Effect of under-reinforcement on the flexural strength of corroded beams

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Published version

O'FLAHERTY, Fin, MANGAT, Pal, LAMBERT, Paul and BROWNE, Elena (2008). Effect of under-reinforcement on the flexural strength of corroded beams. Materials and structures, 41 (2), 311-321.

Effect of under-reinforcement on the flexural strength of corroded beams

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Abstract:

Reinforced concrete beams are normally designed as under-reinforced to provide ductile behaviour i.e. the tensile moment of resistance, $M_{t(0)}$ is less than the moment of resistance of the compressive zone, M_c. The degree of under-reinforcement ($M_{t(0)}/M_c$ ratio) can depend upon the preferences of the designer in complying with design and construction constraints, codes and availability of steel reinforcement diameters and length. $M_{t(0)}/M_c$ is further influenced during service life by corrosion which decreases $M_{t(0)}$. The paper investigates the influence of $M_{t(0)}/M_c$ on the residual flexural strength of corroded beams and determines detailing parameters (e.g. size and percentage of steel reinforcement, cover) on $M_{t(0)}/M_c$. Corroded reinforced concrete beams (100 mm × 150 mm deep) with varying $M_{t(0)}/M_c$ ratios were tested in flexure. The results of the investigation were combined with the results of similar work by other researchers and show that beams with lower $M_{t(0)}/M_c$ ratios suffer lower flexural strength loss when subjected to tensile reinforcement corrosion. Cover to the main steel does not directly influence $M_{t(0)}/M_c$ and, thus, the residual flexural strength of corroded beams is not normally affected by increased cover. A simplified expression for estimating the residual strength of corroded beams is also given.

Keywords:

Under-reinforced Corrosion Flexural Durability Structural

1 Introduction

The current British Standard, BS 8110 [1], for design of reinforced concrete structures is due to be withdrawn by 2010 and replaced by EC2 [2]. The design of flexural elements to both standards is very similar [3] and dictates that the section fails by yielding of the tensile reinforcement. Beams are, therefore, designed as under-reinforced and the tensile moment of resistance, $M_{t(0)}$, is less than the moment of resistance of the compressive zone, M_c. The level of under-reinforcement ($M_{t(0)}/M_c$) depends upon the preferences of the designer in complying with design and construction constraints, codes and availability of steel reinforcement. The amount of tensile reinforcement in an under-reinforced rectangular beam can vary from 0.13% to 4% of the gross cross-sectional area when designing in accordance with BS 8110 [1] or 0.13%bd to 4%bh in EC2 [4]. The designer specifies the number, type and diameter of bars required to provide the area of steel required. The arrangement of the reinforcing bars is constrained by practical considerations such as construction tolerances, clearance between bars and available bar diameter and length. Another criteria to be satisfied is the cover to the reinforcement for durability and fire resistance. Invariably, these factors will have an influence on the moment of resistance of the beam section which in turn will have a direct bearing on the degree of under-reinforcement ($M_{t(0)}/M_c$).

In addition, reinforced concrete is prone to corrosion when subject to attack by chlorides in deicing salt for winter maintenance or through a reduction in the alkalinity through carbonation. There are numerous reinforced concrete beams in-service in structures that are showing signs of distress, initiating as rust stains and eventually leading to longitudinal cracking along the corroding steel reinforcement and spalling of the surrounding concrete. It has been suggested that 10–25% reduction in steel bar section due to corrosion results in failure of serviceability [5]. Despite the majority of reinforced concrete structures meeting or exceeding their intended service life [6], repair and maintenance of reinforced concrete is costly and it is estimated that €1.5 bn is spent annually in Europe [7] to repair deteriorating infrastructure. This figure will continue to rise as the volume of maintenance and repair on corroded reinforced concrete continues to grow. This puts pressure on bridge engineers who are responsible for maintaining deteriorating bridge stock with insufficient maintenance budgets.

The aim of the paper is to determine the influence of $M_{t(0)}/M_c$ on residual flexural strength of corroded beams and to establish the effect of detailing parameters (e.g. bar diameter, percentage of steel reinforcement, cover) on $M_{t(0)}/M_c$. Optimisation of design procedures to accommodate future corrosion of beams may help reduce the enormous repair costs incurred each year. The paper also presents a simplified expression for estimating the residual strength of corroded beams.

2 Research significance

Since it is well established that reinforced concrete deteriorates when subjected to a severe corrosive environment containing chlorides and carbon dioxide, there are a number of methods currently available to minimise deterioration. At the design stage, pozzolanic materials such as silica fume, fly ash and slag can be added to the mixture to enhance the durability of the concrete [8]. Corrosion resistant chemical admixtures are available which increase the resistance of concrete to deicing deterioration [9]. During the in-service stage, electrochemical techniques such as cathodic protection; desalination and re-alkalisation have been used successfully to prevent or slow down the onset of corrosion in reinforced concrete.

The availability of anti-carbonation coatings and corrosion inhibitors also enhance durability and increase the service life of the member.
In addition to the current methods for combating deterioration of reinforced concrete, implementation of the design recommendations identified in this paper will contribute to minimising repair of deteriorated beams at no additional initial cost with the benefit of potential savings on future maintenance costs.

3 Design, manufacture and testing of beams

Reinforced concrete beams were prepared in the laboratory and exposed to accelerated corrosion. Details of test specimens are given in Table 1 and Fig. 1. The beams were 910 mm long, with a cross-section of 100 mm × 150 mm deep. All specimens were designed for flexural failure by providing sufficient links to prevent shear failure. Referring to Table 1, two high yield steel reinforcement bars of 8, 10 and 12 mm were used (2T8, 2T10, 2T12). Beams were subjected to a target corrosion of 0–15% of cross-sectional area in 5% increments. Cover to the main steel was 26, 36 or 56 mm for beams reinforced with main steel 2T8 and 2T10 whereas 56 mm cover only was provided to beams reinforced with main steel 2T12. Beams are identified in the paper by the number and type of main steel bars and cover to the main reinforcement e.g. 2T8/26.

Table 1: Details of reinforced concrete beams

Main steel	Degree of reinforcement corrosion (2RT′/ Ø) (100)%	Main cover (mm)
2T8	0	26, 36, 56
	5	26, 36, 56
	10	26, 36, 56
	15	26, 36, 56
2T10	0	26, 36, 56
	5	26, 36, 56
	10	26, 36, 56
	15	26, 36, 56
2T12	0	56
	5	56
	10	56
	15	56

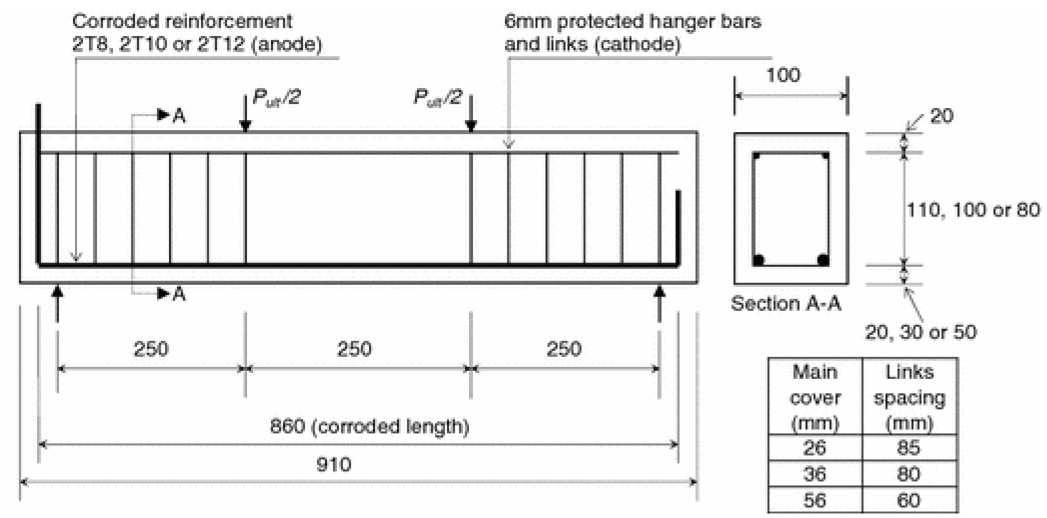

Fig. 1: Beam specimen details

Main reinforcement consisted of high yield (ribbed) bars with a nominal characteristic strength of 460 N/mm^2. Shear reinforcement was 6 mm diameter plain round mild steel bars of nominal characteristic strength 250 N/mm^2 at a spacing of 85, 80 or 65 mm corresponding to the cover of 26, 36 and 56 mm, respectively. Two longitudinal hanger bars for the links were provided at the top of the beam cross-section. These were 6 mm diameter plain round mild steel bars with a nominal characteristic yield strength of 250 N/mm^2. The steel reinforcement was weighed before casting to enable the actual percentage corrosion to be calculated at a later stage. In order to prevent corrosion in the shear reinforcement, shrink wrap tubing was provided at the points of contact with the main reinforcement to break the electrical circuit and hence prevent current flow to the links during the accelerated corrosion process (see Sect. 4). Inspection of the shear reinforcement at the end of the tests showed that this was an effective method of preventing accelerated corrosion of the shear reinforcement. The beams were cast in the laboratory using a concrete with target cube strength of 40 N/mm^2. Mix proportions were 1:1.7:3.8 of ordinary Portland cement:fine aggregate:coarse aggregate. Fine and coarse aggregates were oven dried at 100°C for 24 h. Anhydrous calcium chloride ($CaCl_2$) was added to the mix (1% by weight of cement) in order to promote corrosion of the reinforcement. The concrete was cast in steel moulds in three layers, each layer being carefully compacted on a vibrating table. The specimens were then placed in a mist curing room (20°C and 95% ± 5% Relative Humidity) for 24 h. The samples were demoulded after 1 day and cured in water at 20°C for a further 27 days (28 days in total). The beams were then transferred to a tank filled with a saline solution for accelerated corrosion at 28 days age. Beams were subsequently tested to failure under four point loading.

The control specimens (0% corrosion) were tested in flexure at the age of 28 days but the corroded beams were tested at 42, 56 and 63 days age due to the time taken to reach the target corrosion of 5, 10 and 15%, respectively (Table 1). The loading rate was 5 kN/min.

4 Accelerated corrosion process

The longitudinal tensile steel reinforcement was subjected to an accelerated galvanostatic corrosion process in an electrolytic cell by means of a direct current multi-channel power supply. The accelerated corrosion test arrangement is shown in Fig. 2 with up to three beams electrically connected in series. The system was connected to an ammeter to monitor the cell current. The corrosion process took place in a plastic tank where a 3.5% $CaCl_2$ solution was used as the electrolyte. The solution level in the tank was adjusted to slightly exceed the concrete cover plus reinforcing bar diameter to ensure adequate submersion of the longitudinal reinforcement. The direction of the current was arranged so that the main reinforcing steel served as the anode and the longitudinal hanger bars and the stirrups acted as the cathode. A constant current density of 1 mA/cm^2 was passed through the reinforcement.

This current density was adopted on the basis of pilot tests to provide desired levels of corrosion in a reasonable time. Each degree of corrosion was selected to provide a predefined percentage reduction in the longitudinal bar diameter (excluding the bent-up portion) within the timescale. The relationship between corrosion current density and the weight of metal lost due to corrosion was determined by applying Faraday's law as shown in Eq. 1 [10]:

$$\omega = AItZF \qquad (1)$$

where ω = weight loss due to corrosion in (g); A = atomic weight of iron (56 g); I = electrical current in (A); t = time in (s); Z = valence of iron which is 2; and F = Faraday's constant (96,500 coulombs).

Fig. 2: Reinforced concrete beams undergoing main reinforcement corrosion

The weight loss of metal due to corrosion can also be expressed as:

$$\omega = a\delta\gamma \tag{2}$$

where a = rebar surface area before corrosion (cm^2); δ = material loss (cm); and γ = density of material (7.86 g/cm^3).

The corrosion current can be expressed as:

$$I = ia \tag{3}$$

where i = corrosion current density (amp/cm^2).

Therefore, combining Eqs. 2 and 3 gives:

$$a\delta\gamma = AItZF = AiatZF \tag{4}$$

Substituting known values into Eq. 4 and simplifying gives:

$$\delta = (56)(i)(365)(24)(60)(60)(7.86)(2)(96500) = 1165(i)\text{cm/year} \tag{5}$$

Rewriting Eq. 5, where R is defined as the material loss per year (cm/year), gives:

$$R = 1165(i)\text{cm/year} \tag{6}$$

As an example, for a corrosion rate, i, of 1 (mA/cm^2), R equals 1.165 (cm/year) (from Eq. 6). If, in a reinforced concrete structure, the period of corrosion after initiation is T′ years, then:

$$\text{Metal loss after } T' \text{ years} = RT'(\text{cm}) \tag{7}$$

Therefore:

$$\text{\% reduction in bar diameter in } T' \text{ years} = (2RT'\{\backslash O\})(100) \tag{8}$$

where Ø is the bar diameter. The expression (2RT′/Ø)(100)%, which represents reduction in bar diameter due to corrosion in T′ years, is also defined as the degree of reinforcement corrosion (see Table 1). Preliminary tests were carried out before commencing the program to confirm the reliability of the accelerated corrosion technique.

5 Analysis of beam section

An idealised stress block was used to determine the maximum compressive moment of resistance, M_c, of the section as shown in Fig. 3. Taking moments about the centroid of the tension steel A_s:

$$Mc=(Fcc)(z)+(Fsc)(d-d') \quad (9)$$

where F_{cc} and F_{sc} are the compressive forces in the concrete and steel (hanger bars), respectively.

The maximum value for z is 0.775d as given in BS 8110 [1].

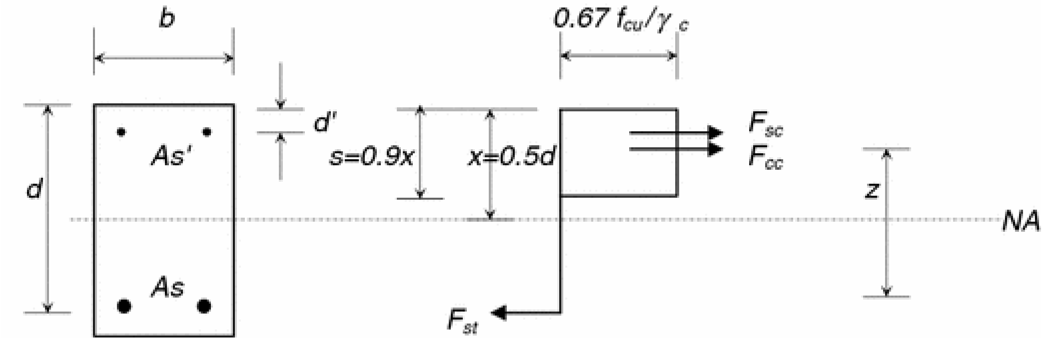

Fig. 3: Section with compression reinforcement

The ultimate concrete design stress is 0.67 f_{cu}/γ_c where the factor 0.67 relates the cube crushing strength to the flexural strength of concrete [11] and γ_c is the partial safety factor for the strength of concrete for designing members cast in situ (normally 1.5). Referring to Fig. 2:

$$Fcc=(0.67fcu/\gamma c)bs \quad (10)$$

and

$$Fsc=\gamma sfyA's \quad (11)$$

Substituting Eqs. 10 and 11 into Eq. 9 gives:

$$Mc=[(0.67fcu/\gamma c)bs](0.775d)+\gamma sfyA's(d-d') \quad (12)$$

For the analysis of laboratory beams of known material properties, partial factors of safety, γ_c and γ_s are taken as unity (the actual yield stress of steel was obtained from tests). Substituting these into Eq. 12 and taking s = 0.45d gives:

$$Mc=[(0.234fcubd^2)+f'yA's(d-d')]10^{-6} \quad (13)$$

Equation 13 was used to calculate the maximum compressive moment of resistance of the beams, M_c. Table 2 gives geometric details and material properties for the beams. Actual values of strength (28 day concrete cube strength and yield strength of steel) given in Table 2 were used to calculate M_c. In addition, the corresponding properties of beams tested by other researchers are also presented in Table 2 to extend the $M_{t(0)}/M_c$ range of beams considered [10, 12, 13]. Some of these beams had tensile reinforcement only with the shear capacity provided by external means. In addition, certain details such as yield strength of the steel reinforcement and depth of concrete cover were not explicitly given and estimates were made from the information available in the papers. The corrosion rates ranged from 0.1 mA/cm^2 to 2 mA/cm^2.

Table 2: Beam properties

	2T8	**2T8**	**2T8**	**2T10**	**2T10**	**2T10**	**2T12**	**Mangat et al. 2T10 [10]**	**Mangat et al. 2T8 [10]**	**Al Sulamani et al. 1T12 [12]**	**Rodriguez et al. 2T10 [13]**
C (mm)	26	36	56	26	36	56	56	20	20	29	20
f_{cu} (N/mm^2)[a]	56.0	52.5	52.2	49.1	59.8	53.6	57.5	40.0	40.0	40.0	50.0
b (mm)	100	100	100	100	100	100	100	100	100	150	150
Ø′ (mm)[b]	6	6	6	6	6	6	6	0	0	0	6
f'_y (N/mm^2)[a]	328	341	345	326	366	328	384	0	0	0	626
A'_s (mm^2)	57	57	57	57	57	57	57	0	0	0	57
d′ (mm)	23	23	23	23	23	23	23	0	0	0	23
h (mm)	150	150	150	150	150	150	150	150	150	150	200
Ø (mm)[b]	8	8	8	10	10	10	12	10	8	12	10
d (mm)	126	116	96	125	115	95	94	125	126	115	175
M_c (kNm)	22.7	18.3	12.7	19.8	20.4	12.6	13.4	14.6	14.9	18.6	59.1
i (mA/cm^2)	1	1	1	1	1	1	1	1	2	2	0.1

[a] Average values of all specimens
[b] Nominal values

6 Test results and discussion

The control and corroded beams failed in a ductile manner with no evidence of shear failure. After testing, the reinforcing bars were removed from the concrete, cleaned using a wire brush and re-weighed. The percentage loss in weight was subsequently calculated. The corrosion was generally spread along the length of the bars. Serious cross-section loss occurred at higher percentages of corrosion.
Control beams representing 0% corrosion were tested for each series of beams (Table 1) and the tensile moment of resistance [$M_{t(corr)} = M_{t(0)}$] was determined. The ratio $M_{t(0)}/M_c$ represents the degree of under-reinforcement of each series of beams. The tensile moment at

failure due to increasing levels of corrosion was obtained from $M_{t(corr)} = 0.25(P_{ult}/2)$ (Fig. 1) where P_{ult} is the ultimate load as described in Sect. 3. The compressive moment of resistance, M_c, calculated from Eq. 13, remains constant for each series of beams (i.e. it is not affected by the degree of corrosion of the tensile reinforcement) since it is based on the properties of both the concrete and steel in the compression zone. The relationship between $M_{t(corr)}/M_c$ and the percentage of corrosion is shown in Fig. 4. In addition, Fig. 5 shows the relationship between $M_{t(corr)}/M_c$ and percentages of corrosion for beams tested by other researchers. The $M_{t(corr)}/M_c$ value at 0% corrosion in Figs. 4 and 5 represents the degree of under-reinforcement $[M_{t(0)}/M_c]$ of each set of beams.

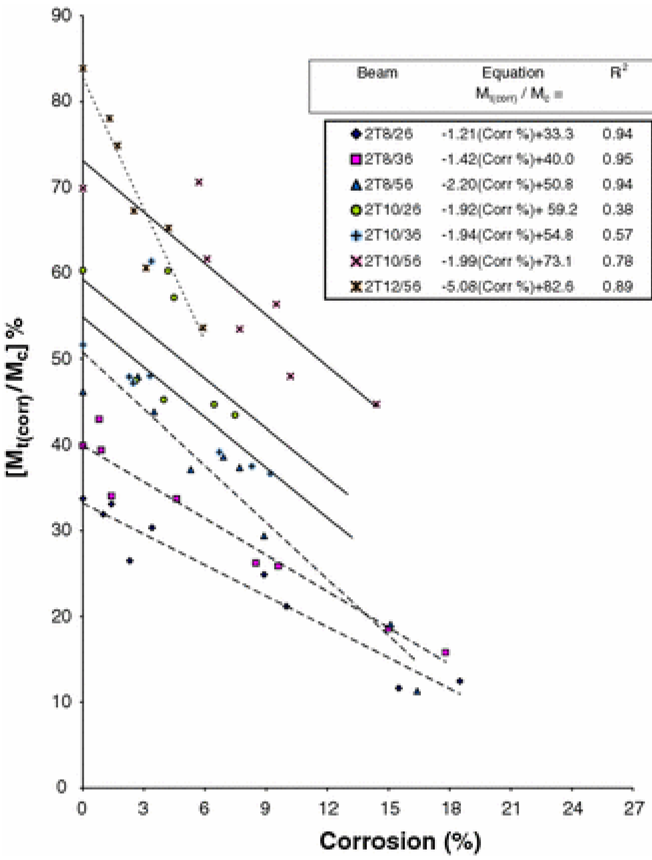

Fig. 4: Degree of under-reinforcement ($M_{t(corr)}/M_c$) against percentage of corrosion

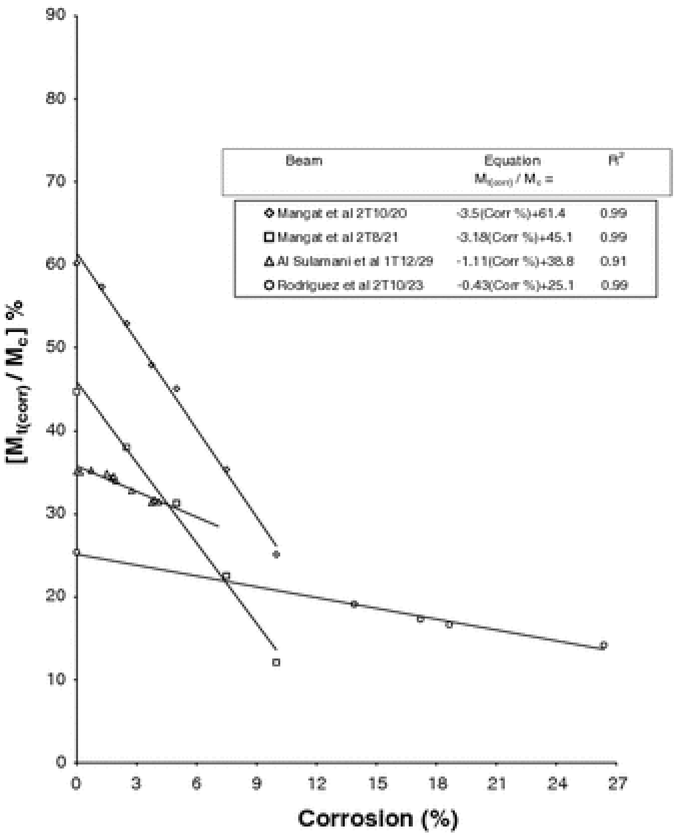

Fig. 5: Degree of under-reinforcement ($M_{t(corr)}/M_c$) against percentage of corrosion for other researchers

Figures 4 and 5 show results from corroded beams reinforced with T8, T10 and T12 main steel and different covers (20–56 mm) to the main steel. The relationship is generally a linear decrease in $M_{t(corr)}/M_c$ with increasing percentages of corrosion. The best fit linear equation for each series of beams is tabulated along with the coefficient of correlation (R^2). Beam 2T10/26 (Fig. 4) exhibits the lowest coefficient of correlation of the series of beams under consideration (0.38). However, the remaining six series of beams in Fig. 4 and the four beam series in Fig. 5 all exhibit a very satisfactory coefficient of correlation. The accelerated corrosion process is a very complex phenomena and considering the different parameters tested by four different researchers (diameter of steel, cover to the main reinforcement, beam dimensions etc., Figs. 4 and 5), there is a very strong correlation between $M_{t(corr)}/M_c$ and the percent of corrosion.

The general relationship between $M_{t(corr)}/M_c$ and percent of corrosion for the data in Figs. 4 and 5 is of the form:

$$(Mt(corr)/Mc)\%=\alpha(Corr\%)+\beta\% \qquad (14)$$

where α is the slope of each line of best fit and β is the intercept ($M_{t(0)}/M_c$).

The values of α and β (together with cover and reinforcement details) for each set of beams are given in Table 3. The relationship between $M_{t(0)}/M_c$ and the slope α is plotted in Fig. 6 and shows that the negative value of α generally increases with increasing $M_{t(0)}/M_c$. This indicates a lower rate of strength loss when higher degrees of reinforcement corrosion occur in beams with higher degrees of under-reinforcement (lower $M_{t(0)}/M_c$). Therefore, use of beams with lower $M_{t(0)}/M_c$ is desirable in practice.

Table 3: Overall comparison

Identification	**Slope[a]**	**$M_{t(0)}/M_c$ (%)**	**Cover**	**Reinforcement (%)**
	α	**β**	**(mm)**	**$100A_s/bh$**
2T8/26	–1.210	33.7	26	0.67
2T8/36	–1.420	39.9	36	0.67
2T8/56	–2.200	46.1	56	0.67
2T10/26	–1.920	60.3	26	1.05
2T10/36	–1.990	51.6	36	1.05
2T10/56	–1.990	69.9	56	1.05
2T12/56	–5.080	83.9	56	1.51
Mangat et al. 2T10/20 [10]	–3.520	60.2	20	1.05
Mangat et al. 2T8/21 [10]	–3.180	44.0	21	0.67
Al Sulamani et al. 1T12/29 [12]	–1.110	38.1	29	0.50
Rodriguez et al. 2T10/20 [13]	–0.430	25.4	23	0.52

[a] Slopes from Figs. 4–5

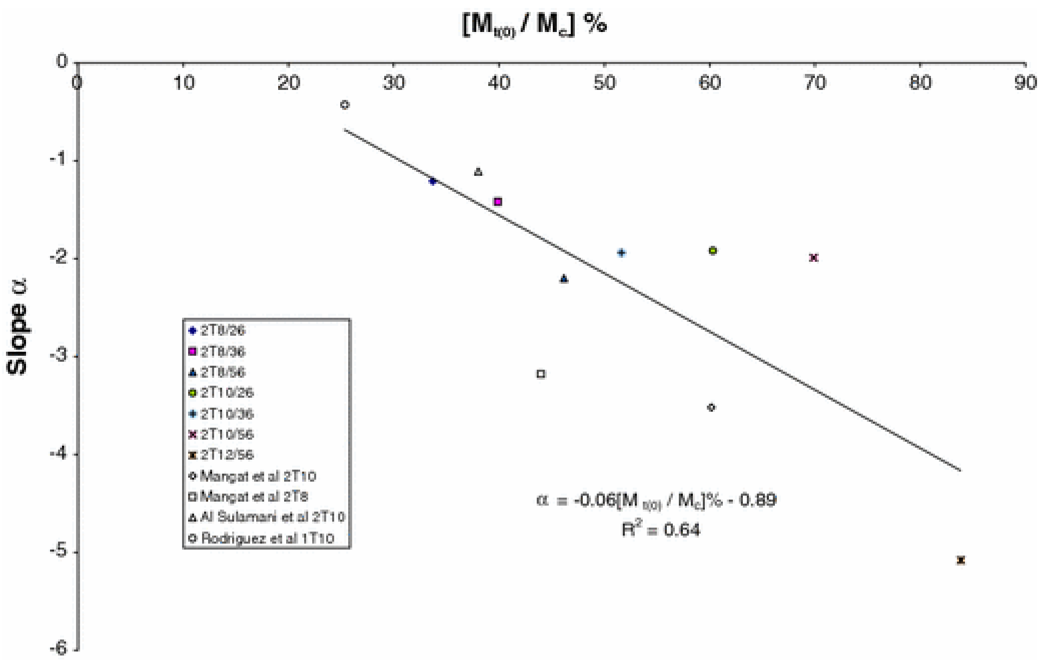

Fig. 6: Relationship between α and degree of under-reinforcement ($M_{t(0)}/M_c$)

7 Designing for durability

In order to optimise the design of beams for enhanced performance in a corrosive environment, the designer should be aware of the main parameters which influence $M_{t(0)}/M_c$. Maximum degree of under-reinforcement should be achieved since this leads to lower loss of strength when reinforcement corrosion occurs. Parameters which control $M_{t(0)}/M_c$ include percentage of main reinforcement, cover to the steel and size and number of reinforcement bars. The percentage of main steel reinforcement ($100A_S/bh$) is plotted against $M_{t(0)}/M_c$ in Fig. 7 for the 11 beams under consideration. The linear relationship shows that lower percentages of main steel result in lower $M_{t(0)}/M_c$, which is beneficial in the event of corrosion of the main steel. Therefore, the designer should aim to reinforce the section with percentages as close as possible to the allowable minimum (0.13%). However, the reduction in reinforcement percentage should not be achieved by simply increasing the cover and, therefore, increasing the section size (bh) since, as shown in Fig. 8, no clear relationship exists between the cover and $M_{t(0)}/M_c$. Hence, the increase in cover to the main steel does not relate to lower $M_{t(0)}/M_c$. Therefore, in design, sufficient cover should be provided to meet the code requirements for durability and fire resistance, but not unnecessarily increased simply to reduce the percentage of reinforcement.

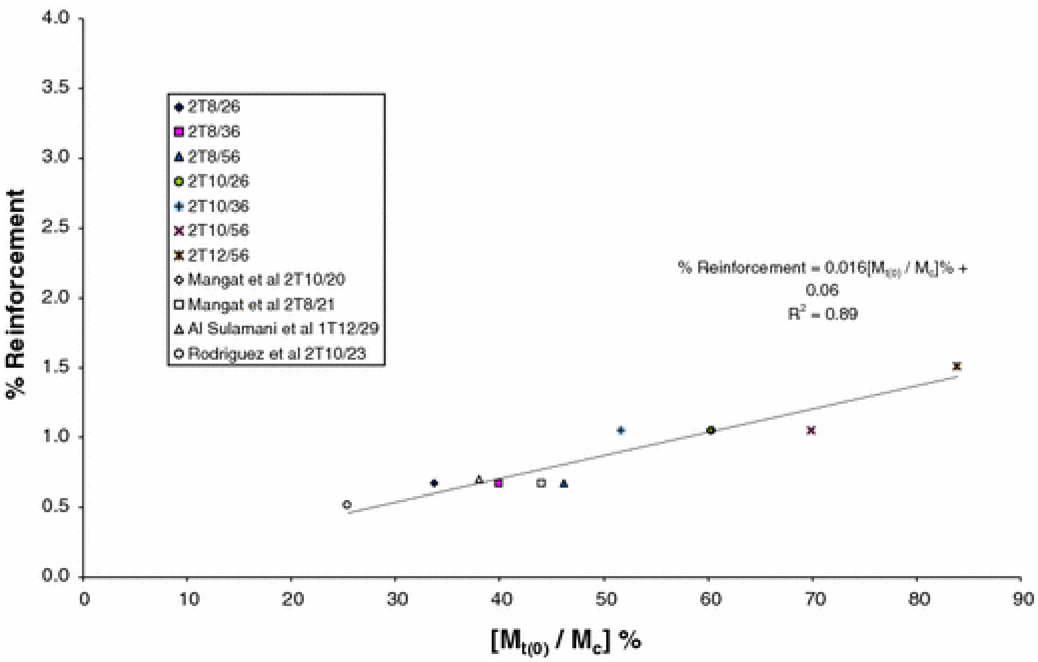

Fig. 7: Percentage reinforcement against degree of under reinforcement ($M_{t(0)}/M_c$)

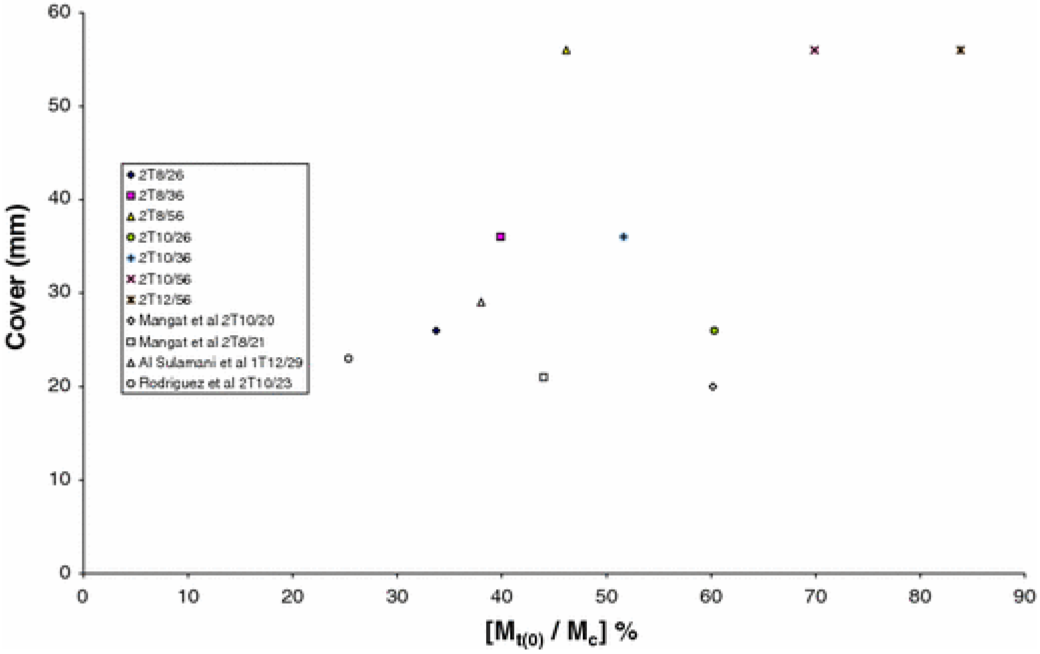

Fig. 8: Cover against degree of under-reinforcement ($M_{t(0)}/M_c$)

It has been established that the bursting radial forces at the interface of reinforcement caused by corrosion products decrease with decreasing rebar diameter. Therefore, the use of a higher degree of under-reinforcement together with low diameters of tensile reinforcement is recommended in design [14]. It has also been reported that bond strength decreases with increased bar diameter, so smaller diameter bars will help minimise the effect of corrosion induced bond failure [14].

8 Residual tensile moment of resistance

The data presented in this paper also allows an estimation of the residual tensile moment of resistance of corroded beams to be obtained. Values of $M_{t(0)}$ and M_c will be available to the design engineer, hence by rewriting Eq. 14, $M_{t(corr)}$ can be estimated from:

$$(Mt(corr)/Mc)\%=\alpha(Corr\%)+(Mt(0)/Mc)\% \qquad (15)$$

where ($M_{t(0)}/M_c$)% is the intercept β in Figs. 4 and 5 and Table 3. Multiplying Eq. 15 by M_c and dividing by 100 gives:

$$Mt(corr)=[Mc\alpha(Corr\%)/100+Mt(0)] \qquad (16)$$

where Corr% is the actual corrosion of the main steel in the in-service beam and α is calculated from the line of best fit representing the data in Fig. 5 as follows:

$$\alpha=-0.06[(Mt(0)/Mc)\%]-0.89 \qquad (17)$$

A factor of safety for concrete (γ_c = 1.5) should also be applied to Eq. 16 to give:

$$Mt(corr)=Mc\alpha(Corr\%)/100+Mt(0)/\gamma c \qquad (18)$$

Equation 18 gives the residual strength (tensile moment of resistance) of a corroded reinforced concrete beam once the degree of corrosion (Corr%) in the tensile steel is determined from a field inspection.

Equation 18 applies within the limits of the parameters covered by test data given in the paper. Further research is required to verify its validity to other parameters and beam sections, for example, Tee sections, which were outside the scope of this investigation.

9 Conclusions

The main conclusions from the results reported in this paper are as follows:

1. A higher degree of under-reinforcement (lower $M_{t(0)}/M_c$) of reinforced concrete beams results in lower loss of strength caused by reinforcement corrosion

2. Lower $M_{t(0)}/M_c$ can be achieved in beam design through specifying areas as close as possible to the required area of steel reinforcement and, therefore, keeping the percentage of main steel reinforcement as close as possible to 0.13%, the minimum permissible in codes of practice. Preference should be given to smaller diameters over larger diameter reinforcement bars

3. Increasing the cover to the steel reinforcement should not be used as a means of increasing the section size simply to reduce the percentage of steel reinforcement. The cover to the main steel reinforcement does not have an influence on $M_{t(0)}/M_c$ and cover should be based on durability considerations only in accordance with the codes of practice

4. An estimate of the residual tensile moment of resistance of corroded beams within the limits of the test data given in the paper can be obtained from the expression

   $Mt(corr)=Mc\alpha(Corr\%)/100+Mt(0)/\gamma c$

Acknowledgements:

The authors gratefully acknowledge the support of CPI Aps in providing the multi-channel power supply for inducing accelerated corrosion.

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