FUNDAMENTAL CHARACTERISTICS OF NEW HIGH MODULUS CFRP MATERIALS FOR STRENGTHENING STEEL BRIDGES AND STRUCTURES

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Abstract

Due to corrosion and the continuous demand to increase traffic loads, there is a great need for an effective, cost-efficient system which can be used for the repair and strengthening of steel highway bridge girders. Recently, research has been conducted to investigate the use of carbon fiber reinforced polymer (CFRP) materials to address this need. This paper describes the details of an experimental program which was conducted to investigate the fundamental behavior of steel-concrete composite bridge girders strengthened with new high modulus CFRP (HM CFRP) materials. The behavior of the beams under overloading conditions and fatigue loading conditions was studied as well as the possible presence of a shear-lag effect between the steel beam and the CFRP strengthening. A series of proposed flexural design guidelines are presented which can be used to establish the allowable live-load increase for a strengthened beam and to design the required HM CFRP strengthening.

Introduction

The use of fiber reinforced polymer (FRP) materials for the repair and strengthening of concrete structures has gained widespread acceptance. Due to the success of this technique, several researchers have investigated the use of externally bonded CFRP materials for the repair and retrofit of steel bridges and structures. A number of different approaches have been investigated to assess the effectiveness of using CFRP materials for the retrofit of steel bridge members including repair of overloaded girders (Sen et al., 2001), repair of naturally deteriorated girders (Mertz and Gillespie, 1996), strengthening of undamaged girders (Tavakkolizadeh and Saadatmanesh, 2003c) and repair of girders with simulated corrosion damage (Al-Saidy et al., 2004). Other research has been conducted to study the fatigue durability of CFRP strengthening systems (Miller et al., 2001).

Early research focused on the use of conventional CFRP materials to repair steel-concrete composite bridge girders which were damaged due to severe overloading conditions (Sen et al., 2001). The CFRP strengthening system helped to increase the yield load and post-elastic stiffness of the beams.

Researchers have also investigated the use of CFRP materials to repair naturally corroded bridge girders (Mertz and Gillespie, 1996). Installation of the CFRP materials restored the elastic stiffness and moment capacity of the girders to levels comparable to those of the undamaged girders.

In another study, three undamaged steel-concrete composite beams were strengthened with one, three and five layers of CFRP strips respectively (Tavakkolizadeh and Saadatmanesh, 2003c). The CFRP materials also increased the ultimate capacity of the strengthened beams by up to 76 percent, however, the increase of the elastic stiffness was minimal. In a companion study, the tension flange of three other steel-concrete composite beams were notched with a 1.3 mm wide notch at midspan to simulate 25, 50 and 100 percent loss of the tension flange due to corrosion (Tavakkolizadeh and Saadatmanesh, 2003b). The repair restored the elastic stiffness and ultimate capacity of the girders to levels comparable to the undamaged state and helped to reduce the measured residual deflections due to overloading.

Other researchers have simulated corrosion damage by removing a uniform portion of the tension flange along the entire length of the girders (Al-Saidy et al., 2004). The repair technique was capable of restoring the lost strength of the damaged beams to levels higher than those of the unstrengthened girders. However, only 50 percent of the lost stiffness of the beams was recovered.

Investigations on the fatigue durability of steel beams strengthened with CFRP materials have been limited. Tavakkolizadeh and Saadatmanesh (2003a) demonstrated that externally bonded CFRP patches can be used to reduce crack propagation rates and increase the fatigue life of cracked steel members. Prestressed CFRP patches can also be installed to help promote crack-closure effects and further extend the fatigue life of cracked steel members (Bassetti et al., 2000). The fatigue durability of naturally corroded steel bridge girders which were repaired with CFRP materials has also been used investigated (Miller et al., 2001).

The majority of the previous research has focused on the use of conventional modulus CFRP materials for the repair of steel bridge members. While substantial strength increases have been achieved, typically large amounts of strengthening were required to achieve substantial increases of the elastic stiffness of the beams. This is due to the relatively low modulus of elasticity of the CFRP as compared to steel and also possibly due to the presence of shear-lag effects between the steel beam and the CFRP strengthening.

Recently, a strengthening system has been developed at North Carolina State University which uses new high modulus CFRP (HM CFRP) materials. These materials have a modulus of elasticity approximately two times greater than that of steel. The effectiveness of using HM CFRP materials to repair steel bridge girders was demonstrated by testing three large-scale steel-concrete composite beams strengthened with different configurations of HM CFRP materials (Schnerch, 2005). The elastic stiffness and ultimate moment capacity of the beams were increased by up to 36 percent and 45 percent respectively. The testing demonstrated that prestressing the HM CFRP strips prior to installation on the steel beam increased the efficiency of utilization of the CFRP and only half the amount of strengthening was required to achieve a comparable increase of stiffness as a beam strengthened with unstressed CFRP laminates.

This paper presents the details and relevant findings of an experimental investigation which was conducted to study the behavior of the strengthened beams under overloading conditions and fatigue loading conditions. Also, the possible presence of a shear-lag effect between the steel and the CFRP, which can limit the effectiveness of the strengthening system, was investigated in detail for a number of different loading conditions. Additional details about the research program are available in Dawood (2005). A series of guidelines are also proposed which can be used by practitioners to design the required HM CFRP strengthening for a given steel-concrete composite beam.

HM CFRP Strengthening System

The carbon fibers used in this study were the DIALEAD K63712 high modulus pitch based carbon fibers produced by Mitsubishi Chemical Functional Products Inc. The fibers were pultruded into 4mm thick, 100mm wide laminates by Epsilon Composite Inc., a French pultrusion company. The modulus of elasticity and ultimate strain of the strips which were reported by the manufacturer are 460 GPa and 0.00334 respectively. The laminates have a fiber volume fraction of 70 percent and were fabricated with a glass fiber peel ply on both faces to minimize the need for surface preparation of the composite material. The strips were bonded to the tension flange of the steel beams using the Spabond 345 two part epoxy adhesive with the fast hardener which is manufactured by SP Systems North America. This adhesive was selected from six adhesives in a previous phase of this research (Schnerch, 2005). The tension flange of the steel beam was grit blasted immediately prior to installation of the strengthening system to remove rust and mill scale from the surface. The surface was subsequently cleaned by air blowing and solvent wiping. After the CFRP strips were installed a wooden clamping system was applied for at least 12 hours until the adhesive had thoroughly set. The adhesive was allowed to cure for at least one week prior to testing.

Experimental Program

A total of seven steel-concrete composite beams were tested to study the effectiveness of the HM CFRP strengthening system. In the first phase of the research three beams were tested to study the behavior of the strengthening system under overloading conditions. The second phase consisted of three beams which were tested under fatigue loading conditions to investigate the durability of the strengthening system. The effect of the bonding procedure on the fatigue behavior of the beams was also examined. In the third phase, the seventh beam was tested to study the possible presence of shear-lag between the steel and the CFRP. The strengthened beams that were tested in the first and second phase of the research were also used to investigate the possible presence of a shear-lag phenomenon. The beams tested in all three phases of the experimental program consisted of scaled steel-concrete composite beams which are typical of most highway bridge construction. The typical cross-section of the tested beams is shown in Figure 1.

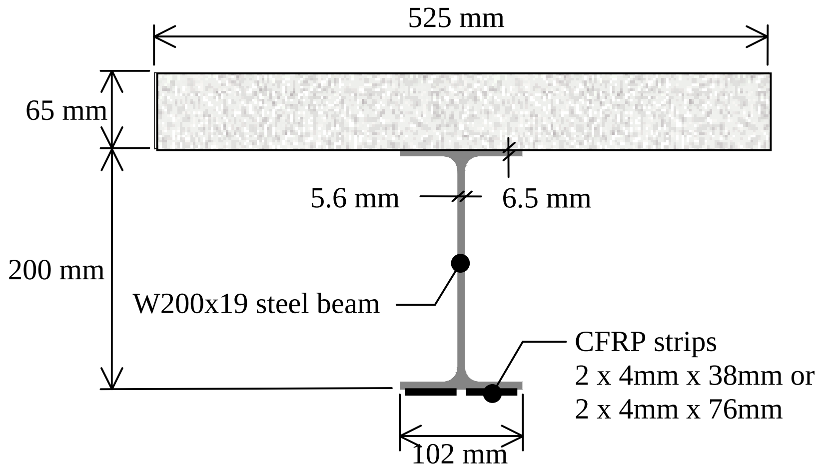


Figure 1: Cross-section of a typical test beam

The beams were strengthened with different levels of HM CFRP materials and tested in a four point bending configuration as shown in Figure 2.

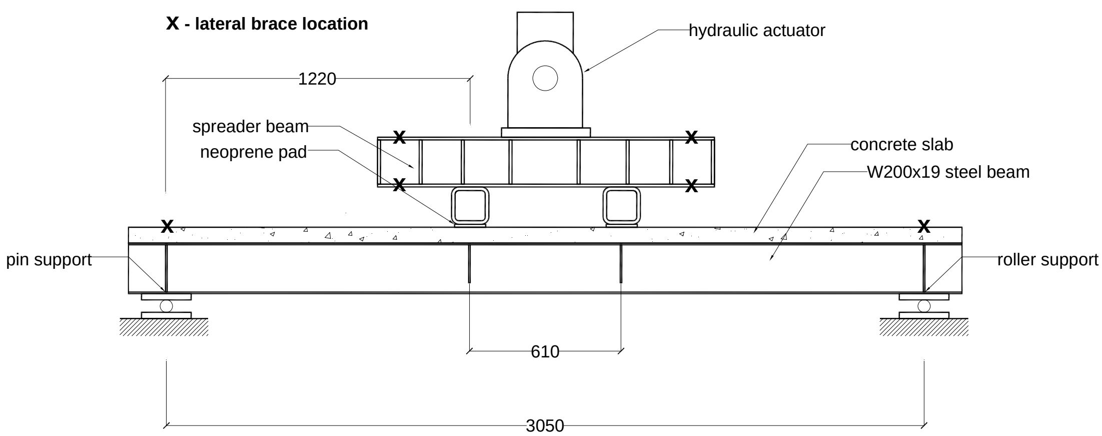


Figure 2: Beam test setup

The test matrix for the three phases of the experimental program is presented in Table 1. In the first phase, the behavior of the beams under overloading conditions was investigated. Two of the test beams were strengthened with two different levels of CFRP while the third remained unstrengthened to serve as a control beam for the overloading study. All three of the beams were unloaded and reloaded at various load levels to simulate the effect of severe overloading conditions.

The second phase was designed to study the fatigue durability of the strengthening system. Two different beams were strengthened with the same amount of CFRP materials, however, using different bonding techniques. The modified bonding technique involved increasing the final thickness of the cured adhesive and additionally including the use of a silane adhesion promoter. The third beam remained unstrengthened as a control beam for the fatigue study. All three beams were subjected to three million fatigue loading cycles with a frequency of 3 Hz. The minimum load in the loading cycle was selected as 30 percent of the calculated yield load of the unstrengthened beams to simulate the effect of the sustained dead-load for a typical bridge structure. The maximum load for the unstrengthened beam was selected as 60 percent of the calculated yield load to simulate the combined effect of dead-load and live-load. The maximum load for the two strengthened beams was selected as

60 percent of the calculated increased yield load of the strengthened beams to simulate the effect of a 20 percent increase of the allowable live-load level for a strengthened bridge.

The final beam was tested under monotonic loading conditions to investigate the possible presence of a shear-lag effect in the absence of more harsh loading conditions. The four strengthened beams which were tested in the first and second phases of the experimental program were also used to study the potential shear-lag effects.

Table 1: Test matrix for the three phases of the experimental program

Beam ID	**Reinforcement Ratio, ρ^***	**Adhesive Thickness, t_a**	**Concrete Strength, f_c'**	**Loading**
Overloading				
ST-CONT	0 percent	N/A	44 MPa	unload/reload
OVL-1	4.3 percent	0.1 mm	44 MPa	unload/reload
OVL-2	8.6 percent	0.1 mm	44 MPa	unload/reload
Fatigue				
FAT-CONT	0 percent	N/A	34 MPa	fatigue: P_{min}=50 kN, ΔP=50 kN
FAT-1	4.3 percent	0.1 mm	34 MPa	fatigue: P_{min}=50 kN, ΔP=60 kN
FAT-1b	4.3 percent	1.0 mm**	58 MPa	fatigue: P_{min}=50 kN, ΔP=60 kN
Shear-lag				
SHL	8.6 percent	0.1 mm	44 MPa	monotonic

*defined as the ratio of the cross-sectional area of the CFRP strengthening, accounting for the fiber volume fraction to the cross-sectional area of the steel beam

**included the use of a silane adhesion promoter

The tensile yield strength and modulus of elasticity of the steel beams were determined by coupon tests according to ASTM A370-02 as 380 MPa and 200,000 MPa respectively. The compressive strength of the concrete used for the concrete deck slabs for the seven test beams was determined from cylinder tests after 28 days in accordance with ASTM C39-03. The measured concrete cylinder strengths are presented in Table 1.

To accurately represent the actual behavior of a typical strengthened highway bridge, the two strengthened beams that were tested in the fatigue study were subjected to a sustained simulated dead-load prior to installation of the HM CFRP strengthening system. The simulated dead-load was applied using an independent loading apparatus as shown in Figure 3. Prior to installation of the CFRP, the simulated dead-load of 50 kN was applied by tightening nuts on a series of threaded rods as shown in the figure. The simulated dead-load was sustained on the beams while the CFRP strips were installed and during the curing process of the adhesive. After the adhesive cured, the load was transferred from the independent dead-load apparatus to the hydraulic actuator and the fatigue loading program was commenced.

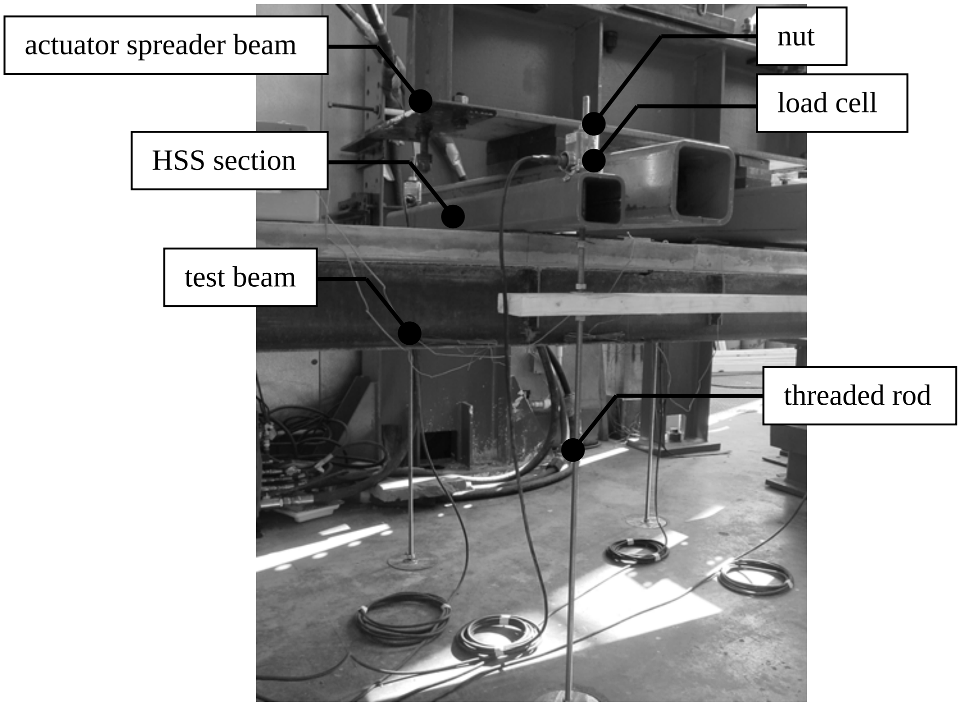


Figure 3: Independent dead-load apparatus

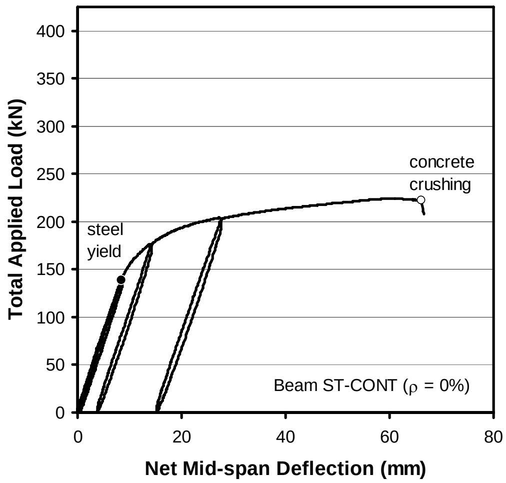
All of the test beams were instrumented to measure deflections at midspan and at the supports as well as to measure strains at various locations on the midspan cross-section of the beam. The measured strains were used to construct the strain profiles for the strengthened beams to investigate the possible presence of a shear-lag effect between the steel and the CFRP strengthening materials.

Experimental Results

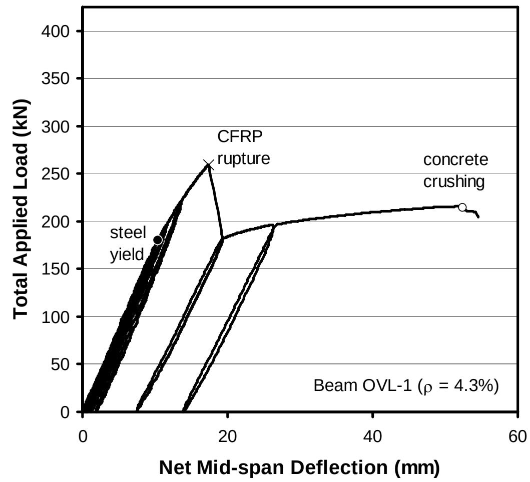
This section presents the results of each of the three phases of the experimental program and discusses the relevant research findings. The three phases of the experimental program are presented separately in the following sections.

Findings of the Overloading Study

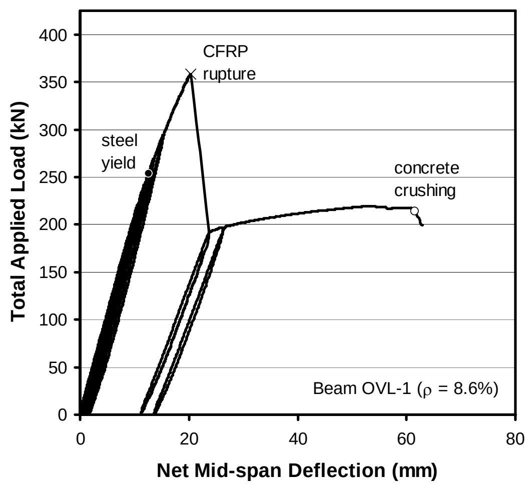
The load-deflection relationships of the three beams that were tested in the overloading study are presented in Figure 4. Beam ST-CONT remained unstrengthened to serve as a control beam for the overloading study while the remaining two beams were strengthened with different reinforcement ratios of CFRP. All three beams were unloaded and reloaded at various loading stages to simulate the effect of overloading conditions.



(a) beam ST-CONT



(b) beam OVL-1



(c) beam OVL-2

Figure 4: Load-deflection behavior of the three overloading test beams

The load-deflection behavior of the three beams was essentially linear up to yielding of the steel. Unloading and reloading was essentially linear and exhibited minimal hysteresis. Prior to yielding of the steel, all three beams exhibited minimal residual deflections upon unloading. However, after yielding of the steel, the unstrengthened beam exhibited a significant increase of the measured residual deflection, as shown in Figure 4(a), while the two strengthened beams continued to exhibit minimal residual deflections up to rupture of the CFRP. After rupture of the CFRP occurred the behavior of the strengthened beams followed a similar trend to that of the unstrengthened beam.

Table 2 presents the elastic stiffness increase, the yield load, the rupture load and the crushing load for the three beams tested in the overloading study.

Table 2: Comparison of the overloading beams

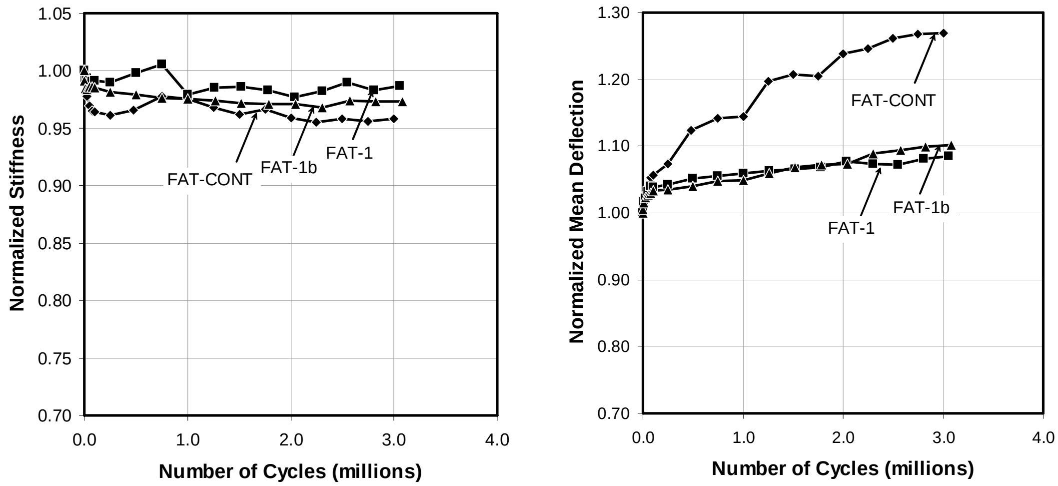
Beam ID	**Reinforcement Ratio**	**Stiffness Increase**	**Yield Load**	**Rupture Load**	**Crushing Load**
ST-CONT	0 %	N/A	137 kN	N/A	222 kN
OVL-1	4.3 %	27 %	181 kN	259 kN	216 kN
OVL-2	8.6 %	46 %	253 kN	358 kN	216 kN

The ultimate capacity of the unstrengthened beam, ST-CONT, was governed by crushing of the concrete while the ultimate capacity for the two strengthened beams, OVL-1 and OVL-2 was governed by rupture of the CFRP. The elastic stiffness, yield load and ultimate capacity of the beams were increased by 46 percent, 85 percent and 61 percent respectively using the higher reinforcement ratio. Inspection of Table 2 indicates that doubling the reinforcement ratio of the applied CFRP, from 4.3 percent to 8.6 percent, approximately doubled the elastic stiffness increase of the beams. Increasing the reinforcing ratio by two times also approximately tripled the increase of the measured yield load and ultimate capacity of the strengthened beams. This demonstrates that increasing the reinforcement ratio did not reduce the efficiency of utilization of the CFRP material. The improved performance of the strengthened beams indicates that it may be possible to increase the allowable live load level of a steel-concrete composite girder strengthened with HM CFRP materials. A proposed methodology for determining the allowable increase of the live load level is described later in this paper.

As mentioned previously, the presence of the HM CFRP materials helped to reduce the residual deflection of the strengthened beams due to the effect of overloading conditions. The residual deflections of the three beams that were tested in the overloading study were compared to evaluate the effectiveness of the HM CFRP strengthening system. Each of the three beams was unloaded at a load level of approximately 175 kN to simulate a severe overloading condition. The average measured strain at the steel tension flange of beams ST-CONT, OVL-1 and OVL-2 at the 175 kN load level were 2.5 ε_y, 1.0 ε_y and 0.6 ε_y respectively, where ε_y is the average yield strain of the steel determined from coupon tests. The beams were unloaded to a load level of 45 kN to simulate the sustained load acting on a structure, due to self-weight, after an overloading event. The plastic component of the residual deflection, after subtracting the initial measured elastic displacement of the beams at the 45 kN load level were compared. The measured residual deflection of beams OVL-1 and OVL-2 were respectively 5 times and 6.5 times lower than the measured residual deflection of beam ST-CONT. Consequently, under severe overloading conditions, an unstrengthened bridge girder may require repair or replacement while a strengthened bridge girder may remain serviceable.

Findings of the Fatigue Study

Three beams were tested in the fatigue study. Beam FAT-CONT remained unstrengthened to serve as a control beam for the fatigue study while beams FAT-1 and FAT-1b were strengthened with a reinforcement ratio of CFRP of 4.3 percent using two different bonding techniques. The two strengthened beams were tested with a 20 percent increase of the applied load range to simulate the effect of increasing the allowable live load for the strengthened beams. All three beams survived a three million-cycle fatigue loading course without exhibiting any indication of failure. Figures 5 (a) and (b) present the degradation of the stiffness and mean deflection respectively of the three beams throughout the three million-cycle loading course. Figure 5 presents the stiffnesses and mean deflections of the beams normalized with respect to the initial values at the beginning of the fatigue loading program.



(a) stiffness (b) mean deflection

Figure 5: Degradation of (a) stiffness and (b) mean deflection for the fatigue beams

All three beams exhibited a minimal degradation of the elastic stiffness of less than 5 percent throughout the three million fatigue loading cycles as shown in Figure 5(a). However, beam FAT-CONT exhibited a nearly 30 percent increase of the mean deflection due to the applied fatigue cycles as shown in Figure 5(b). This was likely due to the fatigue-creep behavior of the concrete deck slab. Both of the strengthened beams exhibited superior performance with an increase of only 10 percent in the measured mean deflection. The observed degradation of the two strengthened beams throughout the three million fatigue cycles was similar which indicates that the bonding technique did not affect the fatigue behavior of the strengthening system.

At the completion of the fatigue program, the three beams were loaded monotonically to failure. The load deflection behavior of the beams followed a similar trend to the observed load-deflection envelope of the three beams that were tested in the overloading study. The ultimate capacity of the two strengthened beams, FAT-1 and FAT-1b, was governed by rupture of the CFRP at a load of 250 kN. After rupture of the CFRP, the load deflection behavior of the two beams followed a similar trend to the load-deflection behavior of the unstrengthened beam, FAT-CONT. Crushing of the concrete for all three beams occurred at a measured load of between 200 kN and 215 kN. The findings of the fatigue study demonstrate the durability of the strengthening system when loaded at an increased live-load level. The allowable increase of the live load should be selected in accordance with the proposed design guidelines outlined later in this paper.

Findings of the Shear-lag Study

One additional beam, SHL, was tested to investigate the presence of a shear-lag effect between the steel beam and the CFRP strips under monotonic loading conditions. The beam was strengthened with a reinforcement-ratio of CFRP of 8.6 percent. The load-deflection behavior of beam SHL followed a similar trend to the load-deflection envelope of beam OVL-2, shown in Figure 4(c), which was strengthened with the same reinforcement ratio of CFRP.

The measured strain profiles at the midspan cross-section of beams SHL and OVL-2, immediately prior to rupture of the HM CFRP strips, are shown in Figure 6. The measured strain profiles for both beams indicate a slight discontinuity of the strain profile between the steel tension flange and the CFRP. However, the opposite sign of the discontinuity suggests that the behavior is not due to the presence of a shear lag effect. The measured discontinuity is likely due to the effect of residual stresses in the steel beam which formed during the manufacturing process or due to possible local instability or lateral movement of the tension flange of the steel beam. The presence of these effects was confirmed by independent strain measurements at different locations on the steel tension flange of the strengthened and unstrengthened test beams. The measured strain profiles for the remaining three strengthened test beams, OVL-1, FAT-1 and FAT-1b, were essentially linear and exhibited minimal discontinuities between the steel and the CFRP. The findings of the shear-lag study indicate that the effect of shear-lag is negligible and the plane sections remain plane assumption is appropriate for the analysis of the strengthened beams.

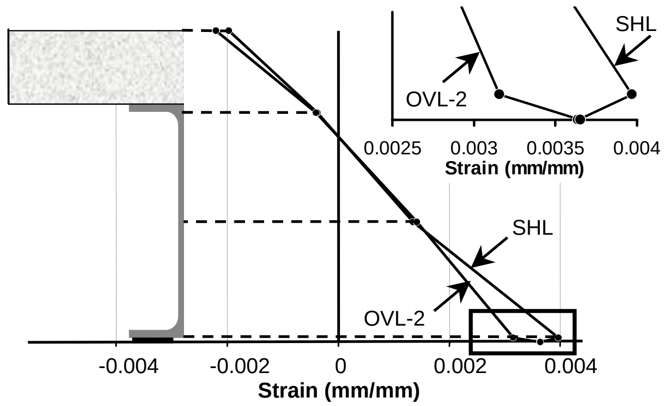


Figure 6: Strain profiles for beam SHL and OVL-2 (ρ = 8.6%)

Design Guidelines

Based on the findings of this research program a series of design guidelines have been developed which can be used by practitioners to design HM CFRP strengthening for steel-concrete composite beams (Schnerch et al., 2005). This section presents a summary of the proposed flexural design procedure including a discussion of the determination of the allowable increase of the live-load level for a strengthened beam.

The flexural analysis and design of a steel-concrete composite beam strengthened with HM CFRP materials are based on a non-linear moment-curvature analysis. The analysis satisfies the requirements of equilibrium and compatibility and neglects the effect of shear-lag between the steel and the CFRP. A non-linear material characteristic is used to represent the stress-strain behavior of the concrete and the steel while the CFRP is assumed to remain linear and elastic to failure.

The moment-curvature behavior of a given cross-section is determined based on the strain at the top level of the compression flange together with an assumed neutral axis depth. The cross-section is broken down into levels corresponding to the concrete deck, the longitudinal steel reinforcement of the concrete deck, the flanges and web of the steel beam, and the HM CFRP strips at the bottom of the cross-section. The strain, ε_x, at any distance, x, from the neutral axis of the section can be calculated using Equation (1) as,

$$\varepsilon_x = \frac{\varepsilon_c}{c} x \qquad \text{Equation (1)}$$

where ε_c is the strain at the top surface of the concrete deck and c is the assumed neutral axis depth.
From the strain profile and the constitutive relationships of the materials, the corresponding stress profile for the beam can be established. The corresponding resultant forces for the different elements of the cross section can be calculated by integration of the stress profile using Equation (2),

$$F = b \int f(x) dx \qquad \text{Equation (2)}$$

where F is the calculated resultant force, b is the width of the element under consideration and f(x) is the constitutive relationship of the material. The assumed neutral axis depth is iterated until horizontal force equilibrium is satisfied. The corresponding moment can then be calculated by replacing the term f(x) in Equation (2) by the term x f(x) and re-evaluating the integrals. The curvature of the section can then be calculated from the strain at the top surface of the concrete deck, ε_c, and the appropriate neutral axis depth, c. This strain is subsequently increased to determine the next increment of curvature and the procedure is repeated. This procedure can be easily extended to predict the load deflection behavior of a beam with a given loading and support configuration by integration of the curvature profile using any commonly accepted method.

Based on the proposed moment-curvature analysis procedure, the allowable increase of live load for a steel-concrete composite beam strengthened with HM CFRP materials should be selected to satisfy three conditions. These three conditions are shown in Figure 7 with respect to the moment-curvature response of a typical steel-concrete composite beam section strengthened with HM CFRP materials. Due to the presence of the additional layer of HM CFRP material, the yield moment of the strengthened beam, $M_{Y,S}$, is greater than the yield moment of the unstrengthened beam, $M_{Y,US}$. This was verified by the findings of the overloading study. Based on the findings of the fatigue study, to maintain the fatigue life of the strengthened beam, the total applied moment acting on the strengthened section under service loading conditions, including the effect of dead load, M_D, and the increased live-load, M_L, should not exceed 60 percent of the increased yield moment of the strengthened section. To satisfy the strength limit state, the total factored moment based on the appropriate dead-load and live-load factors, α_D and α_L respectively, should not exceed the ultimate moment capacity of the strengthened section, $M_{U,S}$. Also, to ensure that the structure remains safe in the case of total loss of the strengthening system, the total applied moment, including the effect of dead-load and the increased live-load should not exceed the residual nominal moment capacity of the unstrengthened section, $M_{n,US}$.

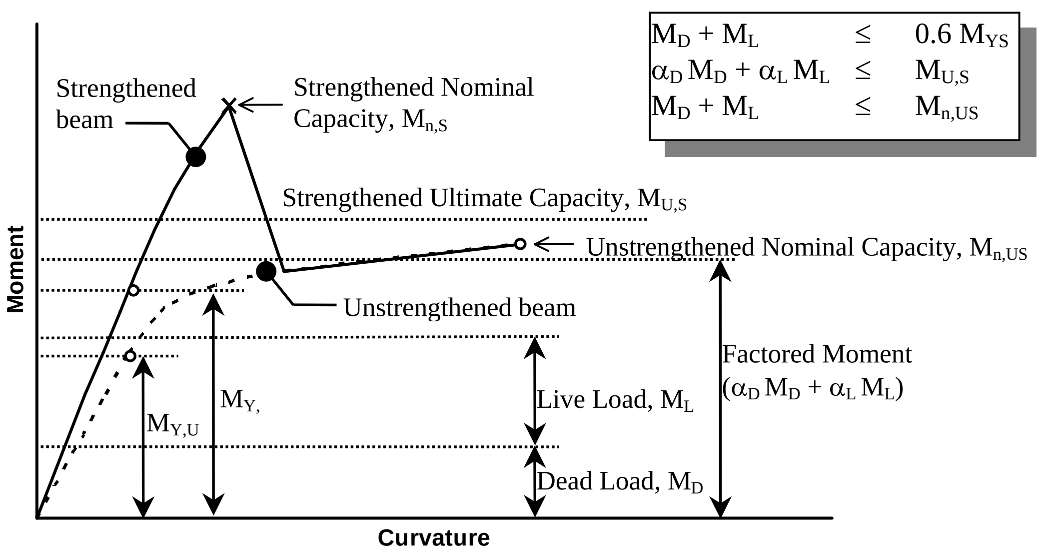


Figure 7: Load levels and moment-curvature behavior for a strengthened beam

While the nominal behavior of the member can be used to predict the behavior under service loading conditions, the design ultimate capacity should incorporate suitable reduction factors to ensure that the member remains safe. These reduction factors should account for the uncertainty of the HM CFRP material properties and should take into consideration the sudden, brittle failure which is typical of most HM CFRP strengthened steel-concrete composite beams. This guideline has adopted the approach outlined in ACI 440.2R-02 for the calculation of the ultimate capacity of concrete beams strengthened with externally bonded FRP materials. However, the approach has been modified to account for the inherent difference between the behavior of steel beams and concrete beams.

To account for the statistical uncertainty of the measured ultimate capacity of the HM CFRP materials, the mean strength of the CFRP reported by the manufacturer, $\bar{f}_{FRP,u}$, should not be used directly in calculating the ultimate capacity of the strengthened section. Rather, the average ultimate strength of the FRP should be reduced by 3 times the standard deviation, σ, as in Equation (3) (ACI 440.2R, 2002).

$$f^{*}_{FRP,u} = \bar{f}_{FRP,u} - 3\sigma \qquad \text{Equation (3)}$$

To account for possible environmental degradation of the CFRP materials throughout the lifetime of the strengthening, ACI 440.2R-02 recommends the use of an environmental reduction factor, C_E. For carbon fiber materials subjected to exterior exposure, which is typical for most bridge structures, a value of C_E of 0.85 should be used. Therefore, the design strength of the HM CFRP material can be calculated as

$$f_{FRP,u} = C_E f^{*}_{FRP,u} \qquad \text{Equation (4)}$$

The design ultimate strain of the CFRP material, $\varepsilon_{FRP,u}$ can be calculated by dividing the calculated design strength of the CFRP by the average elastic modulus, E_{FRP}, reported by the manufacturer.

The nominal moment capacity of the strengthened member, $M_{n,S}$, should be calculated using the proposed moment-curvature procedure and the design strength and ultimate strain of the CFRP. The nominal capacity of a steel-concrete composite beam strengthened with high modulus CFRP materials is typically governed by rupture of the CFRP materials. This type of failure occurs in a sudden, brittle manner without significant warning. To account for the brittle nature of failure, a strength reduction factor, ϕ, of 0.75 is recommended. This reduction factor is consistent with the reduction used in the AISC LRFD Specification (2001) for rupture type limit states. The design ultimate capacity of the strengthened beam, $M_{U,S}$ should be calculated as $\phi M_{n,S}$.

Conclusions

This paper presents the details of an experimental program which was conducted in three phases to investigate the fundamental behavior of steel-concrete composite beams strengthened using high modulus CFRP materials. Based on the findings of the first phase of the experimental program, it is evident that HM CFRP materials can be used to increase the elastic stiffness, yield load and ultimate capacity of steel-concrete composite beams which are typical of most highway bridge structures. Additionally, the presence of the CFRP helps to reduce the residual deflection due to overloading conditions which can help reduce or eliminate the need for future repair or replacement of the structure. The fatigue durability of the strengthening system was demonstrated in the second phase of the experimental program. Two beams were strengthened with a reinforcement ratio of CFRP of 4.3 percent and subjected to three million fatigue loading cycles with an increase of the simulated allowable live-load level of 20 percent as compared to an unstrengthened control beam. Both strengthened beams exhibited superior fatigue performance to the unstrengthened control beam with regards to the fatigue-creep behavior of the concrete deck. Further, the bonding technique did not appear to affect the fatigue behavior of the strengthening system. Based on the measured strain profile of the five strengthened beams which were investigated in the shear-lag study, the effect of shear-lag between the steel beam and the CFRP materials is minimal.

This paper also presents a proposed flexural design procedure, which is based on a non-linear moment-curvature analysis, which can be used to design the required HM CFRP strengthening for a steel-concrete composite beam. Based on the proposed guidelines, the allowable live load increase for a strengthened beam should satisfy three conditions. Particularly,

(i) $M_D + M_L \leq 0.6\, M_{YS}$
(ii) $\alpha_D M_D + \alpha_L M_L \leq M_{U,S}$
(iii) $M_D + M_L \leq M_{n,US}$

The findings of this research demonstrate that new externally bonded HM CFRP materials provide an effective, cost-efficient repair alternative for conventional steel-concrete highway bridge girders.

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