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Fatigue behavior of damaged steel beams repaired with CFRP strips

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ABSTRACT

This paper presents the flexural behavior of damaged steel beams repaired with carbon fiber reinforced polymer (CFRP) strips. The damage is intentionally created by notching the tension flange of the beams. Six beams are tested to evaluate the static and fatigue performance of the repaired beams with emphasis on local plasticity and the CFRP–steel interface. A three-dimensional finite element analysis (FEA) is conducted to predict the experimental behavior. A modeling approach is proposed to simulate the fatigue response of the repaired beams, based on the strain-life method and cumulative damage theory. CFRP-repair results in a recovery of static load-carrying capacity of the damaged beam to that of an undamaged beam. The stress range at the damage influences the fatigue life, damage propagation, and plastic strain development of the repaired beams. Fatigue-crack propagation across the web of the beams is not significant up to 50% of their fatigue life, whereas brittle web fracture follows beyond the threshold. A bilinear fatigue response is observed at the CFRP-steel interface, whose magnitudes are dependent upon the number of fatigue cycles and the applied stress range. An empirical model is proposed to predict the fatigue behavior of the interface.

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1. Introduction

The issue of infrastructure management and rehabilitation is one of the primary interests in civil engineering community. Constructed bridge members deteriorate because of aging, corrosion, increased service loads and traffic volume, use of deicing salts, and collision of heavy trucks [1–3]. Over $180 billion is required to address structurally deficient or functionally obsolete bridges in the United States [4]. Of particular interest are steel bridges that account for more than 43% of the substandard bridges in the nation [5]. Steel bridges are susceptible to corrosion and fatigue cracks. Structural repair of damaged members provides a viable alternative to replacement, reducing downtime and cost. Traditional repair techniques for steel bridges include use of steel plates bolted or welded to damaged girders. Although the performance of such repaired girders is generally acceptable, the repairs themselves may introduce new potential problems associated with increased dead load, corrosion (crevice and galvanic corrosion), and the introduction of fatigue-sensitive details at the junction of existing and repair materials [6]. The application of carbon fiber reinforced polymer (CFRP) composites is proposed as an alternative to steel-plating repair methods. The advantages of CFRP materials include their non-corrosive characteristics, high stiffness- and strength-to-weight ratios, ease and rapidity of erection, and reduced long-term maintenance expenses [7]. A significant advantage of CFRP systems is that they are typically adhesively bonded, rather than mechanically connected, to the substrate steel, resulting in mitigation of the additional stress raisers associated with bolt holes or welds. Although CFRP composites have been primarily used for repairing concrete structures [8], there are relatively limited data available on strengthening damaged steel members using CFRP [9–15]. Nonetheless, this body of work clearly indicates the promise of this technique for strengthening damaged steel members subject to monotonic loads. Cadei et al. [16] provides significant guidance for such applications.

A number of recent studies have investigated the behavior of steel beams having damaged tension flanges repaired with CFRP patches and subject to fatigue loading [5,11,15,17–19]. Tavakkolizadeh and Saadatmanesh [5] assessed the fatigue behavior of intentionally notched steel beams ($L = 1.3$ m) patched with short CFRP sheets ($L = 0.3$ m). A design curve was generated to estimate the fatigue performance of repaired steel beams. CFRP-patching resulted in improvement in fatigue life of damaged beams of up to 3.4 times over that of unrepaired beams. Nozaka et al. [11] considered the performance of combinations of two CFRP materials and five adhesive systems to enhance the fatigue behavior of steel sections. They reported the greatest increase in fatigue strength resulting from the system combining the CFRP and adhesive having the lowest moduli of elasticity of those considered. O'Neill et al. [20] also report improved fatigue performance when

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using softer adhesive systems, attributing this to the greater redistribution of stresses resulting from such systems. Deng and Lee [17] studied fatigue behavior of small-scale steel beams ($L = 1.2$ m) retrofitted with CFRP patches. The initiation and development of cracks along the CFRP patch were monitored during cyclic loading. An empirical stress–fatigue life (S–N) curve was constructed using test results. Bocciarelli et al. [21] tested steel coupons bonded with CFRP plates in fatigue. A variety of stress ranges were simulated to evaluate bond between the CFRP and steel. Progressive bond failure at CFRP-steel interface was observed with increased fatigue cycles. The importance of interface design to improve fatigue performance of CFRP plates bonded to steel surface was discussed. Deng [18] proposed a model for crack initiation and crack growth prediction for CFRP-repaired steel beams in flexure, although this model is calibrated only against cases of moderate cycle fatigue ($N < 2 \times 10^5$). In summary, all studies report an extended fatigue life and a decreased crack growth rate provided the CFRP remained bonded to the steel substrate.

Notwithstanding the foregoing, very limited information is available on the fatigue behavior of steel structures repaired with CFRP composites; indeed the listing given above is believed to be exhaustive at the time of writing. Additionally, there has been no attempt to develop a numerical model for steel members repaired with CFRP composites in order to predict fatigue behavior, particularly the fatigue behavior of CFRP-steel interface. This paper addresses the flexural behavior of damaged steel beams repaired with CFRP strips, including the static and fatigue responses. The focus of the investigation is the local plasticity adjacent to a damage location and the viscoelastic nature of the adhesive bonding agent which is critical to the hysteretic behavior of the CFRP-steel interface. A modeling approach is proposed to predict the fatigue behavior of CFRP-repaired steel beams and is incorporated into a three-dimensional finite element (FE) model.

2. Research significance

Fatigue damage to steel girders is one of the most significant problems affecting the service life of existing bridges. Fatigue-induced cracks are frequently observed in constructed bridge members and corresponding repair is conducted. The application of CFRP composites to repair damaged steel girders is an emerging technique offering an attractive alternative to conventional repair methods such as steel-plating. Limited monotonic test results show the potential of such a repair method, whereas very limited information is available on the behavior of CFRP-repaired steel members subjected to transient (fatigue) loads. No research has been reported from the perspective of numerically modeling fatigue behavior. This paper proposes a systematic modeling approach to predict the behavior of damaged steel beams subject to fatigue loading conditions, based on the strain-life method and cumulative damage theory. An empirical model for the behavior of the CFRP–steel interface in fatigue is proposed. Debonding propagation of the CFRP and crack growth across the steel section are discussed.

3. Experimental program

An experimental program, described in full by Harries et al. [15], was carried out to validate the proposed modeling approach to be explained subsequently. Of particular interest was the behavior of the damaged beams at the crack tip and the efficacy of CFRP-repair, subjected to monotonic and fatigue loads. The following summarizes details of test beams, materials, and instrumentation.

3.1. Materials

W150 × 18 (US designation: W6 × 12) steel beams were used for the test program. The measured yield strength of the beams was 393 MPa with a modulus of elasticity of 200 GPa. The 50 mm wide by 1.4 mm thick CFRP strips used had a tensile strength of 2800 MPa and modulus of 155 GPa. The epoxy adhesives had a tensile strength of 25 MPa and a modulus of 4.5 GPa.

3.2. Specimen details

The beam specimens, shown in Fig. 1, were designed so that both steel fatigue and CFRP bond fatigue damage could be investigated. A total of six beams were tested as indicated in Table 1. A notch [Fig. 1(b)] was machined through the entire tension flange of Beams B through F at a location 152 mm to the right of midspan [Fig. 1(a)]. The notch serves as a stress concentrator to (a) initiate a vertical crack in the steel web and (b) initiate debonding of the CFRP propagating toward the right support [Fig. 1(a)]. The elastic stress concentration factor (typically denoted k) for the notch provided was 3.55 [22]. The corresponding stress concentration factor for fatigue, k_f, is typically less than that for static loading. For the notch geometry provided, the elastic stress concentration factor for fatigue, based on the estimation proposed by Neuber [23], is $k_f = 2.84$. Thus, in this case, the presence of the notch is essentially equivalent to AISC [24] fatigue stress category E.

Prior to bonding the 50.8 mm wide by 1.4 mm thick CFRP strips to Beams C–F, the steel surface was prepared by sanding using a 1500 sfpm (surface feet per minute) belt sander and a 40 grit zirconia alumina belt. This preparation resulted in a sound, slightly striated surface for bonding the CFRP strips. The thickness of the adhesive layer was approximately 1 mm.

3.3. Test set-up and instrumentation

All beams were subject to three-point flexural loading over a simply-supported span of 1830 mm [Fig. 1(a)]. A neoprene pad was placed in between the beam and support to reduce stress concentrations. Beams A–C were loaded monotonically to failure. Beams D–F were subjected to a transient load, cycling from a minimum of 4.4 kN to a maximum of 35.6 kN, 22.2 kN, and 13.3 kN, respectively, as listed in Table 1. The load ranges were equivalent to $0.73P_y$, $0.42P_y$, and $0.21P_y$, respectively, where P_y was the load to cause yield at the root of notch [Fig. 1(b)] of the repaired control Beam C. The loading frequencies varied from 1 Hz to 2.5 Hz (Table 1), which are commonly-used ranges for testing steel or concrete beams in fatigue [17,25] yet low enough to avoid self heating of the CFRP. The applied load was recorded by a load cell in line with the actuator. One strain gage (Gage 1) was located horizontally at the root of the notch to examine the local behavior of the critical section, as shown in Fig. 1(b). Additional gages (Gages 2–7) were bonded along the CFRP strip at 38 mm centers to monitor debonding propagation of the strip in the vicinity of the notch [Fig. 1(c)]. Fatigue crack and CFRP debonding propagation were measured manually at an applied load of P_{max} at pauses in the fatigue loading.

3.4. Results of monotonic tests

For Beams A–C, a monotonic load was applied at midspan of the beams until failure occurred. The ultimate behavior of all three monotonic tests was flexural torsional buckling (which was not restrained due to considerations of companion fatigue tests). The behavior of interest in this experimental program, however, is that in the elastic range and thus the ultimate behavior is not

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