Influence of shear reinforcement corrosion on the performance of under-reinforced concrete beams

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Influence of shear reinforcement corrosion on the performance of under-reinforced concrete beams

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ABSTRACT: Reinforced concrete beams are normally designed as under-reinforced to provide ductile behaviour at failure i.e. the tensile moment of resistance, $M_{t(0)}$, is less than the moment of resistance of the compressive zone, M_c. Since it is well established that the steel in reinforced concrete beams is prone to corrosion, the residual flexural strength is normally the main concern of asset managers. However, concrete cover to the shear reinforcement is less than that to the main steel and therefore, may suffer higher levels of corrosion due to chloride penetration and carbonation. The paper investigates the influence of shear reinforcement corrosion on the performance of reinforced concrete beams. Beams (100mm x 150mm cross section) with two varying degrees of under-reinforcement ($M_{t(0)}$ / M_c ratios) were tested in flexure. The results show that despite exhibiting varying levels of shear reinforcement corrosion (the main steel remained uncorroded throughout), flexure was still the dominant mode of failure. However, all beams did exhibit a decrease in flexural strength with increasing shear reinforcement corrosion levels indicating that the flexural integrity of the beam was influenced by the shear reinforcement corrosion. This was more pronounced for the beams with a higher $M_{t(0)}$/ $M_{c \; ratio}$ (lower degree of under-reinforcement) and this should be taken into account at the design stage.

1 INTRODUCTION

Reinforcement corrosion in concrete structures is the biggest durability problem facing the UK at present. In a survey of 200 randomly selected concrete bridges, corrosion was found in 72% of the sample. It has been suggested that 10 to 25 percent reduction in steel bar section due to corrosion results in serviceability failure (Comite Euro-International du Beton 1983). It is estimated that the direct cost of reinforcement corrosion to the UK economy is around £550m per year (Webster and Clark, 2000). A recent analytical survey for the Highways Agency in the UK (Parsons Brinckerhoff Ltd. 2003) showed that the most common deteriorated elements in the 294 sub-standard bridges analysed on the motorway and trunk road network were slabs, main beams and piers, deck cantilevers and parapets. Within these elements, the most common modes of failure were longitudinal flexure, transverse flexure and general shear. The use of overly conservative or inappropriate methods of analysis was reported to be responsible for the majority of the failures. This may suggest that engineers are attempting to model structural performance with insufficient knowledge of corrosion effects.

2 BACKGROUND

Reinforced concrete flexural elements are designed to fail 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. Sufficient shear reinforcement is included to ensure the shear capacity is greater than the flexural capacity. However, the long term influence of reinforcement corrosion is not taken into account at the design stage. Shear reinforcement has a lower protective concrete cover than flexural reinforcement and will normally corrode first. Premature shear failures are sudden and often with catastrophic consequences.

The aim of this paper is to investigate the influence of shear reinforcement corrosion on the strength of reinforced concrete beams. Recommendations are given to assist the engineer in extending the service life of corroded reinforced beams at the design stage and also help eliminate the possibility of sudden shear failure.

3 EXPERIMENTAL PROCEDURE

3.1 *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 Figure 1. The beams exhibited shear reinforcement corrosion only (main steel remained uncorroded). Beams were 910 mm long, with a cross-section of 100 mm x 150 mm deep. Referring to Table 1, Type 1A beams were reinforced with 2T8 main steel and 6mm mild steel shear reinforcement. Target corrosion of 0%-15% of cross sectional area was applied to the shear reinforcement of these beams in 5% increments. Two specimens were tested per corrosion increment (Table 1). Type 1B beams (Table 1) differed from Type 1A in that the main reinforcement consisted of 2T12. The shear reinforcement was again subjected to a target corrosion of 0% to 15% of cross sectional area in 5% increments, the main steel remained uncorroded (0%). Eight beams were tested in total for Type 1B as shown in Table 1.

Main reinforcement consisted of high yield (ribbed) bars with a nominal characteristic strength of 460 N/mm^2. Shear reinforcement was 6mm diameter plain round mild steel bars of nominal characteristic strength 250 N/mm^2 at a spacing of 65 mm. Cover was 50 mm to the shear reinforcement for the beams presented in this paper, as this also formed part of a larger investigation where the cover was varied. 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 main reinforcement, shrink wrap tubing was provided at the points of contact with the shear reinforcement to break the electrical circuit and hence prevent current flow to the main reinforcement during the accelerated corrosion process. Inspection of the main reinforcement at the end of the tests showed that this was an effective method of preventing accelerated corrosion of the main reinforcement.

The beams were cast in the laboratory using a concrete with target cube strength of 40 N/mm^2. Mix proportions were 1:1.5:2.9 of ordinary Portland cement: fine aggregate: coarse aggregate. Fine and coarse aggregates were oven dried at 100°C for 24 hours. 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 hours. The samples were demoulded after 1 day and cured in water at 20°C for a further 27 days (28 days in total). Electrical connections were made to the steel reinforcement and the beams were then transferred to a tank filled with a saline solution for accelerated corrosion at 28 days age.

The control specimens (0% corrosion) were tested at the age of 28 days. The corroded beams were tested at 42, 48 and 45 days age due to the time taken to reach the target corrosion of 5, 10 and 15% respectively in the shear reinforcement (Table 1). The test span of the beam was 750mm (Figure 1) with symmetrical loads applied at shear spans of 250mm.

3.2 *Accelerated corrosion process*

The shear reinforcement was subjected to an accelerated galvanostatic corrosion process in an electrolytic cell by means of a direct current multi channel power supply. The corrosion process took place in a plastic tank where a 3.5% CaCl$_2$ solution was used as the electrolyte. The direction of the current was arranged so that the main reinforcing steel and hanger bars served as the cathode and the stirrups acted as the anode. 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 bar diameter 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. Further details are available elsewhere (O'Flaherty *et al*, 2008).

4 RESULTS AND DISCUSSION

The reinforcing bars were removed from the concrete after testing, thoroughly 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 reinforcement. Serious cross-section loss occurred at higher percentages of corrosion.

Test results are given in Table 2. Beams are identified by the target amount of corrosion in the shear links reinforcement, for example, 2T8/0+12R6/0 signifies the beam is reinforced with 2T8 main steel exposed to 0% target corrosion and 12R6 links with 0% target corrosion (i.e. control beam, col. 2, Table 2). Main reinforcement steel remained uncorroded (Table 2, col. 3). Actual shear reinforcement corrosion is given in col. 4, Table 2 and varies from the target corrosion in places due to the difficulties encountered in using this technique. However, measured corrosion values are used in the analysis and not the target values. The failure load (P$_{ult}$) is given in col. 5. Despite corrosion being applied to the

shear reinforcement only, the majority of the beams failed in flexure in a ductile manner except both specimens for beams 2T8/0+12R6/15, Type A (Table 2, col. 6).

The tensile moment of resistance, $M_{t(0)}$, was determined for the control beams (0% corrosion on main and shear reinforcement) corresponding to the ultimate loads in Table 2 (col. 5) using the expression $M_{t(0)} = (P_{ult}/2)(0.25m)$. The compressive moment of resistance, M_c, was calculated for the same beams using the equation

$$M_c = [(0.234 \, f_{cu} b d^2) + f_y' A_s' (d - d')]10^{-6}$$

(O'Flaherty et al, 2008).

The ratio $M_{t(0)}/ M_c$, representing the degree of under-reinforcement of the control (uncorroded) beams was then obtained. In order to determine the influence of shear reinforcement corrosion on the structural performance, the tensile moment at failure due to increasing levels of shear reinforcement corrosion ($M_{t(ShearCorr)}$) was also obtained from $M_{t(ShearCorr)}=$ $0.25(P_{ult}/2)$ for P_{ult} values given in Tables 2, col. 5. The compressive moment of resistance, M_c, remains constant for each type of beam (control and corroded) as it is not affected by the degree of corrosion of the shear reinforcement but is based on the properties of the concrete and compression steel in the compression zone i.e. hanger bars. The relationship between $M_{t(ShearCorr)}/M_c$ and the percentage of shear reinforcement corrosion is shown in Figure 2. The $M_{t(ShearCorr)}/M_c$ value at 0% corrosion in Figure 2 represents the degree of under-reinforcement $[M_{t(0)}/ M_c]$ for the control beams.

The relationships in Figure 2 generally show a linear decrease in $M_{t(ShearCorr)}/M_c$ with increasing percentages of shear reinforcement corrosion. The best fit linear equation for each series of beams is tabulated along with the coefficient of correlation (R^2). A very satisfactory coefficient of correlation exists for both tests (> 0.91). Shear failure was evident only at higher degrees of shear reinforcement corrosion for Type 1A (> 18.7%, shown highlighted in Figure 2).

Referring to Figure 2, beam Types 1B, reinforced with 12mm uncorroded main steel, exhibit a higher $M_{t(0)}/ M_c$ ratio (low degree of under-reinforcement) and suffer rapid flexural strength loss due to shear reinforcement corrosion (slope: -0.39). Beam Types 1A, reinforced with 8mm uncorroded main steel have a lower $M_{t(0)}/ M_c$ ratio (i.e. they are more under-reinforced than beam types 1B) and suffer less-rapid flexural strength loss due to shear reinforcement corrosion (slope: -4.80). Therefore, despite beam types A and B both containing corrosion free main steel, the corroded shear reinforcement has an influence on flexural strength. Flexural strength loss due to shear reinforcement corrosion is more rapid in beams with higher $M_{t(0)}/ M_c$.

A similar conclusion was made by the authors (O'Flaherty et al, 2007) for beams undergoing main (tensile) steel reinforcement corrosion only with un-

corroded shear reinforcement. For enhanced performance, beams should be designed with lower $M_{t(0)}/ M_c$ ratios (i.e. higher degrees of under-reinforcement) and guidelines for achieving this are given elsewhere (O'Flaherty, et al, 2007).

The data presented in Figure 2, therefore, indicates that the shear strength of deteriorated reinforced concrete beams is compromised only at high levels of shear reinforcement corrosion (> 18.7% in this investigation). Shear reinforcement corrosion below this level still resulted in flexural failure.

5 CONCLUSIONS

The main conclusions from the results reported in this paper are as follows:
− The predominant failure mode for under-reinforced concrete beams exhibiting low degrees of shear reinforcement corrosion is flexural (<18.7% in this investigation)
− Shear failure occurred only at higher degrees of shear reinforcement corrosion (>18.7% in this investigation)
− Beams with lower $M_{t(0)}/M_c$ ratios suffered a lower rate of flexural strength loss when corrosion was present in the shear reinforcement. Therefore, the recommendation is to design beams with lower $M_{t(0)}/M_c$ for enhanced residual flexural strength but the shear capacity should be increased to guard against the risk of sudden shear failure at higher levels of shear reinforcement corrosion

6 ACKNOWLEDGEMENTS

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