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Modelling of Steel Cantilever With Gfrp Under Cyclic Loading

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Abstract

A new technique to improve the ductility of steel cantilever is presented in this research. Glass fiber reinforced polymer (GFRP) plates bonded to the upper and lower steel cantilever flanges. The cantilever simulates the beam-column joint beam part of the moment-resisting frame subjected to later load. Two validation are conducted to assess the numerical modeling. First, we validate steel cantilever experimentally subjected to symmetric cyclic loading to verify this type of loading. Second, supported steel beam strengthened with GFRP experimentally subjected to monotonic loading to verify GFRP element and the bond between steel and GRFP. Three-dimensional modeling was developed using ANSYS software for steel cantilever without GFRP to study their behavior under cyclic loading. This investigation revealed a significant increase in the strengthened cantilevers' ultimate capacity and ductility with no sign of failure in the bonding adhesive up to GFRP failure.

Keywords — *ANSYS, Steel, GFRP, Cyclic loading.*

I. INTRODUCTION

Moment resistant steel frames are extensively used framing system for steel structures. Steel moment-resisting ductile frames in seismic risk areas must undergo large plastic deformation at critical zones. Lemonis (2018) [1] studied the hysteretic energy dissipation of steel moment numerically resisting frames. The beam is one of the critical zones for the moment-resisting frame that encourages the plastic zone's formulation to absorb and dissipate energy.

Some structural elements under cyclic loading did not satisfy the plastic design requirement. Beam element developed local instabilities under cyclic loading tests, which accelerated by load reversals. Many researchers studied the behavior of structure element under cyclic loading; Popov and Pinkney (1969) [2] tested 24 connection specimens subjected to various cyclic with particular attention to the hysteretic response of the beams under repeated and reversed loadings, Popov and Bertero (1973) [3], study experimentally the behavior of large structural steel cantilevers and their connections, Beamish (1987) [4], noted that under load reversals, local flange buckling can occur at relatively low ductility in members having plate slenderness ratios close to the plastic design limits, POPOV and TSAI (1989) [5], considered the cyclic behavior of unconventional beam-to-column flange moment connections.
W-shaped beam-to-column stub connections subjected to cyclic loading were tested by Korol and Daali (1995) [6] to assess their energy dissipation capabilities and compare the energy absorption of unstiffened beams with locally web-stiffened beams.

FRP is used extensively to rehabilitate structures. Sen et al. (2001) [7] studied the repair of steel concrete composite bridges using CFRP laminates to enhance composite beams' ultimate capacity. Tafsirojjaman et al. (2019) [8] conducted an experimental program to study the strengthening of circular hollow steel members with carbon fiber reinforced polymer (CFRP) subjected to monotonic and cyclic loading.

EL Damatty et al. (2003) [9], and EL Damatty and Abushagur (2003) [10], studied experimentally and analytically the rehabilitation of steel beams using GFRP sheets. Zhan et al. (2015) [11] conducted an experimental program to assess the effectiveness of pultruded glass fiber-reinforced polymer (GFRP) profiles for rehabilitating lattice steel columns. Ascione et al. [12] tested eight full-scale GFRP adhesively bonded beam-column connections were under combined shear and bending. Martins et al. [13] presented a novel connection system for pultruded GFRP tubular profiles using internal steel parts and bolts developed to be used in modular constructions for temporary shelter or emergency scenarios. Vieira et al. (2018) [14], Studied the fatigue loads effect of pultruded glass fiber reinforced polymer (GFRP) composite materials. Ryu et al. (2019) [15] proposed GFRP plates as a strengthening method for steel beams using bolted connections to prevent debonding.

This paper investigates numerically the performance of steel cantilevers strengthened with GFRP subjected to cyclic loading. The strengthened cantilever simulates half the portal frame girder. The two-phase validation process is considered to reach our goal by investigating the effect of steel cantilever

I-beam strengthened by GFRP under cycling loading. First, we validate steel cantilever experimentally subjected to symmetric cyclic loading to verify this type of loading. Second, supported steel beam strengthened with GFRP experimentally subjected to monotonic loading to verify GFRP element and the bond between steel and GRFP. A three-dimensional finite element model combining the validated elements was developed to study the strengthened steel members' behavior under cyclic loads.

II. CYCLIC LOADING OF STEEL CANTILEVER

Korol and Daali (1995) [6] studied the enhancement of W-shaped steel cantilever experimentally by adding different stiffeners types: herring-bone, vertical stiffener. A series of experiments on full-size steel members subjected to quasi-static cyclic loading was undertaken to assess their ductility. The test specimen set-up was shown diagrammatically in figure 1, the cantilever beam projected horizontally from the column-stub.

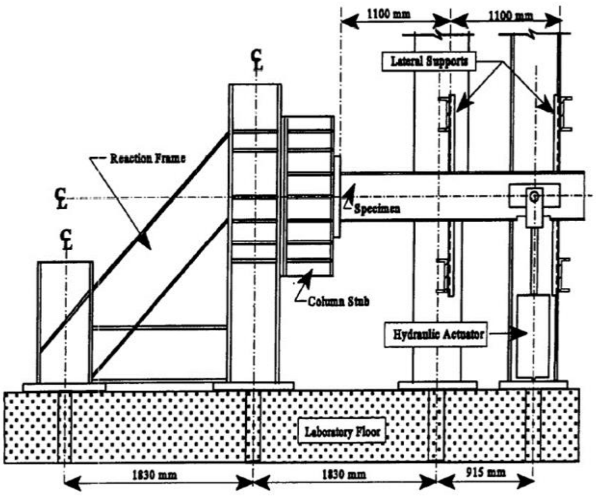

Figure 1 - Korol and Daali (1995) [6], test set-up

A. The sequence of Cyclic Loading

The quasi-static cyclic loading is used to simulate earthquakes loading and high plastic deformations. The quasi-static cyclic loading Korol and Daali (1995) [6], used in their experiments, are cyclic load histories, proposed by Popov and Tsai (l989) [5], governed by deflection control were used in testing the series of specimens. At the beginning of the test, the low cycles help check the test set-up and ensure all data acquisition channels work properly. Cyclic loading gives a partial estimation for the relation between the ductility and time load history. The load histories were applied in terms of the actual initial yield displacement of the beam, as in figure 2.

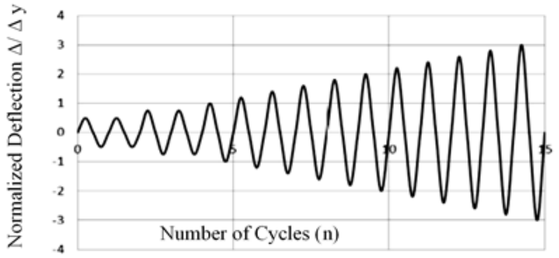

Figure 2 – Symmetric Cyclic Loading

B. Cyclic Loading, Ductility and Rotation Capacity

The beam response can be easily determined in several ways. The beam's deformation was isolated by simply subtracting the product of the connection rotation by the cantilever length from the overall beam-connection deformation. As such, the results to be described involve the applied forces versus beam tip-displacement and the applied moment M at the face of the endplate versus the overall beam rotation θ (elastic and inelastic), normalized by Mp, (plastic moment based on a product of measured yield stress by nominal plastic section modulus) and θp (calculated rotation at Mp).

For a member subjected to bending moment that is laterally braced in accordance with the specification in use, the available ductility, Δu / Δp. or its rotation capacity, Ru, are ductility parameters that determine how effectively internal moments can be redistributed once the plastic tip load, Fp or (Mp) is reached. Ductility is defined as the maximum displacement ratio, Δu, measured from the zero displacement intercept location to the displacement, Δp, associated with Fp. Meanwhile, the rotation capacity defined as the amount of total rotation beyond the plastic limit defined by:

$$R = (\theta_u / \theta_P - 1) = (\Delta_u / \Delta_P - 1)$$

III. VALIDATION OF STEEL CANTILEVER SUBJECTED TO CYCLIC LOADING

One of the unstiffened specimens tested by Korol and Daali (1995) [6] was modeled using ANSYS. A W-shaped steel cantilever (cross-section: W310x21 and length 2100 mm) was selected. Three-dimensional finite element modeling was performed to simulate the tested cantilever beams with symmetric cyclic loading with material properties (yield stress σy = 290 MPa and ultimate stress σu = 415 MPa) complied with ASTM A36.

Eight nodes Shell 181 element was used in numerical modeling, both geometric and material nonlinear effect were included in the analysis with the same mechanical properties of specimen used in the experimental tests to reflect as possible the laboratory conditions during the experiments. Figure 3 shows the finite element mesh for the cantilever assembly.

The finite element analysis, figure 4, shows that buckling of flange and web starts at cycles 8 and 9, respectively. Figure 5 and figure 6 show the force-displacement and normalized moment rotation results. From figures (5 & 6), the maximum force response, Fmax=43.95 kN, also a corresponding positive ductility equal 3.21 and negative ductility equal 2.61. Table 1 summarizes the comparison between F.E results and the experimental results.

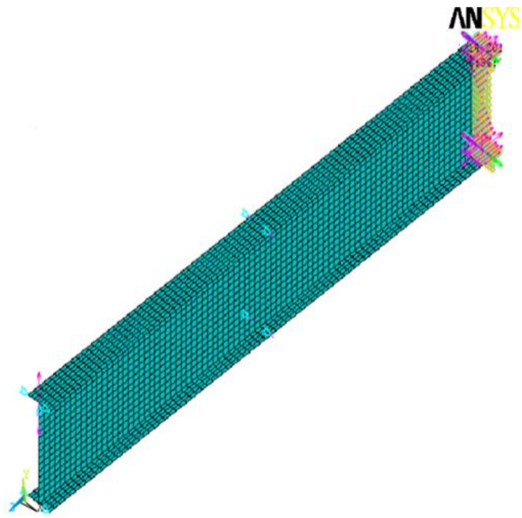

Figure 3 – Steel Cantilever Mesh

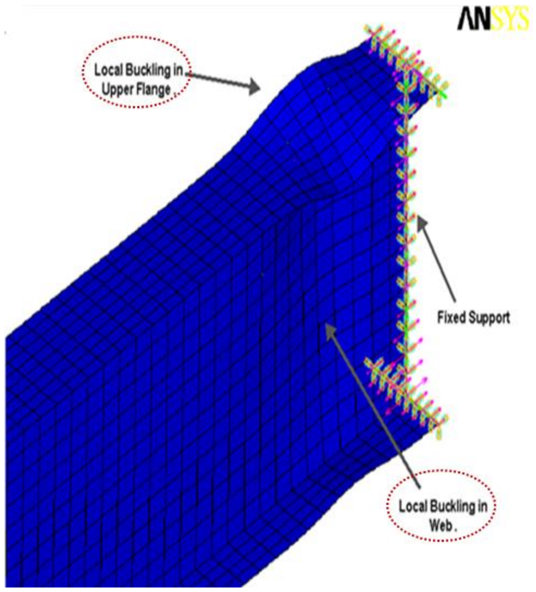

Figure 4 – Steel Cantilever Web and Flange Buckling

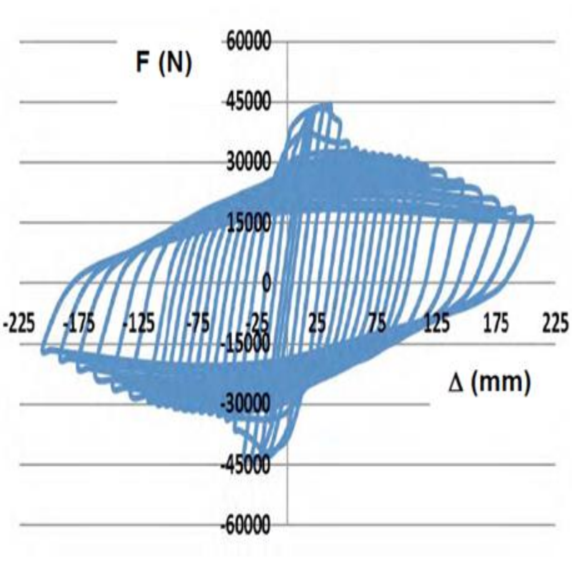

Figure 5 – Steel Cantilever Mesh

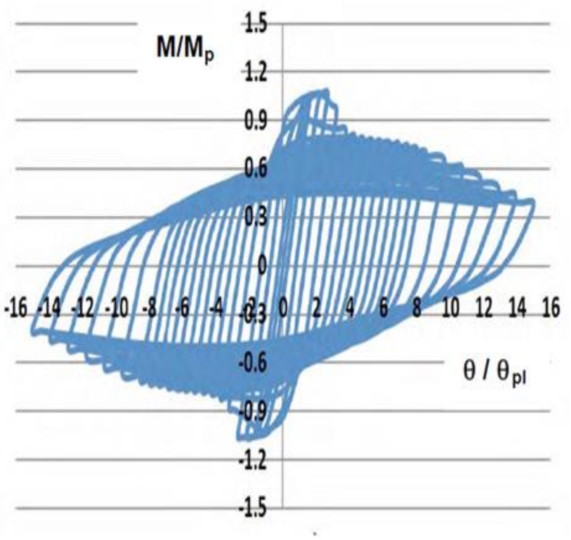

Figure 6 – Web and Flange Buckling

TABLE I
Comparison between F.E. and Experimental Results

	Korol and Daali [6]	Current Study F.E.	Difference %
Maximum Force (kN)	47.12	43.95	6.7
Ductility +	3.44	3.21	6.69
Ductility -	2.69	2.61	2.97
Flange Buckling	Start at cycle 8	Start at cycle 8	- - -
Web Buckling	Start at cycle 9	Start at cycle 9	- - -

IV. VALIDATION OF STEEL BEAM STRENGTHENED WITH GFRP

In this part, the steel beam strengthened by GFRP tested by EL Damatty et al. (2003) [9] was modeled using ANSYS. Supported steel beam (cross-section: W150x37 and length 2800 mm) bonded by GFRP sheets at top and bottom flanges (GFRP thickness 19 mm, 154 mm wide, and length 2400 mm) was selected. The material properties for steel were yield stress σ_y =363 MPa and modulus of elasticity E_s=2x10^5 MPa. The material properties for GFRP were tensile strength 206.85 MPa and modulus of elasticity 1.72 x 10^4 MPa.

Eight nodes Shell 181 element was used in numerical modeling, both geometric and material nonlinear effects were included in the analysis. For steel elements, a bilinear isotropic hardening model with tangent modulus equal to 3% of the elastic modulus was used, while a linear elastic model was assumed for the GFRP to its brittle behavior. The bond between GFRP and steel was modeled by continuous linear springs simulating the shear and peel stiffness of the adhesive with values 21.79 N/mm^3 and 2.26 N/mm^3, respectively, EL Damatty et al. (2003) [9]. Figure 7 shows the finite element mesh for the beam assembly.

The beam in the model was loaded by two-point loads at one-third of the beam length from the two support to simulate the experimental test till failure occurs at GFRP, reach maximum tensile stress. The same failure was found in the experimental results, in figure 8, crushing of GFRP at mid-span without any sign of debonding.

The load-deflection curves from the experimental and our numerical modeling are shown in figure 9. The test specimen experimentally and F.E models' ultimate load values, including springs simulating the adhesive effect, are 432.8, 447.4 kN, respectively, with a difference of 3.37 %.

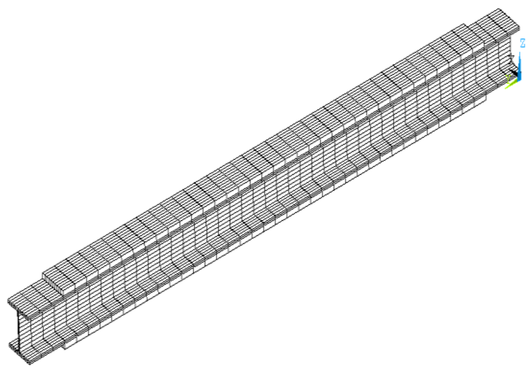

Figure 7 – Strengthened Steel Beam Mesh

Figure 8 – Experimental failure of Strengthened Steel Beam, EL Damatty et al. (2003) [9]

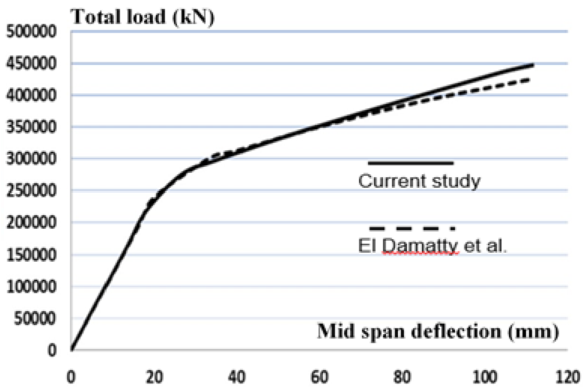

Figure 9 – Strengthened Steel Beam load deflection

V. GFRP STRENGTHENING OF STEEL CANTILEVER SUBJECTED TO CYCLIC LOADING

F.E modeling is used to study the effect of adding GFRP with different thicknesses on the maximum force/moment and ductility/rotation capacity of strengthened steel cantilevers with GFRP sheets.

Two cantilevers with sections W3l0x2l, W3l0x39, and lengths of 2100 mm is subjected to symmetric cyclic loading without and with GFRP. The GFRP sheets are fully bonded with upper and lower flanges and covering the cantilever span L=2100mm. GFRP Thicknesses of 5, 10, 15, 20, and 25 mm were studied. The same material for steel-cantilever (validated model 1) and the same material for GFRP sheets (validated model 2) are used.

Lateral torsional buckling was prevented by adding lateral supports at the mid-span and the cantilever end. The strengthened cantilevers were loaded by symmetric cyclic loaded till failure occur.

A. Steel Section W3l0x2l + GFRP Sheet 5 mm

This model was symmetric cyclically loaded, and the GFRP sheets are bonded with upper and lower flanges and covering the cantilever span L=2100mm. The onset of yielding starts at displacement Δy =13.664 mm with Yielding Force Fy =32.54 kN. It was noticed that the hysteresis loops were stable and maintained the same flexural stiffness up to cycle 11, reaching the maximum force Fmax=56.74 kN; from that point on, the elastic member stiffness started to degrade.

At cycle 12, local buckling appeared in the flange, followed by web buckling, figure 10. To calculate plastic moment for the composite section with GFRP, it was decided to transfer the 5 mm thickness of the GFRP sheet into equivalent steel thickness $t_e = 5/n$ by using modulus ratio $n = E_s/E_{GFRP}$. Therefore, The calculated plastic moment Mp = 90.16 kN.m and plastic load Pp = 42.93 kN.

As a result of the continuously increasing the cyclic loading flange distortions, the member carrying capacity showed a gradual decrease after each cycle. At cycle 13, a second buckling wave appeared in the bottom flange and the web. One can see that the model developed maximum positive and negative ductility's of 4.43 and 3.92 at cycles 10 and 11. Therefore, the rotation capacities for this case may be computed as 3.43 and 2.93, respectively.

Comparing these results with specimen type without GFRP sheets found an enhancement in positive and negative ductility's by 37.69 % and 49.81%, respectively. Also there is increasing in positive and negative rotation capacities by 28.49% and 45.35 %. With a further inelastic rotation, a resistance fall-off started to occur as a consequence of flange and web buckling.

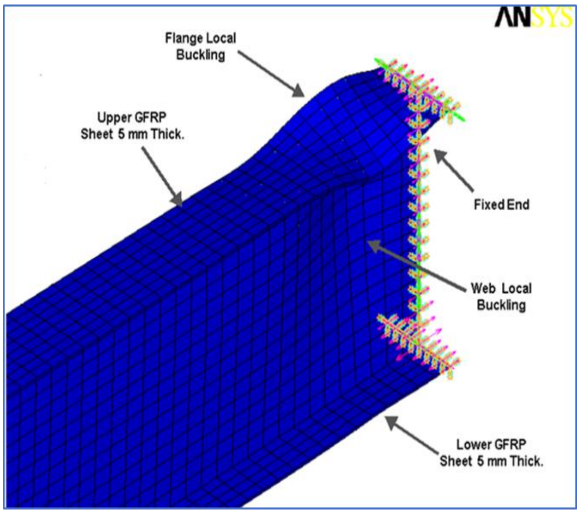

Figure 10 – GFRP Strengthened Steel Cantilever Web and Flange Buckling

B. Parametric Study

After completing all models cyclic loading to failure, the following data has been recorded: yield displacement Δy, yield force Fy, maximum force Fmax, cycle number for local buckling, plastic moment Mp, positive and negative ductility, and positive and negative rotation capacity. A summary of all these results is shown in table 2 for section W3l0x2l and table 3 for the W3l0x39 section.

TABLE 2
Summary of Steel Section W3l0x2l Strengthen with Different GFRP Thickness

	Steel + GFRP (5mm)	Steel + GFRP (10mm)	Steel + GFRP (15mm)	Steel + GFRP (20mm)	Steel + GFRP (25mm)
Δy (mm)	13.66	13.46	13.32	13.18	13.04
Fy (kN)	32.54	33.85	34.9	35.87	36.62
Fmax. (kN)	56.74	69.84	80.76	90.28	100.17
local buckling cycle	12	14	16	18	19
Mp (kN)	90.16	94.19	98.24	102.3	106.35
positive ductility	4.42	5.26	5.61	6.06	6.44
negative ductility	3.91	4.79	5.17	5.6	5.98
positive rotation capacity	3.42	4.26	4.61	5.06	5.44
negative rotation capacity	2.91	3.79	4.17	4.6	4.98

TABLE 3
Summary of Steel Section W3l0x39 Strengthen with Different GFRP Thickness

	Steel + GFRP (5mm)	Steel + GFRP (10mm)	Steel + GFRP (15mm)	Steel + GFRP (20mm)	Steel + GFRP (25mm)
Δy (mm)	15.92	15.33	15.28	15.25	15.19
Fy (kN)	76.91	78.69	79.9	80.85	81.47
Fmax. (kN)	109.59	111.9	115.78	124.74	139.72
local buckling cycle	5	6	6	8	8
Mp (kN)		216.23	223.8	231.38	239.07
positive ductility	2.9	2.99	4.06	5.44	7.37
negative ductility	2.92	3.05	4.01	4.92	7.37
positive rotation capacity	1.9	1.99	3.06	4.44	6.37
negative rotation capacity	1.92	2.05	3.01	3.92	6.37

The percentage of increase for the positive and negative ductility, positive and negative ductility, maximum force compared with the ratio of GFRP thickness to flange thickness are shown in figure 11.

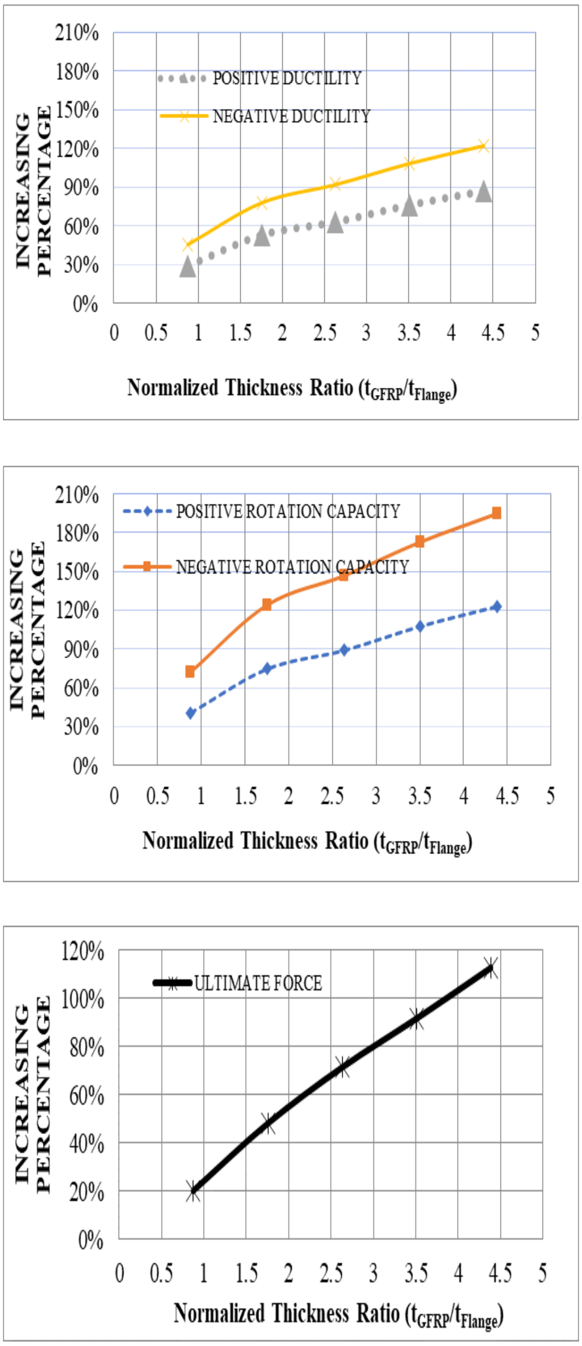

Figure 11 Comparison of W 310x21 with variable GFRP sheet normalized thicknesses subjected to symmetric cyclic loading.

The percentage of increase for the positive and negative ductility, positive and negative ductility, maximum force compared with the ratio of GFRP thickness to flange thickness are shown in figure 12.

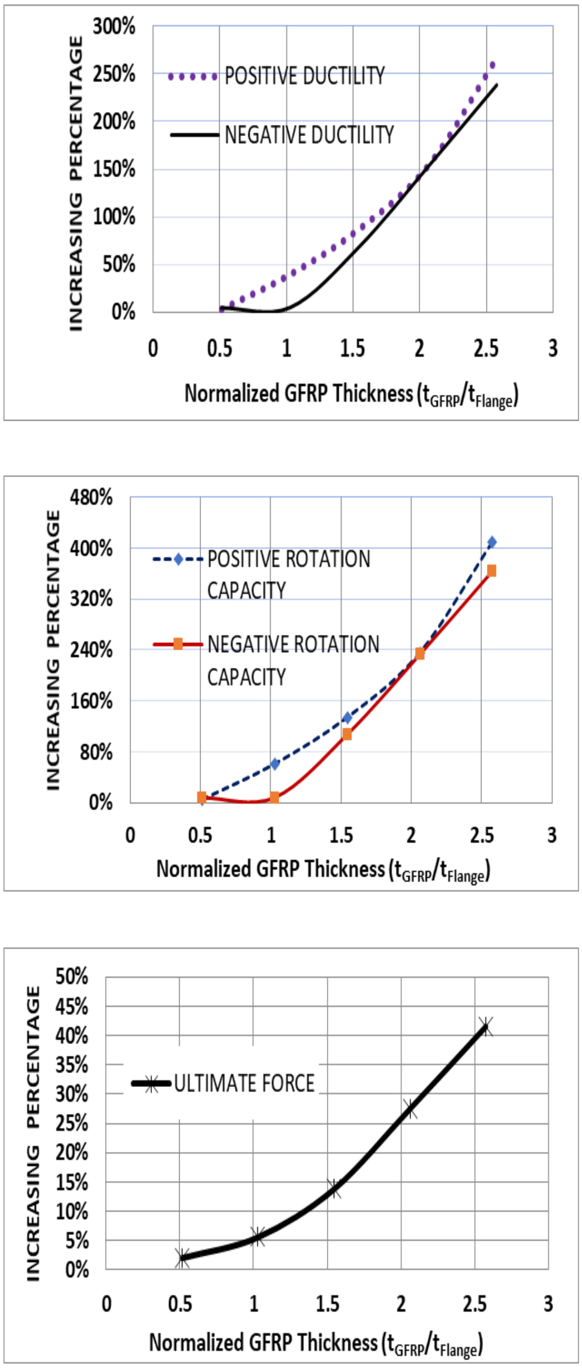

Figure 12 Comparison of W 310x39 with variable GFRP sheet normalized thicknesses subjected to symmetric cyclic loading.

C. Discussion of Results

Results for W310x21 show the maximum force by ratios 29% up to 127 %, increased positive ductility by ratios 37 % up to 100 %, and increased negative ductility by ratios 49 % up to 129 %. Results for W310x39 show the maximum force by ratios 2% up to 41 %, an increase of positive ductility by ratios 2 % up to 267 %, an increase of negative ductility by ratios 4 % up to 238 %. All these percentages of increase are for different GFRP thicknesses compared to without GFRP. The difference between the percentages of increase for two sections type is using the same GFRP thicknesses in strengthening compared to the flange thickness of W310x21 (t=5.7mm) and the flange thickness of W310x39 (t=9.7 mm).

VI. COMPARISON BETWEEN GFRP STRENGTHENING OF VS DIFFERENT STEEL STRENGTHENING

In this part, a comparison between the strengthening types that was experimentally studied by Korol and Daali (1995) [6], and the strengthening by GFRP used in this paper for steel section W310x21. Table 4 shows the beginning of the local buckling for specimen without strengthening either steel or GFRP. The comparison shows the benefit of using the GFRP as a strengthening technique for cyclic loading type.

TABLE 4
Forming of local buckling – steel section W 310x21

Description	Analysis Type	Local Buckling Cycle
Steel without Strengthening	Experimental (validated by ANSYS)	Cycle (8)
Steel with Herring- bone Stiffener	Experimental	Cycle (9)
Steel with Vertical Stiffener	Experimental	Cycle (10)
Steel with GFRP Sheets 5mm	Modeled by ANSYS	Cycle (12)

VII. SUMMARY AND CONCLUSION

The steel beam-column joint has an important role in the load and moment transferring of steel frames. The beam must develop plastic deformation with sufficient ductility/rotation capacity cycle after cycle to permit full redistribution of moments.

This study presents numerically the use of GFRP for strengthening beams subjected to cyclic loading using ANSYS software. Steel cantilever subjected to symmetric cyclic loading is validated by comparing the maximum force, positive and negative ductility, and the cycle buckling start. Simple beam strengthened with GFRP is also validated by comparing the load-deflection curve till failure and maximum failure load. Good agreement is shown for both cases of validation.

A numerical parametric study was performed on different W-shaped steel cantilevers strengthened by GFRP with different thicknesses on top and bottom flanges subjected to cyclic loading. A significant increase in the yield force/moment, maximum force/moment, and positive/negative ductility are shown. From the study of the two different W-shaped sections, the GFRP sheets are affected by the flange thickness and beam depth. Comparison between adding a small thickness of GFRP and adding different steel stiffening positions to the cantilever shows the efficiency of this strengthening technique due to the delay of the local buckling occurrence.

It is believed that the use of a GFRP sheet for strengthening beams in moment resisting frames designed to resist strong earthquakes prove to be beneficial.

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