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FRP Strengthening of Web Panels of Steel Plate Girders against Shear Buckling

Part-I: Static Series of Tests

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ABSTRACT

The result of an experimental programme investigating a novel technique to strengthen web plates against breathing fatigue is presented in this paper; the programme was divided into five phases, including: (1) the development of a novel preformed corrugated FRP panel for strengthening thin-walled steel plate girder webs against buckling, (2) selecting the adequate adhesive and epoxy using double-lap shear and tension specimens, (3) producing the FRP panel, and (4, 5) testing its performance in two main experimental series; the initial (static) series and the final (cyclic) series. Only the initial series which involved tests on 13 steel plates strengthened with the proposed preformed corrugated FRP panel and subjected to in-plane shear will be reported in this paper. This series investigated the performance of different forms of strengthening under static load, in preparation for a subsequent series of cyclic tests to investigate their fatigue performance. Test results showed the efficiency of the technique at increasing the stiffness of the strengthened specimens in comparison to the unstrengthened ones and reducing the critical stresses which will serve as a precursor for the anticipated increase in the fatigue life of the girders.

KEYWORDS:

Plate girders; Steel plates; FRP; Buckling; Shear.

INTRODUCTION

Strengthening of structures has received increased attention within the construction community in recent decades. Structural strengthening may be required due to:

- change of specifications including re-zoning of seismic risk and wind distribution, or changes in design philosophy;
- load increases, especially for bridges and industrial buildings;
- design or construction errors; and
- damage to structural elements caused by deterioration of construction materials or the application of unexpected loads such as fire, earthquake, or impact (Assoodani, 2014).

Steel and composite steel-concrete bridges constitute a large number of the existing bridges worldwide. In 2008, it was reported that about 12% of total number of bridges (72,520 bridges) in the United States were structurally deficient (Bureau of Transportation Statistics, USA, 2008). Steel bridges comprise about 50% of the structurally deficient bridges. The need for adopting durable materials and cost-effective strengthening techniques is therefore self-evident.

Different techniques exist for strengthening structures; all of which have drawbacks. For instance, conventional techniques for strengthening steel structures – like welding additional transverse/longitudinal stiffeners – require heavy equipment during installation, their fatigue performance can be of concern, and may be a need for ongoing maintenance due to corrosion attack. Amongst the available strengthening techniques, the use of fibre reinforced polymers (FRPs) is appealing because of their resistance to corrosion, low weight, and high tensile strength.

The use of these composite materials for structural strengthening of both steel and concrete structures is seen as a promising technology. However, most of the work to date has been focused on concrete structures. The key reason for the less common use of FRPs for strengthening steel structures is their comparatively low tensile modulus of elasticity. This means that either a more costly high modulus carbon fibre reinforced polymer (CFRP), and/or a greater thickness of FRP material is needed for strengthening. By adding externally bonded FRPs, the stress levels in the original member may decrease, thus resulting in a longer fatigue life.

Typically, flexural strengthening with FRP is straightforward as the FRP is bonded along the tensile flange to take full advantage of the strengthening FRP material. However, shear

strengthening is not similarly straightforward because of the multi-axial stress state under shear condition. Most of the shear stress is resisted by the web on the basis of tension-compression diagonals and a smart technique needs to be implemented to take best advantage of the FRP material because it does not perform well in compression in comparison to its superior tensile properties, never mentioning the web buckling problem which might lead to early deboning if not considered in the design.

The project presented in the current paper examines strengthening of the webs of steel plate girders against shear buckling using externally bonded FRPs. In-plane loading of thin web plates close to the shear buckling load results in out-of-plane displacements, which in turn induces secondary bending stresses at the welded web plate boundaries. Under repeated loading the combined state of membrane and secondary bending stresses may result in fatigue cracking and failure. In the current work, an FRP strengthening technique using bonded shapes is applied to resist these out of plane deformations by stiffening the steel plate, rather than to provide additional direct tensile strength (as is the case in flexural strengthening), Fig. (1).

BACKGROUND

Roberts et al. (1995a) noticed that during fatigue tests the girders exhibited considerable web plate breathing, with pronounced shear buckles forming and reforming along the tension diagonals of the web panels during cyclic loading. Fatigue cracks formed along the toes of the welds between the web and boundary members, in regions of high secondary bending stresses caused by out-of-plane (buckling) deformations. The number of load cycles to fatigue crack initiation varied considerably; for higher load ranges the rate of propagation of fatigue cracks was reasonably uniform, while for lower load ranges it was variable.

On the basis of their research, Roberts et al. (1995b) stated that stress ranges at potential fatigue crack locations could be predicted using nonlinear finite element analysis (FEA) or approximate analytical solutions. They also showed that the fatigue assessment procedures recommended in the Eurocodes, based on either principal stress ranges or normal and shear stress ranges, provided conservative estimates of the fatigue life for slender webs subjected to plate breathing.

Skaloud and Zornerova (2005) continued to study fatigue of slender plates. They studied the limit state for the webs of steel plate girders subjected to repeated loading, and stated that the response is affected by the cumulative damage process generated in the web under repeated loading.

Patnaik and Bauer (2004) studied the effect of shear and flexural strengthening in steel beams using CFRP covering. Thirty-percent increase in load bearing capacity was observed in beams which went under tests for flexural strengthening using CFRP on tensile flange while 62-percent increase in shear capacity was observed for beams used for studying the shear of CFRP strips in the beam web.

Limited numbers of researchers have studied strengthening of steel plate girders under short term shear loading. One such study is by Okeil et. al. (2009), who investigated the use of externally bonded GFRP pultruded sections for strengthening steel structures. The GFRP sections were bonded to thin-walled steel plates in orientations that contributed to the out-of-plane stiffness of the plate rather than its in-plane strength, as is the more common practice in FRP strengthening applications. Beam (shear) specimens were tested to explore the out-of-plane strengthening technique, increasing the ultimate capacity of the strengthened specimens by 56%; however reducing their ductility.

Miyashita et al. (2012), also conducted a series of shear buckling tests on seven steel plate girders with CFRP sheets bonded to both sides of the web panel as a substitute for material lost due to corrosion attack. Test variables included the web aspect ratio ($a_w/d = 1$ and 1.5) and the plate slenderness ratio ($h_w/t_w = 133$ and 166). Different numbers of fabric layers and fibre angle orientations were used. An increase in the ultimate capacity of the specimens, between 6.18 and 29.19 percent, was reported, along with a proposed modification to Basler's equation (Basler, 1961) to account for the addition of bonded CFRP showing a good correlation with test results.

Bhutto (2013) presents similar testing on eight CFRP and GFRP strengthened plate girders. The CFRP used was carbon fabric sheets applied in layers by a wet layup process, while the GFRPs used were T and I-sections adhered vertically and diagonally to the web plate to act as substitutes for steel stiffeners. The ultimate loads were increased by 1.20 to 1.54 times due to FRP strengthening.

OPTIMIZATION OF THE STRENGTHENING TECHNIQUE

Analytical Model

Fig. (2) provides details of the analytical model adopted in the current study. The initial plate shown in Fig. (2-a) is a square steel plate of 500×500 mm having a slenderness ratio (h_w/t_w) of 250, where h_w and t_w represent the height and thickness of the steel plate, respectively. Figs. (2-b) and (2-c) show the classical (i.e. flat sheet) and proposed (i.e. profiled) FRP

strengthening techniques, respectively. In the classical technique, several layers (plies) of FRP are bonded using wet-layup process usually over the entire area of the steel plate, to produce a composite section with greater stiffness. The strengthening technique used here is based on a preformed corrugated FRP panel bonded to the compression diagonal of the web plate, which is a more efficient use of FRP material due to the higher flexural rigidity of the profiled shape. Preliminary FEA confirmed that the specimens strengthened with the proposed diagonal corrugated profiled FRP panel had higher stiffness and critical buckling shear stress.

Cross-Section of FRP Profiled Plate

Figs. (3-a and b) show three selected types of corrugated profiled FRP plates; with rectangular, semi-circular, and half-hexagonal profiles, respectively. Several preliminary trials have been made before these ones using hand calculation techniques to optimize the corrugation section with different shapes, heights, widths, and number of corrugations per section. These calculations were based on Euler strut model in axial compression and plates under pure shear conditions. The sectional dimensions of the panel were chosen to consume the least possible FRP material and bonding epoxy quantities, whilst not providing excessive geometrical stiffness, which might lead to brittle failure like most other FRP strengthening techniques. The chosen preformed corrugated FRP panels had typical widths of 195mm, with variable lengths and end cut shapes. Perfect bond was assumed between the FRP and the steel plate in all modelling cases for optimizing the corrugation geometry.

Each of the three profiled shapes was tested numerically using elastic buckling analysis available in the FEA code *Abaqus*. The FRP panels were assumed to be made of GFRP laminate with a thickness of 1.4mm and a modulus of elasticity of 14.4GPa. The FRP section was bonded (tie constrained) to an S275 steel plate having dimensions of 615×245×2mm. Fixed boundary conditions were imposed at both ends of the steel plate, except for the in-plane axial displacement at the loaded edge. This plate was used to represent a compression diagonal strip from the steel plate as shown in Fig. (3-c). A reduced integration linear 4-noded shell element (S4R) was used for both the steel and the FRP.

The results of this preliminary FEA showed that Euler critical buckling loads were 10.3kN, 7.3kN, and 8.4kN for the steel plates strengthened with the rectangular, semi-circular, and half-hexagonal corrugated sections, respectively. In spite of the fact that the rectangular section had the highest buckling load, it was excluded from the results because it is thought that the sharp edges of this section will act like stress concentrators for both FRP composite

and the bonding strips. The semi-circular and half-hexagonal profiled sections were chosen for further numerical investigation.

At this stage it is worthwhile emphasizing that choosing the right geometrical properties for the FRP strengthening panel is not an easy and straightforward process. Different variables need to be taken into considerations for a successful optimization criterion including, but not limited to, type of the FRP used, bonding requirement, size of the specimen, required stiffening level, and some practical considerations with respect to applicability in the field. Based on the observations in this study, it is recommended to limit the second moment of area of the stiffening FRP section so that the ultimate shear capacity of the strengthened composite steel-FRP section does not exceed the shear yield capacity of a similar perfectly flat steel plate (the shear yield capacity can be found by multiplying Mises shear stress ($\sqrt{f_y}/3$) by the height (h_w) and thickness (t_w) of the steel plate).

Convergence Study

Buckling of thin plates is sensitive to the type of element used in an FEA model. Hence, a convergence/sensitivity study was performed. The model adopted in the convergence study was chosen as a typical web plate with practical dimensions of 2000×2000mm. The variables included the type of the element (S4R, S8R, and S9R5) and the slenderness ratio (h_w/t_w) which was taken as 1000, 500, 250, 200, 166.667, 142.857 or 125, corresponding to plate thicknesses of 2, 4, 8, 10, 12, 14, and 16mm, respectively. Both simply-supported and clamped boundary conditions were investigated, and the shear stress was applied to the plate boundaries exactly as in the main model shown in Fig (2-a).

Fig. (4) shows typical curves for the critical buckling shear stress versus the inverse of the degrees of freedom (dof), for specimens with simply supported and clamped boundary conditions and a plate slenderness ratio equal 250. The dof was calculated according to the number of elements used, and the corresponding dof of each node in the element according to compatibility. Using the inverse of dof provides a prediction of the element size that will give the same theoretical value or as close as possible. S9R5 (which have 5 degrees of freedom at each node: three translations and two in-plane rotations, i.e., no rotations about the shell normal) showed superior behaviour with fewer elements, and a size of $h_w/20$ was chosen; this was sufficiently coarse for economy of the model whilst sufficiently accurate (i.e. close to the theoretical value).

Finite Element Analysis

Fig. (5) shows the predicted buckling modes of the FEA model for a $(500 \times 500 \times 2\text{mm})$ steel plate strengthened with flat wet-layup GFRP covering the full surface area of the plate, along with buckling modes for the same plate strengthened with either the proposed semi-circular or half-hexagonal corrugated GFRP panels; both having the same thickness and material properties. The critical buckling shear stresses (τ_{cr}) were 43.5MPa, 114.0MPa, and 122.6MPa, for the flat, semi-circular, and half-hexagonal sections, respectively, in comparison to $\tau_{cr} = 42.5\text{MPa}$ for the initial steel plate with clamped boundary conditions without strengthening.

From the above results, it appears that the new proposed half-hexagonal corrugated FRP panel has the highest buckling stress and therefore it was chosen as the corrugated section to be used throughout this study. The hexagonal section increased the buckling strength by a ratio of approximately three to one in comparison to the control one. A fast parametric study showed that for the classical strengthening technique to achieve the same buckling stress as the proposed one, a 5.9mm thickness GFRP laminate would be required, covering the full surface of the plate. The proposed preformed corrugated FRP panel reduced the required FRP by approximately 8 times (volumetrically) and the required bonding epoxy surface area by 7 times. This did not only reduce the cost of the strengthening process, but it also did not affect the bond strength of the specimen and helped in maintaining the ductile failure associated with un-reinforced steel plate girders.

EXPERIMENTAL PROGRAMME

The objective of the experimental programme was to develop a system to increase the stiffness and strength of web plates. The system in the current work was applied to the web of a presumed end panel in a steel bridge, where high shearing forces are typically exerted. While limited previous work has shown that strength increases are possible for steel structures reinforced with FRPs, relatively little stiffness increase has been observed to date. To provide stiffness increases to thin steel web plates while minimizing material costs, a novel preformed corrugated FRP panel has been introduced (Phase-1). Fig. (6) shows the geometry and orientation of the new preformed corrugated FRP panels.

The experimental programme was conducted in 4 further phases. In Phase-2, saturating resins for wet lay-up of dry fibre sheets and bonding epoxy were selected based on the performance of double lap shear (DLS) coupons that were fabricated and tested in tension. The resin

selected for this process was used in Phase-3 to manufacture the direct tensile specimens which is made to determine the material mechanical properties of the FRP composite.

In Phase-4 (initial test series), thirteen steel plates were strengthened by bonding the new preformed corrugated FRP panel along the compression diagonal of the steel plate, and tested for shear buckling. The in-plane shear loading was applied using a specially designed testing rig “Picture-Frame”, that is capable of holding the steel plate in position, applying realistic boundary conditions and in-plane shear loading, to simulate the case of a steel plate serving as the web of a steel plate girder.

Finally, in Phase-5, six plate girders were built and strengthened with the optimized FRP panel from the initial tests and tested to study the behaviour of the proposed strengthening technique under cyclic loading. Preliminary fatigue analysis indicated that the proposed strengthening technique is likely to succeed in reducing the secondary bending stresses in the web plate welded boundaries and thus prolong the life expectancy of steel bridges in failure modes related to web plate breathing (Al-Azzawi et. al., 2015). This conclusion in addition to the experimental results of the cyclic series shall be presented in a separate paper.

FRP Material and Geometrical Properties

Four types of woven FRP cloth were used: Carbon fibre 2/2 twill 12k 450g, carbon fibre 2/2 twill 12k 650g, biaxial glass cloth 440g, and biaxial glass cloth 600g. Table 1 gives the manufacturer specified properties of the carbon and glass fibres.

Two types of carbon and glass fibre sheets were used so as to maintain the same fibre volume fraction (FVF) for the 2 and 3 layer preformed corrugated FRP panels. Although the longitudinal properties of a composite are dominated by the fibre properties, important parameters such as its ultimate elongation will depend also on the saturating resin used, especially when biaxial fabric sheets are used; this will lower the fibre volume fraction in the longitudinal dimension causing the tensile strength and modulus to be reduced. Despite the reduced tensile strength and modulus of elasticity, this distribution still preferable in the more complex stress states such as for shear in thin steel plates.

Double Lap Shear Tests

Double lap shear (DLS) tests were performed for the epoxy selection phase. Test specimens consisted of two plates fabricated from Grade s275 steel, with dimensions as shown in Fig. (7). These were joined together with preformed GFRP or CFRP sheet panels. Surface preparation for the steel consisted of sanding with 120 grit emery paper to achieve a uniform

surface that was free from surface contamination and mill scale. Schnerch (2005) suggested that for the preparation of small areas this provides good mechanical keying suitable for adhesive bonding to steel. Immediately prior to application of the resin, the surface was cleaned with acetone. The plates were clamped using a special jig to maintain their alignment during curing.

During bonding a uniform coating of epoxy was applied to one side of the steel plate in the bonded region, and the FRP panel was placed onto the steel plates, with 25.4mm of overlap on each of the steel plates. The sheets were then pressed into the resin on the steel plates. A 1.9mm gap was left between the steel plates to minimize the effect of end-to-end bonding. Once the resin had cured sufficiently, the same procedure was repeated on the reverse side. Specimens were cured for at least seven days prior to testing. Fig. (8-a) shows the DLS test setup.

Three different epoxies were trialled to find the best one for the wet lay-up process and the bonding process. Epoxy resins tested were EL2 resin, Tyfo s saturant, and Sikadur-330. Table (2) summarizes these tests and the average shear strength for each one. Each value in the table is the average of three tests. The EL2 resin was chosen for saturation while Sikadur-330 was chosen as the bonding adhesive.

Tension Tests

Tension tests were performed to determine the properties of the CFRP and GFRP sheets using the chosen epoxy from the DLS tests. Test panels were fabricated by wet lay-up whereby the dry fibre sheets were saturated with resin bonded onto a flat release board. The fabric sheets then were covered with a peel ply and a breather layer cloth to absorb the extra resin and then vacuum bagged using a 1.0 bar vacuum pump. Using the vacuum bagging process allowed consistent control of the thickness of the panels and the highest possible fibre volume fraction (FVF) within the section.

After sufficient curing the panels were cut into coupons 50mm in width using a tile saw. Pre-manufactured tapered GFRP tabs were bonded to each coupon to prevent premature failure in the wedge action grips of the testing machine. All tabs were adhered using EL2 resin. After adhering, the tabs and adhesive were allowed to cure at room temperature for at least 48 hours before testing.

The FVF of the specimens was calculated based on the weights of the dry fibre fabric before saturation. Prior to saturation, each length of fabric was carefully weighed. After the FRP panels had achieved sufficient working strength, the entire panel was again weighed to find

the weight of the added epoxy resin. The fibre weight fraction (FWF) was calculated in this manner and the fibre volume fraction could then be determined from:

$$FVF = \frac{1}{\left\{1 + \frac{\rho_f}{\rho_r} \left(\frac{1}{FWF} - 1 \right)\right\}} \quad \dots\dots\dots (1)$$

where:

FVF = Fibre Volume Fraction.

FWF = Fibre Weight Fraction.

ρ_f = Density of fibres.

ρ_r = Density of the epoxy resin.

The thickness of each coupon was measured at six locations; the width was measured at three locations. Averages of these were used to determine the cross-sectional area of the coupons. All coupons were tested using an electro-mechanical Instron testing frame, under crosshead stroke control at a rate 1.0mm/minute with data recorded at 1.0 Hz. Strains were measured using digital image correlation (DIC). Specimens were painted with a high-contrast pattern and a series of images were captured at 0.2 Hz using a digital camera (Fig. 8-b). A bespoke DIC algorithm (REF) was used to track the movement of pixel patches in the images. Table (3) gives the mechanical properties of the GFRP and CFRP used in this study along with some other practical aspects, where F_f is the ultimate tensile strength, $\square_f$ is Poisson's ratio, and E_f is the tensile modulus of elasticity of the FRP section. Each value typically represents the average of three tests.

Manufacturing the FRP Preformed Corrugated Panels

As mentioned earlier, In order to provide stiffness increase to the thin steel plates while minimizing the cost of the applied material, a new preformed corrugated FRP panel was introduced.

The Hex preformed corrugated sections were made by wet layup process of the two types of CFRP and GFRP fabric sheets for two and three layers in an attempt to maintain the same FVF for all specimens. Vacuum bagging was adopted because of the difficulty of moulding the hexagonal section with traditional techniques, see Fig. 9. It is worth mentioning that both open and closed sections were made from GFRP and CFRP as shown in Fig. (6-b), however, the closed section were made with two FRP layers only. This was done to see if the open section would cause any lack of bonding due to its lesser adhesion area.

Development of the Test Method

The classical method to test the shear panel of a plate girder is by testing small or large scale plate girders, combining the two end panels into one plate girder because the end panels represent the most critical shear loading along a plate girder's span; see Fig. (1). However, this is considered a very costly method because each time a variable is changed a new specimen has to be tested.

In the current work a new testing rig (Picture-Frame) is introduced. This picture-frame is capable of holding the steel plate, applying fixed boundary conditions and in-plane shear loading, simulating the web of a steel plate girder; see Fig. (10). It is based on the idea of clamping the steel plate boundaries into a stiff steel frame using bolts that do not pass through the steel plate itself to avoid stress concentrations. It is capable of moving only in-plane using a 4-hinged beam-chain mechanism and thus applies shear force on the steel plate relying only on friction between the picture-frame and the steel plate. The in-plane movement is achieved using 8 mirrored hinges (instead of 4) to avoid cutting the steel plate corners which will affect the stress distribution and to make sure that the loads are applied throughout the plate corners for better simulation of buckling of steel plates under pure shear loading.

The validity of the design for the picture-frame rig was established using finite element simulations that included modelling different scenarios with respect to the boundary conditions applied to the steel plate, the location of the hinges, and the distribution of the stresses developed in each component of the frame and the tested steel plate. Figure (10-b) shows the final finite element model.

Test Variables

The parameters varied in the current study were the type of FRP (CFRP or GFRP), the number of layers of fabric for the same fibre volume fraction, the bonded length along the compression diagonal, the FRP panel section (open vs. closed section), the orientation of the glass fibres, and the shape of the preformed panels' end cut. These are shown in Table (4); along with the ultimate test loads for the specimens tested in the initial static series- refer to Fig. (6) for additional details. Thirteen 500x500x2mm steel plates, having a modulus of elasticity (E_s) of 190GPa and a yield strength (f_y) of 245MPa, were strengthened with the proposed preformed corrugated FRP panels and tested using the picture frame rig. The plates had a constant aspect ratio ($a_w/h_w=1.0$) and slenderness ratio ($h_w/t_w=250$).

Specimen Preparation

Except for the control specimen, which was not strengthened, the following procedure applies for all other specimens (Table 4). First the specimen was grit blasted to the required texture and then cleaned of contamination using acetone. The adhesion energy of the surface was determined using a 20mm dolly with digital adhesion tester. Adhesion tests were performed after 24 hours and 7 days of curing at room temperature, respectively. The adhesion strength varied between an average of 7.65 MPa and 8.21 MPa for the 24 hour and 7 day tests, respectively. The epoxy was then mixed and applied to the steel plate and the FRP panel after removing the peel ply cloth, which helped keeping the FRP panel uncontaminated with a good bonding texture. The FRP panel was attached to the specimen using a special fixture to hold it in position. Finally, a uniformly distributed load was applied to press the panel toward the steel plate to reduce possible air bubbles and maintain a uniform epoxy thickness. The load was maintained for 24 hrs and then removed.

Test Instrumentation

Fig. (11) shows the specimens, testing rigs, and test instrumentation for the initial series of tests. A single strain gauge rosette was used in the centre of the plate to record the vertical, horizontal, and diagonal strains. The strain gauge readings along with the eight displacement gauges (LPs) were recorded at a rate of 1.0 Hz. The location of the LPs and strain gauges are shown in Fig. (11-c) where S refers to strain gauge and LP refers to linear potentiometer. Five of the 30mm LPs (LP1-LP5) were used to determine the plate out-of-plane displacements, while two 30mm LPs (LP6-LP7) ensured that there was no rigid body rotation for the picture-frame, and one 100mm LP (LP-8) was used to determine the deflection at the bottom end of the plate. Tests were performed using a 1000 kN servo-hydraulic actuator at a stroke rate of 1.0 mm/minute.

EXPERIMENTAL RESULTS

Table 4 shows the test results for the thirteen plates tested in this initial series of tests. From this table, it can be seen that for the strengthened specimens, the ultimate capacity is slightly increased between 5.6% and 24.1% depending on the type of the FRP panel used in strengthening them. The lowest strength increment (5.6%) resulted from strengthening the steel plate with GFRP-2L-(45°-45°)-A (SP-2) while the highest increase in strength is achieved using CFRP-2L-(0°-90°)-C (SP-13). This relatively low increment in the ultimate capacity of the FRP strengthened steel plates might be attributed to the fact that the proposed

strengthening technique in the current work is originally designed to increase the stiffness and buckling strength rather than increasing the ultimate capacity of the strengthened specimen as will be discussed in detail later in this section.

The ultimate load is defined as the load where the specimen starts to undergo high deformations without the need to increase the applied shear load significantly. In most cases, the loading curve managed to reach a horizontal plateau representing yielding of the steel plate and in some other cases the loading curve reached to something similar to a yielding plateau but with some minor load increment due to strain hardening of the steel plate and the stiffening effect of the FRP panel. In a few cases the test was stopped for technical and safety reasons when the deformations were very high or when the load reached a level that could affect the testing rig. This latter case took place only once with testing SP-13. However, the buckling load is not very straightforward to determine like the ultimate load and needs more investigation.

For the control specimen, the buckling load is estimated to be around 40.0 kN using the change in slope method compared to the theoretical 42.5 kN for plates with clamped boundary conditions. However, the buckling load for the strengthened specimens could not be detected because the out-of-plane displacement behaviour was significantly altered and neither the change in slope nor Southwell method is applicable for determining the buckling load. This will be discussed and more investigated in the following section.

Central Out-of-Plane Buckling

Fig. (12) shows the load versus out-of-plane displacement for each of the 13 specimens. From this figure, typical patterns of behaviour can be detected. For the control specimen the behaviour is tri-linear where the curve starts linearly at the initial stage of loading then it suddenly changes slope indicating the critical buckling shear stress. After first buckling, the web buckles sideways and only the diagonal in tension is working. This results in the formation of wrinkles, and instead of the pure shear state before buckling occurs; simple tension exists in the wrinkle direction (i.e. diagonal tension). The development of the tension field results in a post-buckling reserve of strength, as illustrated by the non-linear part of the curve for the control specimen in Fig. (12). Since the limit for the steel in tension is its yield strength, the specimen continues to resist shear loading until the diagonal tension yields; this is demonstrated by the upper flat portion of the curve in the figure.

The behaviour of the strengthened specimens is different and from Fig. (12), it can be divided into three categories depending on the type of the FRP panel used. The first category

446 followed the tri-linear behaviour of the control specimen but with a stiffer response, a good
447 example of this type of behaviour is the CFRP strengthened specimens in Fig. (12-a). The
448 second category showed a bilinear response where the linear initial stage of loading
449 continued without showing any sign of buckling until it reached close to the ultimate capacity
450 where it curved dramatically towards the failure plateau. This bilinear type of behaviour can
451 be seen in the GFRP strengthened specimens in Fig. (12-c). Finally, the third category is
452 associated with specimens which altered their predesigned failure mode to a reversed
453 buckling mode in the opposite direction. This reversed buckling mode can be seen in the
454 GFRP strengthened specimens in Figs. (12-c and 12-d).

455 Generally, due to the proposed unsymmetrical way of strengthening the plate from one face
456 only, there is an induced pre-buckling mode which succeeded (most of the tests) in forcing
457 the buckling mode to the FRP face farthest away from the steel plate where the maximum
458 tension stresses exists, and this increases the efficiency of the strengthening technique
459 because FRP is superior in tension. This helped in reducing the out-of-plane displacement
460 and reduced the secondary bending stresses, especially within working stress.

461 For the control specimen, the experimental buckling stress was approximately 40.0 MPa. The
462 value of 40.0 MPa is calculated by dividing the buckling load (40 kN) by the plate sectional
463 area. However, this is just an average number estimated from the zone where the buckling
464 curve extremely changes in slope beyond the initial linear stage, and is not based on any
465 engineering calculations. Nevertheless, this buckling load can be detected from the diagonal
466 tension strain curves as well where a sudden change in the curve slope can be seen around
467 this load announcing the development of the tension field.

468 With respect to the strengthened specimens, no buckling loads could be determined from Fig.
469 (12) due to the significant alteration in the typical behaviour of shear buckling as a
470 consequence of the proposed strengthening technique. This fact will be further supported by
471 the finite element model where no buckling load can be detected as well. In some curves
472 within Fig. (12), there are some indications for the development of the tension field in the
473 tested plates. This is postulated to be caused by a minor slip in one of the tension corners of
474 the picture-frame testing rig in some of the tests.

475 Typical photos for specimens after failure are given in Fig. (13). These photos were taken
476 after the picture-frame was dis-assembled and the steel plate is taken out. From these photos,
477 it can be seen that the proposed strengthening technique succeeded in reducing the out-of-
478 plane displacement and changed the buckling mode.

Effect of the Strengthening Section and FRP Material Type

Fig. (12-a) is a comparison between the open and closed FRP panel sections. It is clear that the closed section FRP panels were not significantly stiffer than the open section ones, even though these had higher second moment of area and much larger bonding area.

The initial response is slightly different and it is postulated that it is due to a minor gripping slip in the tension diagonal, mainly with testing specimens SP-2 (GFRP-2L-(45°-45°)-A) and SP-3 (CFRP-2L-(0°-90°)-A). This may have caused a small slack in the specimen, causing the curves to turn concave down during their initial stages until the slack was taken up. The slip took place because there was no mechanical anchorage between the plate and the picture-frame and the grip was solely dependent on the friction according to the original design. For the remaining tests, eight 16mm diameter bolts were added to the picture-frame to ensure that the steel plate was effectively clamped and the slip prevented in the initial stages. However, this precaution caused the load to fluctuate at the final stages for some of the tests (referred to with dashed lines in the figures).

Fig. (12-a) also shows that the CFRP and GFRP strengthened specimens had similar stiffness in the initial loading stages, while the CFRP ones had higher stiffness during later loading stages. This is expected because in the initial loading stages the plate is not yet buckled and the stiffness of the strengthening material plays a limited role; however for higher loading stages (after the specimen has already buckled), the stiffness of the strengthening material plays a major role as it reduces the out-of-plane displacement.

Effect of the FRP Number of Layers

Fig. (12-b) illustrates the effect of increasing the number of FRP layers for the same fibre volume fraction. From this figure it can be seen that the three-layer GFRP panels were more effective in the initial stages but had the same stiffness in the later stages. However, the three-layer GFRP panel showed more integrity and did not crack at the ultimate load the way the two-layer GFRP one did; this can be seen by comparing the failed specimens' photos in Fig. (13-a). In contrast, the three-layer CFRP panel behaved almost the same as the two-layer one and had even lower ultimate capacity.

Effect of Glass Fibre Orientation with respect to the Axis of Corrugation

Fig. (12.c) shows the effect of the fibre orientation for the GFRP panels. It can be seen that changing the glass fibre orientation from the default of this work, (i.e. 45°/45°) to 0°/90° had no clear effect on the results of the 3-layered GFRP specimen (SP-9) but affected the two-

layered $0^{\circ}/90^{\circ}$ GFRP specimen (SP-8) which had a very stiff but brittle behaviour and led to an alternative mode of failure in the opposite direction to that expected. This could be justified by examining the failed specimen photo in Fig. (13-a) where it can be shown that the FRP panel cracked near the bottom compression corner causing the specimen to alter its buckling mode because this corner usually suffers from severe deformation due to the thrust effect of the sharp triangular end cut associated with type-A FRP panels.

Effect of the End-Cut Shape and Position

Fig. (12-d) shows the effect of the end-cut shape and position for the FRP panels. It can be seen that in the case of GFRP the round cut (Type-B) succeeded in increasing the stiffness of the specimen in both the initial and final stages of loading, in addition to increasing the ultimate load capacity. In the case of CFRP, the round end-cut (Type-B) did not obviously improve the behaviour. The same observation does not hold for the longer FRP panels with triangular end-cuts (Type-C). This type of panel only improved the stiffness of the specimen in the initial loading stages, while the direction of the buckling was reversed in the higher loading stages (i.e. beyond 80 kN). Nevertheless, the CFRP long panel with triangular end-cuts (Type-C) improved both the stiffness and ultimate capacity; however the test was halted when the applied load exceeded 173 kN (for technical/safety reasons).

In-Plane Deflection

Figs. (14-a) to (14-d) show the load-deflection curves in four categories, as was the case for out-of-plane buckling. Fig. (14-a) shows the effect of using closed sections versus open sections, in addition to the bonded area available for bonding the FRP panel to the steel plate. Fig. (14.b) shows the effect of using two-layer FRP panels versus three-layer panels. Fig. (14.c) shows the effect of fibre orientation for GFRP panels, and Fig. (14.d) shows the effect of end cut on the deflection response. From these figures it appears that no obvious trends can be indicated, except that generally the strengthening technique insignificantly increases the stiffness, ductility and ultimate capacity. This leads to the question of whether deflection curves or buckling curves should be used to determine stiffness and energy absorption capacity of the strengthened specimens. By examining the buckling curves and comparing them to the load-deflection curves, it is self-evident that buckling curves are more representative when it comes to shear buckling failure and should be used instead of the usual approach of using the load-deflection curves.

Assessing the Stiffening Effect of the FRP Panel

Despite the fact that there is an obvious buckling behaviour for the control specimen as can be seen from its buckling curve, see Fig. (12); no obvious buckling load could have been estimated from the buckling curves for the other 12 strengthened specimens. This is why there are no values stated for the buckling load in Table 4.

For further investigation of the buckling load and to address the problem of the initial slack in the curves due to the minor slippage at the initial loading stage as mentioned above, the buckling curves were reproduced; see Fig. (15). The two axes in Fig. (15) were transferred into dimensionless ones. For the load axis, this was done by dividing the applied load by the corresponding shear yielding load using Mises criteria where the yielding shear stress can be taken equal to $(f_y/\sqrt{3})$. For the displacement axis, this was performed by dividing the out-of-plane displacement by a limiting displacement (which is believed to be the limit where the behaviour of the curves turns into non-linear). The value of 1.0 mm is chosen as this limit because it represents the average limit between linear and non-linear parts in all the tests.

In Fig. (15), the slack caused by the abovementioned minor slippage was treated by drawing a hypothetical parallel curve removing this problem, as can be seen from the dashed line in the figure.

Looking at Fig. (15), it can be stated that the proposed strengthening technique is effective enough to alter the problem from an Eigen problem with respect to the control steel plate where buckling is a sudden phenomenon followed by the formation of the diagonal tension field, to a typical composite section plate where no sudden buckling is taking place anymore and the deformation process is progressing in a more stable manner.

The above discussion could be justified by the fact that the compression diagonal is not losing its capability in resisting the compression force induced by the applied shear load beyond buckling anymore because the bonded FRP panel is stiffening the steel plate along its compression diagonal. This is a unique case where the design is taking optimum advantage from both the steel plate and the strengthening FRP panel because of the biaxial action where the steel-FRP composite section is forming a composite strut resisting the compression force along the compression diagonal leaving the steel plate alone resisting the tension stress in the perpendicular diagonal. This is due to the fact that, usually, steel does not need strengthening in tension.

The FRP panel is designed to be stiff with respect to its axial direction and weak from its transverse secondary axis, this will allow the FRP panel to act like an accordion where it allows the steel plate to extend with respect to its tension diagonal without having to debond

and at the same time stiffening the compression diagonal of the plate. This is one of the most important reasons that justify why there is no debonding between the steel plate and the FRP panel (even with high deformation at failure stage), and how the typical ductile failure of the steel plate is not only maintained but even improved as will be seen later in this section. Another important observation is that the tension field will act as bracing force for the composite compression strut in the central area leading to more buckling resistance. This means that with the proposed strengthening technique we have both the advantage of the composite strut resisting the compression force and the biaxial action of the steel plate in resisting the shear stress at the same time.

However, the stiffness of the strengthened specimens is obviously much higher than the control one and this need to be quantified. To do so, a relative stiffness increase index is introduced. This stiffness index can be explained by looking at Fig. (15-a), the shaded area between the vertical axis (the load axis) and the modified bucking curve was calculated for the control specimen and then compared to the same area for the other strengthened specimens. For the sake of comparison, this was done with a load equals the ultimate load of the control specimen (140.0kN). The stiffness index then can be written as

$$S.I = A_{ct} / A_{st} \dots\dots\dots(2)$$

where:

- S.I: is the relative stiffness increase index,
- A_{ct} : is the shaded area between the vertical axis (the load axis) and the modified bucking curve for the control specimen,
- A_{st} : is the shaded area between the vertical axis (the load axis) and the modified bucking curve for the strengthened specimens, up to a load equal to 140 kN.

Another important property that needs to be quantified as well is the change in energy absorption capacity (ductility) of the specimens. Usually the increase in the capacity of the strengthened specimen comes at a considerable reduction in the energy absorption (Okeil et al, 2009). In this study, the proposed strengthening technique is targeting the out-of-plane displacement while limited and insignificant effect is observed with the deflection curves. This will lead to the controversial question whether we should use the deflection curves or the buckling curves to estimate the energy absorption capacity. However, this is not the core

of this study; therefore a simple relative energy absorption index is proposed to compare the ductility of the strengthened specimens with the control one.

The increase in energy absorption factor can be taken as the area between the horizontal axis (the displacement axis) and the modified buckling curve of the strengthened specimen divided by the same area for the control specimen, see Fig. (15-a). The limit of 10mm was chosen to be the basis of comparison because most of the tested specimens reached this value; however, the ones who were stopped before this limit for technical reasons were extended to an out-of-plane displacement equals 10mm. Thus, the relative energy absorption index can be stated as:

$$E.I = A_{sb} / A_{cb} \dots\dots\dots(3)$$

where:

- E.I: is the relative energy absorption index,
- A_{sb} : is the area between the horizontal axis (the displacement axis) and the modified buckling curve for the strengthened specimens,
- A_{cb} : is the area between the horizontal axis (the displacement axis) and the modified buckling curve for the control specimen.

Table 5 illustrate the values both for the relative stiffness and energy absorption indices. It can be seen that the proposed strengthening technique succeeded in increasing the stiffness 3 times and that was not on the expense of energy absorption as it is improved up to 50%. It is worth mentioning that these two factors were not calculated for specimens with irregular buckling curves where the mode of buckling is altered (i.e. SP-8, SP-10, and SP-12).

NUMERICAL MODELLING

A non-linear finite element model was used to model the test specimens. These preliminary models are meant to act as indicators and bench marks for the tested specimens and to prove the validity of the proposed picture-frame testing rig; rather than an accurate finite element simulation to be used for a parametric study in the future. More accurate models shall be presented in a separate paper for the cyclic series of tests. Commercial software (*Abaqus* 6.10) was used. The steel plate and the FRP panel were modelled using a nine node reduced integration shell element S9R5, which have five degrees of freedom per node. This element is not available in *Abaqus* standard CAE and can be used only through an *Abaqus* input file.

S9R5 elements are meant for slender plates and were derived originally according to Kirchhoff thin plate bending theory. A Matlab code was written to create the nodes and element incidences to be incorporated in *Abaqus* input files. The size of the web elements was chosen to be 20×20mm based on the aforementioned convergence study. The initial imperfection was found using the elastic Eigen buckling modes; these were initiated using the buckling analysis available in *Abaqus CAE* and then imposed as an initial imperfection using *Abaqus* script commands in the input file.

A bilinear elasto-plastic stress-strain curve was adopted for the steel constitutive model, with a modulus of elasticity (E_s) of 200GPa and yield strength (f_y) of 275MPa. The FRP constitutive material model was simply an equivalent isotropic shell section with a modulus of elasticity of 22.1GPa and 43.9GPa for the GFRP and the CFRP, respectively. Tie constraints are applied between the steel plate and the FRP bonding strips for simplicity, even though it is known that tie constraint will give higher capacity than the same model with cohesive surface interaction instead.

Due to the numerical analysis time cost of the original picture-frame finite element model (Fig. 10-b), which takes usually between 24 to 36 running hours on a core i7 laptop; a simplified model was adopted in this paper, see Figs. (16-a) and (16.b). In Fig. (16-c) the implemented boundary conditions are shown. Table 6 illustrates these boundary conditions in detail. The simplified model is basically the steel plate tied to a surrounding plate frame with an optimized stiffness to simulate the original one. The four corners' hinges were simulated by coupling the translational displacements and freeing the rotational ones. The picture-frame plates were modelled using S4R elements available in *Abaqus CAE*. The loads were applied as shown in Fig. (15-a) and (15-b) to provide the pure shear status and to make sure that the model works properly.

Fig. (17) shows that the model predicts both the strength and the behaviour of the control specimen (without FRP strengthening) with reasonable accuracy. However, the composite model does not show the same accuracy. This may be mostly attributed to the above mentioned slack that took place during the test due to the minor slip at the top left corner of the picture-frame along the diagonal tension field. This took place because there was no mechanical anchorage between the plate and the picture-frame and the grip was solely dependent on the friction according to the original design. This problem was solved in the cyclic series of tests by using a different testing mechanism. Nevertheless, the composite model predictions are acceptable as indicators and not as reference values.

CONCLUSIONS

A novel preformed corrugated FRP panel is introduced in this study to strengthen slender steel plates, such as the web of a plate girder, against breathing deformations of plates leading to fatigue failures. The section of the FRP panel was optimized using finite element modelling accounted for minimizing the cost of the FRP material, the quantity of adhesive being used, workmanship, and the complexity of the multi-axial stress state in the web steel plate.

The subsequent study was divided into five phases. Phase-1 involved optimizing the new strengthening technique through finite element modelling. Phases-2 and -3 involved choosing the adhesive and determining the material mechanical properties for the FRP used through DLS and tension specimens, respectively. Phase-4, which is named the 'initial' series in this paper, was performed to test the efficiency of the proposed panel under static loading. Thirteen steel plates were strengthened with the proposed preformed corrugated FRP panel and tested using a special picture-frame rig. This was designed to hold the steel plate in position with the required boundary conditions while applying in-plane shear without the need to weld the plate into a stiff frame like the case with ordinary plate girders.

The results of the initial series of tests proved the efficiency of the proposed strengthening technique in increasing the stiffness of the steel plate section up to 3.0 times that of the unstrengthened one and consequently increasing its buckling shear stress. In addition to that, the proposed strengthening technique succeeded in preserving the ductile prototype failure associated with intact (unstrengthened) steel plate girders, which is a very important factor in safety usually ignored by other FRP strengthening techniques. As a matter of fact the energy absorption capacities of the strengthened specimens have been shown to exceed that of the unstrengthened ones by a factor of 1.5. No bonding failure could be detected during or after the test even when the steel plate was highly deformed and fully yielded.

The variables studied in this series were, (1) the effect of FRP material which significantly affected the stiffness and buckling behaviour of the strengthened specimens. The CFRP showed more superior behaviour in increasing the stiffness and the shear buckling stress of the strengthened specimens due to its higher modulus of elasticity than the GFRP; (2) the type of section (open versus closed), which showed no significant effect in increasing the stiffness and bonding capacity with respect to the originally proposed open section one; (3) the number of the FRP layers used to make the corrugated panel had a significant effect on the behaviour of the strengthened specimens where the 3-layered ones showed higher capacity and much more stable and ductile behaviour especially for the GFRP ones; (4) the

orientation of the GFRP used, which was taken with respect to the corrugation axis, had a moderate effect on the behaviour of the strengthened specimens. The 0°-90° succeeded in slightly increasing the ultimate strength for the 3-layered specimen while it altered the mode of failure to a more brittle one with respect to the 2-layered one; (5) The effect of the end cut of the FRP panel wasn't very significant in comparing type A (i.e. the triangular cut) and B (i.e. the circular cut) where they almost acted the same but was significant for type C (i.e. the long one) even when the mode of failure was altered for the GFRP specimen. This is believed to be due to the longer dimension which provided less stiffness to the panel and consequently may alter the mode of failure from an even buckling mode (for an aspect ratio of 1.0) to an odd one.

A geometrical and material non-linear analysis was used to model the specimens in this study. A simplified picture frame model was utilized in this study to avoid the more numerically expensive original model. The unstrengthened model was able to realistically simulate the behaviour of the specimens throughout all loading stages, while the strengthened model provided convincing results only up to the failure plateau.

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REFERENCES

- Al-Azzawi, Z., Stratford, T., Rotter, M., and Bisby, L., "FRP Shear Strengthening of Thin-Walled Plate Girder Web Panels Subjected to Cyclic Loading", Proceedings of PROTECT 2015, fifth international workshop on performance, protection & strengthening of structures under extreme loading, East Lansing, MI, USA, June 2015, pp. 350-359.
- Assoodani, Firas Abdul-Razzaq; "Shear behaviour and strength of CFRP retrofitted steel plate girders and steel-concrete composite plate girders", PhD thesis, University of Technology, Iraq, July 2014, 610 pp.
- Bhutto, Muhammad Aslam. "FRP composites for strengthening of steel bridge members," Proceedings of the 15th young researchers' conference, 14 March, 2013.
- Bureau of Transportation Statistics. "Transportation statistics annual report 2008," United States Department of Transportation, Research and Innovative Technology Administration, USA, 2008. National Bridge Inventory Database (NBI), Federal Highway Administration, Wikipedia, https://en.wikipedia.org/wiki/National_Bridge_Inventory.
- Miyashita T., Okuyama Y., Nagai M., Wakabayashi D., Koide N., Hidekuma Y., Kobayashi A., Horimoto W., "Shear buckling tests of steel girder bonded carbon fiber reinforced plastic on its web", Structural faults and repair, Edinburgh, 2012.
- Okeil, A.M., Bingol, Y., and Ferdous, Md.R. "Novel technique for inhibiting buckling of thin-walled steel structures using pultruded glass FRP sections", Journal of Composites for Construction, 13(6), 2009, pp. 547-557.

- Patnaik A.K. and Bauer, C.L. “Strengthening of Steel Beam with Carbon FRP Laminates”, Proceedings of the 4th International Conference on Advanced Composite Materials in Bridges and Structures, Calgary, July 2004.
- Roberts, T. M., Davies, A. W., and Bennett, R. J. H., “Fatigue shear strength of slender web plates”, Journal of Structural Engineering, October 1995, pp. 1369-1401.
- Roberts, T. M., Osman, M. H., Skaloud, M. and Zornerova, M., “Fatigue crack propagation and residual strength of slender web panels”, International Colloquium on Stability of Steel Structures, Budapest, 1995.
- Skaloud, M. and Zornerova, M. “The fatigue behaviour of the breathing webs of steel bridge girders,” Journal of Civil Engineering and Management, Vol. XI, No. 4, 2005, PP. 323-336.
- Schnerch, D.A. “Strengthening of steel structures with high modulus carbon fibre reinforced polymer (CFRP) materials,” PhD Thesis, North Carolina State University, 2005, 265 pp.

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Table 1: Manufacturers' fibre properties		
	PYROFIL TR50S 12K carbon fibre	Glass fibre
Tensile strength (MPa)	4900	2200
Tensile modulus (GPa)	240	80
Ultimate elongation (millistrain)	-	3.5
Typical density g/cm ³	1.82	2.6
Fibre mass per unit length (mg/m)	800	-

Table 2: Double shear lap test series results

Specimen	Load, kN	Stress, MPa
GFRP-600-ES	6.47	5.38
CFRP-650-ES	11.82	9.83
GFRP-600-TS	8.25	6.86
CFRP-650-TS	12.39	10.3
GFRP-600-EE	4.47	3.94
CFRP-650-EE	9.15	7.61
GFRP-600-TE	5.46	4.54
CFRP-650-TE	8.77	7.29
GFRP-600-ET	3.03	2.52
CFRP-650-ET	6.8	5.66
<ul style="list-style-type: none"> - The first 4 letters stands for glass or carbon fibre reinforced polymer. - The numbers stands for the weight of the fabric g/m². - The last 2 letters stands for the resin used, E for EL2, T for Tyfo, and S for Sikadur-330. While the order of them specifies which one of them is the saturating resin and which one is the bonding epoxy, as the first stands for the saturating resin and the second goes for the bonding epoxy. 		

Table 3: Mechanical properties of the FRP used in this study

Specimen	FVF	F_f , MPa	$\square_f$	E_f , GPa	Note
CFRP-650	0.58	527.66	0.28	32.23	2-Layers
CFRP-450	0.59	704.513	0.22	48.12	3-Layers
GFRP-600-45°-45°	0.49	34.45	0.50	7.35	2-Layers
GFRP-440-45°-45°	0.48	61.57	-	18.02	3-Layers
GFRP-600-0°-90°	0.49	238.56	0.52	18.71	2-Layers
GFRP-440-0°-90°	0.48	294.14	0.72	14.34	3-Layers

Table 4: Test variables

Ref.	Specimen	FRP type	Panel type	No. of layers	Fibre orientations*	Buckling load, kN	Ultimate load, kN
SP-1-4	Control Specimen	-	-	-	-	≈ 40.0	140.0
SP-2	GFRP-600-2L	Glass	Type A	2-Layers	+45°/-45°	-	147.8
SP-3	CFRP-650-2L	Carbon	Type A	2-Layers	0°/90°	-	160.5
SP-4	GFRP-600-2L-Closed	Glass	Type A	2-Layers	+45°/-45°	-	149.3
SP-5	CFRP-650-2L-Closed	Carbon	Type A	2-Layers	0°/90°	-	165.0
SP-6	GFRP-440-3L	Glass	Type A	3-Layers	+45°/-45°	-	157.0
SP-7	CFRP-450-3L	Carbon	Type A	3-Layers	0°/90°	-	153.3
SP-8	GFRP-600-2L-0-90	Glass	Type A	2-Layers	0°/90°	-	155.1
SP-9	GFRP-440-3L-0-90	Glass	Type A	3-Layers	0°/90°	-	155.3
SP-10	GFRP-600-2L-B	Glass	Type B	2-Layers	+45°/-45°	-	161.3
SP-11	CFRP-650-2L-B	Carbon	Type B	2-Layers	0°/90°	-	166.2
SP-12	GFRP-600-2L-C	Glass	Type C	2-Layers	+45°/-45°	-	153.0
SP-13	CFRP-650-2L-C	Carbon	Type C	2-Layers	0/90	-	173.8

* The orientation here is taken with respect to the corrugation axis.

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Table 5: Increase in stiffness and energy absorption indices for the initial series

Ref.	Specimen	Stiffness index	Energy absorption index
SP-1-4	Control Specimen	1.0	1.0
SP-2	GFRP-2L-(45°-45°)-A	1.4	1.1
SP-3	CFRP-2L-(0°-90°)-A	2.9	1.4
SP-4	GFRP-2L-(45°-45°)-A-Closed	1.6	1.2
SP-5	CFRP-2L-(0°-90°)-A-Closed	2.6	1.4
SP-6	GFRP-3L-(45°-45°)-A	1.8	1.2
SP-7	CFRP-3L-(0°-90°)-A	3.0	1.3
SP-8	GFRP-2L-(0°-90°)-A	-	-
SP-9	GFRP-3L-(0°-90°)-A	1.6	1.2
SP-10	GFRP-2L-(45°-45°)-B	-	-
SP-11	CFRP-2L-(0°-90°)-B	2.7	1.4
SP-12	GFRP-2L-(45°-45°)-C	-	-
SP-13	CFRP-2L-(0°-90°)-C	3.2	1.5

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Table 6: Simplified picture frame model boundary conditions

Surface	u	v	w	θ_x	θ_y	θ_z
1	1	1	1	1	1	0
2	0	0	1	1	1	0
3	1	1	1	1	1	1
4	0	0	1	1	1	1

1: stands for fixed and 0: stands for free

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(a) FRP Flexural Strengthening (b) Proposed FRP strengthening against shear buckling

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Closed FRP section

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(b): Vacuum bagging consumable materials

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(d): Test instrumentations

Fig. (11): Specimen details and test instrumentations.

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