FATIGUE BEHAVIOR OF CONCRETE BEAMS STRENGTHENED IN FLEXURE WITH NEAR SURFACE MOUNTED CFRP

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1 INTRODUCTION

Increasing the bending moment capacity of existing in-service steel reinforced concrete flexure members can be accomplished with strategically located fiber reinforced polymer (FRP) composite reinforcement materials. In this application, the FRP strengthening reinforcement is either externally bonded to the tension surface of the member or internally located in a near surface mounted location (NSM). As external reinforcement the FRP is typically available as a sheet or fabric, and as NSM reinforcement the FRP is typically solid circular rods of thin rectangular strips. In either of these two cases, force transfer and composite behavior with the existing member is accomplished using an epoxy adhesive to bond the FRP and concrete together, as is shown in Figure 1. The strengthened member's yield and ultimate bending moment capacity will thus be increased due to the additional tensile force provided by the supplemental FRP reinforcement.

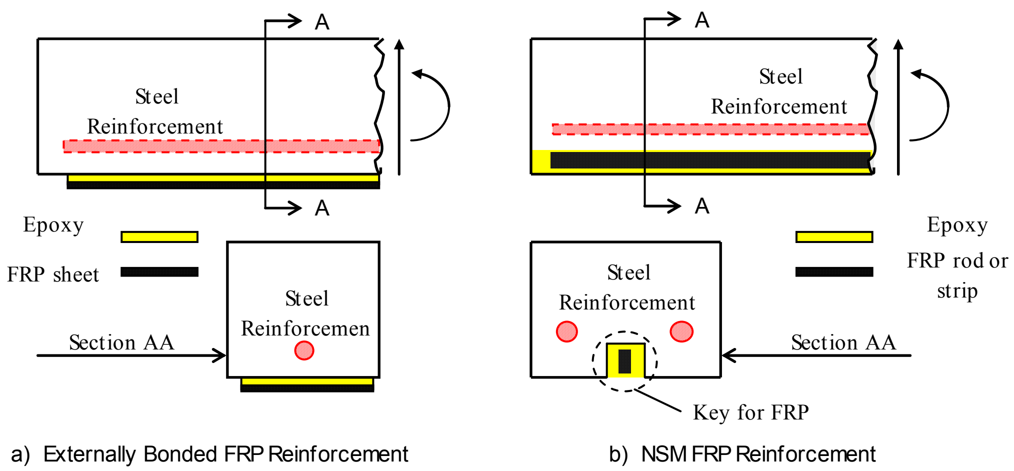


Fig. 1 Flexural strengthening using FRP.

Successful behavior of the FRP strengthened member for the extended service life of the structure will require that the three structural materials (concrete, steel and FRP) function effectively together as a single composite system. Durability of the system will be affected by environmental conditions such as freeze thaw and temperature cycling, and exposure to aggressive chemical agents and ultraviolet light. Additionally, durability of the system could be compromised by fatigue loading, which may result in deterioration and weakening of the individual system components (steel, FRP, concrete) or loss of bond and composite action. Thus, the effects of fatigue loading could be especially problematic when this technology is used for strengthening highway bridges and other concrete structures subjected to millions of load cycles over their service lives. In such cases, the static capacity of the strengthened system may be of no significance if the effects of fatigue loading result in a limit state failure other than that associated with flexural failure. For example, fatigue fracture of the tensile steel reinforcement in concrete beams fatigue loaded between 25% and 50% of ultimate has been reported by Shahawy and Beitelman [1]. For beams strengthened in flexure with externally bonded CFRP plates and load cycled between 4% and 43% of ultimate, Barnes and Mays [2] also reported fatigue fracture of the internal steel reinforcement as the dominant factor governing failure. However, for concrete beams strengthened with NSM CFRP strips and fatigue loaded for 2,000,000 cycles between 10% and 40% of ultimate Quattlebaum et al. [3] reported no failure of the reinforcements (steel or CFRP) and no loss in bond integrity between the CFRP and concrete. These research studies and those of others [4, 5] show that failure limit states associated with fatigue loading must be clearly understood so that the expected static capacity of the strengthened section capacity is achievable.

The objective of this research is to investigate how service level fatigue loading for 2,000,000 cycles affects the static strength and stiffness of simply supported steel reinforced concrete beams strengthened in flexure with NSM carbon fiber reinforced polymer (CFRP) bars and strips. Companion monotonically loaded and fatigue loaded samples provide a basis of comparison that allows for evaluation of how fatigue loading effects failure limit state, yield and ultimate strength, and flexural stiffness.

2 EXPERIMENTAL STUDY

For this research a total of 8 beams were tested, four of which were loaded monotonically to failure and four of which were fatigue loaded for 2,000,000 cycles before being monotonically loaded to failure. Of the four beams in each group (monotonic and fatigue), two were strengthened with NSM CFRP bars, and two were strengthened with thin CFRP strips. The CFRP bars and strips were provided by Hughes Brothers, Inc. [5]. All test beams were simply supported, 2740 mm (108 in) long, and had a cross section of 18.5 mm (7.25 in) high by 229 mm (9 in.) wide. The diameter, tensile strength, and elastic modulus of the CFRP bars are 6.4 mm (0.25 in.), 1725 MPa (250 ksi), and 150 GPa (21810 ksi), respectively. The CFRP strips had a thin rectangular cross section of approximately 15.5 mm by 0.25 mm, and the side surface had a roughened texture to improve force transfer. The cross section area, tensile strength and elastic modulus of the CFRP strips are 0.51 mm^2, 1650 MPa (239 ksi), and 138 GPa (20020 ksi), respectively. Table 1 identifies the test matrix and reinforcement data, and Figure 2 provides all additional relevant experimental details.

Table 1 Experimental Matrix and Reinforcement Properties.

Sample ID		concrete strength f'_c (MPa)	Steel Reinf. (A_s)	CFRP Reinforcement			
Monotonic Samples	Fatigue Samples			type	size	F_{fu} (MPa)	E_f (GPa)
BM1, BM2	BF1, BF2	37	2 x 6.4 mm diam. rebars	2 bars	6.4 mm diam.	1725	150
SM1, SM1T	SF1, SF2			2 strips	15.5 mm x .25 mm	1650	138

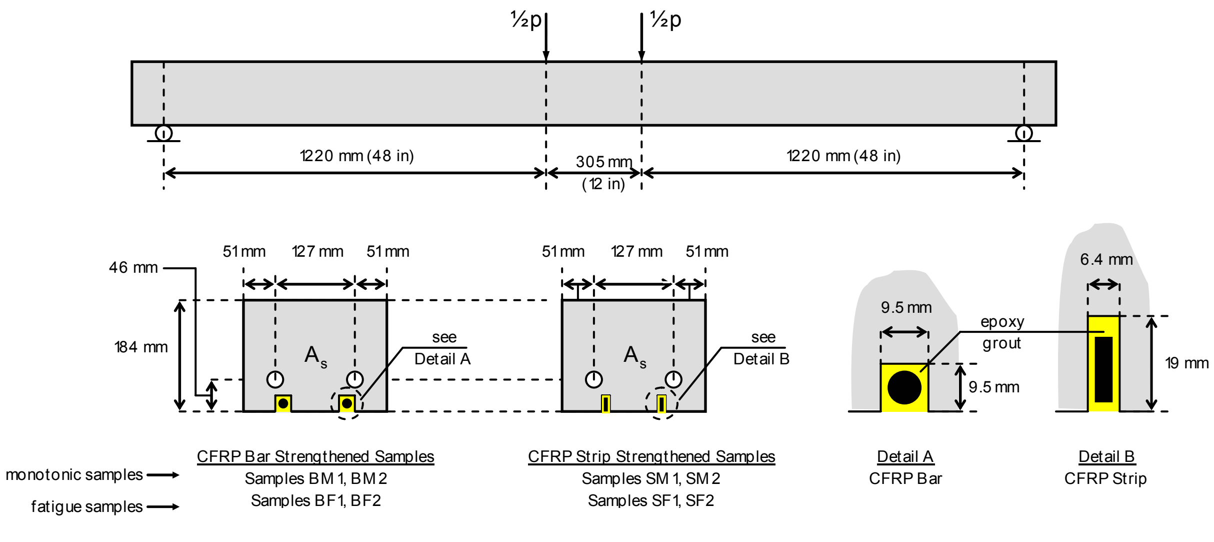


Fig. 2 Experimental Details.

The testing program consisted of the following procedures: 1) sample fabrication, 2) NSM groove cutting, 3) precracking, 4) strengthening, 5) fatigue loading (fatigue samples only), and 6) test-to-failure (all samples). These steps are briefly described in the following paragraphs.

All formwork was constructed using readily available 50 mm x 200 mm (2"x8") dimensional lumber and 20 mm (¾ inch) sheets of plywood. The formwork consisted of two identical pallets with 4 identical beams per pallet. Steel rebar chairs were used to provide 28 mm (1.5 in) clear cover and zip ties were used to hold the steel rebars and rebar chairs firmly in contact with the plywood base. All eight beams were cast simultaneously using concrete provided by a local ready-mix supplier. The concrete slump at the pour was 100 mm (4 in). The concrete used in this investigation was Pennsylvania Department of Transportation (PennDOT) Class AAA concrete [6]. Class AAA concrete is typically used in bridge decks and has a minimum required compressive strength of 28 MPa (4 ksi). It is understood that at this point all eight test beams are identical in size, steel reinforcement, and concrete material strength.

The next step was to cut the NSM grooves into which the CFRP bars and strips would be placed and grouted. For the beams to be strengthened with NSM bars (BM1, BM2, BF1, BF2), two full length longitudinal groves were cut into the tension side of the beams. Each groove had a cross section of approximately 9.5 mm by 9.5 mm (Figure 2). For the beams to be strengthened with NSM strips (SM1, SM2, SF1, SF2), two full length longitudinal groves were cut into the tension side of the beams. Each groove had a cross section of approximately 6.4 mm wide by 19 mm deep (Figure 2). All cuts were made using a circular saw and diamond tip blade with the beams rotated 180 degrees so the tension surface was facing up.

To simulate a flexural member on a highway bridge with many years of service, all eight beams in this study were precracked before strengthening with CFRP. The intent was to simulate the cracked section condition that would likely characterize an in-service candidate structure for strengthening. Precracking consisted of placing a single point load of 15.6 kN (3.5 kips) at four different locations along the span. To simulate a truck traveling across the beam span the moving load sequence consisted of placing the point load relative to the left support at 610 mm (24 in), 1220 mm (48 in), 1525 mm (60 in), and 2135 mm (84 in). The corresponding moment envelop implied that approximately 70% of the span length would be cracked after application of the four precracking load cases. Figure 3 shows details and a typical crack pattern related to the precracking procedure. The maximum moment from the envelope is approximately 45% of the unstrengthened section moment capacity.

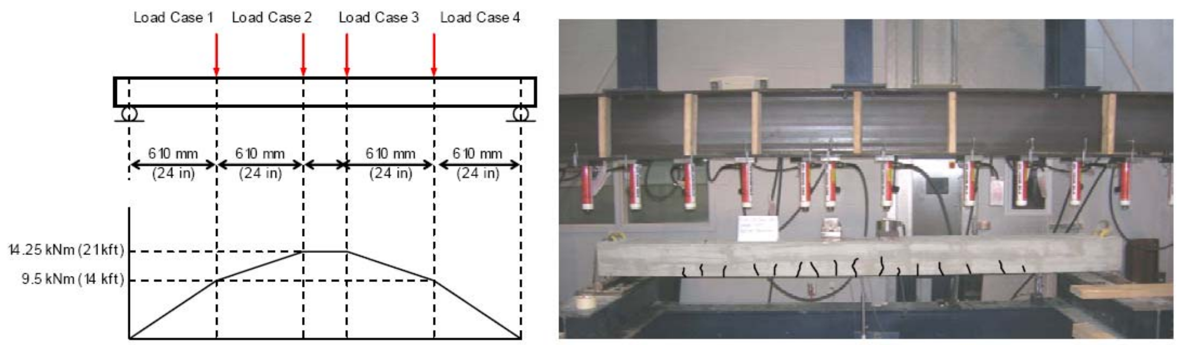


Fig. 3 Precracking Load Cases, Moment Envelop and Typical Crack Pattern.

Following precracking the CFRP strengthening reinforcement was installed into the longitudinal grooves previously cut. The CFRP bar and strip reinforcements were mounted in the near surface position using Concresive 1420 grout manufactured by MBT Protection and Repair Division of Degussa/BASF [7]. This is a two part grout that is applied using an applicator gun designed for a two part grout (Figure 4a). The 300 mm (12 inch) nozzle is designed to mix the grout evenly. The application began by inverting the beam so that the cuts were exposed. The grout was then pumped through the tube and into the groove. The grout was placed so it filled approximately ¾ of the depth of the groove along the entire length of the beam (Figure 4b). The CFRP was lined up at its proper location and pressed down into the grout. The FRP was moved back and forth using a sawing action to make sure there were no pockets where grout was not applied. Care was taken to check that the FRP was in the center of the groove and grout was present on each side. After the FRP was in place, another pass was made with the grout applicator to fill in the top of the cut. A metal trowel was used to remove any excess grout on the surface of the beam. The epoxy grout was allowed to cure for a minimum of two weeks prior to any testing.

Grout applicaltion apparatus — Partially filling groove with grout — Fully installed CFRP reinforcement

Fig. 4 Installation of CFRP Reinforcement.

Fatigue testing and monotonic testing-to-failure consisted of four point loading with a 305 mm constant moment region, as was shown in Figure 2. The fatigue samples (BF1, BF2, SF1, SF2) were loaded in force control between 3.1 kN (0.70 kips) and 20 kN (4.5 kips) at 2.5 Hz for 2,000,000 load cycles. At the conclusion of fatigue loading these samples, together with the monotonic samples (BM1, BM2, SM1, SM2) were loaded statically to failure. During this phase (static test-to-failure) all samples were loaded in force control at 4.448 kN/minute (1 k/minute) up to 35.6 kN (8 kips) and then at 2.22 kN/minute (0.50 k/minute) to failure. This bilinear static load function was selected to maintain a high resolution of data collection during the inelastic phase that was expected at loads of 40 kN (9 kips) and above. Load and deflection data was collected at a sampling frequency of 2 Hz during failure testing.

3 FLEXURAL ANALYSIS

In the unstrengthened condition all beams are underreinforced relative to a balanced strain condition ($\rho_s < \rho_b$). Using compatibility and equilibrium, the ratio of provided steel reinforcement ratio (ρ_s) to that corresponding to a balanced design (ρ_b) is calculated as follows:

$$\rho_s / \rho_b = \left\{ A_s /(bd) \right\} / \left\{ (0.85 f_c^{'} \beta_1 \varepsilon_{cu}) / (f_y (\varepsilon_{cu} + \varepsilon_y) \right\} \quad (1)$$

Using the material properties for steel and concrete given in Section 2 and noting that β_1 is calculated to be 0.785 using ACI 318-05 [8], the result of Eq. (1) for the unstrengthened condition is 0.45 of a balanced design. This relative reinforcement ratio (i.e. 0.45) was intentionally selected to represent the underreinforced design requirement ($\rho_s < \rho_b$) that would be typical for existing concrete structures. The unstrengthened moment strength is calculated using Eq. (2).

$$M_{nu} = A_s f_y \{d - a/2\} \quad where \quad a = \{A_s f_y\} / \{0.85 f_c^{'} b\} \quad (2)$$

Substitution into Eq. (2) yields an ultimate moment strength of 25.2 kNm (223 kin) with a corresponding failure load of 41.33 kN (9.3 k). Thus, the maximum fatigue load of 20 kN (4.5 k) corresponds to approximately 50% of the unstrengthened flexural capacity.

In the strengthened condition two limit states are considered [9]: a) steel yield followed by concrete crushing, and b) steel yield followed by CFRP rupture. Considering the reinforcement ratios and properties of the materials used in this study, all strengthened samples are controlled by steel yield followed by concrete crushing. For this limit state, the neutral axis depth (c) and moment capacity (M_{nu}) for the strengthened condition are calculated using compatibility and equilibrium as follows:

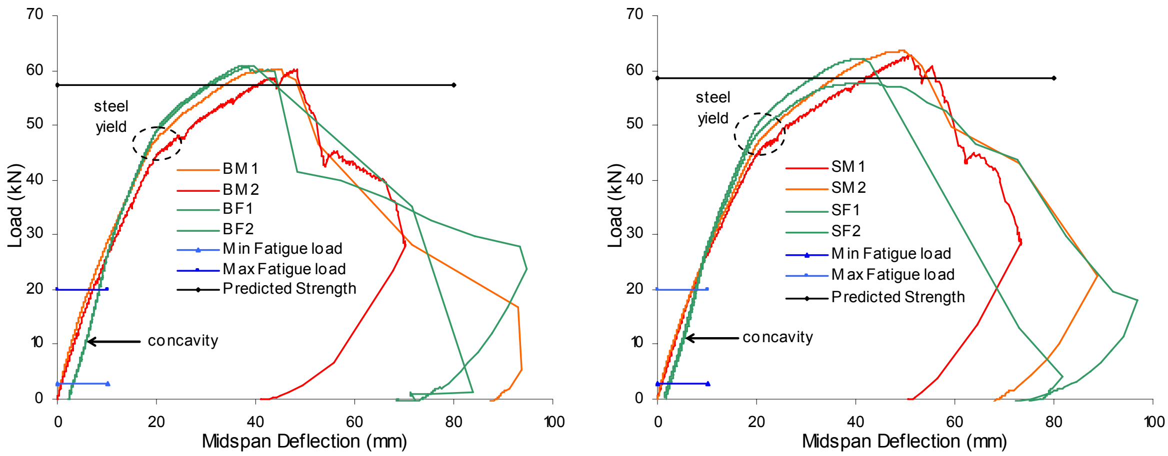
$$c = \frac{\sqrt{(A_f E_f \varepsilon_{cu} - A_s f_y)^2 + 4 \cdot 0.85 \cdot f'_c b \beta_1 A_f E_f \varepsilon_{cu} d_f} - (A_f E_f \varepsilon_{cu} - A_s f_y)}{2 \cdot 0.85 f'_c b \beta_1} \quad (3)$$

$$M_{nu} = A_f f_f \left(d_f - \beta_1 c/2\right) + A_s f_y \left(d_s - \beta_1 c/2\right) \tag{4}$$

Substitution into Eq's (3) and (4) for bar and strip strengthened samples yields an ultimate moment strength of 35 kNm and 35.7 kNm (310 kin and 316 kin), respectively. The corresponding failure loads at concrete crushing for bar and strip strengthened samples are 57.4 kN and 58.6 kN (12.91 kips and 13.17 kips), respectively. Thus, the unstrengthened-to-strengthened ultimate moment strength is approximately 1.40 (i.e. 35.3kNm/25.2kNm).

4 TEST RESULTS

Load-deflection failure test results for CFRP bar and CFRP strip strengthened samples are shown in Figure 5a and 5b, respectively. Also shown in Figure 5 are the minimum and maximum fatigue loads, and the theoretical ultimate strengths calculated using Eq. (4). In Figure 5 the test results for the fatigue loaded samples (BF1, BF2, SF1, SF2) represent the monotonic test-to-failure phase that was executed after these samples were subjected to 2,000,000 load cycles as is described above. From the result presented in Figure 5 it is seen that all fatigue loaded samples survived the fatigue loading phase of the testing program and were loaded to failure with a response similar to the monotonic samples.



a) CFRP Bar Strengthened Samples

b) CFRP Strip Strengthened Samples

Fig. 5 Load Deflection Test Results.

From Figure 5 the effects of flexural strengthening with CFRP are clearly evident in the form of a change in flexural stiffness at yield, where the strain level in the reinforcing steel transitions from elastic to inelastic. The elastic region for the monotonic samples is roughly linear to the yield load. However, for the fatigue loaded samples there is evidence of concavity and a change in stiffness within the elastic load-deflection range. For the fatigue loaded samples (BF1, BF2, SF1, SF2) it is clear from Figure 5 that the stiffness in the fatigue load range is slightly greater than the stiffness at elastic loads above the maximum fatigue load. The concavity and inflection point are the result of load cycling and suggest that fatigue loading results in a slight stiffening of the sample. A similar behavior can be noted in beams fatigue loaded and subsequently tested to failure by Ekenel et al. [10]. At ultimate, all samples in the study failed by steel yield followed by concrete crushing. This is consistent with the predicted mode of failure described in Section 3. Also, in several cases, the beams were deformed after the maximum load was achieved until the CFRP bar or strip ruptured. This behavior demonstrates that there was no evidence of bond deterioration as a result of fatigue load.

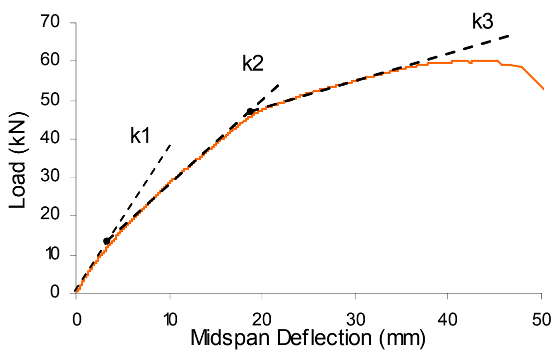
Table 2 provides a comparison of measured yield, measured ultimate and predicted ultimate loads. From Table 2 it is observed that both bar strengthened and strip strengthened samples loaded in fatigue have a slight increase in yield load when compared to the monotonic counterparts. Specifically, the bar strengthened fatigue beams have a 9 percent higher yield capacity than the monotonic counterparts, and the strip strengthened fatigue beams have a 7 percent greater yield capacity than the monotonic counterparts. There is a negligible difference between the ultimate load of the bar fatigue beams when compared to their monotonic counterparts (about 1%). The strip fatigue beams have a 5 percent decrease in ultimate load capacity when compared to the identical control samples. These strength results demonstrate that for the fatigue load schedule used in this study (i.e. 2,000,000 load cycles between 5% and 50% of the unstrengthened moment capacity) there was a slight increase in the yield strength but no appreciable change in the ultimate flexural strength.

The ratio of ultimate strength to yield strength for bar and strip strengthened fatigue beams is 1.20 and 1.22, respectively. This same ratio for bar and strip strengthened monotonic beams is 1.30 and 1.38, respectively. Thus, is a reflection of the slight increase in yield load for the fatigue beams as compared to the monotonic beams. Relative to the predicted moment strength (P_{nu}), the measured values (P_u) are slightly conservative. Referring to Table 2, the measured ultimate strengths ranged from 1.007 (strip strengthened fatigue samples) to 1.066 (strip strengthened monotonic samples) times the predicted strengths. These ratios suggest that the ultimate static moment capacity is accurately predicted using the analysis of Eqs. (3) and (4).

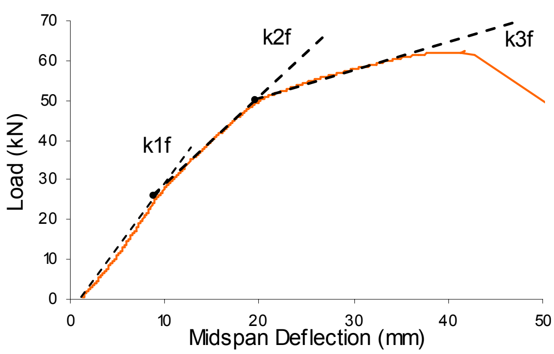
Table 2 Strength Results.

CFRP	Sample ID	Measured Yield Load (P_y)			Measured Ultimate Load (P_u)			Comparison		
		Sample (kN)	Average (kN)	Fat./Mon.	Sample (kN)	Average (kN)	Fat./Mon.	P_u / P_y	P_{nu} (1) (kN)	P_u / P_{nu}
Bars	BM1	44.48	45.88	1.09	59.56	59.78	1.01	1.30	57.4	1.041
	BM2	47.28			60.00					
	BF1	49.19	50.08		59.56	60.16		1.20		1.048
	BF2	50.97			60.76					
Strips	SM1	44.26	45.27	1.07	61.46	62.44	0.95	1.38	58.6	1.066
	SM2	46.28			63.42					
	SF1	48.66	48.52		56.16	59.01		1.22		1.007
	SF2	48.39			61.87					

Using the load-deflection results of Figure 5 it is proposed that the stiffness investigation consider three different values for both the monotonic and fatigue strengthened beams. These are shown in Figure 6 and identified as k1, k2 and k3 for monotonic samples and k1f, k2f, and k3f for fatigue samples. As can be seen, k1 and k1f represent the stiffness between zero load and a moment approximately equal to the precracking level for monotonic samples and the maximum fatigue load for fatigue samples. K2 and k2f represent the stiffness for the remaining elastic region, and finally k3 and k3f represents the stiffness in the inelastic region. Table 3 summarizes the stiffness results for all samples.



a) Monotonic Sample



b) Fatigue Sample

Fig. 6 Definition of Stiffness Values.

Table 3 shows that the three stiffness values characteristic of the fatigue loaded samples (k1f, k2f, and k3f) are in all cases greater than the corresponding stiffness values characteristic of the monotonic samples (k1, k2, and k3). Stiffness is defined is the change in load divided by the change in deflection at midspan. The bar and strip strengthened fatigue stiffness k1f are both 10 percent greater than the bar and strip strengthened monotonic stiffness k1. This is expected due to the permanent deflection incurred in the fatigue beams and because of the concavity of the load-deflection plot. Stiffness k2f for the bar and strip strengthened fatigue beams are both 11 percent greater than the stiffness k2 for the bar and strip strengthened monotonic beams. Stiffness k3f is 10 and 16 percent higher than stiffness k3 for the strip and bar fatigue strengthened beams when compared the strip and bar fatigue monotonic beams, respectively. The increase in stiffness for all three values considered can be predicted just by glancing at the load-deflection plot in Figure 5. The fatigue samples have a higher yield loads, similar yield deflections, similar ultimate loads, and less ultimate deflection than the monotonic samples. With higher loads and similar deflections (at yield), or similar loads and less deflections (at ultimate), the stiffness values will increase.

Table 3 Stiffness Results.

CFRP	Sample ID	K1 or K1f (k/in)		K2 or K2f (k/in)		K3 or K3f (k/in)		Comparison		
		Sample	Avg.	Sample	Avg.	Sample	Avg.	K1f/K1	K2f/K2	K3f/K3
Bars	BM1	16391	17554.5	11081	11451	3251	3269	1.10	1.11	1.16
	BM2	18718		11821		3287				
	BF1	19357	19393	12888	12668.5	3943	3783			
	BF2	19429		12449		3623				
Strips	SM1	17028	16806.5	11764	11299	3518	3482	1.10	1.11	1.11
	SM2	16585		10834		3446				
	SF1	18630	18410.5	12058	12498.5	3717	3832.5			
	SF2	18191		12939		3948				

3 CONCLUSIONS

The following conclusions are deduced from the experimental results:

- All CFRP strip and CFRP bar strengthened beams survived the 2,000,000 load cycles with no observable loss in bond or force transfer. Thus, composite action between the NSM CFRP and concrete appears to be unaffected by fatigue loading.
- The static failure mode for all samples (monotonic and fatigue) was steel yield followed by concrete crushing. Thus, fatigue loading had no effect on the static failure limit state. Also, this limit state was consistent with that predicted according to the flexural theory based on equilibrium and compatibility.
- There was about an 8% increase in the yield load for the fatigue loaded bar and strip strengthened beams when compared to their monotonic counterparts. However, the ultimate capacity of the bar strengthened fatigue samples was nearly identical to the monotonic counterparts, whereas the ultimate strength for the strip strengthened fatigue samples was about 5% less than the monotonic counterparts. The relative ultimate strength numbers, however, are well within expected experimental scatter for this type of research. Therefore, it is suggested that fatigue loading resulted in a slight increase in yield load but had no affect on the section's ultimate static strength.
- For the three stiffness regions considered, there was consistently about a 10% stiffness increase in the load-deflection behavior of the fatigue loaded samples when compared to the monotonic counterparts.

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