Impact of main steel diameter on the flexural capacity of deteriorated reinforced concrete beams

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ABSTRACT: Deterioration of reinforced concrete is principally caused through corrosion of the reinforcing steel when it becomes subjected to a severe corrosive environment containing chlorides and carbon dioxide. It has been estimated that around £500 million is spent annually in the UK alone on concrete repairs to increase the service life of deteriorated structural members. However, before repair is carried out, it is important that the structural capacity of deteriorated members can be estimated to ensure repairs are executed at the appropriate time. The aim of this paper, therefore, is to present information on influence of the diameter of the main steel on the residual strength of corroded reinforced concrete beams.
The paper reports details of laboratory experiments where reinforced concrete beams were tested under four point loading when the reinforcement was corroded along 100% of the span. Results of 26 structural tests with target main steel corrosion ranging from 0% (control) to 15% in 5% increments are presented. The influence of three different diameters of the main steel reinforcement (8, 10 and 12 mm) was investigated and related to the flexural performance. Preliminary results show that corrosion to larger diameter reinforcing steel has a more adverse influence on flexural performance. The ratio of ultimate load to control load (P_{ult}/P_{con}) for a 2T8mm diameter reinforced beam is reduced to approximately 0.53 when 12% of the cross sectional area of the reinforcing bar is lost due to corrosion, this reduces to 0.26 when the same amount of corrosion is applied to a beam reinforced with 2T12mm diameter main steel.

1 INTRODUCTION

Reinforced concrete is widely used in construction due to its versatility and durability when properly designed and placed. In general, the environment provided by concrete protects the reinforcing steel. This is due to the high pH environment present in Portland cement pore solution which passivates the steel (Borgard et al. 1990). Corrosion of the steel reinforcement will not occur unless an external agent changes the normal passive state of the steel in this alkaline environment. When this occurs, corrosion becomes a subject of technical and scientific interest as well as of economic interest. It was reported that over £500 million is spent annually in the UK on refurbishment as the tendency is to repair a structure to increase its design life rather than demolish and replace (Swiss Bank Corporation 1989).

The expansive products of reinforcement corrosion cause cracking, rust staining and spalling of the cover zone which can lead to serviceability failure of structures during their design life (Mangat & Elgarf 1999). Additionally, corrosion results in a loss of reinforcement cross-sectional area and bond, and therefore, a loss in load carrying capacity of the structural element. A survey of bridge stock in the UK revealed that 75% is contaminated with chlorides which in time will cause reinforcement corrosion (Dept. of Transport 1989). In such cases, repair is necessary to increase the service life of the member (Mangat & O'Flaherty 1999, 2000, 2003).

2 BACKGROUND

In this investigation, the main steel in reinforced concrete beams was subjected to an accelerated corrosion technique in the laboratory using one of the several methods available. The galvanostatic method was used in this study to simulate the field conditions. The method involves passing a direct current through the reinforcement to accelerate corrosion. The galvanostatic corrosion is carried out whilst the beam is unloaded, which is different from the corrosion in actual structures. The corrosion by galvanostatic method is general, whereas actual structures have some specific areas that are more prone to corrosion. Thus in the latter case, there is always the possibility of pitting corrosion whereby the cross-sectional area of the reinforcing bars could be significantly reduced, thus reducing the tensile strength of the reinforcing bars. However, to ensure consistency of results in this investigation, the steel reinforcement was subjected to general corrosion only, which allows easier repeatability compared to pitting corrosion.

According to standard corrosion theory, steel embedded in concrete is largely in a protected state because of the alkalinity of the matrix. The corrosion rate depends on the ratio of the cathodic area to the anodic area. In this investigation, the potential was measured every day to ensure that the steel was corroding. The potential cannot be measured directly, as the available measuring devices can measure only a difference in potential. To overcome this limitation, a Saturated Calomel Electrode (SCE) was added to the system by means of a suitable salt bridge. The potential for the steel reinforcement in this study ranged between -750 mV and -500 mV which represents the active state of corrosion process.

3 DESIGN OF THE BEAM SPECIMENS

A total of twenty-six reinforced concrete beams were tested to examine the influence of main steel diameter (*θ)* on the flexural behavior of deteriorated beams. Details of test specimens are given in Figure 1 and Table 1. Beams were 910 mm long with a cross-section of 100 mm wide and 150 mm deep. All specimens were detailed for flexural failure; sufficient links were provided to ensure adequate shear capacity at the anticipated maximum load of the corroded beam. Beams 2T8/0/50 (number, type and diameter of the main steel in mm/target percentage of corrosion/cover in mm), 2T10/0/50 and 2T12/0/50 were tested without corrosion to serve as control specimens (Table 1). The number of specimens tested for each target corrosion percentage is also shown in Table 1.

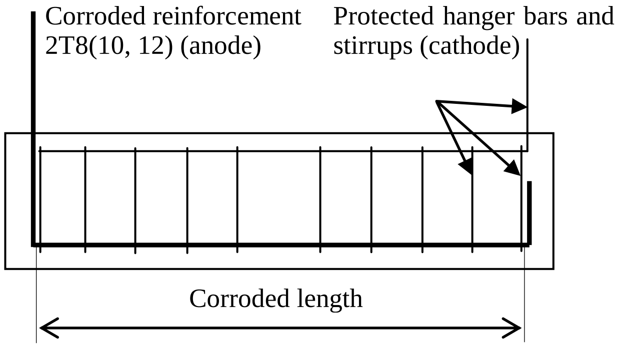

Figure 1. Beam specimens.

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 with yield strength of 250 N/mm^2 at 65 mm spacing for 50 mm cover. Hanger top bars for all beams consisted of two 6 mm diameter plain round mild steel bars with a yield strength of 250 N/mm^2. The main steel reinforcement was weighed before casting to enable the actual percentage corrosion to be calculated at a later stage.

Table 1. Variables in test programme.

Main steel	Target corrosion %	Diameter mm	No. of specimens
2T8	0	8	2
	5		4
	10		2
	15		2
2T10	0	10	2
	5		2
	10		2
	15		2
2T12	0	12	2
	5		2
	10		2
	15		2

Test specimens 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 Portland cement: fine aggregate: coarse aggregate. Fine and coarse aggregates were oven dried at 100°C for 24 hours. Calcium chloride ($CaCl_2$) was added to the mix (1% by weight of cement) in order to promote corrosion of the reinforcement. The material was placed in steel moulds in three layers, each layer being carefully compacted on a vibrating table. The specimens were then placed in the mist curing room (20°C and 95% ± 5% Relative Humidity) for 24 hours. The samples were demoulded after 1 day and cured in water at 20°C for a further 27 days (28 days in total). Specimens were then transferred to a tank filled with a saline solution for accelerated corrosion at 28 days age.

4 ACCELERATED CORROSION PROCESS

The beam specimens were immersed in artificial seawater in a plastic tank at the end of the curing period. A 3.5% $CaCl_2$ solution was used as the electrolyte. The direction of the current was arranged so that the main reinforcing steel served as the anode and the 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. The current supplied to each specimen was checked on a regular basis and any drift was corrected.

The relationship between corrosion current density and the weight of metal lost due to corrosion was determined by applying Faraday's law as is fully detailed elsewhere (Hristova et al. 2003).

Preliminary tests were carried out before commencing the research program to confirm the reliability of the accelerated corrosion technique.

Figure 2 show the reinforced concrete specimens undergoing accelerated corrosion using specialised equipment in the laboratory. The first sign of corrosion was rust staining on the concrete surface, followed by longitudinal cracking in the concrete cover zone.

5 BEAM TESTING

The control specimens (zero percent corrosion) were tested at the age of 28 days but the deteriorated beams were tested at 42, 56 and 63 days for the 5, 10 and 15% target corrosion respectively due to the time taken to reach the desired levels of corrosion. All specimens were tested under four point bending as shown in Figure 3, to determine the ultimate flexural strength.

Premature shear failure was prevented by sufficient shear reinforcement. The testing machine loading rate was set at 5 kN/min.

Figure 2. Reinforced concrete beams undergoing accelerated corrosion.

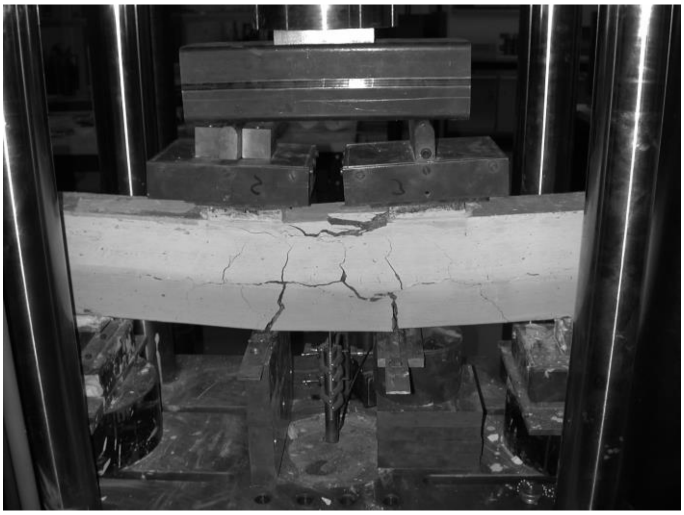

Figure 3. Flexural testing of deteriorated reinforced concrete beams.

The first load tests were carried out on the control specimens and these behaved as expected and in accordance with the design procedures of BS 8110 (British Standards Institutions, 1997). Failure of all 26 beams was in flexure; no shear failure occurred.

Upon completion of the corrosion period and flexural testing, the reinforcing bars were removed from the concrete as shown in Figure 4, cleaned with a wire brush and re-weighed. The percentage loss in weight was subsequently calculated. The resulting degree of corrosion in this investigation, 2RT/D %, ranged between 0% (control) and 16.35 % (Table 2). The corrosion damage was generally spread along the length of the bars. Where serious section loss occurred, it was in the form of localized pitting corrosion rather then general corrosion. This mainly occurred at higher percentages of corrosion.

Figure 4. Deteriorated main steel (anode) with uncorroded links and hanger bars (cathode).

Table 2. Variables in test programme.

Beam identification	Actual corrosion %	Ultimate Load (kN)	Failure mode
2T8/0/50	0	41.4*	Flexure
2T8/5/50	2.7	42.9	Flexure
2T8/5/50	3.5	39.2	Flexure
2T8/5/50	5.3	33.3	Flexure
2T8/5/50	6.9	34.6	Flexure
2T8/10/50	7.7	33.5	Flexure
2T8/10/50	8.9	26.3	Flexure
2T8/15/50	15.1	17.1	Flexure
2T8/15/50	16.4	10.1	Flexure
2T10/0/50	0	62.4*	Flexure
2T10/5/50	5.7	63.0	Flexure
2T10/5/50	6.1	55.0	Flexure
2T10/10/50	7.7	47.7	Flexure
2T10/10/50	9.5	50.3	Flexure
2T10/15/50	10.2	42.8	Flexure
2T10/15/50	14.4	39.9	Flexure
2T12/0/50	0	79.3*	Flexure
2T12/5/50	1.3	73.8	Flexure
2T12/5/50	1.7	70.8	Flexure
2T12/10/50	2.5	63.6	Flexure
2T12/10/50	3.1	57.3	Flexure
2T12/15/50	4.2	61.7	Flexure
2T12/15/50	5.9	50.7	Flexure

* Average of two control specimens.

6 TEST RESULTS AND DISSCUSSION

Table 2 shows the results from testing 26 beams in flexure in the laboratory. Each beam is identified by the amount of main steel, target corrosion and cover (e.g. 2T8/10/50). The cover is constant for all 26 beams (50mm). The actual corrosion (calculated as described in Section 5) is also given along with the ultimate load at failure. Table 2 also shows that the failure of each beam in the three categories (8, 10 and 12mm diameter) was flexural.

It is clear from the ultimate loads given in Table 2 that the strength of the beams decrease with increasing main steel corrosion. For example, the control load of beam 2T8/0/50 (0% corrosion) is 41.4 kN whereas the failure load of beam 2T8/15/50 (16.4% actual corrosion) decreases to 10.1 kN. This is also applicable to the other two categories (10 and 12mm diameter, Table 2) which also show significant reductions in ultimate strength due to increasing corrosion.

To gain a better understanding of the influence of corrosion on the flexural strength of the deteriorated beams, Figures 5-7 show the relationship between P_{ult}/P_{con} and the degree of corrosion of the main steel reinforcement. P_{ult} is the ultimate load obtained from testing the deteriorated beams in the laboratory and P_{con} is the average failure load of the control specimens (0% corrosion to the main steel reinforcement). In all cases, the actual percentage of corrosion was used in the analysis of data as opposed to the target corrosion. This led to a better correlation between flexural performance and degree of corrosion as there was some variation between target and actual values (Table 2).

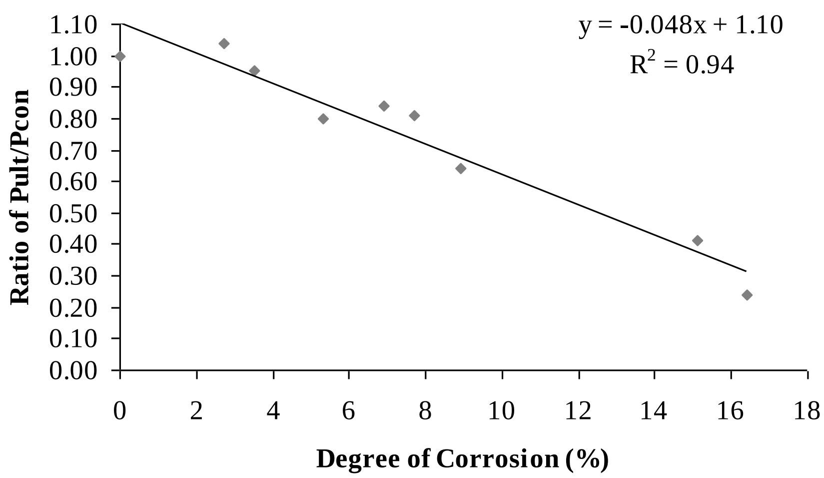

Figure 5. Relationship between P_{ult}/P_{con} and degree of corrosion for beams reinforced with 2T8

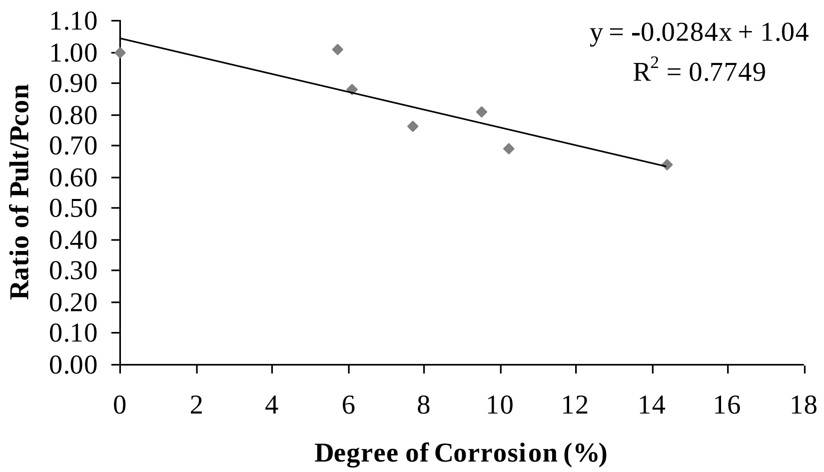

Figure 6. Relationship between P_{ult}/P_{con} and degree of corrosion for beams designed with 2T10

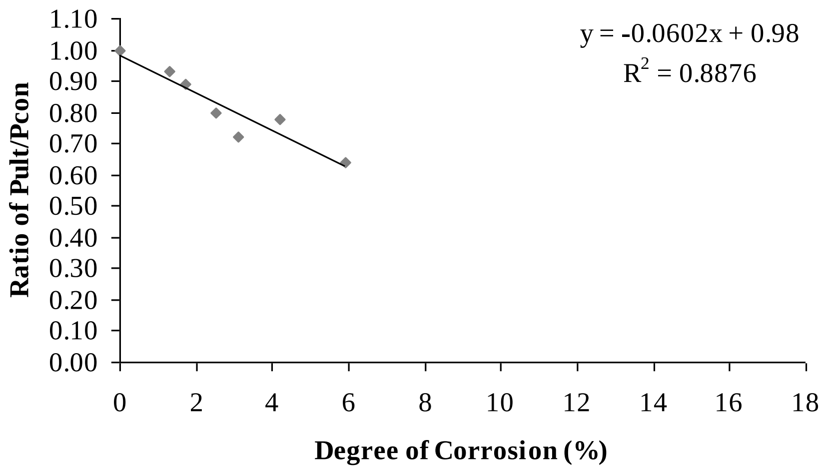

Figure 7. Relationship between P_{ult}/P_{con} and degree of corrosion for beams designed with 2T12

Referring to Table 3, comparisons are made between the ratio of P_{ult}/P_{con} and the degree of corrosion at arbitrary values of 0, 4, 8 and 12% corrosion (calculated from the best fit equations in Figures 5-7, hence the P_{ult}/P_{con} ratios at 0% corrosion do not exhibit a value of 1.00). The data presented in Table 3 is also shown graphically in Figure 8. It is clear from Figure 8 that the beam with 12mm main steel suffers the most severe decrease in strength (P_{ult}/P_{con}) with increasing percent of main bar corrosion. For example, at 12% loss of cross-section, the ratio of P_{ult}/P_{con} for the 12mm reinforced beam is only 0.26, which is approximately half of that for the 8mm reinforced beam (0.53). The 10mm data, however, does not strictly follow the trends offered by both the 8 and 12mm reinforced beams but nevertheless a reduction in P_{ult}/P_{con} is evident with increasing percentages of main steel corrosion (Figure 8).

Table 3. Comparison of residual strength at different diameters and degrees of corrosion.

Degree of corrosion	P_{ult}/P_{con} (ratio)		
(%)	2T8	2T10	2T12
0	1.10	1.05	0.99
4	0.91	0.94	0.74
8	0.72	0.82	0.50
12	0.53	0.70	0.26

A visual survey of the deteriorated beams showed that the diameter of the main steel had an influence on crack widths. Measurements of crack widths were not conducted but it was evident that beams reinforced with 2T12 had similar or higher crack widths at the actual maximum degree of corrosion (4.2 & 5.9%) than the beams designed with 2T8, despite the latter enduring higher maximum degrees of corrosion of 15.1 & 16.4%.

7 CONCLUSIONS

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

– reinforced concrete beams show a loss in residual flexural strength with increasing corrosion of the main steel reinforcement

– beams reinforced with larger diameter main steel experience a higher reduction in flexural strength than beams designed with smaller diameter main steel at similar degrees of corrosion
– the cracking in the cover concrete was more severe in beams reinforced with 2T12 mm compared to those reinforced with 2T8 mm even at lower levels of main steel reinforcement corrosion

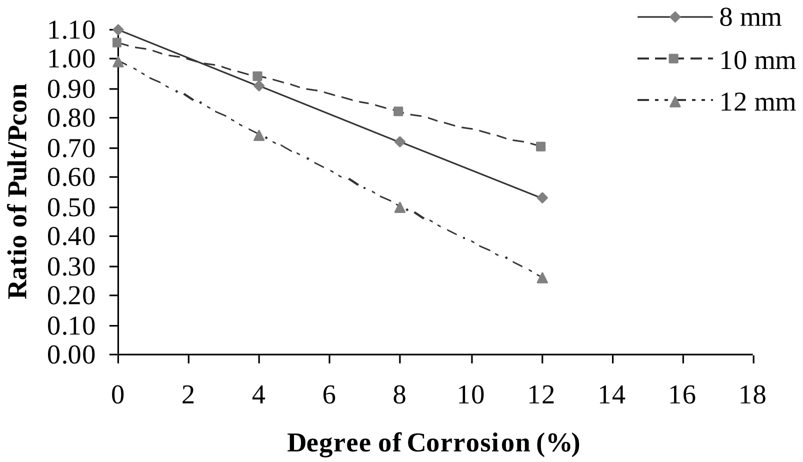

Figure 8. Simplified relationship between Pult/Pcon and degree of corrosion for all beams

8 ACKNOWLEDGEMENTS

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