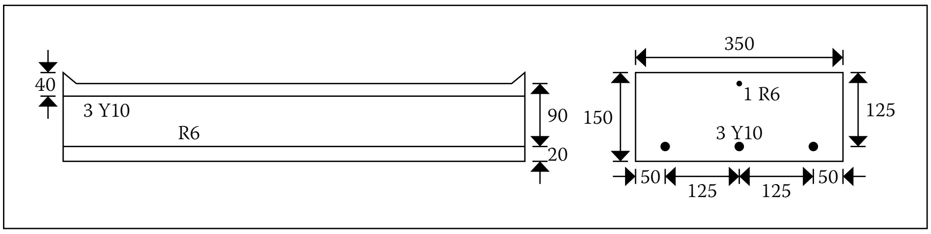

Figure 2 Experimental setup of test slabs (corrosion and flexural testing layouts respectively)

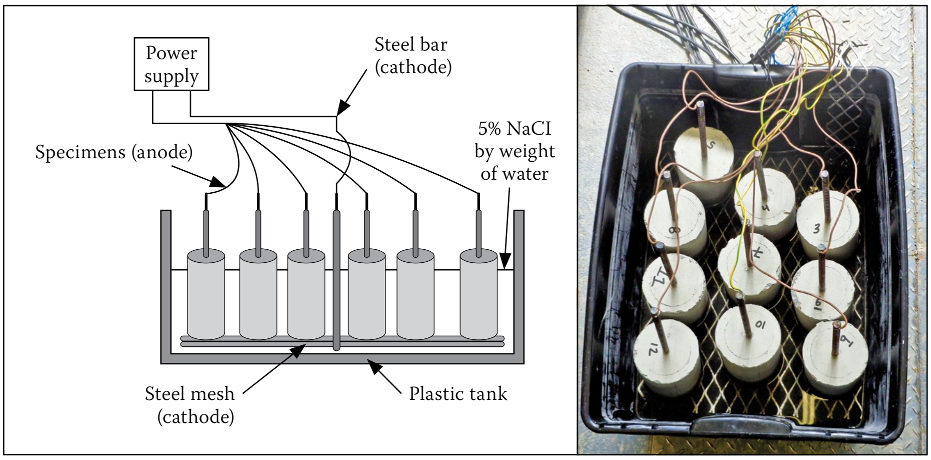

Figure 3 Schematic diagram of the accelerated corrosion test of the cylindrical specimens (Hassan 2003)

level of corrosion increases, without taking into account the phenomenon that occurs at low corrosion levels. It was decided to conduct experimental work to establish whether low levels of corrosion would have a detrimental effect on the bond strength and structural behaviour of reinforced concrete slabs.

EXPERIMENTAL WORK

Manufacturing of test specimen

Structural tests were used to evaluate the effect of corrosion on the bond between the concrete and the reinforcement. Two main effects of corrosion, namely spalling and loss in cross-sectional area, were examined closely as they have a notable effect on structural performance. Two series of tests were prepared, which comprised 20 cylinders and 12 slabs. The cylindrical concrete specimens were 100 mm in diameter, 200 mm high and contained a single Y10 bar protruding at one side only. The mass of the bar was recorded before concrete was cast. The embedded length of the bars was 155 mm with a total bar length of 355 mm (see Figure 1).

A standard structural specimen, resembling a slab, was designed for minimum size to fail in flexure. The specimen was 1 700 mm long, 350 mm wide and 150 mm deep, as seen in Figure 2. A cavity in the bottom (during flexural testing) of the slab was inserted to serve as a reservoir for the salt water solution (during corrosion phase of experiment). Three Y10 bars, with a length of 2 000 mm and a concrete cover of 20 mm, were added to the bottom of the slab, which also contained a single R6 bar with the same length and cover at the top. The mean yield strength of the reinforcing bars was measured as 456 MPa (s.d. = 8.8 MPa) with a modulus of elasticity of 213 GPa (s.d. = 10.2 GPa).

Six 100 mm cube specimens were cast to determine the 7- and 28-day water-cured cube strength of the concrete. Three additional 100 mm cube specimens were cast for each slab and tested on the same day as the corresponding slab. The cement used in this study was a CEM IV / B-V, 32.5R blended cement. Blended cement that can be purchased from any hardware store was used to ensure that the results obtained would be relevant to the behaviour of reinforced concrete typically found in small construction projects, such as double-storey house slabs. The concrete mix composition is indicated in Table 1. The characteristic strength of the concrete was 25 MPa.

Corrosion process

The rate of corrosion of the steel bars in the cylindrical specimens was accelerated by using a plastic container with a steel mesh in the bottom that served as the cathode (Hassan 2003), as illustrated in Figure 3. After the concrete cylinders were placed in the container, it was filled with a 5% NaCl solution to a height of 150 mm. The cylinders were connected in parallel to a DC power supply of 13 V and the current passing through each specimen was recorded. The average current density was 1 087 μA/cm^2 with a maximum applied current of 0.06 A, which is much less than the 0.4 A that Almusallam *et al* (1996) used in previous research. Using Faraday's Law and applying it to the integrated current, the corrosion of the bars was monitored (Stanish *et al* 1999). Two cylinders were corroded to a steel mass loss of approximately 1%, 2%, 5% and 8% each. This group of cylinders would be used to determine if a percentage corrosion smaller than 2% could cause an increase in the bond strength of an RC structure subjected to load. The other cylinders were corroded to a mass loss of approximately 9%, 10%, 11% and 12%. Two specimens from each group were not subjected to corrosion in order to serve as a control.

The method used to accelerate the corrosion of the bars cast into the slabs was based on the work done by Otieno *et al* (2012) where a cavity was inserted at the bottom of the slab to serve as a reservoir for the NaCl solution. Shutter boards were used to construct moulds for the slabs in order to accommodate their size, as well as the 20 mm deep cavity in the slab. During the curing and corrosion phase of the experiment the cavity would be at the top of the slab surface and filled with a 5% NaCl solution. The slabs were exposed to cyclic 3-days wetting and 1-day air-drying in the laboratory for the entire duration of the experiment, as the presence of oxygen is an essential factor for the corrosion of reinforcement in concrete (Shetty *et al* 2011).

After the slabs had been cured for seven days the bars were connected in parallel to the 13 V power supply, as illustrated in Figure 4. With the concrete acting as the electrolyte, the Y10 bars became anodic and the R6 rebar served as the cathode to complete the circuit. The slabs were corroded to different degrees, with percentage corrosion expressed as percentage mass loss. Three pairs of slabs were corroded to an *average* of approximately

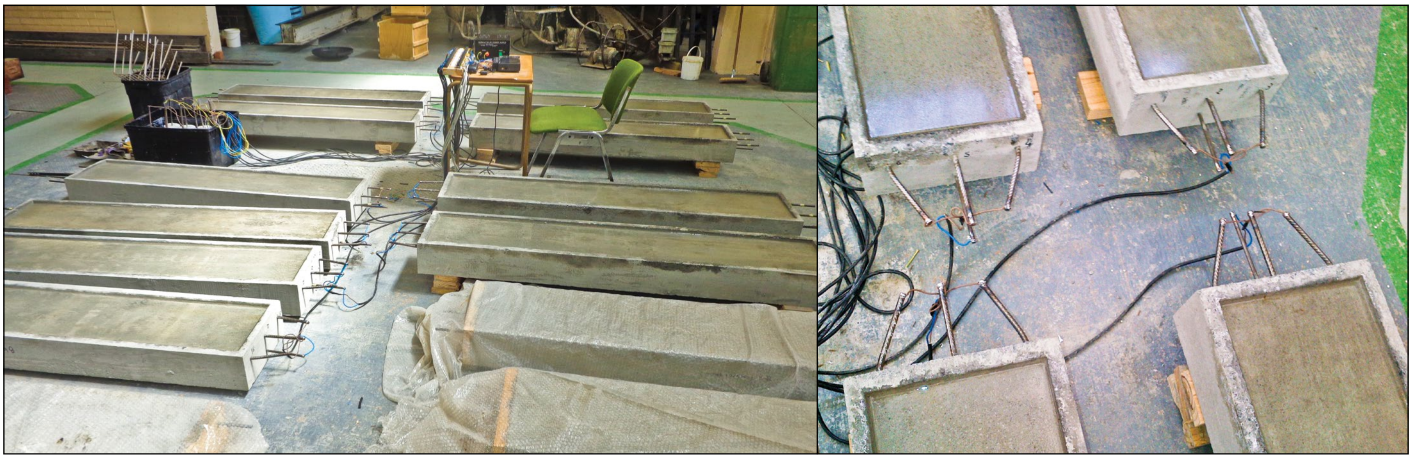

Figure 4 Accelerated corrosion setup of slab specimens

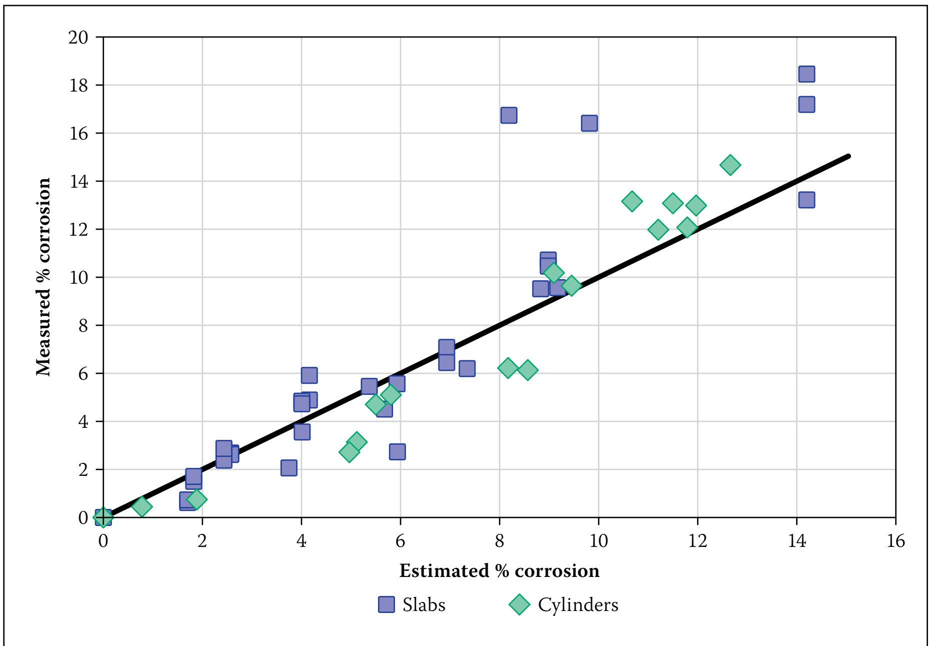

Figure 5 Measured vs Estimated percentage mass loss of the combined slab and cylinder specimens

Table 2 Compressive strength of concrete

% Mass loss	Comprehensive strength of concrete, MPa (s.d.)
0	29.7 (0.6)
2	24.8 (0.4)
3	28.9 (0.9)
6	28.8 (1.7)
6	32.5 (1.4)
8	32.5 (1.4)
9	35.0 (1.6)
14	35.0 (1.6)

2%, 3% and 6%. The remaining four slabs were corroded to 6%, 8%, 9% and 14%, assuming that all three bars had the same percentage corrosion at the time they were tested. By recording the current that passed through each slab and applying Faraday's Law to the integrated current, the corrosion was monitored. The average current density was 211 $\mu A/cm^2$ which is slightly more than the 189 $\mu A/cm^2$ of Malumbela *et al* (2012), but much less than the current density of 10 400 $\mu A/cm^2$ used by Almusallam *et al* (1996).

After the cylinder and slab specimens had been tested, the reinforcing bars were removed from the concrete and cleaned with a steel brush to determine the actual percentage corrosion. The bars were weighed and the corrosion level was calculated as the percentage mass loss for the embedded bar length of each specimen. The non-uniform corrosion over the length of the bar was visible, as is typical of chloride-induced corrosion. Figure 5 is a graphical representation of the actual measured mass loss versus the assumed mass loss estimated using Faraday's Law. It is evident that the estimated values obtained for the cylinder specimens are more accurate than those of the slab specimens. Owing to the varying distance between the cathodic R6 bar and the anodic Y10 bars in the slab, the centre bar corroded much faster than the adjacent Y10 bars. According to Malumbela *et al* (2010), the larger rate of corrosion of the centre bar can also be attributed to the corrosion crack pattern which causes the corrosion agents to reach the centre bar much faster than the exterior bars. This is denoted by the outliers in Figure 5, and resulted in a less accurate estimation of the percentage mass loss for the slab specimens. The uniform concrete cover provided by the cylindrical specimens resulted in more accurate predictions as a result of the homogeneous resistance of the concrete around the reinforcing.

Test procedures

After the cylindrical specimens had reached the desired degree of corrosion they were removed from the accelerated corrosion setup and subjected to a pull-out test. Pull-out tests were conducted on the cylindrical specimen using a hydraulic jack and a specially designed loading frame as indicated in Figure 1. One LVDT (Linear Variable Displacement Transducer) was attached to the steel bar, touching the surface of the concrete, to continuously measure the slip of the bar as the load pulled the bar from its confinement.

Slab specimens were tested using a four-point loading test as seen in Figure 6. The load was applied with a closed-loop Materials Testing System (MTS). The tests were conducted in deflection control, and the applied load, as well as the mid-span deflection, was recorded. Two LVDTs were used to measure the midpoint deflection of the specimen, while a set of four LVDTs was used to determine the average strain in the top and the bottom of the slab in the constant moment region.

RESULTS

The compressive strength of the cube specimens reached an average value of 30.0 MPa at 28 days, while the cube specimens that were tested on the same day as their corresponding slabs had a compressive strength between 24.8 MPa and 35.0 MPa. The slabs with the

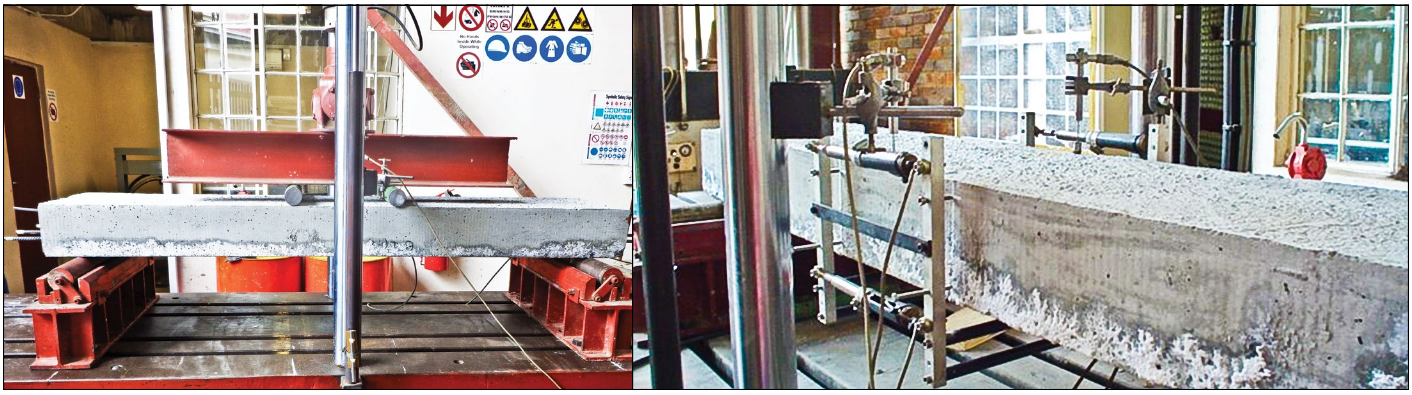

Figure 6 Testing of slabs

lowest corrosion levels were tested two weeks after casting and thus had a lower compressive strength than others on the day they were tested; the structural performance was, however, not significantly affected. The slabs with higher corrosion levels were tested after they had reached strength equal to or more than the target mean strength of 31.6 MPa at 28 days. The compressive strengths of the various slab specimens are summarised in Table 2.

Pull-out tests

The data recorded during the pull-out test was used to determine the bond strength of each specimen. The bond strength of all the cylinder specimens was plotted against percentage corrosion as seen in Figure 7. The first data point on the graph is the average bond stress calculated for the control specimens that were cured for 28 days and not subjected to accelerated corrosion. It is evident that a percentage mass loss less than 2% caused the bond strength to increase before it decreased to a minimum value. This increase could be related to the formation of products of corrosion. These results confirm the observations of Chung (1997) that rust is well adhered to the underlying steel at low levels of corrosion, which helps the bond between steel and concrete. Pressure develops at the concrete–steel interface during the initial stages of corrosion as a result of the larger volume occupied by the corrosion products. This provides some degree of confinement of the steel bar prior to cracking of the concrete cover and contributes to the initial increase in bond strength (Shang *et al* 2010). For corrosion levels higher than approximately 4%, the bond strength was much less than that of the control specimens. When the bars were removed from the concrete specimens it was noted that the corrosion started at the rib section of the bars. As corrosion progressed the height of the ribs decreased until it reached a limit when the corrosion level reached 2%. At this level of corrosion the ribs no longer existed, or a small portion remained (Chung *et al* 2004), thus identifying the 2% corrosion level as the critical corrosion level affecting bond strength.

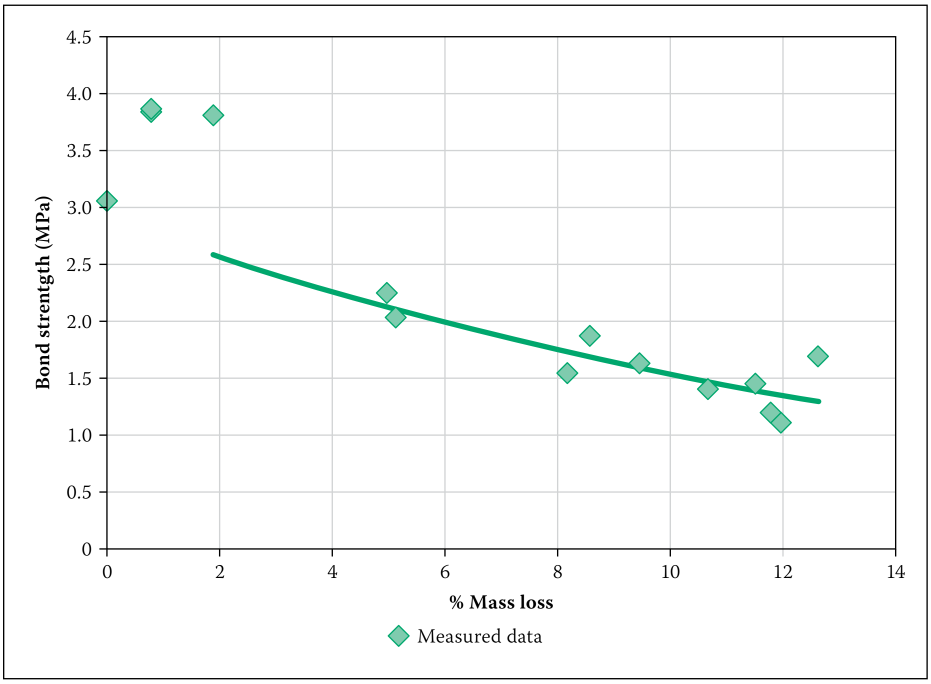

Figure 7 Effect of corrosion on bond strength

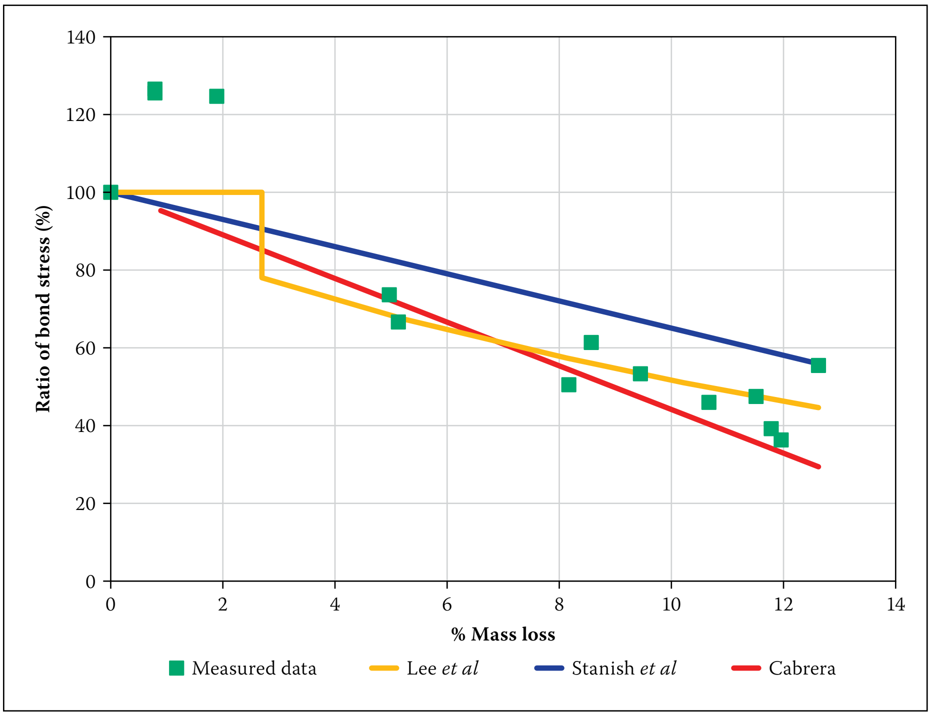

Figure 8 Comparison between the measured data and proposed empirical formulae

Figure 8 illustrates the relationship between bond strength and corrosion level for

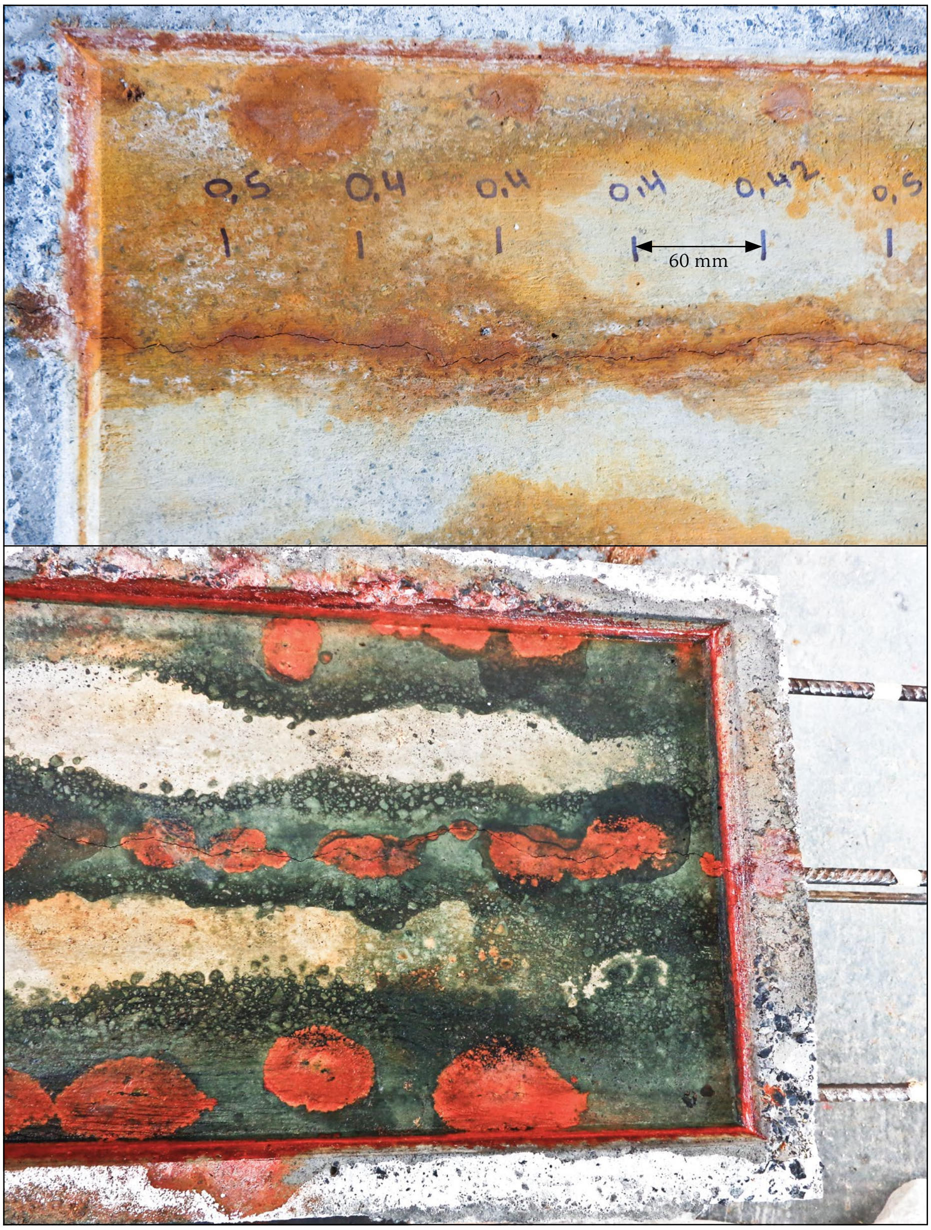

Figure 9 Longitudinal crack along the middle of the slab

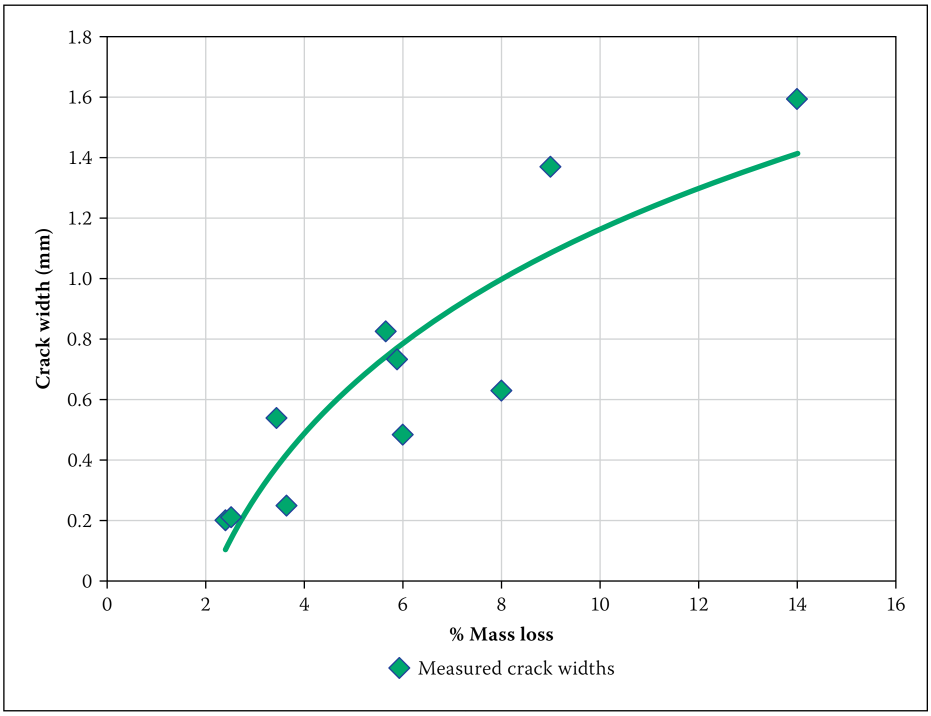

Figure 10 Average crack width with respect to corrosion level

not only the data obtained from this experiment, but also the equations proposed by various researchers. The empirical equation proposed by Stanish *et al* (1999) estimates the bond strength to decrease as the level of corrosion increases. This is not the case, as observed from the experimental data, which suggests that the bond strength is not directly proportional to the corrosion level. For corrosion levels higher than 4% the measured data points fall within the limits presented by the empirical formulae. The data obtained from the experimental procedure is in conformance with the proposed formulae, and suggests some exponential relationship between the decreasing bond stress and the level of corrosion. However, at low corrosion levels (< 2%) the empirical formulae do not represent the behaviour of the cylindrical specimens subjected to a pull-out force. The measured data indicated an increase in the bond stress at 0.8% and 1.9%, which can be interpreted as an increase in the frictional forces at the concrete–steel interface due to a well adhering layer of rust (Chung 1997). These results indicate that the strength of a reinforced concrete structure will not necessarily be negatively affected by low percentages of corrosion, and the useable life of a structure could include the time it takes for corrosion to proceed up to a specified limit.

Slab specimens

A total of 12 slab specimens were subjected to four-point loading to determine the effect of corrosion on bond strength. Varying degrees of corrosion had different effects on the moment capacity of the slabs. This was analysed by plotting the moment of the slab specimen against the curvature using data recorded during structural testing. Various observations were made with regard to colour changes, percentage corrosion and the resistivity of the concrete relating to the corrosion current of the slab specimens. As for the cylinders, the corrosion was non-uniform along the length of the steel bars, which is typical of chloride-induced corrosion. As the steel bars corroded, a longitudinal crack started to form along the middle of the slab due to the increasing corrosion. An accumulation of corrosion products exerts a bursting pressure that causes a crack to form along the length of the bar as indicated in Figure 9.

Cracks lead to more water ingress, which causes the anodic steel bars to corrode even faster, with a higher current passing through the bars. The reason for the longitudinal crack in the centre of the slab is due to the severity and high rate of corrosion of the middle bar. Before the slabs were subjected to four-point loading the crack widths were measured, using a crack measurement microscope, to determine how the crack width varied with respect to the percentage corrosion. The longitudinal crack was measured at 28 positions along the length of the slab, with each measurement approximately 60 mm apart. The relationship of the average crack width with respect to corrosion level is indicated in Figure 10.

As the percentage corrosion increased, the width of the longitudinal crack increased as well. The higher the level of corrosion, the more corrosion products accumulate along the length of the bar. This leads to spalling, which in turn allows more water to penetrate the concrete. Although the crack reached a width of up to 1.6 mm, this did not have an effect on the structural behaviour of the slabs as no transverse forces were applied to the slab. Crack widths up to 0.3 mm are normally deemed to be acceptable in reinforced concrete structures, and the results in Figure 10 confirm that corrosion levels below 2% do not result in excessive surface cracking. These results indicate that wide surface cracks on reinforced concrete elements could be indicative of significant corrosion of reinforcing bars.

Flexural strength tests

The slabs were subjected to four-point loading after they had reached the desired degree of corrosion. The slabs that were not subjected to accelerated corrosion were tested 28 days after casting. The slabs were designed to fail in flexure and did not contain any reinforcement to prevent shear failure. All the slab specimens failed in flexure, with no sign of shear cracks. After a slab specimen had been subjected to four-point loading, the cracks (at failure load) on both sides of the specimen were measured using the crack measurement microscope. It was observed that the number and the size of the cracks, as well as the distance between the cracks that formed due to failure of the slab, varied with respect to the corrosion level of the specimen. Table 3 summarises the average crack widths and crack spacing, as well as the number of cracks and failure load of each of the slab specimens.

The cracks that formed in each member were a function of the level of corrosion of the specimen. The control specimen (0% corrosion) had six cracks, which is more than any of the corroded slabs. As the percentage mass loss increases, the bond between the concrete and the embedded steel becomes weaker. A good bond at the concrete–steel interface results in more cracks that are closely spaced and not as wide. As the slab specimens are subjected to accelerated corrosion, resulting in a decrease in the bond between the concrete and the steel bars, fewer cracks form and they are further apart and have larger crack widths. The average crack spacing and crack width increased from 109 mm and 0.8 mm respectively for the slab with 0% corrosion to 237 mm and 2.9 mm for the slab with a percentage mass loss of 14%. Figure 11 illustrates the relationship of the failure load crack widths and crack spacing with respect to the corrosion level. The failure load was taken as the maximum load reached during testing,

Table 3 Flexural crack measurements and failure load of slab specimens

% Corrosion	No of cracks	Avg crack width (mm)	Avg crack spacing (mm)	Failure load (kN)
0.0	6	0.8	109	46.8
2.4	4	1.0	86	38.1
2.6	4	1.5	114	39.3
3.4	4	1.7	117	41.6
3.6	4	1.3	120	37.6
5.7	4	1.3	130	38.6
5.9	3	1.7	137	38.0
6.0	3	1.5	157	40.9
8.0	2	2.7	193	40.2
9.0	3	2.0	237	33.8
14.0	2	2.9	210	39.3

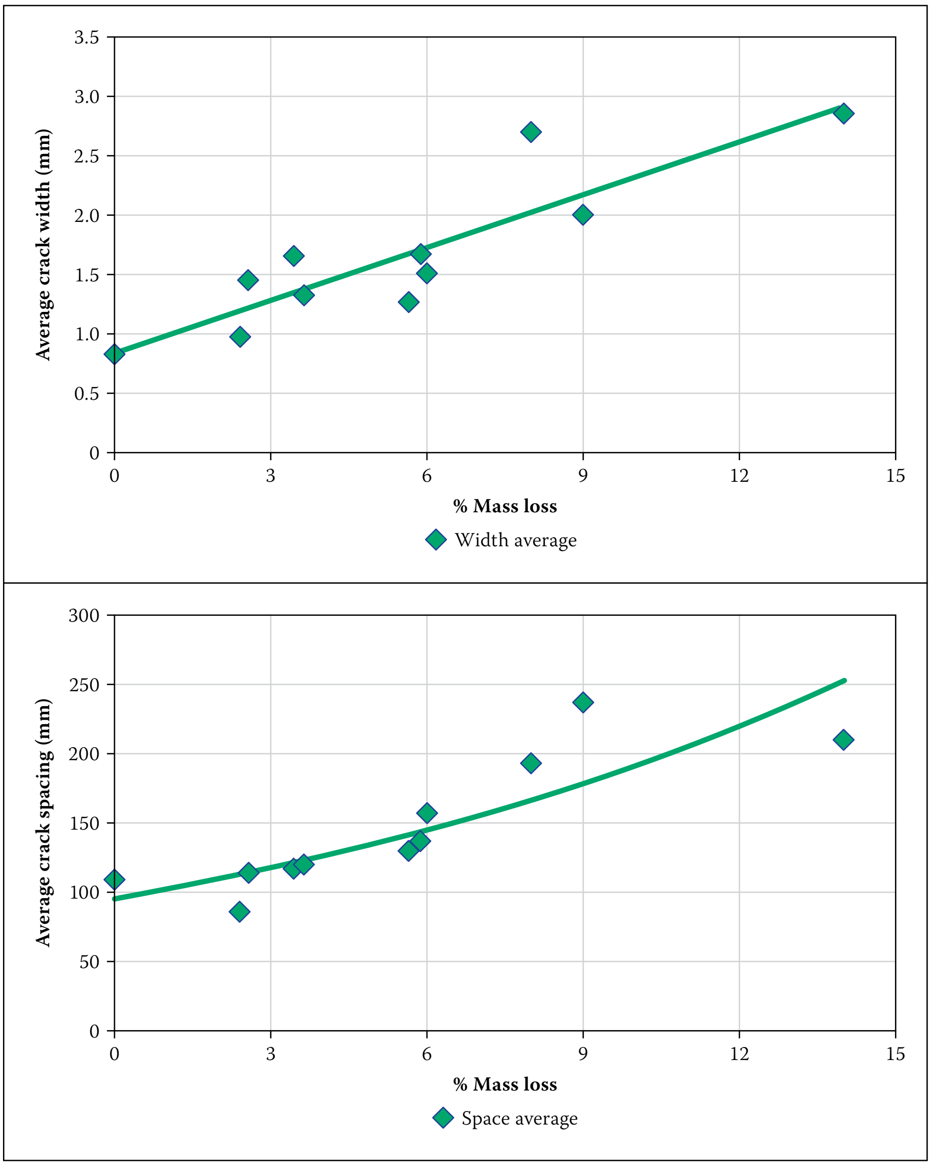

Figure 11 Average failure load crack width and spacing with respect to corrosion level

at which a plateau was observed on the load-deflection graph.

The slabs that were corroded to 6%, 8%, 9% and 14%, where all the bars were assumed to have reached a uniform corrosion level, were used to determine the effect of corrosion on the structural performance of the slabs. The data recorded during the four-point loading test includes the mid-point deflection, the applied load and the data obtained from the LVDTs used to monitor the curvature of the slab. The moment–curvature relationship provides important information with regard to the stiffness of the slab, the slip of the rebar and the cracks that form when the slab is subjected to load. The moment–curvature graph in Figure 12 can be used to illustrate the data obtained from the structural tests. Figure 13 is the moment–deflection graph of the early region, i.e. 0–5 kN.m, which depicts the change in stiffness with respect to corrosion percentage.

The control slab specimen initially contained a very stiff region until the concrete cracked (0-A), which is the expected behaviour of a structural member subjected to load. As soon as the concrete started to crack, the steel at the bottom of the slab began to strain elastically. The steel is now assisting the concrete in carrying the load, due to the weak tensile properties of the concrete. Although the response is less stiff than that of the uncracked section, the slab still contains significant stiffness. A point is reached when the steel begins to yield (B-C), which causes a considerable decrease in flexural stiffness. When designing reinforced concrete structures, provision is made for the steel to yield before the concrete fails in compression to prevent a brittle failure. This provides warning of impending failure and allows the slab to redistribute the load. Although there was a significant loss in flexural stiffness when the steel began to yield, the slab accepted more load due to the strain hardening of the reinforcement. Strain hardening is an increase in the strength of the steel due to plastic deformation (B-C). It was evident from the data obtained to determine the moment–curvature relationship that the neutral axis of the slabs moved upward as the test progressed. This results in a larger tension block below the neutral axis, which allows the slab to accept more load and retain some flexural strength as the load increases. The slab was unloaded when the hydraulic jack reached an extension of 25 mm.

The slab specimens subjected to accelerated corrosion also indicated a ductile response. The slabs that were corroded to 6% and 8% behaved in a similar manner as the control specimen. Prior to cracking of the concrete the slabs contained a stiff region (0-A) similar

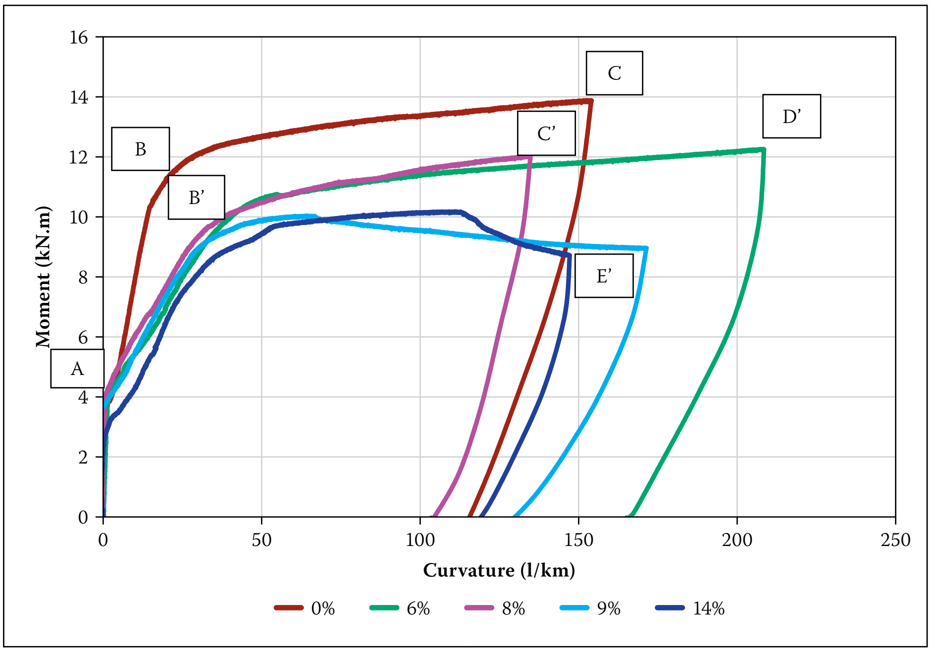

Figure 12 Moment-curvature for slabs with uniform percentage corrosion

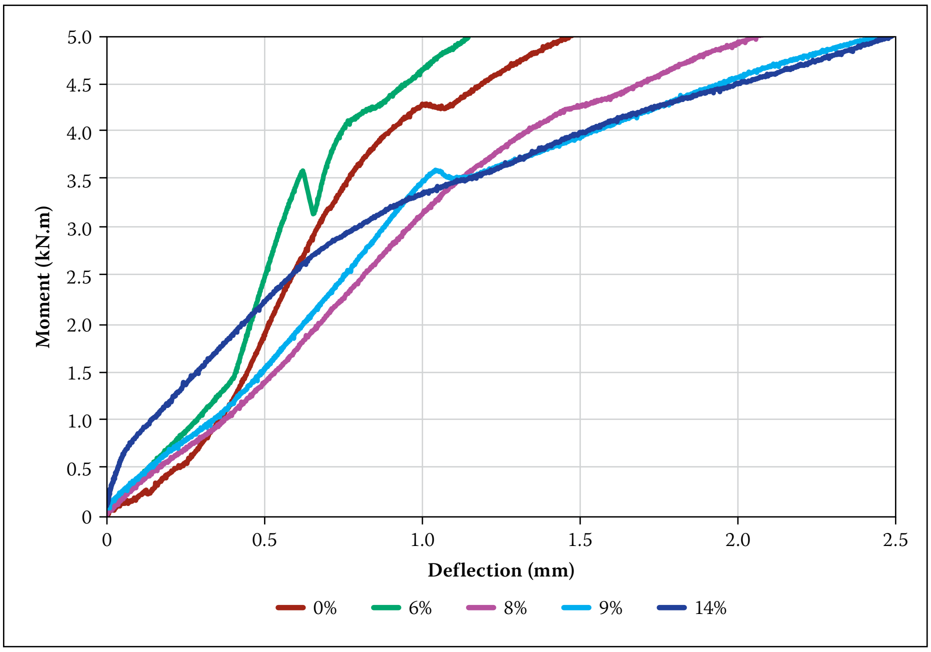

Figure 13 Moment-deflection graph of early region (0-5 kN.m)

to that of the control specimen, as a result of the concrete resisting most of the load until it starts to crack. After the concrete cracks, the structure relies on the stress transfer between the concrete and the reinforcement. The slabs have less flexural stiffness than the control specimen. This can be interpreted as the slip required for the bar to achieve equal stress transfer (Stanish *et al* 1999). When the slab enters the plastic deformation region, the steel begins to yield (B'-C' and B'-D'). From the load response diagram it is evident that the slab corroded to 6% showed similar behaviour to the slab at a corrosion level of 8%, but it had a larger load carrying capacity (B'-D'). This could be due to the corrosion products reducing the bond between the concrete and the reinforcement, thus resulting in more permanent deformation until failure. Additional loss in the load-carrying capacity of the slab corroded to 8% mass loss demonstrates the significant effect corrosion has on the load-carrying capacity of RC structures and confirms the decrease in bond strength as a result of corrosion.

The slabs that were tested at corrosion levels of 9% and 14% respectively had a different response from the other slabs. They reached a much lower moment capacity than that of the control specimen and the slabs corroded to 6% and 8%. These slabs also exhibited a stiff region prior to cracking of the concrete, but after the bars had begun to load elastically the flexural stiffness of the slab specimen decreased significantly. The yield point that was reached for these slabs was much lower than that of the other specimens. In the plastic region, definite slip of the steel

Table 4 Theoretical first cracking moment and ultimate moment of the slab specimens

% Mass loss	Estimated 1st cracking moment (kN.m)	Ultimate moment (kN.m)
0	4.3	13.9
6	3.6	12.2
8	–	12.0
9	3.6	10.0
14	–	10.2

bars is indicated by the sudden decrease in the moment, as seen in region B'-E'. This is where a significant decrease in the moment capacity of the slab occurs with considerable change in curvature, which can be interpreted as the reduction of the stress in the steel due to a decrease in the stress transfer between the steel and the concrete. The ultimate moment capacity that was reached was less than the moment before slip of the bars occurred. The experiment was conducted in deflection control, which means that the deflection of the beam was increased at a constant rate, while the force resisting the movement was recorded. If the experiment had been conducted in load control (where the load placed on the beam is increased at a constant rate), the slabs containing reinforcing bars with high levels of corrosion would have failed suddenly and with no warning of impending failure, as the moment capacity decreases with increased curvature. From the general trend observed in Figure 13 one can expect a sudden failure to occur at higher corrosion levels when a structure is loaded past the point where the steel starts yielding. Table 4 provides data on the estimated first cracking moments and ultimate moments for these members. Comparing the values in the table, it is evident that the first cracking moment, as well as the ultimate moment, decreases from 4.3 kN.m and 13.9 kN.m to 3.6 kN.m and 10.2 kN.m respectively as the percentage mass loss (corrosion level) increases. It can be said that corrosion has a more significant influence on the ultimate moment, as this is where the load-carrying capacity of the specimen relies most on the load transfer mechanism at the concrete–steel interface.

CONCLUSION

Corrosion affects the mechanical properties of reinforcement and the corrosion products lead to cracking and spalling of the surrounding concrete. Severe corrosion may lead to reductions in the load-carrying capacity of structural members.

For the cylindrical specimens subjected to pull-out tests an increase in the bond strength was observed for corrosion levels lower than 2%. The control specimens reached a maximum value of 3.2 MPa, whereas the specimens with an estimated percentage mass loss of 1.9% reached a maximum load of 4 MPa. This 20% increase in bond stress is due to the well adhered layer of rust at low levels of corrosion and can also be attributed to the pressure build-up caused by the corrosion products during the initial stages of corrosion.

The experimental results showed a similar trend to the empirical formulae proposed by Stanish *et al* (1999), Cabrera (1996) and Lee *et al* (2002) to calculate bond strength as a function of corrosion level. However, these formulae only projected the decrease in bond strength for the specimens corroded to a percentage mass loss greater than 2%. These empirical formulae do not take into account the increase in the bond strength at low levels of corrosion.

Slab specimens with low corrosion levels had numerous cracks, with small crack widths up to 1.4 mm, distributed along the length of the slab. Fewer flexural cracks were observed for the specimens subjected to severe corrosion, which is typical of loss of bond. The average crack spacing increased from 109 mm for the slab with 0% corrosion to 237 mm for the slab with a percentage mass loss of 14%. An increase in percentage mass loss causes a decrease in the number of cracks and an increase in crack spacing and crack width.

As for the un-corroded slab specimens, the specimens subjected to accelerated corrosion also had a ductile response to the applied load. The slabs that were corroded to 6% and 8% behaved in a similar manner than the control specimen. Prior to cracking the moment–curvature graph indicated a stiff region where the concrete resisted most of the load. The slab specimens corroded to 9% and 14% had a significant decrease in the moment capacity of the slabs, due to a reduction in the stress transfer between the concrete and the reinforcement. After the ultimate moment capacity of the slabs had been reached, a decrease in the moment resistance was observed. This can be interpreted as slip of the bar, and illustrates the loss in bond strength as a result of corrosion.

Experimental results indicate that at high corrosion levels (more than 8% loss in steel cross-sectional area) the flexural behaviour of the slabs are affected to such an extent that failure occurs suddenly and can be described as a more "brittle" failure.

The severity of corrosion deterioration in RC structures needs to be addressed and methods to rehabilitate and preserve these structures need to be set in place. Further research regarding the effect of corrosion on the structural performance of RC structures is necessary to extend the service life of reinforced concrete structures in future.

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