Fatigue behaviour and design of corroded reinforced concrete beams intervened by ICCP-SS

DOI:

[10.1016/j.compstruct.2020.113295](https://doi.org/10.1016/j.compstruct.2020.113295)

Document Version

Accepted author manuscript

[Link to publication record in Manchester Research Explorer](#)

Citation for published version (APA):

Su, M., Wei, L., Liang, H., Zhu, J., Ueda, T., & Xing, F. (2020). Fatigue behaviour and design of corroded reinforced concrete beams intervened by ICCP-SS. *Composite Structures*, 113295. <https://doi.org/10.1016/j.compstruct.2020.113295>

Published in:

Composite Structures

Citing this paper

Please note that where the full-text provided on Manchester Research Explorer is the Author Accepted Manuscript or Proof version this may differ from the final Published version. If citing, it is advised that you check and use the publisher's definitive version.

General rights

Copyright and moral rights for the publications made accessible in the Research Explorer are retained by the authors and/or other copyright owners and it is a condition of accessing publications that users recognise and abide by the legal requirements associated with these rights.

Takedown policy

If you believe that this document breaches copyright please refer to the University of Manchester's Takedown Procedures [<http://man.ac.uk/04Y6Bo>] or contact uml.scholarlycommunications@manchester.ac.uk providing relevant details, so we can investigate your claim.

Fatigue behaviour and design of corroded reinforced concrete beams intervened by
ICCP-SS

Mei-ni SU ^{a,#}, Liang-liang WEI ^{b,#}, Han-shi LIANG ^b, Ji-Hua ZHU ^{b,*}, Tamon UEDA ^b,
Feng XING ^b

^a School of Mechanical, Aerospace and Civil Engineering, University of Manchester, Manchester,
M1 3NJ, UK

^b Guangdong Province Key Laboratory of Durability for Marine Civil Engineering, College of Civil
and Transportation Engineering, Shenzhen University, Shenzhen, Guangdong 518060, P.R. China

E-mail address: meini.su@manchester.ac.uk (M.N. Su), weiliangliang@email.szu.edu.cn (L.L.
Wei), lianghanshi@email.szu.edu.cn (H.S. Liang), ueda@szu.edu.cn (T. Ueda), xingf@szu.edu.cn
(F. Xing).

* Corresponding author: zhujh@szu.edu.cn (J.H. Zhu), Tel./Fax: +86-755-2653-4021

The first two authors contribute equally

Abstract

To improve the fatigue resistance of degraded reinforced concrete (RC) beams, the present study
employs a dual-functional intervention, impressed current cathodic protection and structural
strengthening (ICCP-SS). This paper presents the results of an experimental and analytical study of
the fatigue performance of corroded simply supported beams with ICCP-SS intervention. The
experimental programme comprises 16 RC beams subjected to accelerated corrosion and ICCP
followed by four-point bending tests. The test results show that the corrosion of the steel
reinforcements in the cathodically protected beams is effectively prevented. More importantly, the
ICCP-SS intervention significantly enhances the fatigue resistance of the flexural members. In

comparison to those of the corroded beams without any additional measures, the fatigue lives of the treated beams are significantly improved, by as much as 202%. The beams with the ICCP-SS intervention fail due to fatigue fracture of the steel reinforcement followed by a combination of slippage and rupture of the carbon fibre meshes. An amount of ductility is observed at the point of failure of the beams subjected to ICCP-SS intervention. In addition, the experimental results are compared with the predictions of the European and Japanese design codes as well as the Chinese design codes.

Keywords: C-FRCM; Fatigue; Impressed current cathodic protection; Reinforced concrete beam; Structural Strengthening.

1. Introduction

Infrastructure such as bridges is continuously subjected to repeated loads from vehicles, which may cause fatigue of the reinforcing bars (re-bar) [1]. Additionally, during the long-term service life of bridges, the corrosion of the steel re-bar in concrete due to the marine environment or de-icing salts is another major problem leading to the early degradation of bridges. The deterioration of reinforced concrete (RC) infrastructure presents severe potential safety issues [2, 3], as tragically demonstrated by the recent collapse of the Morandi Bridge near Genoa in Italy. In addition, a recent increase in traffic volume has highlighted the need for more durable bridges with higher capacities. An effective intervention method for improving the service life of bridges is necessary. Currently, regarding intervention measures, one of the technologies most commonly used to prevent corrosion of steel re-bar in concrete is impressed current cathodic protection (ICCP), whereas one of the most

popular strengthening technologies for improving the structural capacity of reinforced concrete is fibre reinforced polymer (FRP) strengthening. However, the existing structures subjected to chloride-induced corrosion cannot be systematically repaired simply by using ICCP on its own or the structural strengthening (SS) method on its own because ICCP cannot recover the loading capacity of the degraded structures and SS cannot stop the further corrosion of steel re-bar. Thus, this study proposes the use of the impressed current cathodic protection-structural strengthening (ICCP-SS) intervention method [4, 5] to improve the fatigue performance of simply supported corroded RC beams. In the ICCP-SS system, a carbon-fabric reinforced cementitious matrix (C-FRCM), which is a composite of two layers of carbon fabric (CF) mesh and cement-based mortar matrix, is pasted to the soffit of beams as both the anode for ICCP and a strengthening material.

In recent years, there have been a number of studies on the ICCP-SS intervention method. Carbon fibre reinforced polymer (CFRP) has been found to be a dual-functional material, serving as the structural strengthening layer and the anode for ICCP [6]. For the bonding material, Lambert et al. [7] proposed electrically conductive epoxy, which is conductive pigments in epoxy adhesive, while Su et al. [5, 8] developed a high-performance cementitious matrix. Several experimental programmes have been conducted to prove the effectiveness of the ICCP-SS system on degraded structural members subjected to static loads, including RC short columns [4], simply supported beams [5, 7] and continuous beams [9]. Experimental results show that the ICCP-SS intervention can not only effectively prevent the corrosion of steel re-bar in concrete but also significantly improve the static structural performance of the degraded structures when the structures are subjected to corrosion. Thus, ICCP-SS provides a potential cost-effective solution to the durability challenges of existing infrastructures.

67   However, in practical projects, the majority of infrastructure such as bridges and coastal
68   structures is subjected to a variety of cycling loads (e.g., wind loads and traffic loads). The
69   degradation of RC beams during cycling loads has been investigated experimentally and
70   numerically in many studies [10-12]. Carpinteri [13] analysed the energy dissipation in RC beams
71   under cyclic loading. Chang et al. [14] investigated the fatigue behaviour of RC beams to discuss
72   the fracture mode, crack growth and distribution, and the relationship between the loading and mid-
73   span deflection of the beams under fatigue loading. The final fatigue failure was a result of the
74   gradual damage accumulation of the steel re-bar in the concrete. On top of cycling loads, the
75   corrosion of steel re-bar due to a corrosive environment or the use of sea sand would make the
76   durability of these structures a more critical issue [15, 16]. The levels of corrosion loss of steel re-
77   bar are critical issues [17], and the influence of pit corrosion has also been discussed [18]. Ma et al.
78   [15] conducted experiments on the flexural performance degradation of corroded RC beams under
79   fatigue load. The results indicated that the fatigue fracture of the corroded steel re-bar occurs at the
80   largest corrosion pit location. The fatigue life of the beam at 9.03% corrosion loss is approximately
81   65% of that of the un-corroded beam.

82   Regarding the improvement of the fatigue behaviour of RC structural members, several recent
83   studies have investigated the effect of a CFRP sheet/plate on the fatigue behaviour of RC beams
84   [19-22]. Barnes et al. [23] investigated the fatigue performance of concrete beams strengthened with
85   CFRP plates. The strengthened beams exhibited an enhanced fatigue life when the same stress range
86   was applied to the reinforcing bars of strengthened and un-strengthened beams. The fatigue fracture
87   of the internal steel reinforcement is the dominant factor governing failure in CFRP-strengthened
88   beams. Gheorghiu et al. [24] studied the responses of CFRP-strengthened beams under fatigue

loading with different load amplitudes and found that increasing the fatigue load amplitude significantly increased the cracking strain in the CFRP-concrete joint. Al-Rousan et al. [25] studied the effects of various parameters on the fatigue life of RC beams strengthened with CFRP sheets. The results indicated that the applied stress range had the strongest impact on the fatigue life. The fatigue life increased with the number of layers of CFRP and the size of the CFRP-concrete bonding area, both of which facilitate the transfer of stress from the steel re-bar to the CFRP. In addition, the fatigue performance of RC beams strengthened with CFRP under severe environments was considered. Huang et al. [26] studied the effects of temperature on the fatigue behaviour of CFRP-strengthened beams. The results showed that the fatigue life decreased with increasing temperature. Moreover, the failure modes of the strengthened beams shifted from steel yielding to interface debonding as the temperature rose.

However, the studies mentioned above do not cover the fatigue performance of simply supported beams strengthened with C-FRCM composites; furthermore, none of them consider the effect of ICCP on the fatigue performance of simply supported beams. This topic definitely needs investigation. It is well known that the loading response of C-FRCM composites is different from that of CFRP-epoxy resin with more ductility; i.e., a great number of fine cracks occurred during the tensile process, and the final tensile failure was a progressive rupture of the fibre filaments and the slippage of the fibre bundles within the mortar matrix [27, 28]. Therefore, although this study can refer to the fatigue performance of beams strengthened by FRP-epoxy resin, there is still a need to carry out experimental studies to explain the fatigue behaviour of FRCM-strengthened beams.

This paper presents the first investigation of the fatigue performance of simply supported corroded beams with ICCP-SS intervention. In this study, a total of 16 RC beams were prepared and

subjected to pre-corrosion for 270 days, followed by the bonding of a C-FRCM layer on the soffit of the corroded beam. Afterwards, ICCP was applied to 10 beams for 450 days using the C-FRCM as the anode material. Four-point bending fatigue tests were conducted on the beams to investigate the effects of the protection current density and applied load level on the fatigue life of the beams. The predictions of the fatigue life according to the international design standards - fib Model Code [29] and Japan Society of Civil Engineers (JSCE) Standard Specifications for Concrete Structures [30] as well as the GB 50010 [31] in China were also discussed in comparison to the experimental results.

2. Experimental programme

2.1 Test specimens

The experimental programme includes a total of 16 RC beams. The dimensions of the beam and configuration of the steel reinforcement are shown in Fig. 1. Detailed information on the specimens and the corresponding labelling system is presented in Table 1.

To accelerate the corrosion of the steel re-bar in the concrete beams, 3% NaCl (by the mass of the cement) was added to the concrete mix; the 28-day compressive strength of the concrete was 43.1 MPa. The steel re-bar embedded in the concrete was all HRB400 grade [32]. To simulate the corrosive environment in real applications, after 28 days of curing, the specimens were placed outdoors for 270-day dry-and-wet cycles. After the accelerated corrosion procedure, the soffit of the beam was sandblasted, cleaned, and pre-wet to bond it with the C-FRCM composite. The C-FRCM composite comprised a cementitious matrix [4] and two layers of CF meshes (12,000 filaments for one bundle, mesh size of 10 mm × 10 mm). The CF mesh used in this study was from Jiangsu

Dingfeng Company, with a measured tensile strength of 2125 MPa. The cementitious matrix was developed by Zhu et al. [4] and had a compressive strength of 54.4 MPa and a flexural strength of 7.2 MPa. Please note that strain gauges were attached to the CF meshes at critical locations – the mid-span and loading points of the beams. The C-FRCM composite was cured for 28 days before the application of ICCP to 10 beams for 450 days. As indicated in Table 1, four beams were protected by a current density of 20 mA/m², three beams by 60 mA/m², and three beams by 100 mA/m². The ICCP system has been described in detail in Su et al. [5]. Key electrochemical parameters were measured during the ICCP process.

2.2 Static and fatigue properties of the steel reinforcement

The material properties of the steel re-bar were determined by static and fatigue tensile tests. The length of the specimen was 500 mm for both the static and fatigue tests. For the static loading test, an extensometer with a gauge length of 50 mm was used to measure the strain of the specimen during loading. The specimen was loaded at 0.2 mm/min by displacement control. Regarding the fatigue tests, the distance between the two clamps was equal to 350 mm. The testing frequency was 5 Hz.

The static mechanical properties of the steel in each beam are shown in Table 2. A typical static stress-strain curve is shown in Fig. 2. The experimental results of the stress range of the steel re-bar (S) and the fatigue life (N) are presented in Fig. 3, where S is the fatigue stress range $S = \sigma_{s\_max} - \sigma_{s\_min}$, σ_{s_max} is the upper limit of the fatigue stress, σ_{s_min} is the lower limit of the fatigue stress, and N is the fatigue life. The relationships of the stress range and fatigue life were fitted by linear curves, as shown in Fig. 3 and Eq. (1). The coefficient of determination (R^2) was 0.925.

$$\log S = -0.204 \log N + 3.54 \quad (1)$$

2.3 Four-point bending tests

The test configuration of the beam, with a pure bending region of 400 mm and a shear span of 400 mm, is shown in Fig. 1. To determine the parameters for the fatigue tests, three static four-point bending tests were first performed. The test was displacement-controlled with a loading rate of 0.3 mm/min. A 10 kN load was applied to the specimen to eliminate the gap between the beam and instrument before the test. During the loading process, the loads, mid-span deflections, and the strains in the steel re-bar, concrete, and CF meshes were measured.

A total of 13 beams were tested by means of fatigue tests under sinusoidal load cycles with a frequency of 5 Hz. The applied load ratio ($R_L = P_{\min}/P_{\max}$, where $P_{\min}$ is the minimum applied load and $P_{\max}$ is the maximum applied load) that the specimen was subjected to was 0.2. The applied load levels ($S_L = P_{\max}/P_u$, where P_u is the ultimate load of the reference beam F0-I0-S) were 0.85, 0.75, 0.65, and 0.55. Three cycles of static loading from zero up to $P_{\max}$ were first performed before fatigue loading. In addition, static tests were carried out after specific cycles of fatigue loading to measure the deflections, crack development, and strains of the concrete, steel re-bar, and CF meshes.

3. Results and discussion for ICCP

During the 450-day cathodic protection of the beams, it is necessary to collect electrochemical signals such as the feeding voltages and 4-hour decay values to assess the protection effectiveness of the steel re-bar in the concrete, as shown in Fig. 4. The feeding voltages show the change in resistance between the anode and cathode since a constant current was applied. The 4-hour decay value is defined as the difference between the potential of the steel (cathode) measured after a 4-

hour interruption of the current and the instant-off potential, indicating the reduction in corrosion. Fig. 4 (a) shows that as expected, the feeding voltages of the specimens protected by 100 mA/m² (indicated by the specimen names with I100) are the highest. In comparison, the specimens protected by 20 mA/m² (indicated by the specimen names with I20) have the lowest feeding voltages. During the ICCP operation, the feeding voltages are found to be rather stable within the range of 1 – 4 V, which are rather low values and are safe for access. Fig. 4(b) shows that at the beginning of the ICCP process, the 4-hour decay values of all the specimens are lower than the 100-mV cathodic polarization criterion, which means that the corrosion of the steel is still active outside the immune zones. After 10 days of application of ICCP, the 4-hour decay values of the specimens protected by 100 mA/m² first reach 100 mV, indicating that these specimens have been protected cathodically. Later, the specimens protected by 60 mA/m² and by 20 mA/m² also generally reach the polarization line after 50 days and 100 days, respectively. This shows that a higher protection current leads to more efficient protection of steel re-bar. During the whole ICCP period, the 4-hour decay values of all specimens increased steadily and finally remained stable at approximately 400 mV, 200 mV, and 100 mV for specimens protected at 100, 60, and 20 mA/m², respectively. Please note that some minor discrepancies occurred due to variations in the weather and humidity since the specimens were placed outdoors. Even so, it is clear from Fig. 4 that the steel re-bar in beam specimens under ICCP are all well-protected, while a larger protection current leads to more pronounced protection effects.

In addition, to better verify the protection effectiveness of ICCP on the steel re-bar in the beam, the steel re-bars were removed from the tested beams after the fatigue tests. These steel re-bars were cleaned according to ASTM G1-03 [33]. The corrosion rate was measured, and the results are

presented in Table 1. In general, the corrosion rate of the steel re-bar protected by ICCP is lower than that of the unprotected reinforcements.

4. Results and discussion of the four-point bending tests

4.1 Failure modes

The failure modes of all the tested beams are shown in Table 3, and some typical failure modes are presented in Fig. 5. Under static loads, the un-strengthened unprotected beam (F0-I0-S) fails by the yielding of the steel re-bar and crushing of concrete in the compression zone. The FRCM-strengthened beam (F2-I0-S) and the beam with the ICCP-SS intervention (F2-I60-S) fail by the yielding of the steel re-bar and a combination of slippage between the carbon fibre meshes and cementitious matrix and the partial rupture of carbon fibres.

Under fatigue loads, flexural cracks are initiated from the bottom of the beam after the first static test cycle and subsequently propagate to the compression zone. The failure mode of the un-strengthened beams (F0-I0-0.65 and F0-I20-0.65) is brittle flexure failure due to fatigue fracture of one tensile steel re-bar. The width and depth of the cracks on the concrete surface are more significant than those on the specimens strengthened with FRCM composite, as shown in Fig. 5. For the 11 FRCM-strengthened beams, including those protected by ICCP and those not, the general failure mode is flexural failure by the fatigue fracture of one or two steel re-bars in the pure bending span, followed by the failure of the FRCM composite due to load redistribution. The stress level of the FRCM composite increases significantly after the fracture of the steel re-bar, leading to a debonding failure at the FRCM/concrete interface (for specimen F2-I0-0.55) (see Fig. 5(b)) or the partial fracture of the carbon fibre meshes together with the slippage between the CF meshes and

217 cementitious matrix (for the rest of specimens). As the test continues, the concrete in the
218 compressive zone is finally crushed. It is also observed that the vertical cracks that appear on
219 FRCM-strengthened beams generally are thinner and spaced further apart. Upon the appearance of
220 vertical cracking in the pure bending region, the stress of the tensile longitudinal steel re-bar
221 increases significantly around the crack region. For the non-cathodic protected beams, the
222 reinforcement fractures where corrosion pits are located.

223 A comparison of the failure modes of the beams strengthened by the FRCM composite and FRP-
224 epoxy resin shows that the FRCM-strengthened beams generally fail in a more ductile manner.
225 Before the failure of the FRCM-strengthened beams, more tiny cracks form on the FRCM composite;
226 as the applied load and deflection of the beam increases, one or two cracks exhibited a larger width
227 than the others. A slippage between the carbon fibre meshes and cementitious matrix occurs in these
228 regions. With the accumulation of the slippage, the load resisted by the FRCM composite moves
229 towards the steel re-bar, and the deflection of the beam increases. Upon failure, the re-bar fractures,
230 followed by the fracture of the external carbon fibre meshes in contact with the cementitious matrix.
231 Before fatigue failure, the pronounced deformation of the FRCM-strengthened beams signals the
232 final collapse of the member, which is catastrophic in terms of the safety design considerations. For
233 the FRP-epoxy resin strengthened beams, following the fatigue failure of the re-bar and an extension
234 of the flexural cracks, the CFRP sheets commonly suddenly fail by debonding or fracture under
235 tension [25, 34]. The FRP-epoxy resin strengthened beams generally fail in a brittle manner without
236 any advance warning.

237 *4.2 Strain distributions of different components*

238 To measure the strain development in the concrete, reinforcement, and FRCM composite,

strain gauges were attached to these materials at the loading points and mid-span of the beams. Fig. 6(a) shows the strain in the re-bars of an FRCM-strengthened beam (F2-I20-0.65) and a non-strengthened beam (F0-I20-0.65) after different fatigue cycles. The strain values up to 500 cycles are presented in Fig. 6(a). In the presence of C-FRCM, the deformation of the strengthened beam is smaller, and the strain and stress level of the steel re-bar are clearly lower. As the number of cycles increases, the stress on the steel reinforcement keeps increasing gradually due to the upward movement of the neutral axis of the cracked section. Most of the steel re-bar is found to yield before the tension fracture. Similarly, the stress level of steel re-bar in the FRP-epoxy resin strengthened beams is reduced due to the presence of FRP [25, 34]. The strain development of the concrete in the compression zone at the mid-span of the beam is shown in Fig. 6(b). As expected, a higher load level leads to more considerable strain in the concrete in the compression zone, and the strain in the concrete increases as the fatigue tests continue. Fig. 6(c) shows the strain development in the carbon fibre meshes embedded in the cementitious matrix. The values of the strain gauges decrease due to the stress release after the occurrence of slight deformation and slippage at the CFRP and matrix interface.

4.3 Hysteretic curves and deflection

For each specimen, after some fatigue cycles, a static loading test was conducted to generate a loading/unloading curve, as shown in Fig. 7. It can be seen that the area of the load-deflection curve in the first loading/unloading cycle was much larger than that of the following cycles due to the greater energy consumption in the first cycle. The main difference between the first and subsequent cycles is cracking (i.e., crack initiation in the first cycle and crack propagation in the subsequent cycles) in the concrete, which consumes energy. This phenomenon is observed in any RC beam [35,

36].

The protection current density and the load level influence the deflection of the beam, as shown in Fig. 8. In general, under higher protection current, the deflection of the beam is found to be larger, which is believed to be attributed to the more relaxed mechanical bond between the carbon fibre meshes and the cementitious matrix. However, this is not true for specimens F2-I20-0.75 and F2-I60-0.75; unfortunately, the reason for these differences is not yet clear; the possibilities include the discrepancies associated with concrete materials. As explained by Su et al. [5], an amount of gaseous yellow liquid appears on the surface of the CF during the process of ICCP, which may cause the CF to separate from the concrete interface. The levels of the cyclic loads also affect the deflection in the component. The experimental results show that a beam subjected to a higher load level deforms more than the others because under a higher load level, especially the load level of 0.75, the stress on the tensile steel re-bar is relatively high, and the steel almost yields, resulting in large deformation in the steel re-bar and the beam. The deflection of the beam during the cycling load process can be divided into three stages (see Fig. 8 (b)) – in the initial stage of loading (i.e., the first tens of thousands of fatigue cycles), the deflection in the beam increases fast; as the cycling loads continue, the cracks on the concrete become stable, as did the stiffness and deflection of beam; in the final loading stage before failure, the main cracks on the concrete widen and deepen rapidly, so the deflection in beam suddenly increases. In addition, compared to that in the non-strengthened beams, the deflection in the FRCM-strengthened beams is smaller. The development of deflection in simply supported beams strengthened with traditional epoxy resin and FRP plates is close to that in this experiment. The deflection of CFRP-strengthened beams increases almost linearly with the number of cycles when plotted on a log scale until near failure, where it increases rapidly [37, 38]. As

explained by Song and Yu [37], the increase in deflection with the cycles is attributed to the degradation of the constituent material properties and the increase in the depth of the flexural cracks with repeated loading.

4.4 Fatigue life

The fatigue lives of 13 beams under fatigue loading tests are summarized in Table 4 and Fig. 9. Under the same load level and protection current density (F0-I0-0.65 versus F2-I0-0.65; F0-I20-0.65 versus F2-I20-0.65), the fatigue lives of the strengthened beams are approximately 250% of those of the non-strengthened beams. The improvement in the fatigue behaviour in the presence of the C-FRCM composite is pronounced since the C-FRCM composite decreases the stress level of the steel re-bar.

The relation between the load level and the fatigue life of the beams is shown in Fig. 9(a). The results indicate that the fatigue life is reduced with increasing load level. For example, when the protection current density is 20 mA/mm², the fatigue life values of the strengthened beams at load ratios of 0.55, 0.65 and 0.75 are 111.3, 52.8 and 19.4 × 10⁴ cycles, respectively. It is also found that the fatigue life of the RC beams is largely dependent on the steel re-bar. As shown in Table 5, the fatigue cycles of the beam obtained from the four-point bending tests are generally close to the fatigue cycles derived from the stress level of the re-bar. It is therefore recommended that one of the criteria for the fatigue design of FRCM-strengthened beams should be the limit of the stress level of the steel re-bar.

The relation between the protection current density and the fatigue life of the beam is shown in Fig. 9(b). Under the same applied load level, the fatigue lives of the beams cathodically protected by different currents are similar. As illustrated in Section 3, the steel re-bar of all the cathodically

protected beams are found to be protected from corrosion well; therefore, the corrosion conditions of the cathodically protected beams are similar. In addition, although the applied current causes anode polarization at the C-FRCM interface, this effect has not caused significant degradation of the C-FRCM anode; therefore, the strengthening performance of the C-FRCM composite is similar as the applied currents varied from 20 mA/m² to 100 mA/m².

Compared to those of specimens F0-I0-0.65 and F0-I20-0.65, the fatigue lives of specimens F2-I0-0.65 and F2-I20-0.65 increase by 150% and 156%, respectively. This is due to the contribution of FRCM to the reduction in the steel stress during the cycling loads. However, the improvement in the fatigue life performance of FRCM-strengthened beams is not very prominent, as expected, although the percentage of improvement can be larger when the reinforcement ratio of the beams is smaller. The possible explanation for this result is the premature cracking of the FRCM at the beginning of cycling loading followed by fabric slippage, which results in the limitation of reduction in the steel stress. In future studies, to improve the contribution of the FRCM to the fatigue performance of concrete structures under flexure, the ductility of the FRCM composites can be enhanced. Leung et al. [39] concluded that the high ductility and capability to form multiple fine cracks of engineered cementitious composites (ECCs) are responsible for an impressive improvement in the fatigue performance of RC beams. It is believed that high ductility and multiple fine cracks can help release the energy in the process of cycling loading on concrete structures.

The available fatigue test data of FRP-epoxy resin strengthened beams in the literature [23, 37, 40-43] are also collected (see Table 6) for comparison with the fatigue behaviour of the FRCM-strengthened beams generated in this study (see Table 5), as shown in Fig. 10. Under a similar stress range and load ratio, the fatigue life of the FRP-epoxy resin strengthened beams is generally similar

to that of the FRCM-strengthened beams except for the beams subjected to a low stress level. Under a low load level, the fatigue life of the beam tested by Papakonstantinou et al. [40] (load level of 0.36) was 2×10^6 cycles, that of the beam tested by Barnes et al. [23] (load level of 0.32) was 1.89×10^6 cycles, and that of beam F2-I20-0.55 tested in this study could reach 1.11×10^6 cycles. The S-N relationship of the specimens in which steel reinforcement fatigue fracture occurs is more or less the same despite the differences in the strengthening method (FRP external bonding, FRCM overlaying). The corrosion of steel re-bar in concrete has specific influences on the fatigue life of RC beams. The fatigue life decreased from 1346 to 304 thousand cycles at the identical fatigue load ratio of 38.9% when the corrosion of steel re-bar increased from 8.3% to 16.7% (i.e., the average mass loss of steel) [37]. In contrast, the ICCP-SS intervention method could be helpful for controlling the corrosion of steel re-bar, demonstrated by the fact that the fatigue life of beam F2-I20-0.55 reaches more than 1000-thousand cycles at the fatigue load ratio of 55%.

5 Comparison with design predictions

Three international design standards - the fib Model Code for Concrete Structures [29], JSCE Standard Specifications for Concrete Structures [30] and Chinese Standard GB 50010-2010 [31] - are used to predict the fatigue life of the beams considered in this study. The predicted results are compared with the experimental fatigue life results in the present paper to assess the accuracy of the existing standards for the fatigue of beams repaired by the ICCP-SS intervention method. The experiment in the present paper uses a single load level, and the fatigue damage of the specimens is controlled by the steel re-bar. All the safety factors are set to be unity for comparison purposes.

5.1 fib Model Code 2010

The fatigue design procedure is codified in Section 7.4.1 of the fib Model Code [29] through semi-empirical design curves based on existing fatigue test data. The fatigue design in this code is applicable to non-low cycle fatigue members with fatigue lives greater than 10^6 cycles. When the test is a single load level, the failure mode is controlled by the fatigue life of the steel re-bar:

$$(\Delta\sigma_{Rsk})^m N^* = (\Delta\sigma_{Es})^m N_f \quad (2)$$

When the diameter of the steel re-bar $\phi \leq 16$ mm, $\Delta\sigma_{Rsk} = 160$ MPa at $N^* = 10^6$, and the stress exponent $m = 5$, while $\Delta\sigma_{Rsk} = 95$ MPa at 10^8 cycles, and the stress exponent $m = 9$; therefore, the fatigue life N_f can be obtained based on the relevant stress range $\Delta\sigma_{Es} = \sigma_{max} - \sigma_{min}$,

5.2 JSCE Standard Specifications

The JSCE Standard Specifications for Concrete Structures - Design [30] provide the fatigue design of concrete and steel re-bar in Sections 5.2 and 5.3 of the Standard Specifications, respectively. The fatigue life of this experiment is controlled by the steel re-bar, and the specific calculation is as follows:

$$f_{srd} = 190 \frac{10^a}{N^k} \left(1 - \frac{\sigma_{sp}}{f_{ud}} \right) / \gamma_s \quad (3)$$

When $N \leq 2 \times 10^6$, a and k are determined by the following formula:

$$a = k_{0f} (0.81 - 0.003\phi) \quad (4)$$

$$k = 0.12 \quad (5)$$

5.3 GB 50010-2010

Apart from fib and JSCE, the Chinese Code for the design of concrete structures standard GB

50010-2010 [31] recommends the limit stress ranges of steel reinforcement in Section 4.2.6, as shown in Table 7. When the stress range of steel reinforcements is within this range, RC structures are expected to have a fatigue life greater than 2×10^6 cycles. However, GB 50010-2010 [31] does not provide design equations for predicting the fatigue life of RC structures when the stress range of the reinforcements exceeds the values given in Table 7. Two types of steel reinforcements were investigated to obtain the S-N relationships, i.e., HRB 335 and HRB 400 hot-rolled ribbed bars.

5. 4 Result comparisons

The fatigue lives of the steel reinforcements obtained from fatigue tests (see Fig. 3) are compared to the predictions by fib [29], JSCE [30] and GB 50010 [31], as shown in Fig. 11, where the test data are the measured stress range. Note that GB 50010 only provides the limit stress range of steel under fatigue when it sustains more than 2×10^6 fatigue cycles. When the fatigue life reaches the specified number of cycles at 2×10^6 , the maximum stress predicted by JSCE is 181 MPa, followed by $S = 179$ MPa based on the proposed model (i.e., Eq. (1) in the present paper). In Table 7, it can be found that $S = 156$ MPa per GB 50010. The result predicted by the fib Model Code has the lowest limit stress range of 147 MPa. To consider a possible long cycle at 2×10^7 , the predicted limit stress values of steel based on the proposed model in this paper and fib Model Code are 128 MPa and 123 MPa, respectively. In addition, as shown in Table 8, the nominal yielding and ultimate strength of the HRB 335 steel are 335 MPa and 455 MPa, respectively, and those of the HRB 400 steel are 400 MPa and 540 MPa, respectively [31]. The diameter of the steel re-bar is taken as 12 mm. The fatigue lives of the two kinds of steel re-bar can be predicted by the fib model code using Eq. (2) and by the JSCE Standard Specifications using Eq. (3). It is found that the prediction results per fib and JSCE are slightly longer than that of GB 50010. It can be seen that the predictions by

the fib Model Code are more sensitive to the type steel reinforcement than those of GB and JSCE.

The key results obtained from fatigue tests are presented in Table 4, and the fatigue lives predicted by fib [29], JSCE [30], and the S-N curve of steel reinforcement (see Fig. 3) are summarized in Fig. 12 and Table 5. The calculation was carried out based on both the strain value obtained from the beam theory (i.e., the plane-remain-in-plane hypothesis) and the strain value obtained from the strain gauges in the tests. Fig. 12 indicates that the predictions based on the measured strain of the steel re-bar are generally more accurate and consistent than those based on beam theory. All three design methods are generally conservative when following beam theory. Among the three design methods, the predictions by the steel S-N curve are the closest to the test results, followed by the predictions per the fib Model Code [29], while the JSCE Standard Specifications [30] provide the most conservative predictions. It is indicated that the fatigue lives of these beams are generally governed by the fatigue life of the steel reinforcements. When using the fib Model Code for the design of fatigue beams, the predicted fatigue life is lower than the experimental results regardless of whether the predictions are carried out based on beam theory or the measured strain values, except for specimen F2-I0-0.75. Regarding the JSCE Standard Specifications [30], the predicted fatigue life is significantly conservative compared to the measured fatigue life from tests when the load ratio is 0.75, and the stress level of the re-bar is near the yielding level (more than 90% of the yield). JSCE seems unsuitable for the fatigue design of beams when the steel re-bar has almost yielded. For load ratios of 0.65 and 0.55, although the predicted fatigue life is generally conservative, it is not far from the experimental results except for those of specimen F0-I20-0.65. In the fib Model Code [29], the diameter of the steel reinforcement is taken into consideration, while the JSCE Standard Specifications considers the ultimate tensile strength of the

steel reinforcement. It is possible to explain the deviation between the fatigue lives of the experimental and predicted results.

The key differences associated with the beams considered in this study are that the beams have been subjected to accelerated corrosion and ICCP treatment. The results show that for beams with corroded steel reinforcements, the current prediction formulas of the fatigue life can be adopted when the corrosion degree of the steel is lower than 5%. In Table 4, both the maximum stress and the stress range of the steel in beam F2-I20-0.55 are higher than those in beam F2-I0-0.55. Similarly, the stress of the steel in the beams with ICCP-SS intervention is also slightly higher than that in the solely strengthened beams at the applied load levels of 0.65 and 0.75. Accordingly, compared with the those of the beams strengthened by C-FRCM, the counterpart beams subjected to ICCP treatment have comparable steel stress conditions, resulting in the similar fatigue lives of the treated beams. This result might be due to the negligible corrosion of the steel in the concrete before application of the ICCP treatment. However, for the ICCP-SS-treated beams, the C-FRCM layer is degraded at the interface after cathodic protection, resulting in an unmatured failure of the C-FRCM layer, and more stress is thereafter transferred to the steel reinforcement. Thus, the actual strain distribution might not satisfy the linear proportional relationship assumed by the plane-remain-plane hypothesis. The difference between the experimental and predicted results could be partly attributed to the discreteness of the measured strain values of the re-bars due to the stress concentrations at the corrosion pits.

6. Conclusions

A total of 16 simply supported beams were tested under static and cycling loads. Before four-point bending tests, these beams were subjected to 150-day accelerated corrosion and were repaired

by ICCP-SS with 450-day cathodic protection. The test results show that corrosion of the steel re-bars in the cathodically protected beams is effectively prevented and that ICCP-SS intervention can significantly enhance the fatigue resistance of flexural members. Under static load, the failure mode of the ICCP-SS-treated beams is the yielding of the steel reinforcement followed by a combination of the slippage and partial rupture of the carbon fibre meshes. During cyclic loading, the crack width and spacing on the concrete surface are smaller for C-FRCM-strengthened beams than for non-strengthened beams for the same load. The stress level of the steel tension reinforcement generally increases as the neutral axis of the cross-section moves up due to the development of cracks in the concrete. The failure mode of un-strengthened beams is brittle flexure failure due to fatigue fracture of one tensile steel re-bar; the general failure mode of the FRCM-strengthened beams is flexural failure by fatigue fracture of one or two steel re-bars in the pure bending span, followed by failure of the FRCM composite due to load redistribution.

The fatigue life of the beam obtained is generally close to that of the fatigue cycles derived from the stress level of the reinforcing bars using the S-N relationship of the reinforcing bars taken from the beam repaired by ICCP-SS. The experimental results are also compared with the predictions by the JSCE and fib design codes. In general, both design standards underestimate the fatigue lives of ICCP-SS beams because the S-N relationships provided by the standards underestimate the fatigue lives of the reinforcing bars in the beams; the predictions calculated based on the measured steel strain are closer to the experimental results.

Notations

S = fatigue stress range of steel in the tensile loading tests;

455 σ_{s_max} = upper limit of the fatigue stress of steel in the tensile loading tests;
 456 σ_{s_min} = lower limit of the fatigue stress of steel in the tensile loading tests;
 457 N = fatigue life of steel in the tensile loading tests;
 458 R_L = applied load ratio of beams under cycling loads;
 459 P_{max} = maximum applied load of beams;
 460 P_{min} = minimum applied load of beams;
 461 S_L = applied stress level of beams under cycling loads;
 462 P_u = ultimate load of the reference beam F0-I0-S;
 463 $\Delta\sigma_{Rsk}$ = stress range relevant to N^* cycles;
 464 m = stress exponent in the characteristic fatigue strength function;
 465 N^* = characteristic number of cycles;
 466 $\Delta\sigma_{Es}$ = steel stress range under the acting loads;
 467 σ_{max} = maximum steel stress corresponding to the maximum applied load in the fatigue tests of
 468 beams;
 469 σ_{min} = minimum steel stress corresponding to the minimum applied load in the fatigue tests of beams;
 470 N_f = predicted fatigue life in the stress range $\Delta\sigma_{Es}$;
 471 f_{srd} = design fatigue strength of a deformed bar;
 472 f_{ud} = design tensile strength of steel re-bar;
 473 σ_{sp} = stress in the deformed bar due to permanent load, which is assumed here to be as the minimum
 474 stress during the cyclic loading;
 475 γ_s = material factor of the steel re-bar, which is taken here as 1.0;
 476 a = a factor depending on the shape of the lug and the diameter of the steel re-bar;

477 k_{of} = a factor depending on the shape of the lug, which may generally be taken as 1.0;

478 ϕ = diameter of the steel re-bar (units: mm);

479 i = protection current density (mA/m²);

480 ρ = corrosion degree (%).

481

482 **Declarations of interest**

483   The authors declare that they have no known competing financial interests or personal
484 relationships that could have appeared to influence the work reported in this paper.

485

486 **Data availability statement**

487   The raw/processed data required to reproduce these findings cannot be shared at this time as
488 the data also forms part of an ongoing study.

489

490 **Acknowledgements**

491   We would like to thank the support from the Key-Area Research and Development Program of
492 Guangdong Province (2019B111107002), the National Key Research and Development Program of
493 China (2018YFE0124900), the National Natural Science Foundation of China
494 (51538007/51778370/51861165204), the Natural Science Foundation of Guangdong
495 (2017B030311004), and the Shenzhen Science and Technology Project
496 (GJHZ20180928155819738).

497 **References**

498 [1] Pimentel M, Brühwiler E, Figueiras J. Fatigue life of short-span reinforced concrete railway

499 bridges. *Struct Concr* 2008;9:215–22.

500 [2] Al-Hammoud R, Soudki K, Topper TH. Fatigue flexural behavior of corroded reinforced
501 concrete beams repaired with CFRP sheets. *J Compos Constr* 2010;15:42–51.

502 [3] Rteil A, Soudki K, Topper T. Mechanics of bond under repeated loading. *Constr Build Mater*
503 2011;25:2822–7.

504 [4] Zhu J, Su M, Huang J, Ueda T, Xing F. The ICCP-SS technique for retrofitting reinforced
505 concrete compressive members subjected to corrosion. *Constr Build Mater* 2018;167:669–
506 79.

507 [5] Su M, Wei L, Zeng Z, Ueda T, Xing F, Zhu J-H. A solution for sea-sand reinforced concrete
508 beams. *Constr Build Mater* 2019;204:586–96.

509 [6] Zhu J-H, Wei L, Moahmoud H, Redaelli E, Xing F, Bertolini L. Investigation on CFRP as
510 dual-functional material in chloride-contaminated solutions. *Constr Build Mater*
511 2017;151:127–37.

512 [7] Lambert P, Van Nguyen C, Mangat PS, O’Flaherty FJ, Jones G. Dual function carbon fibre
513 fabric strengthening and impressed current cathodic protection (ICCP) anode for reinforced
514 concrete structures. *Mater Struct* 2015;48:2157–67.

515 [8] Su M, Wei L, Zhu J-H, Ueda T, Guo G, Xing F. Combined Impressed Current Cathodic
516 Protection and FRCM Strengthening for Corrosion-Prone Concrete Structures. *J Compos*
517 *Constr* 2019;23:4019021.

518 [9] Su M, Zeng C, Li W, Zhu J-H., Lin W, Ueda T, Xing F. Flexural performance of corroded
519 continuous RC beams rehabilitated by ICCP-SS. *Compos Struct* 2020;232.
520 <https://doi.org/10.1016/j.compstruct.2019.111556>.

521 [10] Bruno D, Olivito RS, Spadea G. Fracture in reinforced concrete beams under repeated loads.
522 *Eng Fract Mech* 1990;35:269–75.

523 [11] Kwak H-G, Kim S-P. Nonlinear analysis of RC beam subject to cyclic loading. *J Struct Eng*
524 2001;127:1436–44.

525 [12] Carpinteri A, Spagnoli A, Vantadori S. Mechanical damage of ordinary or prestressed
526 reinforced concrete beams under cyclic bending. *Eng Fract Mech* 2005;72:1313–28.

527 [13] Carpinteri A. Energy dissipation in RC beams under cyclic loadings. *Eng Fract Mech*
528 1991;39:177–84.

- 529 [14] Chang D-I, Chai W-K. A study on the fatigue strength behavior of reinforced concrete
530 structures. *Int J Press Vessel Pip* 1989;40:51–75.
- 531 [15] Ma Y, Wang G, Su X, Wang L, Zhang J. Experimental and modelling of the flexural
532 performance degradation of corroded RC beams under fatigue load. *Constr Build Mater*
533 2018;191:994–1003.
- 534 [16] Wu J, Diao B, Xu J, Zhang R, Zhang W. Effects of the reinforcement ratio and chloride
535 corrosion on the fatigue behavior of RC beams. *Int J Fatigue* 2019;105:299.
- 536 [17] Azad AK, Ahmad S, Azher SA. Residual strength of corrosion-damaged reinforced concrete
537 beams. *ACI Mater J* 2007;104:40.
- 538 [18] Guo Z, Ma Y, Wang L, Zhang J. Modelling guidelines for corrosion-fatigue life prediction
539 of concrete bridges: Considering corrosion pit as a notch or crack. *Eng Fail Anal*
540 2019;105:883–95.
- 541 [19] Brena SF, Benouaich MA, Kreger ME, Wood SL. Fatigue tests of reinforced concrete beams
542 strengthened using carbon fiber-reinforced polymer composites. *ACI Struct J* 2005;102:305.
- 543 [20] Aidoo J, Harries KA, Petrou MF. Full-scale experimental investigation of repair of
544 reinforced concrete interstate bridge using CFRP materials. *J Bridg Eng* 2006;11:350–8.
- 545 [21] Toutanji H, Zhao L, Deng Y, Zhang Y, Balaguru P. Cyclic behavior of RC beams
546 strengthened with carbon fiber sheets bonded by inorganic matrix. *J Mater Civ Eng*
547 2006;18:28–35.
- 548 [22] Harries KA, Reeve B, Zorn A. Experimental evaluation of factors affecting monotonic and
549 fatigue behavior of fiber-reinforced polymer-to-concrete bond in reinforced concrete beams.
550 *ACI Struct J* 2007;104:667.
- 551 [23] Barnes RA, Mays GC. Fatigue performance of concrete beams strengthened with CFRP
552 plates. *J Compos Constr* 1999;3:63–72.
- 553 [24] Gheorghiu C, Labossiere P, Proulx J. Response of CFRP-strengthened beams under fatigue
554 with different load amplitudes. *Constr Build Mater* 2007;21:756–63.
- 555 [25] Al-Rousan R, Issa M. Fatigue performance of reinforced concrete beams strengthened with
556 CFRP sheets. *Constr Build Mater* 2011;25:3520–9.
- 557 [26] Huang P, Zhou H, Wang H, Guo X. Fatigue lives of RC beams strengthened with CFRP at
558 different temperatures under cyclic bending loads. *Fatigue Fract Eng Mater Struct*

2011;34:708–16.

[27] Arboleda D, Carozzi FG, Nanni A, Poggi C. Testing procedures for the uniaxial tensile characterization of fabric-reinforced cementitious matrix composites. *J Compos Constr* 2015;20:4015063.

[28] Wei L, Zhu J-H, Ueda T, Su M, Liu J, Liu W, et al. Tensile behaviour of carbon fabric reinforced cementitious matrix composites as both strengthening and anode materials. *Compos Struct* 2020;234. <https://doi.org/10.1016/j.compstruct.2019.111675>.

[29] fib. *fib Model Code for Concrete Structures*. Berlin, Germany: Ernst & Sohn, Wiley, 2013. Print. 2010.

[30] JSCE. *Standard Specifications for Concrete Structures – Design*. Japan Society of Civil Engineers, JSCE Guidelines for Concrete No. 15. 2007.

[31] GB 50010. *Code for design of concrete structures*. Beijing, China: China Building Industry Press. 2010.

[32] GB/T 1499.2. *Steel for the reinforcement in concrete - Part 2: Ribbed bars*, NEQ. Beijing, China: Standards Press of China. 2018.

[33] ASTM G1-03(2017). *Standard Practice for Preparing, Cleaning, and Evaluating Corrosion Test Specimens*. ASTM International, West Conshohocken, PA. 2017.

[34] Al-Hammoud R, Soudki K, Topper TH. Bond analysis of corroded reinforced concrete beams under monotonic and fatigue loads. *Cem Concr Compos* 2010;32:194–203.

[35] Dong JF, Wang QY, Guan ZW. Structural behaviour of RC beams externally strengthened with FRP sheets under fatigue and monotonic loading. *Eng Struct* 2012;41:24–33.

[36] Katakalos K, Papakonstantinou CG. Fatigue of reinforced concrete beams strengthened with steel-reinforced inorganic polymers. *J Compos Constr* 2009;13:103–12.

[37] Song L, Yu Z. Fatigue performance of corroded reinforced concrete beams strengthened with CFRP sheets. *Constr Build Mater* 2015;90:99–109.

[38] Ferrier E, Bigaud D, Clement JC, Hamelin P. Fatigue-loading effect on RC beams strengthened with externally bonded FRP. *Constr Build Mater* 2011;25:539–46.

[39] Leung CKY, Cheung YN, Zhang J. Fatigue enhancement of concrete beam with ECC layer. *Cem Concr Res* 2007;37:743–50. <https://doi.org/10.1016/j.cemconres.2007.01.015>.

[40] Papakonstantinou CG, Petrou MF, Harries KA. Fatigue behavior of RC beams strengthened

- 589 with GFRP sheets. *J Compos Constr* 2001;5:246–53.
- 590 [41] Masoud S, Soudki K, Topper T. Postrepair fatigue performance of FRP-repaired corroded
591 RC beams: Experimental and analytical investigation. *J Compos Constr* 2005;9:441–9.
- 592 [42] Charalambidi BG, Rousakis TC, Karabinis AI. Fatigue behavior of large-scale reinforced
593 concrete beams strengthened in flexure with fiber-reinforced polymer laminates. *J Compos*
594 *Constr* 2016;20. [https://doi.org/10.1061/\(ASCE\)CC.1943-5614.0000689](https://doi.org/10.1061/(ASCE)CC.1943-5614.0000689).
- 595 [43] Aidoo J, Harries KA, Petrou MF. Fatigue behavior of carbon fiber reinforced polymer-
596 strengthened reinforced concrete bridge girders. *J Compos Constr* 2004;8:501–9.
597 [https://doi.org/10.1061/\(ASCE\)1090-0268\(2004\)8:6\(501\)](https://doi.org/10.1061/(ASCE)1090-0268(2004)8:6(501)).

Figure Captions:

Fig. 1. Detailed dimensions and test set-up of beam specimens (all dimensions in mm).

Fig. 2. Typical stress-strain curve of reinforcement obtained from static tensile test.

Fig. 3. S-N fitting curve for reinforcement fatigue test results.

Fig. 4. Electrochemical parameters of the beam measured during ICCP protection: (a) Feeding voltage; (b) 4-hour decay potential.

Fig. 5. Failure modes of beams under fatigue loading: (a) Fatigue failure of non-strengthened beam (F0-I0-S); (b) Fatigue failure of strengthened beam – debonding between the CF meshes and cementitious matrix after reinforcement fracture (F2-I0-0.55); (c) Slippage and partial fracture of CF meshes after reinforcement fracture.

Fig. 6. Strain of reinforcement in fatigue loading: (a) Steel reinforcement; (b) Concrete; (c) Carbon fibre meshes.

Fig. 7. Typical load-deflection curves from fatigue tests: (a) F2-I20-0.55; (b) F2-I20-0.75.

Fig. 8. Deflection of beams after different fatigue cycles: (a) Effects of current density; (b) Effects of load level.

Fig. 9. Relationship between fatigue life and current density in ICCP and load level during cycling loads: (a) Effects of load level; (b) Effects of current density.

Fig. 10. Comparison results of S-N relation between FRP system and FRCM in this paper.

Fig. 11. Comparison results of fatigue life of steel in tension between the proposed model and international codes.

Fig. 12. Comparison results of fatigue life of beams: (a) Comparison of fatigue life based on the steel stress calculated from the beam theory; (b) Comparison of fatigue life based on the steel stress calculated from the measured strain of steel reinforcement.

Fig. 1. Detailed dimensions and test set-up of beam specimens (all dimensions in mm).

Fig. 2. Typical stress-strain curve of reinforcement obtained from static tensile test.

Fig. 3. S-N fitting curve for reinforcement fatigue test results.

(a) Feeding voltage

(b) 4-hour decay potential

Fig. 4. Electrochemical parameters of the beam measured during ICCP protection.

(a) Fatigue failure of non-strengthened beam (F0-I0-S)

(b) Fatigue failure of strengthened beam – debonding between the CF meshes and cementitious matrix after reinforcement fracture (F2-I0-0.55)

(c) Slippage and partial fracture of CF meshes after reinforcement fracture

Fig. 5. Failure modes of beams under fatigue loading.

(a) Steel reinforcement

(b) Concrete

(c) Carbon fibre meshes

Fig. 6. Strain of reinforcement in fatigue loading.

(a) F2-I20-0.55

(b) F2-I20-0.75

Fig. 7. Typical load-deflection curves from fatigue tests.

(a) Effects of current density

(b) Effects of load level

Fig. 8. Deflection of beams after different fatigue cycles.

(a) Effects of load level

(b) Effects of current density

Fig. 9. Relationship between fatigue life and load level during cycling loads and current density in

Fig. 10. Comparison results of S-N relation between FRP system and FRCM in this paper.

Fig. 11. Comparison results of fatigue life of steel in tension between the proposed model and international codes.

(a) Comparison of fatigue life based on the steel stress calculated from the beam theory

(b) Comparison of fatigue life based on the steel stress calculated from the measured strain of steel reinforcement

Fig. 12. Comparison results of fatigue life of beams.

677 **Table 1**

678 Labelling of specimens and corrosion degree.

Specimens	ICCP, i (mA/m ²)	CF mesh (layers)	Corrosion degree, ρ (%)
F0-I0-S	0	0	3.89
F2-I0-S	0	2	4.04
F2-I60-S	60	2	3.09
F0-I0-0.65	0	0	3.56
F0-I20-0.65	20	0	2.31
F2-I0-0.55	0	2	3.42
F2-I0-0.65	0	2	4.24
F2-I0-0.75	0	2	4.37
F2-I20-0.55	20	2	3.13
F2-I20-0.65	20	2	2.27
F2-I20-0.75	20	2	2.61
F2-I60-0.65	60	2	2.72
F2-I60-0.75	60	2	2.14
F2-I100-0.65	100	2	3.02
F2-I100-0.75	100	2	1.75
F2-I100-0.85	100	2	1.73

679

Table 2

Mechanical properties of steel in each beam.

Specimens	Static performance				Fatigue performance			
	f_y (MPa)	f_u (MPa)	ε_y (%)	ε_u (%)	σ_{s_max} (MPa)	σ_{s_min} (MPa)	S (MPa)	N ($\times 10^3$)
F2-I60-S	521	635	2.2	7.8	-	-	-	-
F0-I20-0.65	506	610	2.1	12.7	316	63	253	375
F2-I0-0.55	485	580	2.2	8.4	261	52	209	706
F2-I0-0.65	482	584	2.2	13.5	241	48	193	1792
F2-I0-0.75	495	574	2.7	5.3	251	50	201	702
F2-I20-0.55	475	575	2.3	8.6	291	58	233	899
F2-I20-0.65	490	575	2.2	11.7	326	65	261	191
F2-I20-0.75	466	571	2.1	13.4	462	92	370	79
F2-I60-0.65	507	611	2456	12.1	296	59	237	545
F2-I60-0.75	501	606	2.6	9.2	512	102	410	38
F2-I100-0.65	501	605	2.3	9.9	-	-	-	-
F2-I100-0.75	511	593	2.2	5.2	-	-	-	-
F2-I100-0.85	506	603	2.1	7.5	-	-	-	-

683 **Table 3**

684 Failure modes of tested beams.

Specimens	Number of fractured steels	Condition of concrete in compressed zone	Condition of FRCM	Location of main crack
F0-I0-S	0	Crushed	-	Midspan
F2-I0-S	0	Crushed	Slippage	Midspan
F2-I60-S	0	Crushed	Slippage	Midspan
F0-I0-0.65	1	Crushed	-	Midspan
F0-I20-0.65	1	Uncrushed	-	Midspan
F2-I0-0.55	2	Uncrushed	Interfacial debonding	Loading point
F2-I0-0.65	2	Uncrushed	Slippage & fracture	Loading point
F2-I0-0.75	2	Crushed	Slippage & fracture	Loading point
F2-I20-0.55	1	Uncrushed	Slippage & fracture	Loading point
F2-I20-0.65	2	Uncrushed	Slippage & fracture	Loading point
F2-I20-0.75	1	Crushed	Slippage & fracture	Midspan
F2-I60-0.65	1	Uncrushed	Slippage & fracture	Midspan
F2-I60-0.75	2	Uncrushed	Slippage & fracture	Midspan
F2-I100-0.65	1	Uncrushed	Slippage & fracture	Midspan
F2-I100-0.75	1	Uncrushed	Slippage & fracture	Loading point
F2-I100-0.85	2	Uncrushed	Slippage & fracture	Midspan

685

686 **Table 4**

687 Key results from fatigue tests.

Specimens	Load ratio (P_{max}/P_{u1})	Load ratio (P_{max}/P_{u2})	Max. steel stress, σ_{max}^1 (MPa)	Min. steel stress, σ_{min}^1 (MPa)	Stress range, S^1 (MPa)	Max. steel stress, σ_{max}^2 (MPa)	Min. steel stress, σ_{min}^2 (MPa)	Stress range, S^2 (MPa)	Experimental fatigue life ($\times 10^3$)
F0-I0-0.65	0.65	0.63	410	85	325	397	100	297	175
F0-I20-0.65	0.65	0.63	410	85	325	502	100	402	206
F2-I0-0.55	0.55	0.53	310	42	268	198	40	158	1300
F2-I0-0.65	0.65	0.63	372	54	318	260	39	221	437
F2-I0-0.75	0.75	0.73	432	67	365	412	101	311	285
F2-I20-0.55	0.55	0.53	310	42	268	227	39	188	1113
F2-I20-0.65	0.65	0.63	372	54	318	246	100	206	528
F2-I20-0.75	0.75	0.73	432	67	365	422	99	323	194
F2-I60-0.65	0.65	0.63	372	54	318	265	40	225	447
F2-I60-0.75	0.75	0.73	432	67	365	421	101	320	151
F2-I100-0.65	0.65	0.63	372	54	318	314	40	214	360
F2-I100-0.75	0.75	0.73	432	67	365	414	40	314	152
F2-I100-0.85	0.85	0.83	494	79	415	502	40	402	62

688 Note: P_{u1} is the ultimate bearing capacity of reference beam (F0-I0-S); P_{u2} is the ultimate bearing capacity of reinforced beam (F2-I0-S); S is the stress range of steel
689 in fatigue loading, $S = \sigma_{max} - \sigma_{min}$; ¹ Obtained from beam theory; ² Obtained from the measured strain of steel reinforcement.

690 **Table 5**

691 Tested and predicted fatigue life of beams (unit: $\times 10^3$ cycles).

Sequence number	Specimens	Fatigue life obtained from tests	Predicted fatigue life by JSCE ¹	Predicted fatigue life by fib ¹	Predicted fatigue life by S-N curve ¹	Predicted fatigue life by JSCE ²	Predicted fatigue life by fib ²	Predicted fatigue life by S-N curve ²
1	F0-I0-0.65	175	9.7	28.9	107.0	16.4	45.5	166.9
2	F0-I20-0.65	206	9.2	28.9	107.0	1.2	10.0	37.9
3	F2-I0-0.55	1300	85.2	75.3	273.2	7301.9	4050.0	3681.9
4	F2-I0-0.65	437	17.2	32.2	119.1	446.2	199.8	710.6
5	F2-I0-0.75	285	4.3	16.1	60.3	9.4	36.0	132.8
6	F2-I20-0.55	1113	84.7	75.3	273.2	1718.1	451.3	1578.2
7	F2-I20-0.65	528	16.9	32.2	119.1	787.8	282.7	998.1
8	F2-I20-0.75	194	4.3	16.1	60.3	6.8	29.9	110.6
9	F2-I60-0.65	447	17.8	32.2	119.1	391.8	181.8	648.0
10	F2-I60-0.75	151	4.6	16.1	60.3	8.2	31.4	116.2
11	F2-I100-0.65	360	17.7	32.2	119.1	230.3	232.5	824.4
12	F2-I100-0.75	152	4.5	16.1	60.3	9.1	34.2	126.3
13	F2-I100-0.85	62	1.3	8.5	32.2	1.2	10.0	37.9

692 Note: ¹ Calculated based on the beam theory; ² Calculated based on the measured strain of steel reinforcement.

Table 6 Available test data in literature.

Sources	Corrosion of steel (%)	Strengthening	Load level, S_L	Maximum steel stress, σ_{max} (MPa)	Stress range of steels in tension, S (MPa)	Experimental cycles to failure, N ($\times 10^3$ cycles)
Barnes et al. [23]	0	CFRP plates	0.39	303	272	508
	0		0.32	247	222	1889
	0		0.26	198	180	>2000
Song et al. [37]	8.3	CFRP sheets	0.39	-	-	1346
	13.8		0.39	-	-	459
	15.0		0.39	-	-	610
	16.7		0.39	-	-	304
Papakonstantinou et al. [40]	0	GFRP sheets	0.36	231	212	>2000
	0		0.41	258	239	685
	0		0.43	256	244	880
	0		0.45	268	250	800
	0		0.49	287	269	570
	0		0.53	331	312	235
	0		0.59	378	349	126
	0		0.73	427	402	31
Masoud et al. [41]	5.5	CFRP sheets	-	378	336	118
	9.0		-	389	346	95
	10.5		-	392	348	101
	5.5		-	316	281	232
	9.0		-	321	285	209
	10.5		-	322	286	216

694

695 **Table 6** Cont.

Sources	Corrosion of steel (%)	Strengthening	Load level, S_L	Maximum steel stress, σ_{max} (MPa)	Stress range of steels in tension, S (MPa)	Experimental cycles to failure, N ($\times 10^3$ cycles)
Charalambidi et al. [42]	0	CFRP plates	0.77	402	352	990
	0		0.78	414	365	1450
	0		0.73	403	352	807
	0		0.74	416	373	696
	0		0.70	396	340	774
Aidoo et al. [43]	0	CFRP plates	0.65	380	338	309
	0		0.51	279	279	1280
	0		0.65	380	306	193
	0		0.51	279	279	960

696

697 **Table 7**

698 Limit stress range of steel reinforcement.

Steel stress ratio ($\sigma_{min}/\sigma_{max}$)	Limit steel stress range ($\sigma_{max} - \sigma_{min}$), (MPa)	
	HRB 335	HRB 400
0	175	175
0.1	162	162
0.2	154	156
0.3	144	149
0.4	131	137
0.5	115	123
0.6	97	106
0.7	77	85
0.8	54	60
0.9	28	31

699

Table 8

Comparison of predicted fatigue life on HRB 335 and HRB 400 steels rebars.

Descriptions of items		Type of steel rebars	
		HRB 335	HRB 400
Information of steel	Nominal yielding strength, f_{sy} (MPa)	335	400
	Nominal ultimate strength, f_{su} (MPa)	455	540
	Diameter, ϕ (mm)	12	12
	Steel stress ratio, $\sigma_{min}/\sigma_{max}$ (MPa)	0.2	0.2
	Stress range, $\sigma_{max} - \sigma_{min}$ (MPa)	154	156
	Minimum stress, σ_{min} (MPa)	38.5	39
	Maximum stress, σ_{max} (MPa)	192.5	195
Prediction of fatigue life, N ($\times 10^6$ cycles)	fib model code [29]	10.1	7.1
	JSCE [30]	7.7	7.8
	GB 50010 [31]	2.0	2.0