Fatigue strength of fibre-reinforced-polymer-repaired beams subjected to mild corrosion[1]

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Abstract: Infrastructure corrosion is an expensive problem worldwide. In the case of reinforced concrete (RC) structures, corrosion reduces the steel cross sectional area and thus decreases the capacity of the corroded RC members. The expansion of the corroded steel also induces tensile stresses in the concrete causing the concrete cover to crack and spall, thus reducing the bond capacity between concrete and steel. This paper reports on a research program conducted at the University of Waterloo that studied the effect of corrosion on flexural and bond fatigue strength. The effect of the addition of fibre-reinforced-polymer (FRP) sheets on the fatigue life of corroded RC beams was also assessed. Eighteen beams (152 mm × 254 mm × 2000 mm) were tested in two groups, with each group consisting of three sets of tests. Group F was designed to study the fatigue flexural behaviour; the repaired beams in this group were strengthened with a flexural FRP sheet along their tension side and confined by intermittent U-shaped FRP sheets along their length. Group B was designed to study the fatigue bond behaviour; hence, the repaired beams in this group were confined with U-shaped FRP sheets in the anchorage zone. The variables in each group were the percentage of corrosion (0% and 5% theoretical mass loss), the load range, and the use or omission of a FRP repair method. Results showed that a mild level of corrosion (5% theoretical mass loss) caused on average 10% and 20% reductions in flexural and bond fatigue strength, respectively. Strengthening the corroded beams with FRP sheets enhanced the fatigue behaviour of the beams. In both groups, the fatigue strength was on average 15% higher than that of the corroded unrepaired beams.

Key words: corrosion, fibre-reinforced-polymer (FRP) sheets, fatigue strength, steel–concrete bond, flexural performance, durability.

Résumé : La corrosion des infrastructures est l'un des problèmes mondiaux les plus coûteux. Dans le cas des structures en béton armé, la corrosion réduit la superficie de la section transversale, réduisant ainsi la capacité des éléments de charpente en béton armé corrodé. L'expansion de l'acier corrodé induit des contraintes en tension dans le béton, engendrant la fissuration et l'éclatement de l'enveloppe de béton, réduisant l'adhérence entre le béton et l'acier. Le présent article fait le compte rendu d'un programme de recherche à l'Université de Waterloo qui examinait l'effet de la corrosion sur la résistance à la fatigue en flexion et de l'adhérence. L'effet de l'ajout de feuilles de polymères renforcés de fibres (PRF) sur la longévité à la fatigue des poutres en béton armé corrodées a également été évalué. Dix-huit (18) poutres (152 mm × 254 mm × 2000 mm) ont été mises à l'épreuve en deux groupes, chaque groupe comportant trois ensembles d'essais. Le groupe F a été conçu pour étudier le comportement à la fatigue en flexion; les poutres réparées dans ce groupe ont été renforcées par une feuille de PRF en flexion le long de leur côté en tension et confinées par des feuilles de PRF en U placées de manière intermittente sur toute la longueur. Le groupe B a été conçu pour étudier le comportement à la fatigue de l'adhérence; ainsi, les poutres réparées de ce groupe ont été renforcées par une feuille de PRF en U placée dans la zone d'ancrage. Les variables de chaque groupe étaient le pourcentage de corrosion (de 0 à 5 % de perte théorique de masse), la plage de charge et l'utilisation ou l'omission d'une méthode de réparation impliquant des PRF. Les résultats montrent qu'un niveau faible de corrosion (5 % de perte théorique de masse) a causé des réductions moyennes de 10 à 20 % en résistance à la fatigue en flexion et de l'adhérence respectivement. Le renforcement des poutres corrodées par des feuilles de PRF a amélioré le comportement à la fatigue des poutres. Dans les deux groupes, la résistance à la fatigue était de 15 % plus élevée en moyenne que dans les poutres corrodées non réparées.

Mots-clés : corrosion, polymère renforcé de fibres (PRF), résistance à la fatigue, adhérence acier-béton, rendement en flexion, durabilité.

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Written discussion of this article is welcomed and will be received by the Editor until 31 July 2007.

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Introduction

Fibre-reinforced polymers (FRP) have recently emerged as an alternative to traditional methods for the repair and rehabilitation of concrete structures. They are of interest to rehabilitation engineers because of their high strength to weight ratio, high fatigue resistance, and corrosion resistance (ACI 1996, 2002).

Corrosion is considered to be the main durability problem in reinforced concrete structures. Several methods and (or) materials have been used to repair corroded structures. The recent increase in the use of FRP encouraged the Intelligent Sensing for Innovative Structures (ISIS) Canada network to sponsor studies of the viability of this material as a new repair and strengthening method for corroded reinforced concrete structures. A University of Waterloo (UW) research group has been working in this area for several years (Badawi and Soudki 2004; Craig and Soudki 2005; El Maaddawy et al. 2004, 2005*a*, 2005*b*; Masoud et al. 2004, 2001; Soudki and Sherwood 2003, 2000).

The majority of this research has been focused on the performance of FRP repaired corroded beams under monotonic loading, but structures such as bridges and marine constructions are subjected to repeated loading rather than monotonic loading. Repeated loading can cause the fatigue failure of structural elements, even if the applied load is well below the static capacity of those elements (ACI 2001*a*). At the same time these structures are vulnerable to corrosion from salt, either from atmospheric exposure (marine structures) or from de-icing agents used in the winter season on bridges. One of the direct consequences of corrosion is a reduction of the cross-sectional area of the reinforcing steel. In the case of localized corrosion, this steel reduction takes the form of pits (pitting corrosion) in the reinforcing steel that can dramatically reduce the fatigue strength of structures because the pits act as stress raisers (Masoud et al. 2004). In addition, the volume expansion of corroded steel leads to tensile stresses and longitudinal cracking of the concrete along its reinforcing bars (bond-splitting cracks), which reduces the bond fatigue strength of structures (ACI 2001*b*).

This paper presents experimental results obtained in a research program that examined the effect of strengthening by FRP sheets on the flexural and bond fatigue strength of beams subjected to mild corrosion. In the case of flexural fatigue, the beams were corroded in their middle third, whereas the bond beams were corroded in the anchorage zone (shear zone). It should be noted that the cross sectional dimensions, number, and size of the tension steel bars of the beams reported in this paper were kept the same as those used by Masoud et al. (2004), Badawi and Soudki (2004), El Maaddawy et al. (2004), and Craig and Soudki (2005) for comparison purposes.

Experimental program

Eighteen reinforced concrete beams (152 mm × 254 mm × 2000 mm) were constructed and tested. The variables of the study were failure mode (flexure or bond), corrosion damage (0% or 5% theoretical mass loss), the load range applied, and use or omission of FRP for beam strengthening. The beams were divided into two main groups F and B. The aim of tests in group F was to study the effect of corrosion and FRP repair on the fatigue flexural strength of RC beams. Therefore, the beams in this group were corroded in the constant moment region (the anchorage zone was kept uncorroded) and strengthened in flexure by FRP sheets with their fibres parallel to the reinforcing bar. The flexural FRP sheet was placed along the tension side of the beam and confined with U-shaped FRP sheets. The objective of the group B tests was to study the fatigue bond behaviour of corroded beams. Hence, beams for this group of tests were corroded in the anchorage zone (flexural zone was kept uncorroded) and were wrapped with U-shape FRP sheets with their fibres transverse to the longitudinal axis of the beam to increase the concrete confinement in the bond-critical zone. Each group was further divided into three sets: control (C), corrosion damaged (CD), and corroded repaired (CR). The complete test matrix is shown in Table 1.

Details of flexural beams (group F)

Group F beams were reinforced in tension with two 15M deformed bars. Shear reinforcement in the form of 8 mm hoop stirrups placed at a spacing of 100 mm in the shear zone and a spacing of 250 mm in the flexure zone was provided. Two 8 mm smooth bars were placed in the compression zone. Figure 1 shows the details of group F specimens.

Details of bond beams (group B)

Bond beam specimens were reinforced with two 20M bars. Stirrups in the form of 8 mm smooth bars were placed at a spacing of 125 mm and two 8 mm smooth bars were used to hold them in place in the compression zone. To ensure bond failure in the anchorage length at the ends of the beam, a low-density polyethylene tube was used to cover the reinforcing bars in the middle portion of the beam, thus creating an unbonded zone. To promote a bond failure, the anchorage length was limited to 250 mm, which is less than the development length required by the Canadian Standards Association (CSA) standard CAN/CSA-A23.3-94 (CSA 1994). Details of the bond specimens are shown in Fig. 2.

Induced corrosion

The level of corrosion damage in this study was chosen to correspond to a 5% theoretical mass loss. The theoretical mass loss was calculated using Faraday's law $m = Ita/ZF$, where m is the mass loss (g), I is the corrosion current (A), t is the corrosion time (s), a is the atomic weight (56 g for iron), Z is the valence of the corroding metal (2 for iron), and F is Faraday's constant (96 500 A.s).

To attain the required level of corrosion in a reasonable time, an accelerated corrosion technique was used. In this technique the corrosion process is activated by chloride salts and accelerated by electrical polarization of the steel reinforcement. To ensure that the corrosion in the tension reinforcement is activated in the required zone, salt was added in the amount of 2.15% chlorides by weight of cement to the concrete mix placed in the middle zone (over a region of 700 mm length × 100 mm depth) for specimens in group F and in the shear zone (over a region of 300 mm length × 100 mm depth) for specimens in group B (Fig. 3). This amount of chlorides was sufficient to depassivate the reinforcement and initiate corrosion, since the chloride concentration was larger than the corrosion threshold value (ACI

Table 1. Test matrix test results.

Group	Set[a]	Beam notation[a]	Corrosion (%)	CFRP[b]	Min. load (kN)	Max load (kN)	Load range[c] (%)	Fatigue life
F	C	F-C-72	0	No	10.5	105.4	72	120 800
		F-C-57				85.6	57	311 748
		F-C-47				72.5	47	681 089
	CD	F-CD-72	5	No	10.5	105.4	72	73 744
		F-CD-57				85.6	57	198 669
		F-CD-47				72.5	47	399 448
	CR	F-CR-72	5	Yes	10.5	105.4	72	114 604
		F-CR-57				85.6	57	386 648
		F-CR-47				72.5	47	827 956
B	C	B-C-53	0	No	10	63	53	53 969
		B-C-55				65	55	34 202
		B-C-65				75	65	3 080
	CD	B-CD-40	5	No	10	50	40	222 263
		B-CD-45				55	45	245 318
		B-CD-55				65	55	340
	CR	B-CR-50	5	Yes	14	83	50	142 208
		B-CR-55				90	55	17 731
		B-CR-65				104	65	113

[a] X-Y-Z: X is F (flexure) or B (bond); Y is C (control), CD (corrosion damaged), or CR (corrosion repaired); Z is percentage of load range applied.
[b] CFRP, carbon-fibre-reinforced polymer.
[c] Load range is a percentage of the static load capacity of the un-corroded beams.

Fig. 1. Longitudinal and cross-sectional details for group F beam specimens, all dimension in millimetres.

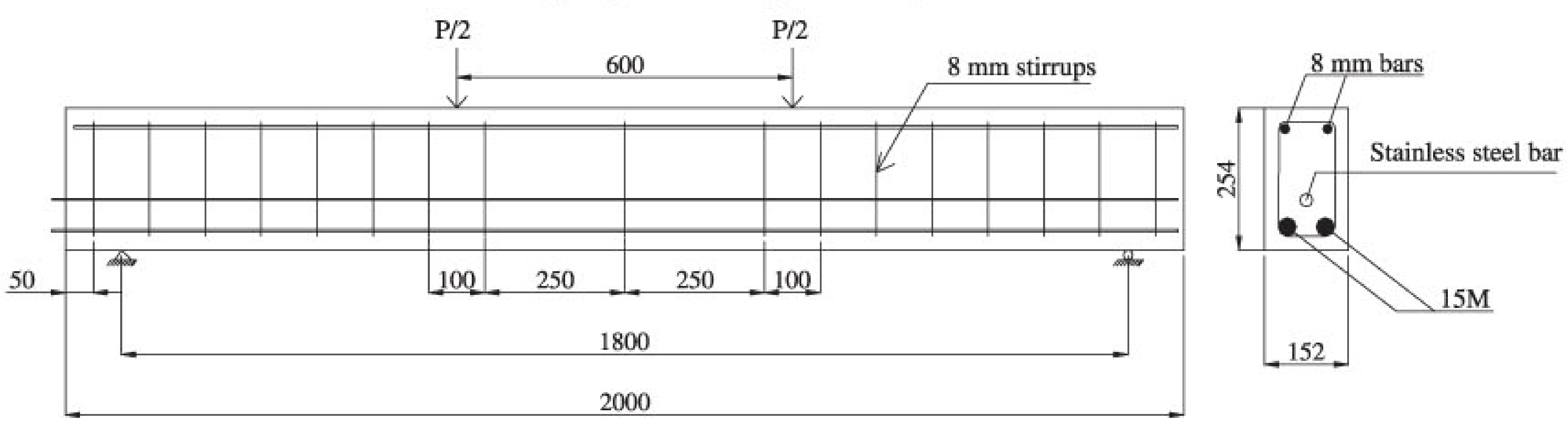
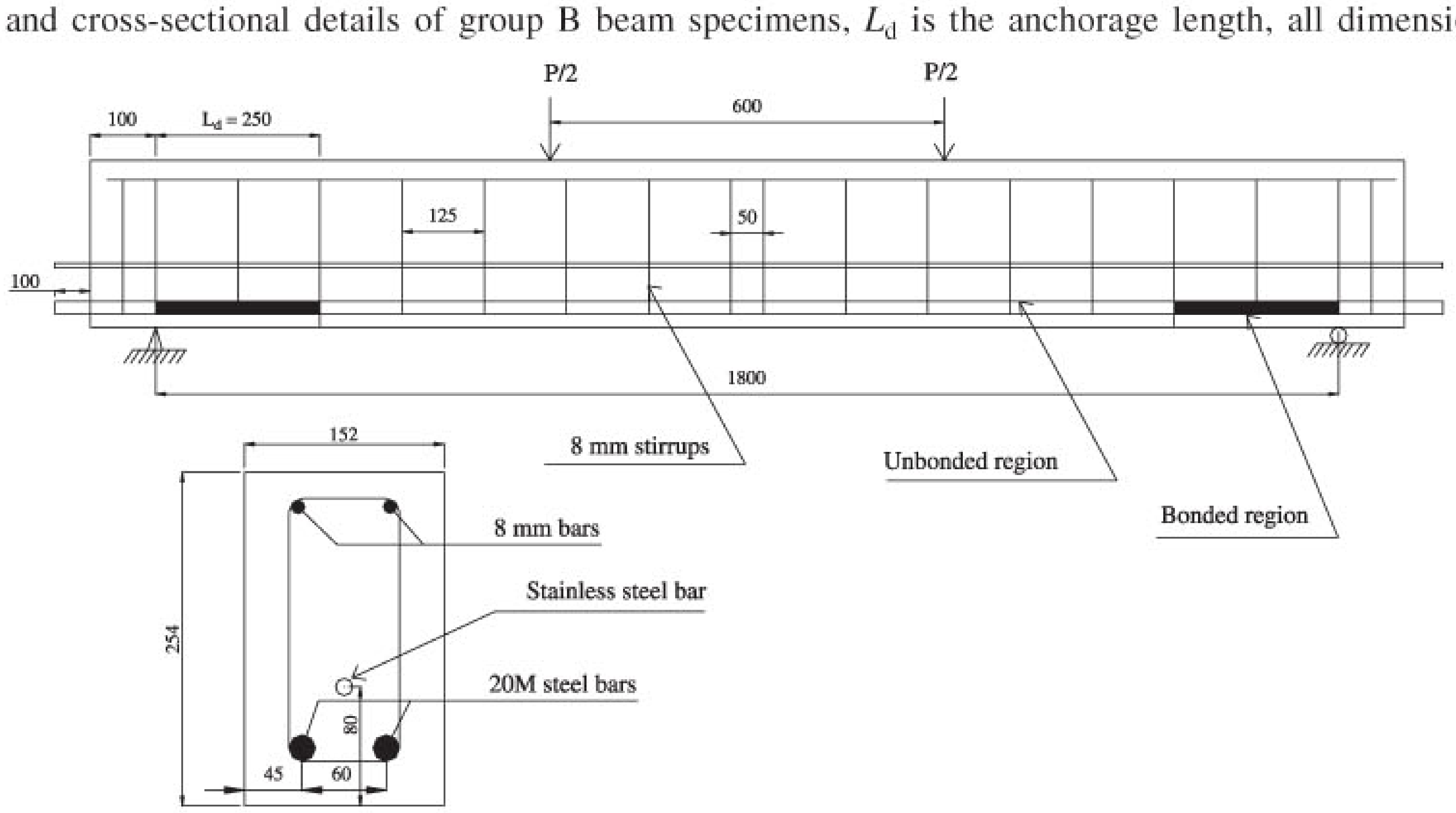
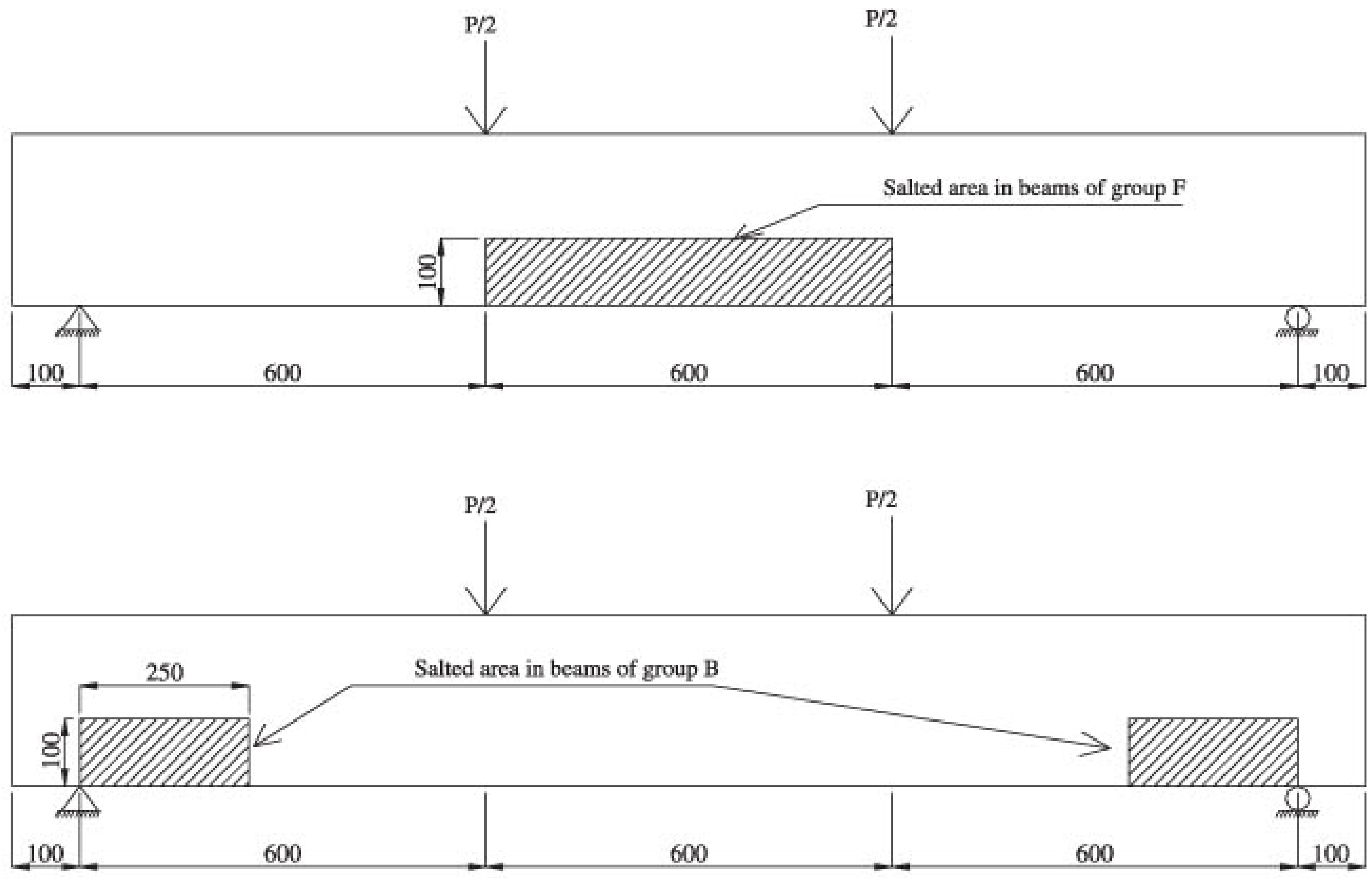


Fig. 2. Longitudinal and cross-sectional details of group B beam specimens, L_d is the anchorage length, all dimensions in millimetres.



2001*b*). No salt was added to the concrete mix of the rest of the specimen. Stirrups placed in the salted zone were epoxy coated to protect them from corrosion. In addition, one 8 mm stainless steel tube (type 304) was placed in the bottom third of the beam (80 mm from the bottom) to be used as a cathode (Figs. 1 and 2).

Fig. 3. Corroded zones for group F and B beam specimens, all dimensions in millimetres.



A constant current density of 150 μA/cm^2 was supplied, as recommended in an earlier study (El Maadawy and Soudki 2003), to the specimens. To ensure a constant current in the beams, they were connected in series, and the direction of the electrical current was adjusted so that the tension steel served as the anode and the stainless steel served as the cathode. The corrosion process started no earlier than 28 d after casting. The beams were kept in a special chamber, where the required moisture and oxygen for the corrosion process were provided constantly, for 51 d (calculation based on Faraday's law).

Fibre-reinforced polymer repair

Both groups were strengthened by carbon-fibre-reinforced polymer (CFRP). For group F a 150 mm wide × 1800 mm length CFRP sheet was used for flexural strengthening along the tension bottom of the beam. The fibre orientation for this flexural FRP sheet was parallel to the steel reinforcement. This sheet was confined by intermittent U-wraps (100 and 150 mm wide) along the length of the beam. The fibre orientation of the intermittent FRP U-wraps was perpendicular to the steel reinforcement. Details of the wrapping for group F are shown in Fig. 4. The CFRP system used to repair group F (commercially known as SIKWRAP 103C) was kept the same as that used by Masoud et al. (2004) for comparison purposes.

For group B, a one layer 250 mm wide U-shaped CFRP flexible sheet was placed transversely in the bonded region so that the fibre orientation was perpendicular to the steel reinforcement. The CFRP system to repair this group is commercially known as SIKAWRAP 230C. It was chosen because its stiffness was less than that used for group F to ensure bond failure.

In both groups the CFRP repair was applied after the corrosion process had halted. The application procedure followed was in accordance with the manufacturer's guidelines. The CFRP repair was left to cure for at least 7 d prior to testing.

Material properties

All the flexural steel used in this study was Grade 400, with a specified average yield strength of 440 MPa. The 8 mm smooth bars had a yield strength of 340 MPa. The concrete was supplied by a local ready mix plant and had a compressive strength ranging between 30 and 40 MPa at the time of beam testing. The CFRP sheets used in group F were SIKAWRAP HEX 103C laminated with SIKADUR HEX 300. For group B, SIKAWRAP 230C with SIKADUR 330 epoxy were used. Table 2 summarizes the mechanical properties, as supplied by the manufacturer, of the cured CFRP systems.

Fatigue loading tests

Figure 5 shows the test setup. Both groups were tested using the same procedure, and the specimens were tested in four point bending under load control. The loading system produced a constant moment region in the middle third of the beam specimen. Load was applied manually until the desired maximum load was reached, and was then decreased to the mean load. Thereafter a sine wave load cycle was applied about the mean load, using a MTS 407 controller with a frequency between 1.5 and 3 Hz. The minimum load for group F was 8% of the static strength of the beams, while for group B it was 10%. The maximum load levels were chosen so that fatigue failure occurred at lives between 10^3 and 10^6 cycles.

A linear variable displacement transducer (LVDT) was used to measure the deflection at midspan. In group B, additional LVDTs were mounted to the end of the steel bars (which extended outside the concrete beam) to measure the free-end slip of the bar relative to the concrete. Load was measured using a 333 kN (75 kips) load cell, and all the readings were saved on a computer through a data acquisition system.

Experimental results

The static load capacity of the uncorroded beams was determined experimentally for both groups F and B. The group F uncorroded unrepaired static beam failed in flexure (steel yielding followed by concrete crushing) and sustained a maximum load of 132 kN. The uncorroded beam specimens in group B failed by concrete splitting in the anchorage

Fig. 4. Repair scheme for group F beam specimens, all dimensions in millimetres.

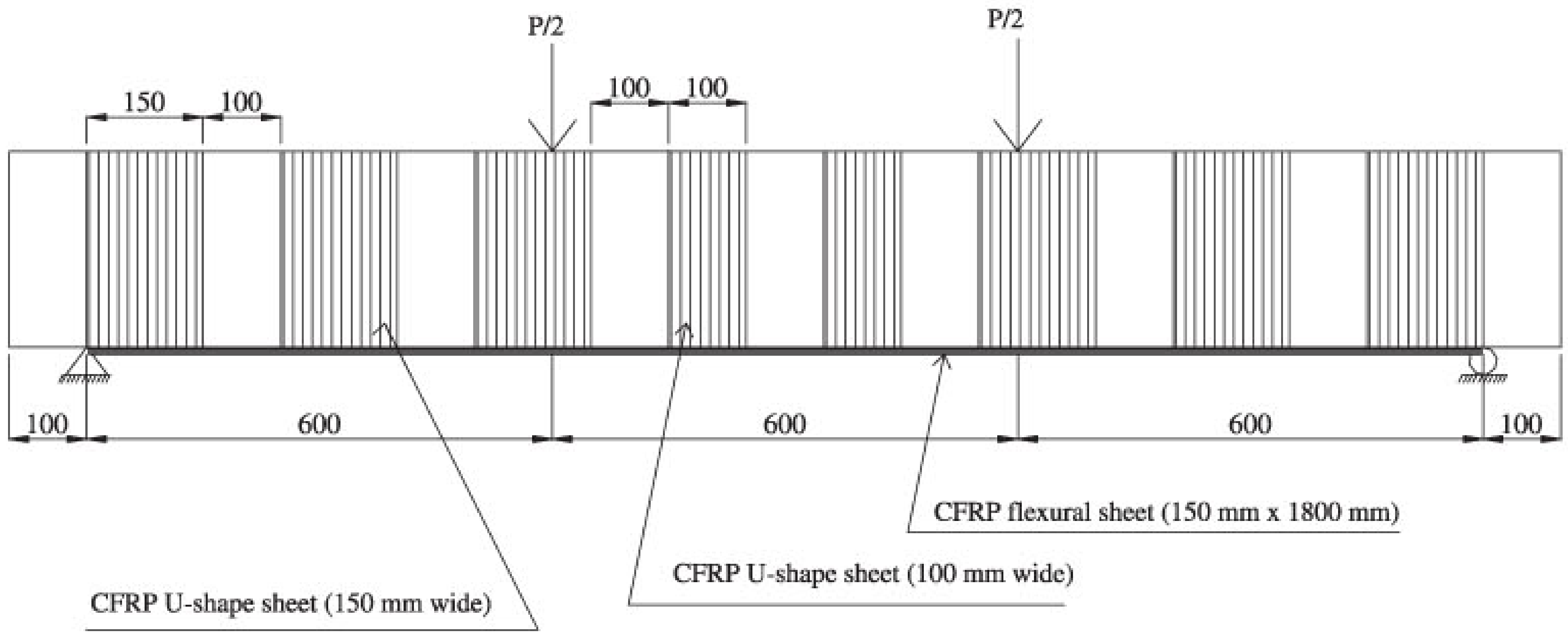
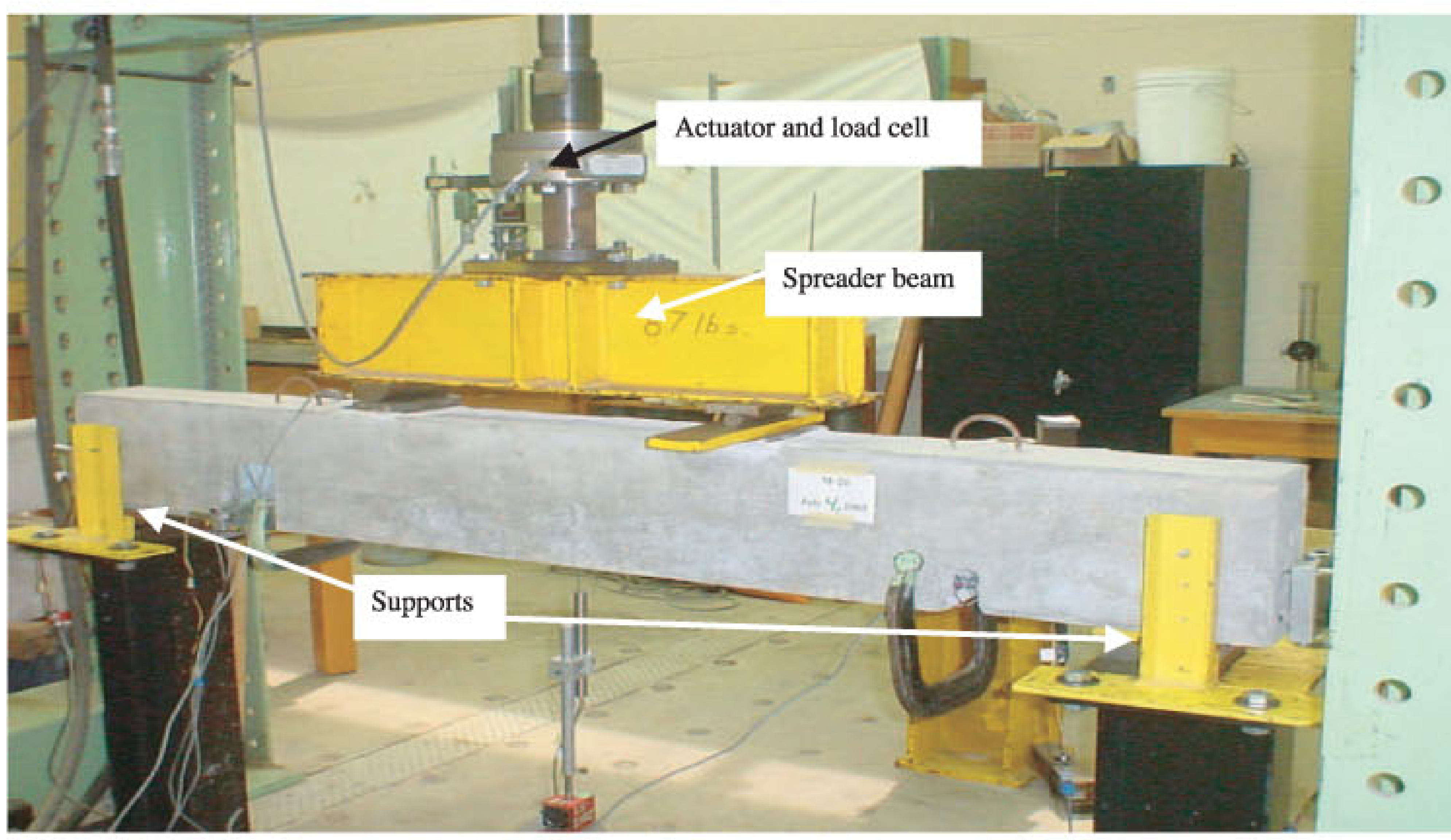


Table 2. Carbon-fibre-reinforced polymer tensile properties (as supplied by the manufacturer).

Type	Ultimate strength (MPa)	Modulus of elasticity (MPa)	Elongation at break (%)	Thickness (mm)
SIKAWRAP 103C	717	65 087	0.98	1.016
SIKAWRAP 230 C	715	61 012	1.09	0.381

Fig. 5. Test setup.



zone. The maximum loads were 100 and 138 kN for the unwrapped and wrapped beams, respectively.

All specimens tested in this study failed in fatigue. In group F, the failure was in flexural fatigue, which is characterized by rupture of the steel reinforcement (Fig. 6). In group B, the failure was by concrete splitting along the bar in the anchorage zone (Fig. 7). Table 1 summarizes the results for all specimens tested in this study.

All specimens in group F had fatigue cracks that started in the tension steel reinforcement, near one of the ribs in the constant moment region. The cracks then propagated under fatigue loading until the tension bar ruptured. In sets F-C and F-CD (control and corrosion damaged unrepaired beams, respectively), rupture of the first bar caused the beam to fail. However, in set F-CR (corroded and repaired) the rupture of one of the steel bars resulted in redistribution of the stresses, by the transmission of additional load to the CFRP flexural sheets. The beam continued to hold the maximum load until the second bar failure, and rupture of the CFRP flexural sheet at the same time.

The load–deflection hysteresis curves of the three sets of tests in group F exhibited similar behaviour (Fig. 8). For all beams, the stiffness reduced with life because of concrete softening, and the deflection increased with increasing life. The presence of CFRP sheets reduced the increase in deflection of the repaired beams, in comparison to the unrepaired beams (Fig. 9).

Figure 10 shows the variation of the fatigue life with the load range applied on each beam. The load range was expressed as percentage of the static capacity of the uncorroded unrepaired beam. Figure 10 clearly shows that for all specimen groups, the measured fatigue life data followed perfectly

Fig. 6. Typical bar rupture of beams in group F (beam F-CD-72).

Fig. 7. Typical bond failure of beams in group B (beam B-C-55).

linear trend lines in a log-scale. As the load range was reduced, the fatigue life increased (Table 1). The fatigue strength in set F-CD (corroded and unrepaired) flexural beams was reduced by approximately 10% compared with those of set F-C. Repairing the 5% corroded beams with CFRP (set F-CR) increased the fatigue strength of the beams by 10% at 100 000 cycles and by 18% at 750 000 cycles compared with the corroded unrepaired beams (set F-CD). It should be noted that repairs that used CFRP sheets restored the fatigue flexural strength of the corroded beams to a level equal to that of the original uncorroded beams.

All the bond beams tested in group B failed by concrete splitting along the reinforcement bar. For sets B-C and B-CD (control and corroded unwrapped beams, respectively) longitudinal cracks initiated along the anchored zone of the reinforcing bars during the first few hundred cycles, and continued to grow in width and length until failure. In set CR, because of the presence of the CFRP sheets, it was not possible to see the crack patterns and crack propagation during cycling. After the failure of the specimens the CFRP sheets were removed, and an inspection of the cracks under the CFRP sheets showed that cracks were longitudinal. This indicated that failure was also accompanied by bond splitting. However, the width of these cracks was significantly smaller than the ones observed in the unrepaired beams.

Figures 11 and 12 show the variation of the free end slip with the number of cycles for corroded unwrapped and corroded wrapped beams, respectively. For corroded unwrapped

Fig. 8. Typical load–deflection curves for specimens in group F (beam F-CD-57). Note: ◆, cycle 1; ●, cycle 100 000; ■, cycle 198 000.

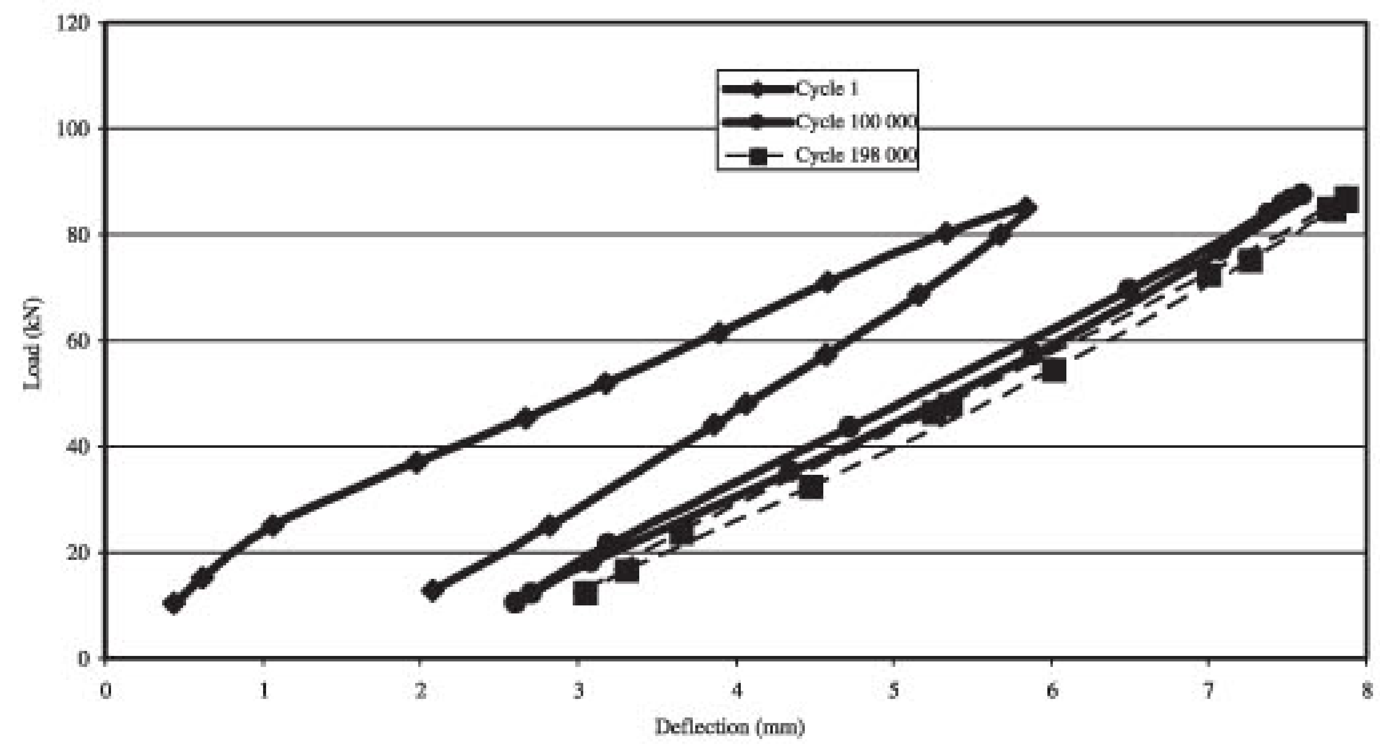


Fig. 9. Typical behaviour of deflection versus cycles for beams in group F. Note: ◆, F-C-7; □, F-CD-57; △, F-CR-57.

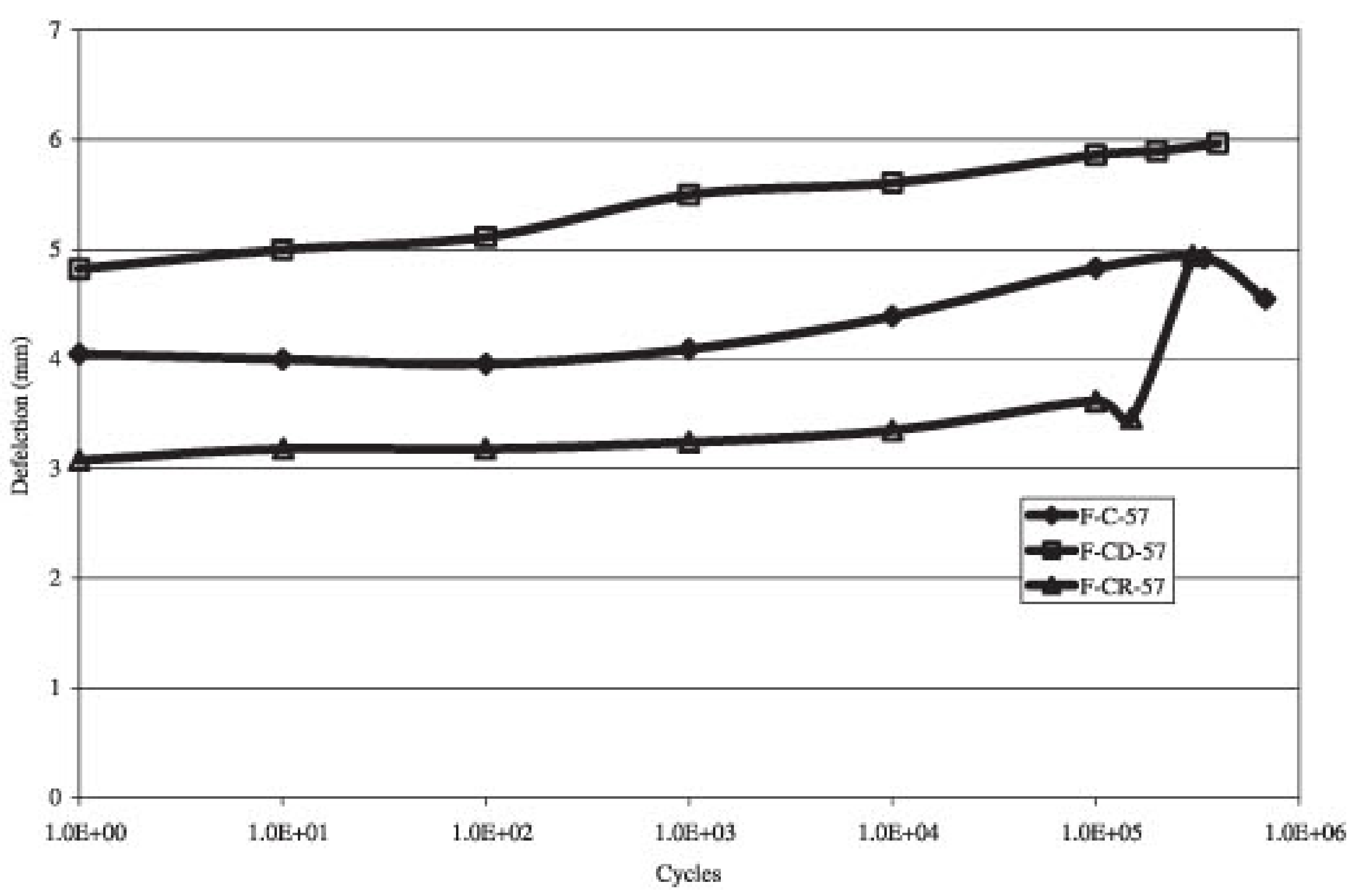
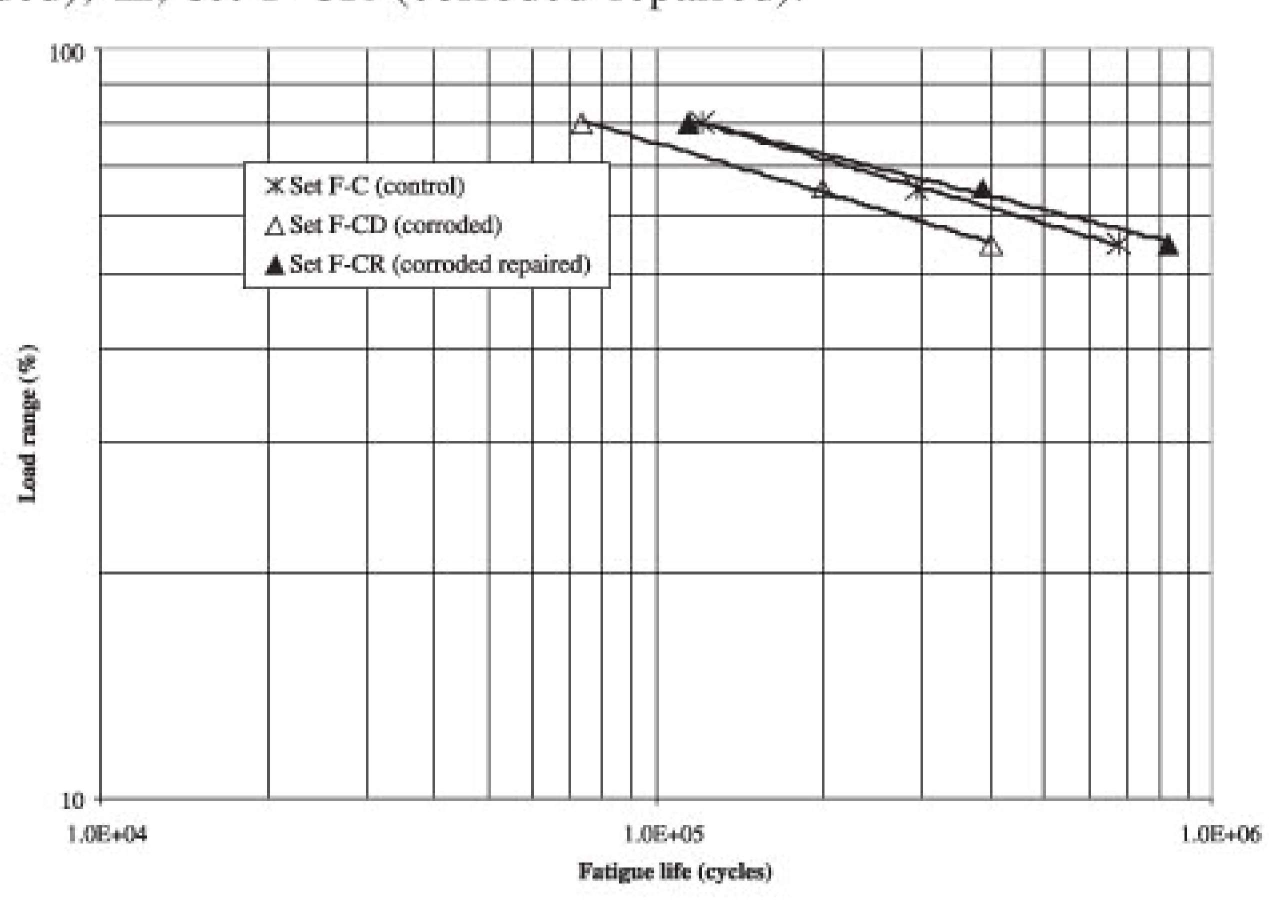


Fig. 10. Variation of beam fatigue life with load range for specimens in group F. Note: ✳, set FC (control); △, set F-CD (corroded); ▲, set F-CR (corroded repaired).



beams (set B-CD) the slip increased at a slow constant rate (Fig. 11) for most of the life, then just before failure the slip increased exponentially. This slip behavior was accompanied by a brittle type of bond failure. The beams wrapped with CFRP sheets (set B-CR) shown in Fig. 12 had an initial high rate of slip, then there was a constant rate of slip increase until failure. The confinement supplied by the CFRP

Fig. 11. Typical variation of the free-end slip with fatigue life for corroded unwrapped specimens (beam B-CD-55).

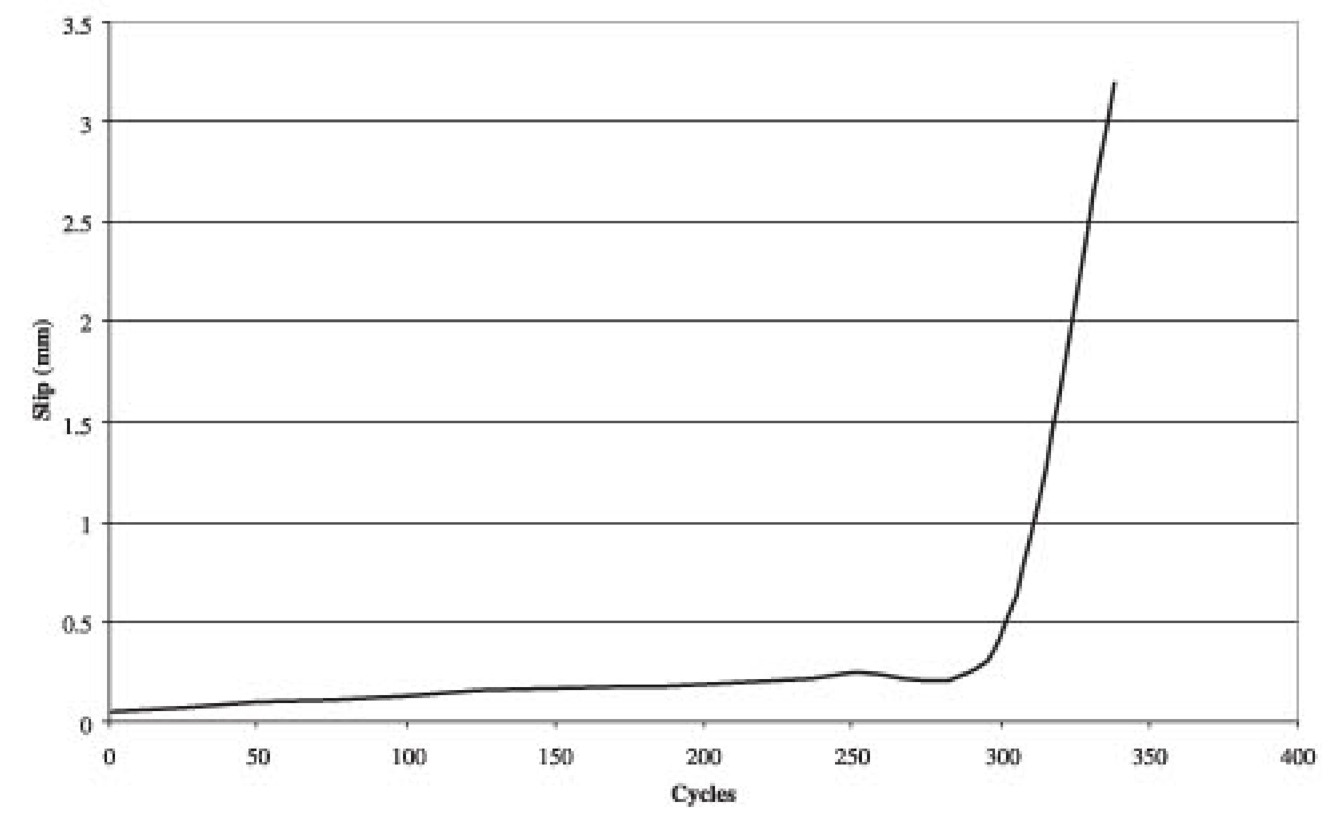
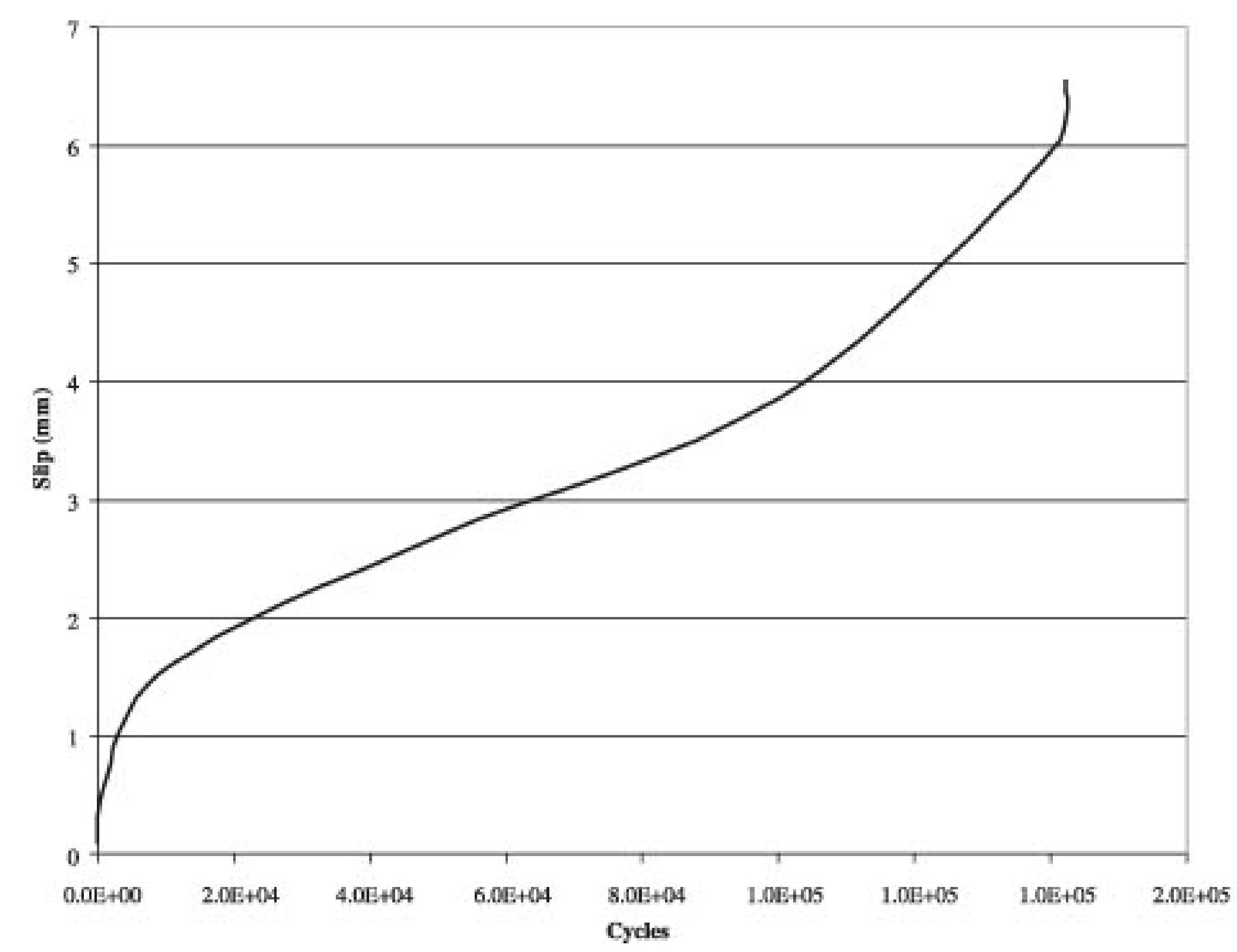


Fig. 12. Typical variation of the free end slip with fatigue life for corroded wrapped specimens (beam B-CR-50).



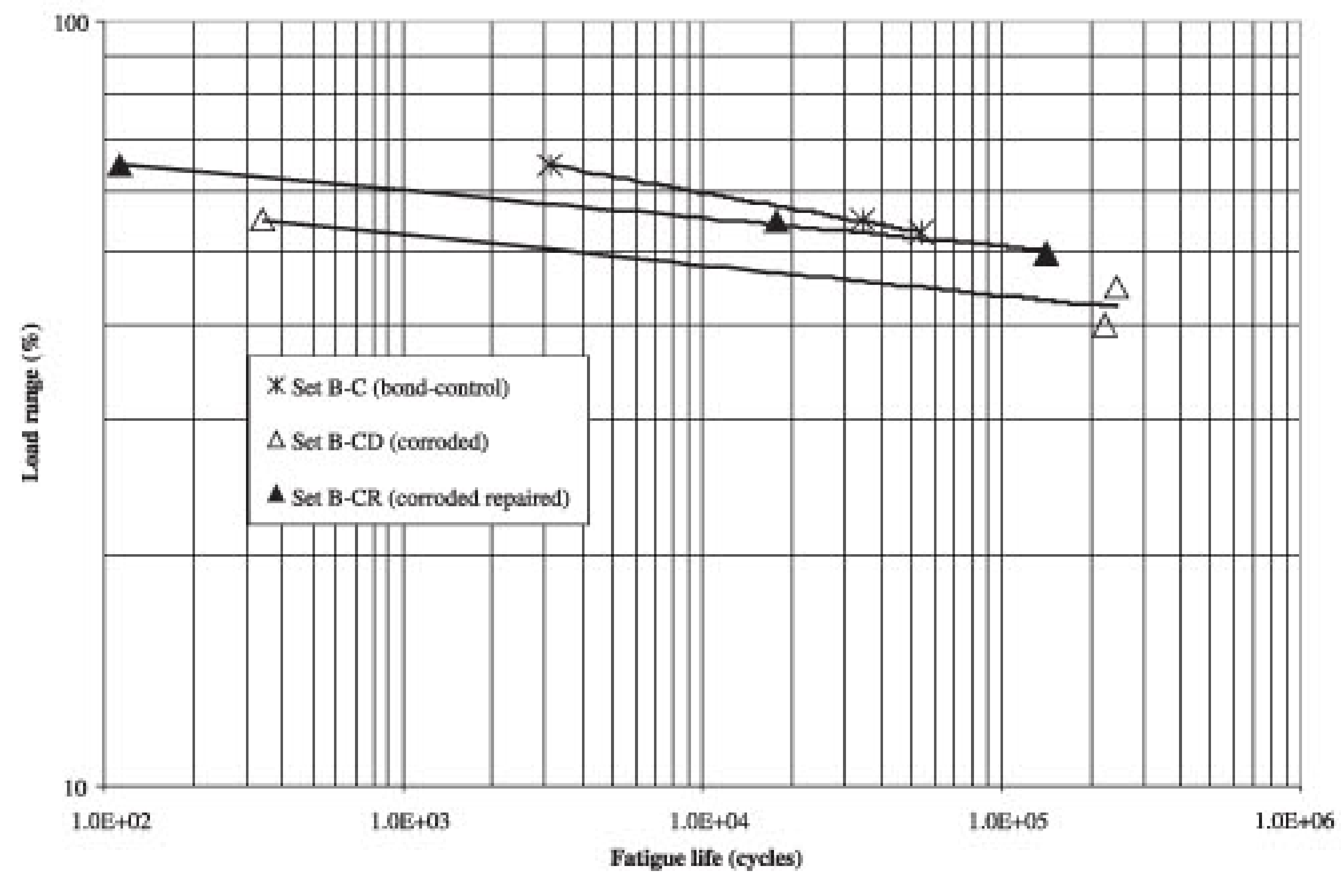
sheets greatly increased the beam ductility by increasing the slip at the onset of failure from ~2.5 to ~6.0 mm.

The life–load range variation of this group is presented in Fig. 13. It is evident that all fatigue data followed linear trend lines. In set B-CD (with a 5% mass loss because of corrosion), the fatigue bond strength decreased by 23% and 15% compared with the control (uncorroded) beams at 3 000 cycles and 50 000 cycles, respectively. The decrease was less at low load ranges (high fatigue lives) than at higher load ranges (lower fatigue lives). It should be noted that specimen B-CD-45 endured more cycles than B-CD-40, because of the scattering nature of fatigue failures. When CFRP sheets were added to strengthen the beams, an increase of 15% in fatigue bond strength (compared with the fatigue strength of the corroded unstrengthened beams) was achieved. This increase was approximately the same at all fatigue lives. It should be noted that at long fatigue lives, the original uncorroded fatigue strength of the beams repaired with CFRP sheets was restored.

Conclusions

Based on the results presented in this paper, the following conclusions can be drawn.

Fig. 13. Variation of the fatigue life with load ranges for group B specimens. Note: ✳, set B-C (bond-control); △, set B-CD (corroded); ▲, set B-CR (corroded repaired).



(1) The fatigue failure of all flexural beams was by rupture of the steel, whereas that of the bond beams was by splitting of concrete along the reinforcement bar in the anchorage zone.
(2) Corrosion resulting in a 5% theoretical mass loss decreased the average fatigue strength of flexural beams by approximately 10% and the average fatigue bond strength of bond beams by approximately 20%.
(3) Flexural beams were repaired by the application of CFRP flexural sheets to the tension side of the beam, and application of intermittent U-shaped FRP sheets along the beam length. Bond beams were repaired through the use of a CFRP U-wrap in the anchorage zone. The use of CFRP sheets in these repairs increased the fatigue flexural and bond strengths on average by 15% compared with the corroded unrepaired beams. In some cases, the fatigue strength of the corroded beams repaired with CFRP sheets increased to that of the control beams.
(4) The use of CFRP sheets for beam repairs showed that slip at the onset of failure for group B corroded beams increased by approximately 2.4 times.

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