Finite element investigation of the structural response of corroded RC beams

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A B S T R A C T
Rebar corrosion is the most commonly observed deterioration mechanism in reinforced concrete (RC) structures, severely affecting their performance and potentially resulting in premature in-service failures. In this paper, the structural performance of a series of RC beams—damaged by corrosion at different locations—is studied numerically using two-dimensional (2D) non-linear FE analysis (NLFEA). The FE models are developed for the corrosion damaged RC members and validated using experimental data. The results of a parametric study are presented, which examined the influence of concrete compressive strength ($f_c$), bond deterioration and corrosion damage in the compressive region of the beams on the predicted response. The importance of system based models, incorporating key aspects of corrosion related damage, is highlighted and discussed in relation to both strength and serviceability requirements.

1. Introduction

Rebar corrosion is identified as the major deterioration cause of RC structures, commonly affecting bridges, car parks and various marine/coastal structures. Corrosion of steel in concrete causes internal damage to RC elements, owing to the loss of steel area and the formation of associated expansive corrosion products. The severity of this damage, however, depends on the nature (i.e. uniform or/and pitting) and extent of corrosion and the location of its occurrence, i.e. tension, compression and/or shear rebars in RC beams. Accurate performance evaluation of corrosion prone structures could allow extension of service life, where appropriate, and may contribute to a more consistent safety level across a network of structures. This would improve the efficient use of scarce resources, and minimize the impact of indirect costs through optimised inspection, maintenance and repair works [2]. As a result, several experimental, analytical and numerical studies have been devoted to this topic, aiming to clarify the influence of different forms of corrosion damage on the structural response of RC elements such as beams and slabs, e.g. [3–14].

Physical effects of corrosion, which may ultimately lead to inadequate performance, include loss of steel area, loss of bond between steel rebars and concrete, reduced concrete strength due to cracking, loss of concrete section due to spalling and reduced mechanical properties of the affected rebars, i.e. strength and ductility, mostly due to pitting formation [2,13].

A number of studies have been devoted on corrosion-induced bond deterioration, and its effect on the strength of RC beams and slabs, (e.g. [10,11,15]). Under-reinforced beams with extensive loss of bond at the tensile rebars, are likely to exhibit small reductions of their ultimate load capacity compared to elements without impaired bond, provided that the bars are well anchored at their ends [4]. Other studies [16–20], have highlighted the importance of confinement (usually provided as transverse steel stirrups) on the residual bond performance of corroded rebars. The effects of bond deterioration on the performance of beams under serviceability conditions, however, are not well established. This aspect has become increasingly relevant in recent years due to the gradual shift and tightening of infrastructure management objectives, i.e. in maintaining a predefined level of service over the network rather than focusing simply on structural integrity.

A limited number of studies have investigated the effect of corrosion damage in the compressive region of RC beams [9,14]. Du et al. [9], investigated experimentally the effect of corrosion in the compression region of beams, though results are presented only for over-/balanced-reinforced sections. On the other hand, corrosion damage on all rebar types (tension, compression and shear links) in the same beams is examined in the experimental study of Rodriguez et al. [5]. Consequently, the impact of corrosion in the compressive region cannot be readily isolated. A methodology to include corrosion related concrete damage in compression was proposed in [11]; several types of corrosion were assumed in beams which were then analysed in order to quantify performance degradation, though degradation in the compression region alone was not explicitly investigated.

In the present study, non-linear finite element analysis (NLFEA) is used to investigate the potential effects of corrosion damage on...
2. Finite element modelling of beams

The behaviour of corroded under-reinforced beams, from two experimental studies, is simulated herein [5,9]. In each of these studies, beams with different steel ratios were tested. For example...
Table 1
Geometrical details and corrosion damage of the examined beams (Maximum pit depths, \( P_{\text{max}} \), are given in brackets).

<table>
<thead>
<tr>
<th>Beam designation</th>
<th>Steel ratio (%)</th>
<th>( L ) (mm)</th>
<th>Loads spacing ( X_1/X_2 ) (mm)</th>
<th>Tension</th>
<th>Compression</th>
<th>Stirrups</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ref. [5]</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ref. [9]</td>
<td>T280</td>
<td>0.87</td>
<td>1800</td>
<td>111.10(2.06)</td>
<td>0.0</td>
<td>33.00(0.735)</td>
</tr>
</tbody>
</table>

\( a \) Local maximum.

Table 2
Material and bond properties for 11-type beams.

<table>
<thead>
<tr>
<th>FE model designation</th>
<th>Concrete strength, ( f_c ) (MPa)</th>
<th>Residual yield strength of rebars (MPa)</th>
<th>Bond</th>
<th>Damaged concrete of top cover</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tension</td>
<td>Links</td>
<td>( R ) (Eq. (3a))</td>
<td>Bond strength, ( u_{\text{max}}^0 ) (MPa)</td>
</tr>
<tr>
<td>111a (control)</td>
<td>50</td>
<td>575</td>
<td>626</td>
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<tr>
<td>114a</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>115a</td>
<td>34</td>
<td>569</td>
<td>496</td>
<td>0.691</td>
</tr>
<tr>
<td>116a</td>
<td></td>
<td>551</td>
<td>430</td>
<td>0.479</td>
</tr>
<tr>
<td>111FC34</td>
<td>34</td>
<td>575</td>
<td>626</td>
<td>–</td>
</tr>
<tr>
<td>115FC25</td>
<td>25</td>
<td>569</td>
<td>503</td>
<td>0.775</td>
</tr>
<tr>
<td>115FC50</td>
<td>50</td>
<td>551</td>
<td>430</td>
<td>0.479</td>
</tr>
<tr>
<td>116FC25</td>
<td>25</td>
<td>551</td>
<td>430</td>
<td>0.479</td>
</tr>
<tr>
<td>116FC50</td>
<td>50</td>
<td>551</td>
<td>430</td>
<td>0.479</td>
</tr>
<tr>
<td>115NB</td>
<td>34</td>
<td>569</td>
<td>503</td>
<td>0.775</td>
</tr>
<tr>
<td>116NB</td>
<td></td>
<td>551</td>
<td>430</td>
<td>0.479</td>
</tr>
<tr>
<td>114LC</td>
<td>568</td>
<td>496</td>
<td>0.691</td>
<td>5.81</td>
</tr>
<tr>
<td>115LC</td>
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<td>551</td>
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<td>0.479</td>
<td>4.03</td>
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<tr>
<td>114ND</td>
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<tr>
<td>116ND</td>
<td>551</td>
<td>430</td>
<td>0.479</td>
<td>4.03</td>
</tr>
</tbody>
</table>

Table 3
Material and bond properties for T-type beams.

<table>
<thead>
<tr>
<th>FE model designation</th>
<th>Concrete strength, ( f_c ) (MPa)</th>
<th>Residual yield strength of rebars (MPa)</th>
<th>Bond</th>
<th>Damaged concrete of top cover</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tension</td>
<td>Links</td>
<td>( R ) (Eq. (3b))</td>
<td>( u_{\text{max}}^0 ) (MPa)</td>
</tr>
<tr>
<td>T280 (control)</td>
<td>35.8</td>
<td>489</td>
<td>526</td>
<td>–</td>
</tr>
<tr>
<td>T282a</td>
<td>44.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T282FC25</td>
<td>25</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T282FC35</td>
<td>35</td>
<td>479(489)</td>
<td>520</td>
<td>0.94</td>
</tr>
<tr>
<td>T282NB</td>
<td>44.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T282LB</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T282T5</td>
<td>44.5</td>
<td>479(489)</td>
<td>520</td>
<td>0.94</td>
</tr>
<tr>
<td>T282T10</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T282T15</td>
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<tr>
<td>T282T20</td>
<td></td>
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</tr>
<tr>
<td>T282T25</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>T282TC30</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

are obtained from MC90 [23] based on concrete's compressive strength \( f_c \) and maximum aggregate size. A small element size is used for concrete (approximately 10 x 10 mm). This allows distinct cracks to be obtained by visualization of the softened (cracked) concrete elements, without the need to introduce discontinuities to the FE models (i.e. discrete crack approach).

The behaviour of concrete in compression is modelled using a parabolic curve [21], as shown in Fig. 2(a). Linear-elastic response is considered for stress levels up to 30% of the compressive strength, \( f_c \). The post-peak response of concrete in compression is modelled using compressive fracture energy, \( G_c \), based on the recommendations of Nakamura and Higai [24]. In beams affected by corrosion in their compressive region, reduced values of compressive strength and \( G_c \) are used for the elements of the top cover to consider the loss of strength and ductility of the cracked concrete (see Section 3.3).

3. Corrosion damage models

3.1. Steel rebars

Uniform corrosion has an insignificant effect on the stress–strain properties of the affected rebars and it is modelled simply
by reducing the cross-sectional area [25]. The presence of pitting corrosion, however, may cause significant degradation of the mechanical properties due to localized stress concentrations [13, 26]. In the present study a linear reduction in yield strength of the corroded rebars at increasing levels of pitting corrosion is considered, using Eq. (2) [1]:

$$f_y^D = [1.0 - \alpha_f(A_{\text{pit}}/A_{\text{strom}})] 100 f_y$$  \hspace{1cm} (2)

where, $f_y^D$ and $f_y$ are the reduced and initial yield strengths respectively, $A_{\text{strom}}$ is the area of the uncorroded rebar, $D_0$ is the initial rebar diameter, $\alpha_f = 0.005$ [26] and $A_{\text{pit}}$ is the area of the pit, which is a function of pit depth ($P_{\text{max}}$), pit width and initial diameter $D_0$ [1]. In the absence of pit depths data, these are obtained from uniform corrosion data and a pitting factor $R_{\text{pit}} = P_{\text{max}}/P_{\text{av}} = 4–8$, taken as 6 for this study [27].

3.2. Bond deterioration model

3.2.1. Bond strength of corroded rebars

Small amounts of corrosion, prior to the development of visible concrete cracking, tend to cause an increase of bond strength between the rebars and concrete [28]. Bond strength begins to deteriorate with the formation of corrosion cracking in concrete, typically along the rebar’s length. Despite the widely scattered bond test results of corrosion affected rebars [20], the presence and extent of confinement is identified as a critical factor which determines the residual bond strength (and overall bond performance) of corroded rebars [17, 18, 29, 30]. In the present study, bond strength is calculated using Eq. (3a) [8], which considers separately the contribution of concrete and steel stirrup confinement on bond strength.

$$u_{\text{max}}^D = R[0.55 + 0.24 (c/d_b)] \sqrt{f_p} + 0.191 (A_{\text{st}}f_{yt}/s_b)$$

$$R = A_1 + A_2 X$$  \hspace{1cm} (3a)

$$u_{\text{max}}^D = R[0.55 + 0.24 (c/d_b)] \sqrt{f_p} + 0.191 (A_{\text{st}}f_{yt}/s_b)$$

$$R = A_1 + A_2 X$$  \hspace{1cm} (3b)

where, $u_{\text{max}}^D$ is the reduced bond strength, $c$ is the thickness of concrete cover, $d_b$ is diameter of the anchored rebar, $f_p$ is concrete compressive strength, $A_{\text{st}}$ is area of shear rebars, $f_{yt}$ is the stirrup yield strength, $s_b$ is stirrup spacing, $R$ is a factor to account for the reduced contribution of concrete towards bond strength with factors $A_1$ & $A_2$ being a function of the rate of corrosion used in the accelerated corrosion process: For a corrosion current of $i = 0.09 \text{ mA/cm}^2$, $A_1 = 1.104$ and $A_2 = -0.024$ while for $i = 0.35 \text{ mA/cm}^2$, $A_1 = 1.079$ and $A_2 = -0.0123$ [8]. $X$ is the amount of corrosion expressed as the percentage of mass loss of the steel rebar. The reduction in confinement, and hence bond strength, due to stirrup corrosion is considered by reducing the stirrup area and yield strength as described in Section 3.1. Advantages of this bond strength model, over other empirical models, e.g. [18], include its ability to predict the experimentally observed [28] increase of bond strength for low amounts of corrosion, and its explicit dependence on the rate of the corrosion process.

3.2.2. Local bond stress–slip law

Experimental and numerical studies suggest that the relation between bond stress and slip is controlled by both the amount of corrosion attack in the longitudinal rebar and the amount of the confinement. A slightly modified version of the local bond stress–slip law developed by Harajli et al. [31], for beams confined with steel stirrups, is adopted here; see Fig. 2(d). The initial part of the curve, for values of bond stress up to 70% of bond strength, is given by Eq. (4), which is similar to the relation proposed in CEB [23], with a slightly different coefficient. Beyond this point, bond stress continues to increase with a reduced stiffness, until the peak value corresponding to the bond strength is reached. A linear post-peak reduction of bond stress is assumed up to slip $s_2 = 0.35c_0$, where $c_0 =$ rib spacing = 8 mm (assumed). The slips at which bond stiffness changes and bond strength is mobilised are given from Eqs. (5) and (6), respectively.

$$u = u_1 (s/s_1)^{0.3}$$  \hspace{1cm} (4)

$$s_u = s_1 (u_{\text{max}}/u_1)^{1/0.3}$$  \hspace{1cm} (5)

$$s_{\text{max}} = s_1 e^{(1/0.3)\ln(u_{\text{max}})} + s_0 \ln(u_1/u_{\text{max}})$$  \hspace{1cm} (6)

where, $\alpha = 0.7$, $u_1 = 2.57 f_p^{0.5}$, $f_p$ is the compressive strength of concrete, $s_1 = 0.15c_0$ and $s_0 = 0.15$ and 0.4 mm for plain and steel confined concrete, respectively. The majority of studies on bond performance of corroding rebars are dealing with the prediction of residual bond strength rather than the bond stress–slip relationship characteristics. As a result, the above model represents a good approximation of the bond stress–slip relationship for corroded members given the insufficient experimental data. More details regarding the selected bond stress–slip relation, which is in rational agreement with trends from recent experimental studies (e.g. [29, 30]), can be found in [32]. Further research is required to

Fig. 2. Constitutive models for (a) concrete in compression and (b) tension, (c) steel rebars, (d) Bond stress–slip law, (e) typical mesh discretization of half beam and support conditions and (f) assignment of zero thickness to interface elements.
develop bond stress–slip relationships for corroded rebars, considering the effect of different influencing parameters, for instance the amount of active/passive confinement, specimen geometry and loading history [20,33].

3.3. Residual strength of corrosion damaged concrete

Cracked concrete, due to corrosion, subjected to compressive stresses exhibits reduced performance compared to undamaged concrete [14]. A reduced value of compressive strength is calculated for the beams with their compression rebars affected by corrosion, as suggested by Coronelli and Gambarova [11], using Eq. (7):

\[ f'_c = f_c / [1 + k (\varepsilon_{ps}/ \varepsilon_{co})] \]

where, \( f'_c \) is the compressive strength of the damaged concrete, \( f_c \) is the compressive strength of the undamaged concrete, \( k = 0.1 \) [11], \( \varepsilon_{ps} \) is the concrete strain at peak load and \( \varepsilon_{co} \) is the smeared transverse strain due to corrosion cracking, which is a function of the number of corroded longitudinal (top) rebars in the section, the volumetric expansion of the rust product and the average corrosion attack penetration, for details see [11]. The reduced value of compressive strength is assigned to the concrete elements of the top cover assuming that no cover spalling has occurred.

4. Analysis cases and parameter variation

The FE models are verified using experimental data [5,9], i.e. models 111a–116a and T280–T282a. Subsequently, they are used in a parametric study to examine the influence of various parameters on the predicted response, such as concrete’s compressive strength, bond performance and corrosion damage in the compressive region of the beams. Tables 2 and 3 summarise the residual material and bond properties used for all the analyses of 11- and T-type models. Since the tensile rebars of beam T282 are only partially corroded in their tension region, two values for bond strength are given in Table 3, with the values in brackets used for the non-corroded regions of the beam. A reduced compressive strength, \( f'_c \), considered along the entire top cover for models T282T5–T282T25 (Table 3) is calculated based on the amount of corrosion in the top rebars. Finally, both the experimentally tested specimens [5,9] and the numerical models assume that all rebars within the same rebar group (e.g. all tension rebars) in a beam, are affected by the same amounts of corrosion loss.

5. Results and discussion

5.1. Model validation and performance of corroded beams

The predicted (numerical) and experimental load–deflection curves for 11-type and T-type beams can be seen in Figs. 3 and 4, respectively. A good correlation is observed between the experimental [5,9] and the numerical results, on the basis that the serviceability limit, as well as yield and ultimate loads (\( P_y \) and \( P_u \), respectively) were predicted with reasonable accuracy (see Table 4). The yield and ultimate loads correspond to the load at yielding of the tension rebars and the highest predicted load value, respectively. The yield and ultimate loads are similar (i.e. difference between predicted yield and ultimate loads < 2%, in Table 4) due to the elastic–perfectly plastic model used for the rebars. For the same reason the results indicate that models 111a and 115a underestimated slightly the ultimate capacities of the beams. This difference is observed to be smaller for the relatively high corroded beams 114a and 116a. This behaviour can be explained by considering the performance of highly pitted tension rebars of beams 114 and 116, which in turn resulted in localised stress concentrations and, eventually, local yielding and rupture in the corroded rebars [5]. Pitting is also responsible for the experimentally observed loss of beam ductility, defined as the ratio of maximum deflection over the deflection at yielding. The loss of ductility due to pitting corrosion (not quantified in this paper) is currently an area of ongoing research. The difference in initial stiffness of the experimental and numerical load–deflection curves of the examined beams at low load levels can be attributed to the thermal and shrinkage cracking in the experimental beams, and the experimental set-up where some loads are applied initially to test the instrumentation. This may result in some cracking leading to a reduced stiffness not apparent in the FE models (i.e. numerical models are uncracked at the beginning of the analysis).

The results for 11-type corroded beams indicate that their structural performance significantly deteriorates for increasing amounts of corrosion (beams 115, 114 and 116 respectively) in comparison to the control beam 111. Notable reduction of load corresponding to the serviceability limit (taken as L/500 [34]), yield and ultimate load capacities is apparent in Fig. 3 and Table 4. For example, the load capacity at both SLS (i.e. loss of bending stiffness) and ULS of model 116 (with 26.4% steel loss of the rebars in tension) reduced by approximately 40%. The performance of 11-type corroded beams, however, is a function of complex interaction between different corrosion damage types at various locations.
The numerical results for T-type beams indicate that 11% corrosion-induced steel loss (model T282a), within a central portion of the tensile rebars, caused an approximately 12% reduction of its ULS load capacity. However, no noticeable bending stiffness deterioration is observed in this case (i.e. almost the same load is calculated at deflections which correspond to the serviceability limit deflection for both control beam T280 and T282a). The predicted response is in agreement with the experimental data of Du et al. [9], where a similar behaviour is observed. The effect of bond deterioration on the response of corroded beams is discussed later in this paper.

The numerical models predicted flexural failures, for all 11-type and T-type beams analysed in this study, (i.e. no shear/anchorage failure is predicted). The control beams 111a and T280, as well as the corroded beam T282a (corrosion in tension rebars only), failed by yielding of their tensile rebars.

The numerical models for severely corroded beams 114a and 116a (with 24.3% and 22.5% loss of top rebars, respectively) predicted that the flexural failure is initiated by gradual crushing of the weakened top concrete cover. The conventional wisdom dictates that such behaviour leads to a brittle failure and the tensile rebars are unable to reach their yield strength. The compressive rebars may undergo buckling in between the transverse restraints provided by two adjacent shear links. This has not been incorporated in the numerical models because under-reinforced beams are used in this study and the contribution of compressive rebars on the overall behaviour of these beams is expected to be insignificant. This is also supported by the experimental study of [5] where no buckling of compressive rebars is reported in corroded beams 114 and 116 and the crushing of concrete in the top cover precedes the yielding of tensile rebars. The corresponding numerical models also predicted a ductile failure mode, i.e. the beam continued to resist the applied loads well beyond the yielding of tensile rebars until the crushing of undamaged core concrete, for which a higher concrete strength is used. This is due to the fact that the compressive stresses redistributed to, and resisted by, the core concrete below the level of the top rebars, as demonstrated in Fig. 14.

Concrete crushing of the top cover of the less severely corroded beam 115 (with 12.6% loss of top bars), is initiated almost at the same time as yielding of its tensile rebars, with most crushing, however, occurring at later stages.

The predicted behaviour for all beams analysed in this study is in agreement with previous experimental and numerical studies [5,9,11].

5.2. Effect of $f_c$ on the flexural performance of corroded beams

Since compressive strength is a basic variable in the models used to calculate the residual concrete and bond properties, its influence on the predicted response is analysed for both beam groups (i.e. 11- and T-type models). Rodriguez et al. [5], used a higher compressive strength for concrete for the control beam 111a ($f_c = 50$ MPa) than the corroded beams 114–116 ($f_c = 34$ MPa). The control model 111a is re-analysed using $f_c = 34$ MPa (model 111FC34); hence revising all other input parameters, i.e. $f_t$, $G_C$ and bond strength, accordingly. Similarly, the performance of models 115a–116a and model T282a (partially corroded), is studied for different compressive strengths (models 11-type FC and T282FC in Tables 2 and 3, respectively). The load–deflection curves for these models are presented in Figs. 5–9.

The yield and ultimate load capacities for the control models (111a and 111FC34) are relatively unaffected by the adopted range of $f_c$ values (see Fig. 5). The reduction of bending stiffness in model 111FC34 is relatively small. A slight decrease in the load capacity (difference < 5%) at the SLS can be seen for 111FC34 model. It is for this reason that the same flexural cracks form at a relatively lower load in model 111FC34 compared to model 111a (see Fig. 5), causing an earlier shift of the neutral axis away from the tensile face of the beam. Hence, the yielding load in model 111FC34 is reached at a slightly higher deflection, compared to model 111a.

Similar behaviour can be observed for the members corroded only in the middle part of their tensile region, i.e. T282-type models in Fig. 6. No concrete crushing is observed prior to the yielding of tensile steel in any of these models. The change in overall bending stiffness of the models is small, following a similar trend with 11-type control model. Although the compressive strength of concrete

\[ \text{Table 4} \]

Comparison of experimental and numerical results.

<table>
<thead>
<tr>
<th>Model</th>
<th>$P^1_f$ (kN)</th>
<th>Yield load, $P_0$ (kN)</th>
<th>Ultimate load, $P_u$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FE Exp. Dif</td>
<td>FE Exp. Dif</td>
<td>FE Exp. Dif</td>
<td></td>
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<tr>
<td>111</td>
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<td>38.46</td>
</tr>
<tr>
<td>115</td>
<td>20.03</td>
<td>28.28</td>
<td>28.92</td>
</tr>
<tr>
<td>114</td>
<td>17.21</td>
<td>26.63</td>
<td>27.48</td>
</tr>
<tr>
<td>116</td>
<td>14.75</td>
<td>23.54</td>
<td>23.76</td>
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<tr>
<td>T280</td>
<td>30.98</td>
<td>49.26</td>
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<tr>
<td>T282</td>
<td>30.52</td>
<td>44.14</td>
<td>44.16</td>
</tr>
</tbody>
</table>

\[ {}^1 \text{Load at the serviceability limit deflection taken as L/500} \]
The load–deflection responses for the 11-type corroded beams (with all rebars corroded along the full member length) are sensitive to $f_c$, as shown in Figs. 7 and 8. A few key observations from these figures are:

- The bending stiffness of the member reduces considerably with a reduction in compressive strength of concrete, resulting in reduced load capacities at SLS, e.g. 115FC25 predicted 17% and 22% lower loads than the corresponding loads for 115a and 115FC50, respectively.
- The bending stiffness changes abruptly at approx. 17 and 22 kN for model 115FC25 and 115a respectively (Fig. 7). Similarly the stiffness changes rapidly at 9 kN, 13 kN and 19 kN for models 116FC25, 116a and 116FC50 respectively (Fig. 8). These align with the initiation of crushing at the corrosion damaged top cover in the models.
- Yield load is higher for model 115FC50. Concrete crushing in this model initiates after yielding of the tension steel, at about 14 mm deflection, at which point a reduction of the load capacity can be seen in Fig. 7.
- For relatively high corroded bars, i.e. 116-type models with approx. 23% section loss in the top bars, the crushing of concrete in the top cover initiates prior to yielding. This in effect means a reduction of the beam’s effective depth for an equivalent sectional analysis. Hence, the yield and ULS load capacities are same for the three models in Fig. 8.
- The ULS load capacity remains unaffected in all models, for a given corrosion related damage.
- Prior to crushing initiation in corrosion damaged top cover, the slope of load–deflection curve for model 115a follows the bending stiffness profile of its corresponding 115FC50 numerical model (see Fig. 7). In contrary, the highly corroded model 116a, where crushing in the top cover initiates prior to the serviceability limit deflection, follows the trend of its respective FC25-type models (i.e. 116FC25 in Fig. 8) up to the service limit deflection.

Two possible factors contributing to the described behaviour are; the reduced bond performance and the reduced residual compressive strength of the corrosion damaged top cover of these models. These factors are individually examined in following sections. The above results indicate that the influence of concrete compressive strength, $f_c$, is more predominant on the performance of beams affected by corrosion in their compressive zone, causing increased deflections at the serviceability and reductions of yield/ultimate capacity as shown in Figs. 7–9. The value of $f_c$ together with the amounts of corrosion in the top bars determine effects on the residual bond properties of the corroded rebars, the overall performance of the beams with moderate corrosion in a central part of their tensile region is relatively insensitive to the value of $f_c$ adopted in the analysis.

Fig. 7. Load–deflection curves for 115a ($f_c = 34$ MPa), 115FC50 ($f_c = 50$ MPa) and 115FC25 ($f_c = 25$ MPa).

Fig. 8. Load–deflection curves for 116a ($f_c = 34$ MPa), 116FC25 ($f_c = 25$ MPa) and 116FC50 ($f_c = 50$ MPa).

Fig. 9. Effect of $f_c$ on the performance of 11-type corroded beams: (a) at serviceability deflection (L/500) and (b) at yielding.
the time of concrete crushing in the top cover and defines failure mode of the beams, i.e. ductile (yielding of the tensile rebars) or brittle failure (concrete crushing of the corrosion damaged top cover). In contrary, the load–deflection response is relatively unaffected by concrete compressive strength for beams where corrosion damage is concentrated only in their tension side (small changes of bending stiffness are observed due to reduction of bond between the tension rebars and concrete).

5.3. Effect of bond deterioration

The effect of bond deterioration is examined for two different corrosion scenarios, namely; (a) partial bond deterioration, over a central part of the tensile rebars (i.e. models T282a, T282NB and T282LB) and (b) impaired bond along the entire length of the tensile rebars (i.e. models 115a, 115NB, 116a and 116NB). The notation NB indicates no bond deterioration in these models (i.e. bond is assumed to be undamaged, while all other effects of corrosion are considered; see Tables 2 and 3). An additional analysis case of corroded beam T282 (i.e. model T282LB) has been considered, to investigate the effect of complete loss of bond – in the central portion of beam – on structural performance. Corrosion activity (i.e. anodic areas in a macrocell) is likely to be at its highest near flexural cracks (due to loading), where the access of chlorides, water and oxygen to the rebars, is easier (e.g. [35]). Hence, the aforementioned corrosion damage scenarios of corroded T-type beams can be considered as a close representation of simply supported beams exposed to the environment, since flexural cracking is expected to occur at mid-span locations.

The results for models T282a, T282NB and T282LB are shown in Figs. 10 and 11, while Fig. 12 shows the numerical results for 115 and 116 models.

Bond deterioration did not affect the yield/ultimate load capacity in any of the examined beams (Figs. 10 and 12). Consequently, the reduction of yield and ultimate loads in Fig. 4 (about 12% in both cases) for beam T282a in relation to the control beam T280, are attributed entirely to the loss of steel area. In 11-type corroded beams, the reduction of yield/ultimate loads is due to the loss of steel area and the corrosion damaged concrete in compression, which is discussed in detail in the next section. This observation, which is in agreement with previous studies (e.g. [4,36]), allow to safely conclude that bond deterioration may be omitted when performing ultimate load capacity calculations, provided that the ends of the rebars are well anchored.

Fig. 10 (T282a and T282NB), and Fig. 12 indicate that the bending stiffness, of T-type and 11-type beams, respectively, is relatively unaffected by the impaired bond performance due to corrosion. In contrary, model T282LB, (loss of bond in a central region), exhibits an 11% reduced load at serviceability limit deflection as shown in Fig. 10. In previous experimental and numerical studies, where a notable deterioration of bending stiffness is reported, very low residual bond strength (or complete loss of bond) have been considered over substantial lengths of the tension rebars [11,36]. This implies that the length of tension rebars associated with the bond deterioration is equally important (together with the residual bond strength) to establish the performance of corroded members at SLS.

Significant alterations are observed in the predicted cracking patterns of the corroded beams at the serviceability limit deflection. As expected, fewer but wider flexural cracks formed in the corroded beams, in comparison to their respective uncorroded (control) beams. Fig. 11(a)–(d) show a progressive reduction of cracks (accompanied by increase in their widths) for T-type beams.
models, for increasing levels of bond damage. For example, only one wide crack formed within the corroded region of model T282LB (i.e. crack no. 1 in Fig. 11(d) is more than twice as wide as the widest crack predicted for T282a). Furthermore, at increasing loss of bond, the predicted cracking sequence is altered, as shown in Fig. 11(d), where new cracks form outside the central region of the T282LB (in comparison to models T280 and T282a). Based on these results it can be concluded that although the load–deflection curves are relatively insensitive to the impaired bond, the crack widths and spacing are significantly altered; thus bond deterioration may result in reduced serviceability loads in relation to both deflection and crack width limitations.

5.4. Effect of compressive rebar corrosion on the flexural behaviour of beams

The compressive strength of concrete is a key factor in defining the timing of compressive concrete crushing for beams damaged by corrosion in their compressive zone. As shown in Section 5.2 (and Figs. 7 and 8), this affects the flexural performance of the beams; hence this aspect is investigated in detail here.

For the tension rebars corroded in the central part of the beams, T282a, a varying degree of corrosion damage in the compressive rebars is considered (i.e. $f_D^{cw}$ is calculated assuming a range of 5%–30% area loss in compressive rebars; see Table 3, models T282T5–T282LC30). A gradual loss of yield/ultimate capacity and bending stiffness is observed for increasing corrosion loss, as shown in Fig. 13. There is a clear trend of a shift in the failure mode, i.e. from ductile to brittle failure, with the increasing in corrosion related damage.

It has been established earlier that the compressive cover concrete crushing prior to the yielding of tensile steel causes a reduction in the yield capacity. This jump in the ULS capacity is apparent at 10% corrosion loss in compressive rebars in Fig. 13(b). Further damage in the compressive concrete cover does not significantly alter the ULS capacity of the member.

Fig. 13(b) also highlights that the SLS capacity of the member remains unaffected until the crushing of concrete occurs prior to this limit, which is at about 15% corrosion in the compressive rebars. A parabolic reduction in the SLS capacity beyond this corrosion damage is apparent from Fig. 13(b).

The results of an extreme case, T282LC30 assuming a complete cover delamination due to 30% corrosion in the compressive rebars and stirrups (see Table 3) is also presented in Fig. 13(a). The significant difference in the bending stiffness between this and the other models leads to the conclusion that there is a sufficient capacity in the section after the initiation of compressive concrete crushing. This is due to the fact that the concrete crushing in the top cover is localised in small zones, (which is an acceptable behaviour from a physical point of view [24]), and the compressive loads are redistributed to the surrounding (undamaged) concrete. This behaviour can be seen in Fig. 14 (24.3% loss of top rebars), where the crushing at a section leads to higher stresses in the adjacent sections, leading to an increase in the crushing zones when the applied loads are progressively increased.

Two extreme cases are analysed for the 11-type beams. For the first case the concrete cover is completely removed, representing cover delamination (see Table 2 for 114LC–116LC). In the second case, the concrete top cover is assumed to be undamaged. In both cases, all other corrosion effects are considered in the analysis similarly to 11-type-a models; see Table 2. Note that the width of the section in 116a and 116LC models is reduced assuming complete loss of the vertical covers due to high stirrup corrosion (stirrups are damaged by approximately 40% uniform corrosion loss [5]). Figs. 15 and 16 clearly demonstrate that the performance of the experimental beams, (and their corresponding 11-type-a models), are placed between the two extreme cases.
The higheryield/ultimate loads in all 11-type-ND models are due tothe delayed crushing in the compressive concrete cover. Similartothe T-typeT models, the timing of the initiation of crushing governsthe model behaviour; the bending stiffness of 11-type-a models is very similar tothe 11-type-ND models up tothe crushing point. Beyond this point, the bending stiffness ofthe 11-type-a models is reduced but stays higher than the 11-type LC models, for the similar reasons previously stated for T-type models.

It can be concluded that the assumption of undamaged concrete in compression, leads to escalating over-estimation of bending stiffness and capacity at yield and ULS. In contrary, the assumption of complete cover loss from the beams leads to a significant under-estimation of the capacity at SLS, especially in the less corroded beam 115 (see Fig. 16).

The numerical results also indicated that assuming no damage for the concrete cover in compression leads to higher bond stresses atthe SLS. For example, extensive failure of bond locally was observed in model 116ND in comparison with model 116a (no local bond failure). Finally, although no shear failure is predicted in these analyses, the loss of concrete strength and/or concrete section is likely to affect the shear behaviour of members, especially in the presence of high stirrup corrosion. This further highlights the limitations of sectional analysis methods for the assessment of existing structures, which may be suitable for the traditional ULS assessment but should be used with extreme caution for the performance assessment of in-service structures.

6. Conclusions

Rebar corrosion is the most common deterioration mechanism affecting the performance of RC structures. A 2D non-linear FE model is developed in this paper to assess the structural performance of a series of RC beams damaged by varying degree of corrosion at different locations. Key parameters affecting the performance of corroded members are identified and their influence on the behaviour of under-reinforced beams is quantified. These include the influence of compressive strength of concrete, impaired bond performance (due to corrosion in the tension rebars) and corrosion damage in the compressive regions of under-reinforced beams. The following conclusions are drawn from this study:

(1) Loss of steel area and associated concrete damage/section loss (due to the accumulation of expansive corrosion products) are found to be the main causes for loss of strength and bending stiffness.

(2) Compressive strength appears to have little influence on the predicted load–deflection response of uncorroded beams, or beams damaged by corrosion only in their tension region. By contrast, the load–deflection response of beams damaged by corrosion in their compressive region is particularly sensitive to the compressive strength; this of course depends on the corrosion intensity in the affected top rebars.

(3) The effect of bond deterioration was examined for two corrosion damage scenarios; one with partial bond damage (model T282) and one with bond deterioration occurring alongthe entire length (11-type models), of the tensile rebars. The load–deflection response in the both cases was found to be relatively unaffected by the impaired bond. This indicates that bond damage may be omitted for ULS strength predictions of corroded beams, provided that the tension bars are well anchored at their ends.

(4) The bond deterioration is responsible for changes in cracking patterns and widths, i.e. increasing bond deterioration results in increasing crack spacing and widths. Consequently, modelling bond deterioration is highly significant for performance assessment at the serviceability limit state.
(5) Performance is sensitive to cover concrete damage in the compressive zone; ignoring this leads to over-estimation of structural performance at the both the serviceability and ultimate limit states. Excluding the effect of the damaged cover from the analysis, a simplification necessary in sectional analysis, results in under-estimation of the serviceability performance. In both cases, the magnitude of the error depends on the degree of corrosion damage in the compressive rebars and the concrete strength $f_c$ of the beams. Hence, a detailed system based model, such as the one developed in this paper, is desirable in assessing both strength and serviceability performance of corroded members as well as in capturing changes in the failure modes.

The use of such numerical models can not only serve as a tool for the assessment of existing structures, but also as a complementary tool for planning and optimisation of future experimental studies by establishing an a priori parameter sensitivity (even though some are difficult to control in physical experimentation).

The performance of in-service RC beams, affected by spatially varying corrosion intensities, is a function of complex interactions of the various damages in the system and their interdependencies. This underlines the limitations of conventional sectional analysis methods in advanced assessment and performance prediction studies, and hence the need to develop system based models such as the one presented in this paper. The effect of spatial variations of corrosion damages on the structural performance is currently under investigation and will be presented in near future.

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References


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