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Influence of steel reinforcement corrosion on the stiffness of simply supported concrete beams

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ABSTRACT: The in-service performance of reinforced concrete beams can be severely affected through corrosion of the steel reinforcement when it becomes subjected to harsh corrosive environments containing chlorides and carbon dioxide. In such instances, corrosion is likely to occur in the steel reinforcement, with the expansive nature of the corrosion products likely to induce cracking and spalling of the concrete. A loss of structural integrity (stiffness) will occur and this can severely influence the serviceability of the member.
The purpose of this paper is to investigate the relationship between degree of corrosion and loss of stiffness in corrosion damaged under-reinforced concrete beams. Beams (100mm x 150mm cross section) were subjected to accelerated corrosion in the laboratory and subsequently tested in flexure to failure. The paper reports on the results of these tests and relates the degree of corrosion in the main steel to the percentage loss in stiffness in the concrete beams.

1 INTRODUCTION

The increasing age of the highway bridge stock throughout Europe has led to increased deterioration in existing structures. Surveys have indicated that the main reasons for deterioration, besides normal wear and tear, are the increasing weights and volume of traffic using the road network, and adverse environment conditions such as exposure to chlorides and freeze-thaw attack. The magnitude of the capital investment in European bridge stock requires that effective maintenance is provided to ensure that the bridges are kept in safe service at optimum cost. Reliable assessment methods are urgently required to assist the bridge engineer to evaluate the residual capacity of deteriorated elements.

When steel reinforcement corrodes, tensile stresses are generated in the concrete as a result of the expansive corrosion products. Since concrete can endure much less tensile stress than compressive stress, tensile cracks are readily nucleated and propagated as a result. The development of corrosion products along the bar surface may affect the failure mode and ultimate strength of flexural members due to two causes: firstly, due to a reduction in the degree of bar confinement caused by an opening of longitudinal cracks along the reinforcement and, secondly, due to significant changes at the steel-concrete interface caused by changes in the surface conditions of the reinforcing steel (Al-Sulaimani et al. 1990). In all corrosion cases, repair is necessary to increase the service life of the member (Mangat & O'Flaherty 1999, 2000) and it is estimated to cost approximately €1.5bn in Europe each year (Davies 1996).

2 BACKGROUND

In this investigation, the 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, in reality, localised pitting corrosion extends to many sites resulting in extensive and relatively uniform levels of corrosion covering large areas of reinforcement (Mangat & Elgarf 1999). In these tests, the aim was to subject the steel reinforcement to general corrosion.

Concrete structures which have suffered from corrosion to the steel require maintenance to increase their service life. However, it is difficult to make a decision as to when repair is required without knowing how much the capacity has been reduced due to deterioration. Shear reinforcement normally corrodes first as it is nearer to the concrete surface but main steel, too, can be affected simultaneously if the corrosion inducing elements penetrate deep into the concrete. One of the objectives of the laboratory work was to evaluate the influence of corrosion to the main and shear reinforcement on the stiffness of simply supported beams.

3 EXPERIMENTAL PROCEDURE

A total of twenty reinforced concrete beams were tested to examine the influence of reinforcement corrosion on the stiffness. Details of test specimens are given in Figure 1 and Table 1. Beams were 910 mm long with a cross-section of 100 mm x 150 mm deep. Span was 750 mm. 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 are identified as follows (Table 1): e.g. 2T8/5+12D6/10 gives the number, type, and diameter of the main steel in mm/target percentage of corrosion + number, type, and diameter of the links in mm/target corrosion. 2T8/0+12D6/0 were tested without corrosion to serve as control specimens (Table 1). Cover throughout was 50 mm to the shear reinforcement and 56 mm to the main steel.

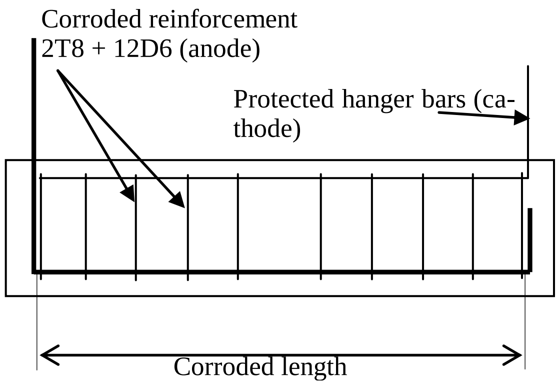

Figure 1. Beam specimens.

Table 1. Variables in test programme.

Beam ID (main steel /shear steel)	Target Corrosion (main and shear) (%)	No. of specimens
2T8+12D6	0	2
	5	6
	10	6
	15	6

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 a nominal yield strength of 250 N/mm^2. Shear reinforcement was spaced at 65 mm centres for 56 mm cover to main steel reinforcement. 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 steel reinforcement was weighed before casting to enable the actual percentage corrosion to be calculated at a later stage. Corrosion of the top reinforcement (hanger bars) was prevented by providing shrink wrap tubing at the points of contact with the shear steel reinforcement to break the electrical circuit and hence prevent current flow during the accelerated corrosion process. Corrosion of the main and shear reinforcement was conducted simultaneously using a multi channel power supply. Shrink wrap tubing was again provided at the points of contact between the shear and main steel to enable each bar to be corroded separately using the multi channel power supply.

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. A 3.5% $CaCl_2$ solution was used as the electrolyte. The direction of the current was arranged so that the main and shear reinforcing steel served as the anode and the hanger bars and additional copper, which was added to the tank, acted as the cathode.

A constant current density of 1 mA/cm^2 was passed through the main and shear 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 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 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.

The rebars were retrieved from the concrete after the testing program and reweighed to establish the precise level of corrosion.

4 RESULTS AND DISCUSSION

The load-deflection curves for the beams under investigation are given in Figure 2 and include 18 corroded beams and the average of two control beams. The effect of the degree of corrosion on the overall stiffness of corroded flexural member is tabulated in Table 2 and is taken from the data presented in Figure 2. Referring to Table 2, beams are identified in column 1. The actual percentage of reinforcement corrosion, measured as described in Section 3, is given in column 2. Column 3, Table 2, shows the stiffness of the beams after testing in the laboratory. The stiffness was determined via the straight line portion of the graph from the origin in Figure 2. The percentage difference in loss of stiffness between the control beam (2T8/0+12D6) and the corroded beams is given in column 4.

To gain a better understanding of the performance of the beams when both main and shear reinforcement are subjected to corrosion, Figure 3 shows the effect of corrosion on the stiffness. Referring to Figure 3, the beam stiffness, obtained from the slope of the load-deflection curves in Figure 2, is plotted against the degree of main reinforcement corrosion only, as this is considered the main parameter with respect to loss of stiffness (the shear reinforcement was also corroded in the test as this better represents actual conditions). Generally, an increase in corrosion leads to a decrease in stiffness. At very high levels of corrosion (>25%), the mode of failure changes from flexure to shear.

Table 2. Experimental data.

1	2	3	4
Beam ID	Actual Main Bar and Shear Links Corrosion %	Stiffness kN/mm	% difference in stiffness
2T8/0+12D6*	0; 0	9.3	-
2T8/5+12D6/5	6.5; 4.3	9.5	2
2T8/5+12D6/5	2.9; 3.0	8.5	-9
2T8/10+12D6/5	4.8; 5.6	10.0	8
2T8/10+12D6/5	5.3; 6.5	9.1	-2
2T8/15+12D6/5**	25.7; 24.7	4.5	-52
2T8/15+12D6/5**	25.7; 27.6	4.3	-54
2T8/5+12D6/10	8.3; 6.3	6.1	-34
2T8/5+12D6/10	3.1; 3.8	7.7	-17
2T8/10+12D6/10	4.1; 3.9	8.9	-4
2T8/10+12D6/10	4.7; 8.2	8.1	-13
2T8/15+12D6/10	7.8; 4.5	6.8	-27
2T8/15+12D6/10	8.8; 9.1	6.8	-27
2T8/5+12D6/15	10.7; 12.0	5.3	-43
2T8/5+12D6/15	11.8; 14.2	6.8	-27
2T8/10+12D6/15	6.7; 9.4	11.4	23
2T8/10+12D6/15	7.9; 8.2	6.9	-26
2T8/15+12D6/15	6.9; 7.4	9.0	-3
2T8/15+12D6/15	6.9; 4.0	8.1	-13

* average of two specimens ** failed in shear

4.1 Influence of corrosion on stiffness

Referring to Table 2, col. 4, the percentage difference in stiffness between the control beam (2T8/0+12D6) and the corroded beams is given. This data is plotted against the degree of corrosion of the main steel in Figure 4 but excludes three positive data point from column 4 which are considered to be erroneous - 2T8/5+12D6/5, 2T8/10+12D6/5 and 2T8/10+12D6/15 and the two beams which failed in shear (2T8/15+12D6/5, 2T8/15+12D6/5). Referring to Figure 4, a linear correlation is evident with R^2=0.58. The decrease in stiffness is equal to 2.83 times the degree of corrosion of the main steel.

The failure load of the control beam was 41.4 kN (Figure 2). However, if the service load is estimated as 40% of the ultimate (control) load to account for factors of safety included in the design (dead load: 1.4; live load: 1.6; materials: 1.5 for concrete and 1.05 for steel), the service load would be approximately 16.6kN. Therefore, the deflection of this beam in service, assuming a stiffness of 9.3 kN/mm (Table 2) and a service load of 16.6 kN would be 1.8 mm (deflection = 16.6 kN / 9.3 kN/mm). However, the allowable deflection is span/250 (British Standards Institution), or 3mm for these beams meaning only a further 1.2mm of deflection is required before the beam is outside of allowable deflection limits. To estimate the percentage of corrosion required to cause this increase in deflection, the service load is assumed to remain at 16.6 kN but the deflection is assumed to reach its maximum allowable limit of 3mm. Therefore, since stiffness = service load / deflection, stiffness = 16.6 kN / 3mm or 5.53 kN/mm.

This means that the stiffness of the beam would have to decrease to 5.53 kN/mm before it exceeds serviceability limits, a decrease of 40% ((9.3 - 5.53)/9.3). Applying this percentage loss of stiffness to Figure 3, main steel corrosion would have to exceed 14% before deflection criteria becomes an issue (40%/2.83).

5 CONCLUSIONS

The main conclusions from the results reported in this paper are as follows and apply within the limit of the parameters covered by the test data in the paper.

- reinforced concrete beams show a loss in stiffness with increasing corrosion of the main and shear steel reinforcement;
- an equation linking the decrease in stiffness to main steel corrosion was determined as follows:
  - % decrease in stiffness = 2.83 x degree of main steel corrosion

– main steel corrosion would have to exceed 14% before deflection criteria are exceeded.

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