Flexural Ductility Behavior of Strengthened Reinforced Concrete Beams Using Steel and CFRP Plates

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ABSTRACT

Reinforced concrete beams are commonly retrofitted using Carbon Fiber Reinforced Plastic (CFRP) plates as the technique is both inexpensive and unobtrusive. However, tests have shown that CFRP plates tend to de-bond at low strains, which can severely limit the ductility. Structural strengthening of beams subjected to flexure can be achieved using different retrofitting materials. The mostly used other retrofitting materials are; high strength galvanized steel plates (HSGS plates) and normal strength steel plates (NSS plates). This paper reports the behavior of retrofitted beams with each of these three materials. The experimental results of this study suggest that (HSGS plates) can be used to increase the strength of reinforced concrete structures with little, if any, loss of ductility.

KEYWORDS: Retrofitting, Strengthening, Flexure, Reinforced concrete beams, Ductility, CFRP plates.

INTRODUCTION

The low modulus of elasticity and high strength of CFRP plates posed a disadvantage due to their brittle behavior, thus reducing the ductility of the member, and the incompatible strain behavior with the reinforcement embedded in the reinforced concrete beams may cause peeling-off at large flexural cracks (Task Group 9.3 FRP, 2001). Therefore, failure by deboning of CFRP plates is the main concern when retrofitting flexural members, implying that the added strength can be at the cost of loss of ductility of retrofitted members. As the need rises for retrofitting and strengthening of flexural members, different materials are being used. Each claims to be the best retrofitting material. Two of the mostly used materials are: high strength galvanized steel plates (HSGS plates); a material that combines high strength to weight ratio and corrosion resistance due to galvanization, and normal strength steel plates (NSS plates) which have a more ductile behavior.

Researchers found that the application of CFRP plates for flexural strengthening is very effective, provided that proper bond is observed. However, it was observed that the additional strength in the strengthened beam tends to decrease for the same retrofitting material if the area of reinforcing steel in the unretrofitted beam is higher (Duthinh and Starnes, 2001). Such reduction in strength is believed to be due to the incompatibility of the stress-strain relationship between reinforcing bars and CFRP plates. Shehata et al. (2001) reported brittle failure of beams strengthened in flexure with two and three CFRP plates (Shehata et al., 2001).

The behavior of FRP-strengthened RC structures is often controlled by the bond strength of the interface

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between the FRP and the concrete. However, due to the premature debonding failure, the strength utilization ratio of FRP is often only 15–35% of the material strength of the FRP plate (Wu et al., 2009). Liu et al. (2006) showed that beams retrofitted with near surface mounted steel plates reached a strain of 0.042 at ultimate load compared to 0.014 strain for beams retrofitted with CFRP plates, they concluded that it is better to retrofit beams that require ductility with high tensile steel plates (Liu et al., 2006).

Ductility is a property of a beam that is important for seismic design, and since retrofitting is sometimes concerned with upgrading a structure to resist seismic forces, identifying the retrofitting material which gives better ductility is important. Moreover, ductility allows moment to be carried at constant magnitude when deformation under plastic hinge conditions takes place; i.e., to assure that the plastic hinge has sufficient capacity to permit load redistribution while undergoing further deformation.

Ductility is generally measured by the ratio of the ultimate deformation to that at the first yielding of steel reinforcement (strain = 0.002).

Test Program

Ten beams of cross sectional dimension 200mm, 250 mm and a span of 1500 mm reinforced with 3#12 bars (A_s = 339 mm²) with an effective depth of 220 mm were tested in this investigation. The beams were adequately reinforced for shear using #10 stirrups placed at 150 mm spacing (Fig. 1). These beams were tested using a 1000 kN test frame (TONI-MFL) under flexural loading of two equal one-third loading over a 1500 mm span, to determine the maximum strain at the extreme fiber at failure load. Modes of failure have been reported and analyzed. Beams were divided into three groups, each group retrofitted with one of the three materials under investigation, and the 10^{th} beam was kept as a control specimen and tested without retrofitting.

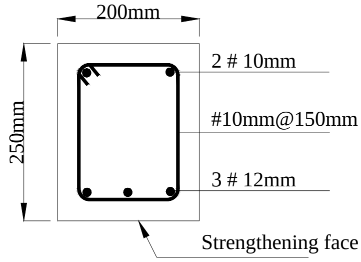

Figure 1: Details of test specimens

Materials

Reinforcing Steel: grade 60 deformed bars of strength f_y= 420 MPa.

Concrete: concrete used for the beam specimens was normal concrete with slump of 58 mm using Pozzolanic Portland cement, concrete tested cylinders had a strength of f_c'= 25 MPa. Crushed lime stone was used for coarse aggregates and crushed sand stone was used for fine aggregates. The concrete mix was designed to achieve the required strength. Beam specimens were cured under normal conditions as the structures they are supposed to represent and hoisted to the testing machine using an overhead crane.

Materials Used for Strengthening Beam Specimens

- High Strength Galvanized Steel (HSGS) Plates.
- Normal Strength Steel (NSS) Plates.
- Sika CarboDur Carbon Fiber Reinforced Plastics (CFRP) Plates S512/80.

Their properties are shown in Table 1.

The epoxy used to fix all three types of plates to the beams was BASF Concresive 1414, which is equivalent to Sikadur resin, since both are made up to ASTM C881, to eliminate the effect of the fixing material on the performance of the specimens under loading. First, a non-slump epoxy bedding BASF Concresive 2200 was used and after the bedding reached its full strength (7 day curing), the epoxy was applied and the plates were fixed in place and allowed to cure for other 7 days.

Table 1. Properties of materials used for strengthening beam specimens

Material	E-modulus [GPa]	Tensile strength [MPa]	Thickness [mm]	Width [mm]	Length [mm]
HSGS Plates	204	550	0.89	120	1000
NSS Plates	204	420	3	80	1000
CFRP Plates	165	3100	1.20	50	1000

Experimental Work and Results

Beam specimens were tested in flexure under two equal one-third loading over a 1500 mm span, using a 1000 kN testing machine. Linear variable displacement transducers (LVDT) were used to monitor the mid-span deflection. Displacement strain gauge transducers (PI-gauges) were installed on the compression, middle and tension sides at the mid-span of the beams. Tension PI-gauge was positioned at the level of the internal reinforcement to determine the magnitude of strain in the reinforcement bars under increased loading as shown in Fig. 2.

Figure 2: Experimental setup

Control Beam: The behavior of the control beam under loading was typical of a beam subjected to flexural stresses, failure load was at 145 kN. Crack pattern showed that uniformly spaced cracks were formed in the bending region which formed concrete blocks connected by the dowel action of the bottom steel, at the point of maximum diagonal shear cracks the mid-span strain at the compression side was 0.00121 and at the tension side it was 0.0148. Maximum deflection at mid-span was 10.9 mm.

Strain values of the control beam are shown in Table 2, the failure mode is shown in Fig.3 and the strain distribution over the cross-section is plotted for the 50kN, 100kN and failure loads as shown in Fig.4. The ductility value for the control beam is 7.35.

Figure 3: Failure of the control beam

Beams Retrofitted with CFRP Plates: Failure was typical of a brittle compression failure, debonding at the cracked region followed by crushing of the concrete at the top. The maximum load reached at failure was 190 kN, the mid-span strain at the compression side was 0.0028, while the strain at the tension side at failure load was 0.0064. Maximum deflection at mid-span was 13mm. Average of the beams' experimental data for this group is shown in Table 3, failure mode is

Table 2. Experimental data for the control beam

Load(kN)	Deflection(mm) Δ	Mid-Span Strains ε		
		Top	Center	Bottom
0	0	0	0	0
50	1.24	-0.00019	-0.00066	0.000977
100	3.06	-0.0003	-0.00028	0.002659
110	3.46	-0.00033	-0.00024	0.002989
120	4.41	-0.00036	-0.00018	0.003322
130	5.86	-0.00047	-0.00004	0.004519
140	7.16	-0.0005	0.000096	0.008031
145	10.95	-0.00121	0.000858	0.014796

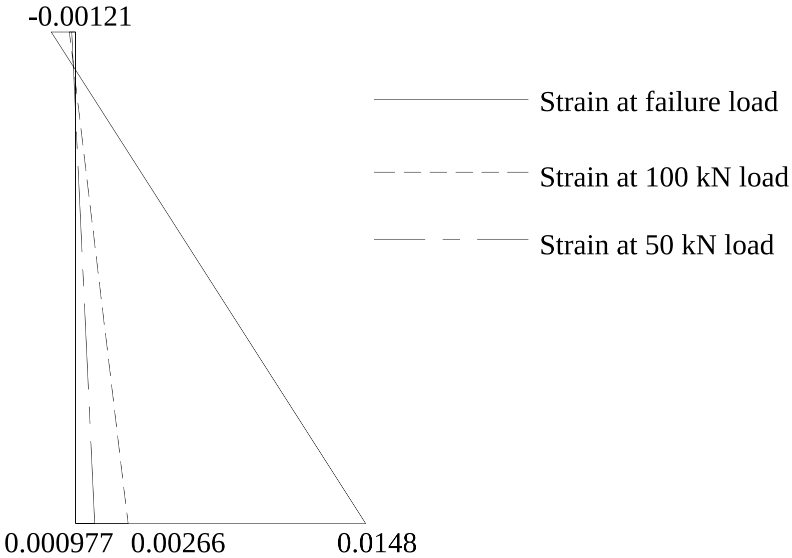

Figure 4: Strain distribution for the control beam

shown in Fig. 5, and the strain distribution over the cross-section is plotted for the 50kN, 100kN and failure loads as shown in Fig.6. The ductility value for this group of beams is 3.2, where we notice a drop in ductility for this group of beams.

Beams Retrofitted with HSGS Plates: Failure was an excellent example of balanced failure. The maximum load reached at failure was 192kN, the mid-span strain at the compression side was 0.0022, while the strain at the tension side at failure load was 0.0257 and maximum deflection at mid-span was 10.82 mm (Table 4). Failure mode is shown in Fig. 7, and the strain distribution over the cross-section is plotted for the 50kN, 100kN and failure loads as shown in Fig.8. The ductility value for this group of beams is 12.85, where we notice an increase in ductility for this group.

Figure 5: Failure mode of beam retrofitted with CFRP plates

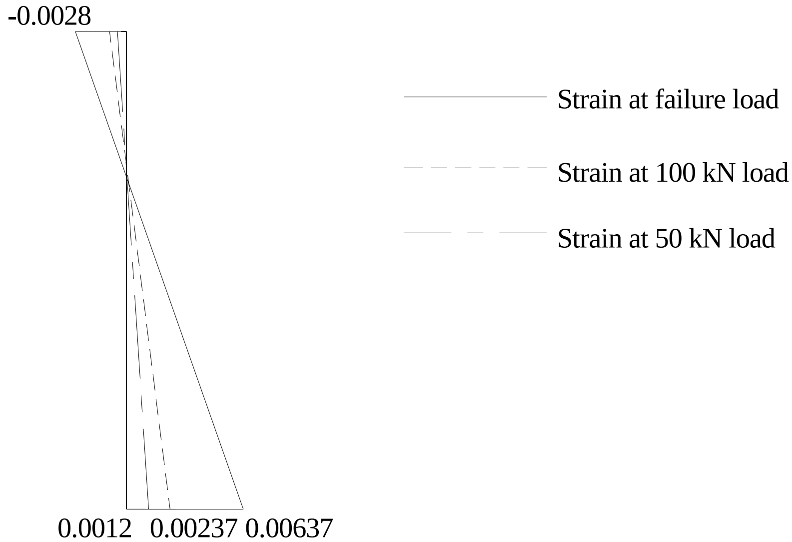

Figure 6: Strain distribution for beam retrofitted with CFRP plates

Beams Retrofitted with NSS Plates: Failure was an excellent example of tension failure. The maximum load reached at failure was 188kN, the mid-span strain at the compression side was 0.0014, while the strain at the tension side at failure load was 0.018 and maximum deflection at mid-span was 9.8mm (Table 5). Failure mode is shown in Fig. 9 and the strain distribution over the cross-section is plotted for the 50kN, 100kN and failure loads as shown in Fig. 10. The ductility value for this group of beams is 8.85, which is a little higher than that of the control beam.

Discussion of Experimental Results

Ductility of a member is defined as its ability to sustain inelastic deformations prior to failure without substantial loss of strength. A ductile system displays sufficient warning before catastrophic failure. Looking at Fig. 7, it is obvious that the beams retrofitted with HSGS plates showed the highest deformation before collapse, the second would be the beams retrofitted with NSS plates, the un-retrofitted beam comes third and in the last place are the beams retrofitted with CFRP Plates. This result was expected beforehand, because of the stress-strain characteristics of both steel and CFRP.

Figure 7: Failure mode of beam retrofitted with HSGS plates

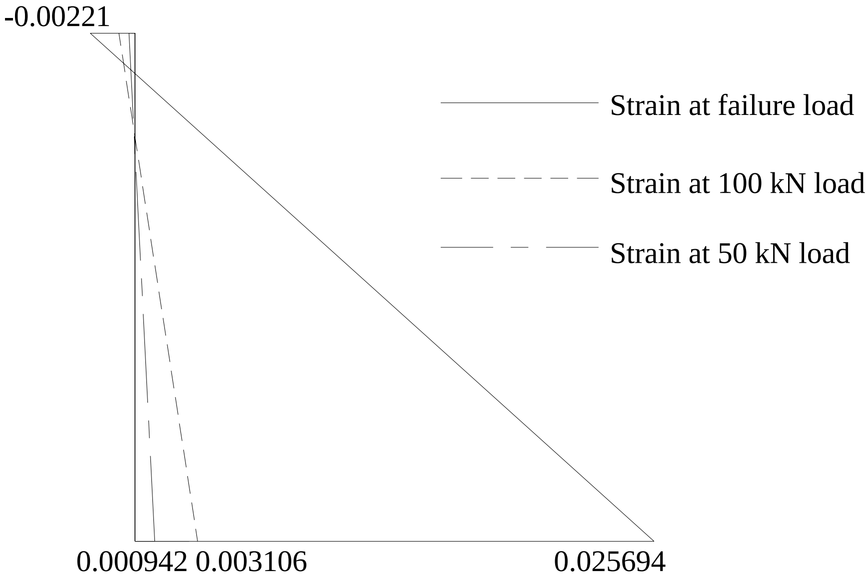

Figure 8: Strain distribution for beam retrofitted with HSGS plates

Ductility values (the ratio of the ultimate deformation to that at the first yielding of steel reinforcement) are shown in Table 6 in descending order, we notice that the beams retrofitted with HSGS plates have the highest ductility value (12.85) compared to that of the other beams, beams retrofitted with NSS plates come second with a ductility value of 8.85, the control beam comes third with a ductility value of 7.35, while beams retrofitted with FRP plates come in the last place with a ductility value of 3.2. These values are presented graphically in Fig. 11.

The modes of failure reported for each of these materials indicate that the balanced failure mode for the HSGS plates yields the economic use of material compared with the NSS plates that fail in tension and the CFRP plates that fail by debonding without reaching full strength.

Table 3. Experimental data average of the 3 beams retrofitted with CFRP plates

Load(kN)	Deflection(mm) Δ	Mid-Span Strains ε		
		Top	Center	Bottom
0	0	0	0	0
50	1	-0.00051	0.000369	0.001202
100	2.95	-0.00092	0.000722	0.002367
115	3.8	-0.00109	0.000834	0.002865
130	4.5	-0.00129	0.000929	0.003378
145	5.3	-0.00148	0.001011	0.003937
160	6.1	-0.00157	0.001151	0.004449
170	8.2	-0.00207	0.001185	0.004655
180	10.3	-0.00247	0.001217	0.004851
190	13	-0.00278	0.001246	0.006371

Table 4. Experimental data for one of the 3 beams retrofitted with HSGS plates

Load (kN)	Deflection(mm) δ	Mid-Span Strains ε		
		Top	Center	Bottom
0	0	0	0	0
50	0.6	-0.00034	0.000112	0.000942
100	1.6	-0.0008	0.000432	0.003106
125	2.22	-0.00102	0.000672	0.004064
150	3.02	-0.00125	0.001072	0.004992
165	3.57	-0.00136	0.001312	0.005424
175	4.42	-0.0017	0.001552	0.008384
185	7.82	-0.00192	0.001392	0.021472
192	10.82	-0.00221	0.001472	0.025694

Figure 9: Failure mode of beam retrofitted with NSS plates

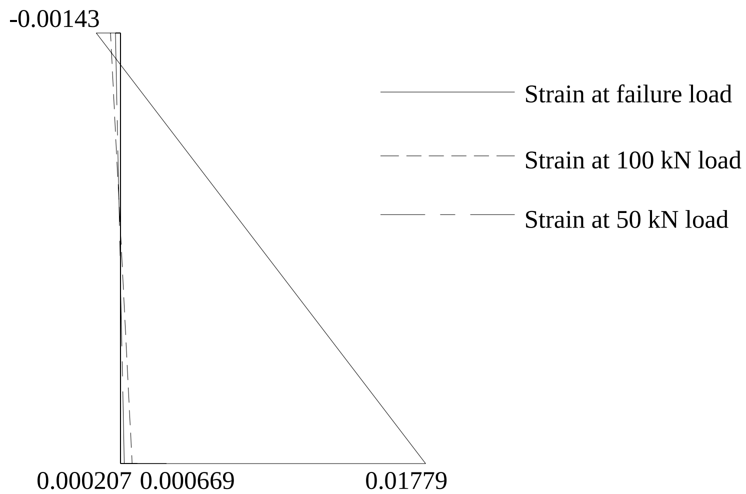

Figure 10: Strain distribution for beam retrofitted with NSS plates

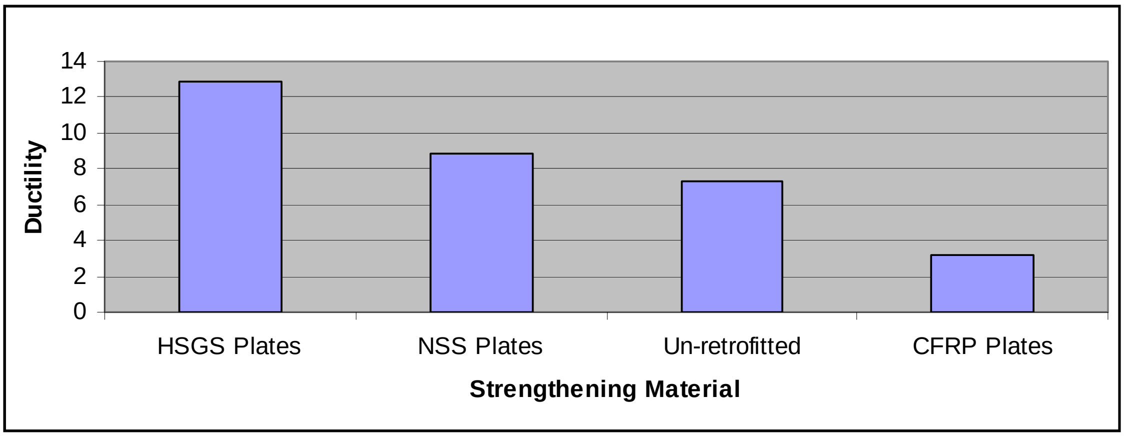

Figure 11: Ductility value of each strengthening material

Table 5. Experimental data for one of the 3 beams retrofitted with NSS plates

Load (kN)	Deflection(mm) δ	Mid-Span Strains ε		
		Top	Center	Bottom
0	0	0	0	0
50	1.8	-0.00021	-0.00018	0.000207
100	3.8	-0.00061	-0.0003	0.000669
125	4.9	-0.00102	-0.00036	0.004450
150	5.7	-0.00105	-0.0004	0.00626
165	7	-0.00110	-0.00044	0.00689
180	8	-0.00120	-0.00065	0.00899
188	9.8	-0.00143	-0.00093	0.01779

Table 6. Ductility value of each strengthening material

Retrofitting Material	Ductility	Improvement on Ductility (%)
HSGS Plates	12.85	75%
NSS Plates	8.85	20%
Un-retrofitted	7.35	0
CFRP Plates	3.2	–57%

CONCLUSIONS

- CFRP plates are known for their brittle behavior and their stress-strain relation. They are not compatible with the reinforcing steel embedded inside the concrete, therefore the ductility of beams retrofitted with these plates is low.
- HSGS plates have the same stress-strain relation as the reinforcing steel embedded inside the concrete member undergoing the strengthening procedure, therefore the ductility value is higher.
- Retrofitting of existing concrete members using CFRP plates can achieve the desired strength, but will not enhance the ductility of the member, on the contrary it may reduce it by as much as –57%.
- Using HSGS plates can achieve the same goal while enhancing the ductility of the member by as much as 75%.
- Modulus of elasticity of NSS plates is the same as that of the reinforcing steel of the retrofitted beams, and so is the stress-strain relation, therefore beams retrofitted with these plates achieve a little higher ductility than the un-retrofitted one; i.e., the ductility is increased by 20%.
- Although the strength to weight ratio of CFRP plates is higher than that of the HSGS plates, the cost of repair using the light gauge high tensile galvanized steel plates is lower than that of the CFRP plates, since the CFRP plates fail by debonding and do not reach full strength. Only 15-35% of their strength is utilized.
- HSGS plates are also more cost effective than NSS plates in addition to their corrosion resistance due to galvanization.

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