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Authors

Mosallam, A
Elsanadedy, HM
Almusallam, TH
et al.

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Structural evaluation of reinforced concrete beams strengthened with innovative bolted/bonded advanced frp composites sandwich panels

Ayman Mosallam^{a,*}, Hussein M. Elsanadedy^b, Tarek H. Almusallam^b, Yousef A. AL-Salloum^b, Saleh H. Alsayed^b

^a Civil & Environmental Engineering Department, University of California, Irvine, CA 92697-2175, USA

^b Specialty Units for Safety & Preservation of Structures, Department of Civil Engineering, King Saud University, Riyadh, Saudi Arabia

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ABSTRACT

This paper presents and evaluate the performance of a novel, non-conventional fiber-reinforced-polymeric (FRP) composite system for repair strengthening reinforced concrete (RC) beams. This strengthening system is in the form of thin sandwich FRP panels that are bonded/bolted to concrete. In evaluating this innovative system, experimental and numerical assessment were conducted. In the experimental phase of this study, service and ultimate flexural behavior of three groups of RC beams were evaluated. The three beam specimens of the first group were designated as control specimens and their performance was used as baseline for comparison to the performance of the other two strengthened RC beams groups. The second and third groups comprised of three beam specimens for each group that were externally strengthened with bolted/bonded carbon/phenolic sandwich panels. A nonlinear finite element analysis was performed using LS-DYNA code to simulate and predict the flexural behavior of all RC beam specimens. A numerical parametric study was performed for different simulated cases to assess the effect of several parameters. Numerical results agreed well with the experimental results and test observations. The results of this study indicated that the FRP sandwich panel strengthening systems are effective in enhancing flexural behavior of the strengthened RC beams.

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1. Introduction

Few decades, polymer composites were introduced to the construction industry as new alternative structural materials and strengthening system for existing structures. One of the applications that attracted the attention of civil engineers was the external strengthening of existing structures constructed from different conventional materials such as steel (e.g. [21], reinforced concrete (e.g. [3,4,13,23]), masonry (e.g. [22,26] and wood (e.g. [25,1]). Due to the popularity and affordability of reinforced concrete, many of these structures have been in service for decades in almost every country worldwide. For the last two decades or so, two forms of FRP strengthening became popular, namely: (i) externally bonded cured-in-place and precured composite laminates (e.g. [20,25], and (ii) near-surface-mounted (NSM) composite strips, grids and rebars [5,24]. Until recently, little development to such systems was reported and the majority of research and applications focused on the two main common FRP strengthening

methods stated earlier with some exceptions of using prestressing composites elements [31]. Lately, Mosallam et al. [27] and Kim et al. [15] evaluated a hybrid high-performance composite retrofit system comprised of both CFRP laminates and high-performance mortar.

2. Related work

The concept of sandwich structure has been known for decades for its superiority to resist both out-of-plane as well as in-plane loads. However, most of the applications were focused in utilizing sandwich panels as stand-alone members, especially in aerospace industry and recently in bridge applications. The majority of the FRP bridge deck applications in USA are in the form of sandwich construction (e.g. [23], and [9]. In 1994, Mosallam [19] developed the H-Lam¹ system that is composed of thin sandwich panels with pre-drilled central, edge and staggered holes that provide the capability of inserting mechanical fasteners in addition to adhesive in attaching the H-Lam to different structural members. The H-Lam

* Corresponding author.

E-mail address: mosallam@uci.edu (A. Mosallam).

¹ US Patent Pending # 60-146,830.

system provides a prefabricated sandwich composite honeycomb panel that is comprised of two fiber composite faces, separated by and connected to a structural lightweight core that are less stiff and less rigid than the composite top and bottom faces. The faces and core are connected by an adhesive that provides structural continuity across the panel depth.

In 2004, Qiao, Yang, and Mosallam [29] discussed the implementation of functionally-degraded sandwich panels for over-height collision protection of highway bridges that was developed by the Mosallam in 1994 [19]. The I-Lam sandwich panel that is made of a composite sandwich construction with multi-layer aluminum honeycomb core and top and bottom aluminum thin face sheets, and they are developed/designed specifically for impact damage protection of bridge girders (e.g., reinforced concrete girders). In this study, the impact behavior of the I-Lam system was

analyzed using the higher-order sandwich beam theory, and the validity was demonstrated by comparisons with the numerical finite element simulations using ABAQUS and LS-DYNA software.

The behavior of beams made of multiple layers of sandwich panels with E-glass fiber-reinforced polymer face sheets and phenolic core was evaluated by Manalo and Aravinthan [17]. Belouettar et al. [6] reported the results of an experimental investigation on static and fatigue behavior of composites honeycomb materials. Experimental and numerical investigation focusing on assessing the interface crack propagation in foam-cored sandwich beams was presented by Berggreen et al. [8]. Results of the study indicated that for low core densities, where the crack propagates in the interface immediately below the face sheet, there is fair agreement between experiments and numerical simulated results. However, for higher density cores, cracks tend to propagate in the laminate itself with extensive fiber bridging leading to a conservative numerical predictions.

A literature review has revealed no studies on the use of honeycomb sandwich panels as externally bonded reinforcement of RC beams for flexural strength enhancement. Contribution of this new material technology in the area of rehabilitation of concrete structures was attempted in this study, in which CFRP-honeycomb as well as GFRP-honeycomb composite plate systems were used to strengthen RC beams in flexure. As mentioned earlier, the strengthening system comprises of a sandwich panel with FRP faceplates having a sinusoidal honeycomb core within, and is pre-equipped for mechanical anchorages. The system is glued using epoxy, or

Table 1
Test matrix.

Beam ID	Strengthening system	No. of specimens
CB	Control beam	3
B90CP	Beams strengthened with CFRP-honeycomb composite plate of 15 cm width and 90 cm length	3
B90GP	Beams strengthened with GFRP-honeycomb composite plate of 15 cm width and 90 cm length	3
Total no. of specimens		9

Fig. 1. Typical geometrical and reinforcement details RC beam specimen.

other room-temperature cured adhesives, as well as anchored mechanically using concrete screws to the soffit of the RC beams, which assists in precluding or even delaying the unfavorable interfacial debonding failure modes commonly encountered in RC beams upgraded with externally bonded FRP laminates.

In this study, a total of three groups of nine beams were tested under four-point bending. A numerical investigation utilizing non-linear finite element (FE) analysis was also conducted and a comparison was made between the experimental and numerical results. The numerical analysis was extended to evaluate the effect of several important parameters that were not evaluated experimentally, that included: (i) plate length, (ii) bond condition, and (iii) end anchorage on the flexural performance of strengthened beams.

3. Merits of the H-Lam composite retrofitting system

The H-Lam retrofit system provides a new generation of strengthening techniques where maximum quality control and quality assurance are maintained for the reinforcing elements. This due to the fact that the composite sandwich panels are fabricated and cured at the shop and the positions of the mechanical fasteners and the associated stiffeners are predrilled and pre-positioned at the shop. This eliminates a great deal of construction flaws that

are experienced with the existing composite strengthening systems. In addition, by providing both adhesives and mechanical fasteners in attaching the panels to the existing member surface, higher shear strength is achieved at the interface, and a safety factor against any environmental attacks to the adhesives at the bondline is achieved. The use of metallic or non-metallic mechanical fasteners in attaching the composite sandwich panels will also help in squeezing the excess adhesive, which in turn will assure a complete bond between the plate and the member to be repaired or upgraded.

In addition, this new strengthening system provides an easy and simple methodology of applying a finished product at the member surface without the need of the conventional wet lay-up process and the associated technical disadvantages. The length of these panels are flexible and can be cut to specified dimensions at the site using simple tooling that are commonly used for wood.

The presence of core material in the H-Lam system provides another mean of increasing the energy absorption capability of the reinforcing panel. This is a major concern especially in highway bridge application due to the high probability of sudden impact by vehicles and trucks and in industrial facilities where cranes and forklifts are presence as well as in the case of harbor and port application where impact generated from heavy tankers and ships are inescapable.

Fig. 2. Dimensions and tensile coupons for H-Lam-C and H-Lam-G sandwich panels.

Fig. 3. Typical tensile stress–strain curves for Nomex™ core materials: (a) fiber direction, (b) transverse direction [13].

4. Experimental program

4.1. Test matrix

The experimental program was designed to investigate the feasibility of using FRP-composite sandwich panels in increasing the flexural load capacity of RC beams. The test matrix consisted of 9 beams divided into 3 groups consisting of three beams each. All beams were rectangular in cross-section (150×150 mm) and 1200 mm long with an effective span of 1000 mm. The criterion for selection of the beam dimensions was based on available resources in the laboratory. The beams were reinforced with 2010 longitudinal steel bars in the tension side. The shear reinforcement comprised of $\phi 8$ stirrups at a spacing of 50 mm, to ensure that failure would be controlled by flexural yielding. All beam specimens evaluated in this study were designed to have singly reinforced sections with tension reinforcement only and hence, compression steel was not used. Three beams (the first group “CB”) were used as control specimens. Three beams (the second group “B90CP”) were externally upgraded by CFRP-composite sandwich panels in the tension side; whereas the last three beams (group “B90GP”) were externally strengthened with GFRP-composite sandwich panels. The test matrix for all the RC beam specimens evaluated in this study is listed in Table 1. Details and dimensions of test beams are presented in Fig. 1.

4.2. Materials properties

4.2.1. Concrete and steel reinforcement

Ready-mix concrete was used to fabricate the RC beam specimens. Six standard cylinders (150×300 mm) were fabricated from

Table 2
Mechanical properties of H-Lam composite sandwich panels.*

Property	CFRP-honeycomb plate	GFRP-honeycomb plate
Thickness of FRP faceplate (mm)	0.65	0.75
Thickness of honeycomb core (mm)	8.0	8.5
Thickness of composite plate (mm)	9.3	10.0
Sandwich panel's tensile strength		
Average (MPa)	489	269
Standard deviation (MPa)	31.07	21.92
COV (%)	6.4	8.2
Sandwich panel's elastic modulus		
Average (GPa)	37.80	18.07
Standard deviation (GPa)	1.12	0.29
COV (%)	2.9	1.6

* Results are based on test coupons; COV = coefficient of variation.

Table 3
Mechanical properties of epoxy adhesive.*

Property	Typical test value
Glass transition temperature (T_g)	82 °C
Tensile strength	72.4 MPa
Tensile modulus	3.18 GPa
Elongation percent	5.00%
Flexural strength	123.4 MPa
Flexural modulus	3.12 GPa

* Based on manufacturer's datasheet.

the same concrete patch for evaluating the 28-day compression strength of the concrete. The average compressive strength of the six cylindrical specimens of the concrete mix was 44 MPa. In order to determine the actual characteristics of steel reinforcement, three specimens of steel rebars from each diameter were tested under uniaxial tension. The average values for yield strength are 425 and 528 MPa for $\phi 8$ mm and $\phi 10$ mm steel bars, respectively. The average experimental values for tensile strength are 638 and 620 MPa for $\phi 8$ mm and $\phi 10$ mm steel bars, respectively.

4.2.2. H-Lam composite sandwich panels

In this study, both CFRP-honeycomb (H-Lam-C) and GFRP-honeycomb (H-Lam-G) composite panels were used as strengthening systems (refer to Fig. 2a–c). The H-lam panels were manufactured using heated press manufacturing process shown in were used to strengthen RC beams in flexure. The system is composed of low-density honeycomb Nomex™ core that is made of aramid fiber paper coated with heat-resistant phenolic resin. The total thickness of the core is 9.05 mm and 8.94 mm for the carbon/phenolic and glass/phenolic H-Lams, respectively. The fiber architecture of the composite face sheets are in the form of cross-ply $[0^\circ/90^\circ/c]_s$ for both H-Lam-G and H-Lam-C with a unit ply thickness of 0.279 (face sheet laminate thickness = 0.56 mm) and 0.305 mm (face sheet laminate thickness 0.61 mm) for carbon/phenolic and glass/phenolic H-Lams, respectively. The total thickness of all the H-lam systems is 10.16 mm. As mentioned before, and in order to ensure complete bond to H-Lam and concrete substrate, 5 mm diameter holes with molded inserts at each hole (refer to Fig. 2c) were pre-drilled at an equal spacing of 30.48 cm at the centerline of each sandwich panel. For characterizing mechanical properties of composite sandwich panels, three ASTM standards are commonly

used: (i) Standard Practice for Determining Sandwich Beam Flexural and Shear Stiffness ASTM (D7250/D7250M-06-2012) and (ii) Standard Test Method for Flatwise Tensile Strength of Sandwich Constructions (ASTM C297/C297M-04-2010, and (iii) Standard Test Method for Edgewise Compressive Strength of Sandwich Constructions (ASTM C364/C364M-07-2012). However, another test method, similar to that employed by Moody and

Anthony [18], was modified and employed to determine the mechanical properties for each panel in the length-wise direction. In order to obtain the mechanical properties of the sandwich panels, five standard coupons prepared from each composite system were fabricated and tested to determine the different mechanical properties for each H-Lam system (refer to Fig. 2f and g). As shown in these figures, and in order to prevent crushing of the soft core at

(a) Saturating concrete surface with epoxy after sandblasting

(b) Bonding CFRP Sandwich Panel to Concrete Using Epoxy Plus Three Concrete Screws

(c) Tightening concrete screws

(d) Applying pressure on the I-Lam-C Using Clamps

Fig. 4. Application sequence of composite sandwich panels.

the grips locations, the honeycomb materials were removed at the gripped ends and the space between the face sheets at these locations was filled with a rectangular steel solid core of 100 mm in length, 152.4 mm in width and with the thickness associated of each H-Lam panel. The specimens were subjected to in-plane tensile loads up to failure. Fig. 2h and i show the typical ultimate failure modes of test coupons. Typical load–strain curves for the core materials are developed by Foo et al. [12] and are presented in Fig. 3. The average geometrical mechanical properties of each system are presented in Table 2.

4.2.3. Epoxy adhesives

A two-part, room-temperature epoxy adhesives was used in bonding all the sandwich panels to the beam specimens' soffits. As shown in Table 3, the tensile strength and modulus of the room-temperature cured epoxy adhesive used in bonding the

H-Lams to the soffit of the RC beam specimens are 72.4 MPa and 3.18 GPa, respectively. Other thermo-mechanical properties are also presented in Table 3.

4.2.4. Mechanical fasteners

As mentioned earlier, mechanical fasteners in the form of low-carbon concrete screws were used to attach the H-Lam panels to the RC beam specimens, in conjunction with the epoxy adhesives. The yield and tensile strengths of these screws were 240 MPa and 400 MPa, respectively. The total length of each metal screw was 100 mm.

4.3. Installation of H-Lam sandwich panels

The installation sequence of the H-Lam panels is presented in Fig. 4. After the curing period for the RC concrete beams, the

(a) Instrumentation layout

(b) Test setup

Fig. 5. Typical beam test setup and instrumentation.

bottom sides of specimens B90CP and B90GP were sand-blasted to remove dirt and any loose materials and were then cleaned. Markings were made on the specimens to outline the H-Lam edges. As discussed earlier, both epoxy adhesives and low-carbon steel screws were used to attach the H-Lam panels to bottom (tension)

side of the strengthened beam specimens. Prior to bonding composite plates to concrete, a hammer drill was employed to make a pilot hole for each concrete screw. Three concrete low-carbon steel screws with 3-mm diameter were installed at 30.48 cm equal spacing to mechanically fasten each H-Lam panel. The tension

Table 4

Material properties used in the FE modeling.

<i>Concrete</i>		
Material model	MAT_CSCM_CONCRETE	
Uniaxial compressive strength (MPa)	44.0	
Max aggregate size (mm)	10	
<i>Steel rebars</i>		
Material model	Ø10 main tension steel bars	Ø8 transverse reinforcement
Young's modulus (MPa)	MAT_PIECEWISE_LINEAR_PLASTICITY	MAT_PIECEWISE_LINEAR_PLASTICITY
Poisson's ratio	200,000	200,000
Yield stress (MPa)	0.3	0.3
Tangent modulus (MPa)	528	425
Plastic strain to failure	784	0
	0.1169	0.11788
<i>Composite laminate used for H-Lam*</i>		
Material model	CFRP	GFRP
Thickness per layer (mm)	MAT_ENHANCED_COMPOSITE_DAMAGE	MAT_ENHANCED_COMPOSITE_DAMAGE
Young's modulus in long. dir. (MPa)	0.65	0.75
Young's modulus in transverse dir. (MPa)	37,800	18,070
Longitudinal tensile strength (MPa)	1754	838
Transverse tensile strength (MPa)	489	269
	48.9	26.9
<i>Honeycomb core materials used in the composite plate†</i>		
Material model	MAT_PIECEWISE_LINEAR_PLASTICITY	
Young's modulus (MPa)	50	
Poisson's ratio	0.4	
Yield stress (MPa)	5	
Tangent modulus (MPa)	0	
Plastic strain to failure	1.00E-05	
<i>Carbon fiber reinforced polymer (CFRP) Ply used as end anchorage</i>		
Material model	MAT_ENHANCED_COMPOSITE_DAMAGE	
Thickness per layer (mm)	1.0	
Young's modulus in long. dir. (MPa)	77,300	
Young's modulus in transverse dir. (MPa)	3380	
Longitudinal tensile strength (MPa)	846	
Transverse tensile strength (MPa)	40.6	

* Based on testing standard composite plate coupons.

† Assumed properties.

Table 5

Experimental vs. FE results with key points of load–deflection curves.*

Beam ID	Results	P_y (kN)	P_u (kN)	Δ_y (mm)	Δ_u (mm)	K_s (kN/m)	μ_Δ	ε_{su} ($\mu\epsilon$)	$\varepsilon_{FRP,u}$ ($\mu\epsilon$)	Failure mode
CB	EXP (1)	42.99	52.19	2.54	31.71	23387	12.49	31403	–	CC
	EXP (2)	43.06	55.66	2.49	28.27	21816	11.35	NA	–	CC
	EXP (3)	45.26	53.72	3.11	35.55	19569	11.43	NA	–	CC
	EXP-AVG	43.77	53.86	2.71	31.84	21591	11.76	31403	–	
	FE	43.09	50.20	3.30	32.00	19231	9.70	31878	–	CC
	EXP-AVG/FE	1.02	1.07	0.82	1.00	1.12	1.21	0.99	–	
B90CP	EXP (1)	54.39	90.72	2.68	8.41	27450	3.14	8547	9205	DL
	EXP (2)	58.32	100.78	2.75	11.34	30326	4.12	9177	NA	DL
	EXP (3)	58.39	98.85	2.80	9.44	31890	3.37	9350	10355	DL
	EXP-AVG	57.03	96.78	2.74	9.73	29888	3.55	9025	9780	
	FE	54.93	90.69	3.00	10.50	30549	3.50	8844	9138	DL
	EXP-AVG/FE	1.04	1.07	0.91	0.93	0.98	1.01	1.02	1.07	
B90GP	EXP (1)	58.86	86.99	3.04	11.42	34242	3.76	13173	14515	FR
	EXP (2)	49.39	85.32	2.51	10.42	25755	4.15	15221	NA	FR
	EXP (3)	48.19	78.92	2.60	12.65	26988	4.87	17640	NA	FR
	EXP-AVG	52.15	83.74	2.72	11.49	28995	4.26	15345	14515	
	FE	51.59	86.06	3.00	13.00	26552	4.33	14515	14829	FR
	EXP-AVG/FE	1.01	0.97	0.91	0.88	1.09	0.98	1.06	0.98	

* P_y and Δ_y = load and mid-span deflection at yielding of main steel; P_u = ultimate load; Δ_u = mid-span deflection at ultimate; K_s = effective stiffness at service load level; μ_Δ = deflection ductility ratio = Δ_u/Δ_y ; ε_{su} = main steel strain at ultimate load; $\varepsilon_{FRP,u}$ = FRP strain at ultimate load; CC = concrete crushing; DL = concrete cover delamination; FR = plate rupture; NA = not available data.

sides of strengthened beam specimens and the bonding surface of the H-Lams were coated with epoxy adhesive as illustrated in Fig. 4a and b. The sandwich panels were then bonded to the concrete surface and the screws were tightened using screw driver as shown in Fig. 4c. In order to ensure optimum bond quality, pressure was applied to the sandwich panels using metal clamps as seen in Fig. 4d.

4.4. Flexural tests setup

Epoxy adhesive was allowed to cure at the laboratory ambient temperature, after which each of the nine RC beam specimens were subjected to a 4-point loading protocol. All beam specimens had a clear flexural span of 1.0 m and a shear span (distance from the support reaction force to the nearest applied concentrated load as shown in Fig. 1) of 0.425 m ($a/d = 3.15$). The load was applied using a rigid steel transfer beam (see Fig. 5) that was connected to a calibrated 2000-kN AMSLER testing machine. As shown in the Fig. 5, a calibrated load cell was securely positioned between the moving crosshead and the rigid load-transfer steel beam to capture the applied incremental load during each test. In all tests, the load was applied monotonically in the form of a linear displacement-control ramp with a displacement rate of 1 mm/min up to failure. A Linear Variable Displacement Transducer (LVDT) was attached at the bottom side of each specimen at the mid-span to measure vertical deflections at these locations. In addition, electrical strain gages were bonded to both internal tensile steel rebars, as well as, to outer face (tension side) of each H-Lam to measure strains for each beam specimen.

5. Test results and discussion

Table 5 lists a summary of the flexural behavior of all test groups in terms of: (i) yield load (load corresponds to the first measured yield strain of tensile steel), (ii) flexural ultimate strength, (iii) mid-span deflection at yield load (Δ_y), (iv) mid-span deflection at ultimate load (Δ_u), (v) effective stiffness at service load, (vi) steel strain at peak load, (vii) FRP strain at peak load, (viii) failure mode and (ix) deflection ductility. In this paper, deflection ductility is weighted using the deflection ductility ratio (μ_Δ) defined as:

$$\mu_\Delta = \frac{\Delta_u}{\Delta_y} \quad (1)$$

Fig. 6. Load–deflection curves for tested beams.

Due to the fact that the load–deflection curves generated for the specimens of each group were very similar, load versus mid-span deflection curves for one representative beam specimen of each group are shown in Fig. 6. However, all load–deflection curves for all specimens of each group are presented later in Fig. 12. The ultimate failure mode for each beam specimen are illustrated in Fig. 7.

(a) Control group CB

(b) Strengthened group B90CP

(c) Strengthened group B90GP

Fig. 7. Modes of failure for tested beams.

As shown in Fig. 6, the typical load-deflection curve for the control group CB displayed typical nearly-bilinear response of under-reinforced concrete beams. The ultimate failure of all the control beam specimens was due concrete crushing during the yielding of the internal steel reinforcement (see Fig. 7a). However, strengthened beam specimens failed either by peeling of the concrete cover from the high-modulus/high strength carbon/epoxy H-Lam cutoff points (cohesive failure) as shown in Fig. 7b or due to tensile rupture of the low-modulus/low-strength glass/epoxy H-Lam (refer to Fig. 7c). The failure of the strengthened specimen was brittle. Initially, uniformly distributed flexural cracks were observed at the location of the bonded/bolted composite panels. As compared to cracks observed in the control beam specimens, the cracks were narrower for the strengthened beams. As the load increased, the size of the flexural increased leading to a flexural failure at the mid-span region. The ultimate mode of failure was in the form of concrete crushing at mid-span region top surface as shown in Fig. 7a. The average ultimate load (P_u) and deflection ductility (μ_Δ) for the three control beam specimen were 53.86 and

11.76 kN, respectively. For the strengthened group B90CP, the three beams evaluated in this study displayed similar behavior. At an average ultimate load of 96.78 kN, beam specimens failed suddenly due to concrete cover delamination at the right end as shown in Fig. 7b. However, for group B90GP, the beams attained a relatively lower average peak load of 83.74 kN at which failure occurred due to sandwich panel rupture near the mid-span as seen in Fig. 7c. For both strengthened groups B90CP and B90GP, the flexural capacity of the beams dropped suddenly upon failure to values close to those of control specimens. In general and based on experimental results, experimental results indicated that the use of bolted/bonded H-Lam composite system not only succeeded in increasing the flexural capacity of the as-built (control) RC beams, but also enhanced the flexural stiffness of the strengthened RC beams. For example, the average values of flexural strength gain were 80% and 55% for H-Lam strengthened beam groups B90CP and B90GP, respectively (refer to Table 5). Also, Table 5 shows that as compared to the control (unstrengthened) beams, the flexural stiffness of the two H-Lam strengthened groups B90CP and

Fig. 8. Finite element mesh in FEM software showing the modeled components.

B90GP have increased by 38% and 34%, respectively. The average deflection ductility values for groups B90CP and B90GP are 3.55 and 4.26, respectively. It should also be noted that no interfacial debonding between the sandwich panel and the concrete was observed for any of the six upgraded beams (see Fig. 7). This can be attributed to the superiority of using the combined bolted/bonded joint between the H-Lam and the concrete.

6. Numerical analysis

LSTC [16], a general-purpose finite element program, was utilized for the numerical modeling of the test beams. The 3-D finite element model was developed using a general-purpose

pre-processor FEMB. Only one half of each RC beam was modeled accounting for its symmetry.

6.1. Model geometry

In order to model the real behavior of the RC beams evaluated experimentally in this study, it is imperative that the concrete volume be modeled using solid elements. For this reason, 8-node reduced integration solid hexahedron elements were used to model both concrete and honeycomb core. Both the longitudinal steel rebars and the transverse steel stirrups were modeled using 2-node Hughes-Liu beam elements [14] and [30]. The carbon/phenolic and glass/phenolic face sheets of the H-Lam sand-

Fig. 9. Comparison between experimental and FE results for all beam specimens.

wich panels were modeled using 4-node shell elements. The Belytschko and Tsay [7] element formulation was used for shell elements.

In the FEM model, the concrete solid elements were $12.5 \text{ mm} \times 12.5 \text{ mm} \times 12.5 \text{ mm}$ and $12.5 \text{ mm} \times 12.5 \text{ mm} \times 25 \text{ mm}$ in size; whereas, honeycomb solid elements were $8.65 \times 12.5 \times 12.5 \text{ mm}$ and $9.25 \times 12.5 \times 12.5 \text{ mm}$. Numerical convergence study showed that further decrease in the mesh size has little effect on the numerical results but leads to the risk of computer memory overflow and substantially increases the computing time. Fig. 8 shows the typical mesh of an upgraded beam, which consists of 6192 solid concrete elements, 432 solid honeycomb elements, 436 beam elements for the reinforcing bars and 864 shell elements for the FRP laminates to give a total of 7924 elements. The FE analysis was based on perfect-bond assumption between steel rebars and surrounding concrete, between Nomex™ honeycomb core and FRP face sheet laminates as well as between top FRP face sheet laminate and concrete substrate. The latter assumption is justified and confirmed by experimental observations where debonding failure mode was not observed in all strengthened beam specimens due to the use of both adhesives and metal screws.

6.2. Materials modeling

The material model Type 159, MAT_CSCM_CONCRETE was employed to model the concrete volume. This is a smooth or continuous cap model available in LS-DYNA for solid elements, with a smooth intersection between the shear yield surface and the hardening cap. The current version of concrete material model 159 has up to 37 input parameters, with a minimum of 19 parameters that must be fit to data. Default material model parameters are provided for the concrete model based on three input specifications: the unconfined compression strength (*grade*), the aggregate size, and the units. The parameters are fit to data for unconfined compression strengths between about 20 and 58 MPa (2901 to 8412 psi), with emphasis on the midrange between 28 and 48 MPa (4061 and 6962 psi). The unconfined compression strength affects all aspects of the fit, including stiffness, three-dimensional yield strength, hardening, and damage. The fracture energy affects only the softening behavior of the damage formulation. Softening is fit to data for aggregate sizes between 8 and 32 mm (0.3 and 1.3 inches). In this model, the initial damage surface coincides with the yield surface. Concrete cracking is considered using the traditional smeared crack approach.

The material model Type 24 (Piecewise Linear Plasticity), MAT_PIECEWISE_LINEAR_PLASTICITY was utilized to model steel

bars and honeycomb core. This material is suited to model elastoplastic materials with an arbitrary stress versus strain curve and an arbitrary strain rate dependency. For steel rebars, the minimum information needed to model rebar is the nominal yield strength. In this analysis, the Nomex™ honeycomb core material was assumed to be linear elastic up to failure and hence, a very small value of $1.00\text{E-}05$ was assigned for the plastic strain to failure as identified in Table 4.

In order to model the FRP laminates used in the composite sandwich panels, the material model Type MAT_054-055, MAT_ENHANCED_COMPOSITE_DAMAGE was employed. An orthotropic material with optional brittle failure can be defined using this material card. Various failure criteria are possible for this card. The failure criterion of Chang and Chang [10] was used in this analysis. A summary of the material properties used in the FE analysis is presented in Table 4.

6.3. Erosion

The erosion option provides a way of including failure to the material models. This is not a material or physics based property; however, it lends a great means to imitate concrete spalling phenomena and produce graphical plots which are more realistic representations of the actual events. By activating this feature, the eroded solid element is physically separated from the rest of the mesh. Material failure was simulated by element erosion at a specific plastic strain; thus, whenever an element reaches this critical value, it is removed from the computation. This erosion model represents a numerical remedy to distortion, which can cause excessive and unrealistic deformation of the mesh. The erosion option used in the FE analysis is associated with the constitutive material model. For concrete model (material model type 159), the erosion card has to be input along with the material card. In this study, concrete elements were allowed to erode when the maximum principal strain reached 0.05. For other constitutive models used in the FE analysis, the erosion option is not required as the failure criteria are enough to designate material failure in the model. For instance, the failure criterion of Chang and Chang [10] was used to define the failure of FRP laminates. For steel bars and Nomex™ honeycomb core, the plastic strain to failure was sufficient to signify material failure.

6.4. Boundary conditions

Due to the symmetry of the RC beams, only one half of the beam was modeled in LS-DYNA. A node set was created which consisted of nodes at support location of the beam which had to be restricted

Fig. 10. FE simulated mode of failure for control beam CB.

for the displacement in the global *Z-direction thus representing a roller support near the beam end. Symmetric boundary conditions were applied for the nodes in elements for the plane representing the continuation of the beam in reality. This included restriction of displacement in the global X-direction and the rotation about the global Y- and Z-directions for those nodes. Since the loading was displacement controlled, another node set was created which comprised of nodes along the loading plane which were controlled to have the same Z-displacement throughout the test.

6.5. Loading strategy

LS-DYNA uses explicit time integration algorithms for solving the problems, which are less sensitive to machine precision than

other finite element solution methods. The load application process in LS-DYNA is time-history dependent. Since the testing procedure involved displacement-controlled quasi-static loading, the inertia effects were removed from the dynamic equation by assigning a constant velocity to the displacement controlled node set. This will lead to zero acceleration and hence zero inertia force. The rate of change of displacement was defined as 1 mm/min to match with the experimental loading.

7. Finite element results and discussion

A summary of both FE and experimental results is listed in Table 7. A graphical comparison between experimental and FE analysis results for all RC beam specimens is presented in Fig. 9. As

(a) Contours of Maximum Principal Strain – Whole model

(b) Contours of X-stress –Outer CFRP Face Sheets

(c) Contours of X-stress – Honeycomb Core

Fig. 11. FE simulated mode of failure for strengthened beam B90CP.

(a) Contours of maximum principal strain – Whole model

(b) Contours of X-stress – Outer GFRP face sheets

(c) Contours of X-stress – Honeycomb core

Fig. 12. FE simulated mode of failure for strengthened beam B90GP.

shown in Fig. 9, a good agreement was achieved between the experimental and numerical results. The following paragraph discuss the results of the FE numerical results.

7.1. Modes of failure

Figs. 10–12 present the simulated modes of failure for RC beams evaluated in this study using LS-DYNA software. It should be noted that these failure modes are based on contours of mid-surface maximum principal strains. Figs. 10 and 11 presents the X-stress contours for sandwich panel outer composite face and honeycomb core, for specimens B90CP and B90GP, respectively. Comparing the simulated failure

modes with those observed from the laboratory tests, one can conclude that the results of the FE analysis agree very well with test results. For example, numerical analysis results indicated that the control beam CB failed due to concrete crushing after the formation of wide flexural cracks in the maximum moment region as depicted in Fig. 10. As shown in Fig. 11, the failure of specimen B90CP is due to concrete cover delamination after reaching a maximum stress of 345 MPa in the outer CFRP faceplate (70% of the CFRP tensile strength). The maximum corresponding X-stress in the honeycomb core was about 0.66 MPa (about 0.2% of the maximum CFRP stress). However, as seen in Fig. 11, B90GP failed by plate rupture at midspan after the outer GFRP faceplate attained a maximum stress of

(a) Beam CB

(a) Beam CB

(b) Beam B90CP

(b) Beam B90CP

(c) Beam B90GP

(c) Beam B90GP

Fig. 13. Experimental and numerical load–deflection curves for tested beams.

Fig. 14. Experimental and numerical load versus longitudinal steel strain for tested beams.

268 MPa (99.6% of the GFRP tensile strength). Yet, the maximum corresponding X-stress in the honeycomb core was about 1.0 MPa (about 0.4% of the maximum GFRP stress). The very low stress levels in the honeycomb core are attributed to its low tensile modulus.

7.2. Load–deflection curves

Fig. 13 presents a comparison between the load–deflection curves obtained from the experimental and the FE studies for all the beam specimens. One can see from this figure that the experimental load–deflection curves are in good agreement with the test results especially in predicting the ultimate load carrying capacity of both control and strengthened RC beams (refer to Table 5). As seen in Table 5, the maximum deviations of 4% and 7% were found for the numerical results for yield and ultimate loads, respectively. Yet, compared with the experimental results, maximum deviations of 18%, 12% and 21% were observed for mid-span deflection at yield load, mid-span deflection at ultimate load and deflection ductility, respectively. The stiffness of the beam specimens was also predicted accurately by the FE models in comparison with the test results. Fig. 13 also shows that the FE models were effective in simulating the softening behavior which reveals the accuracy of the material model. The FE analysis also revealed the effectiveness of using FRP-honeycomb composite sandwich panels in upgrading the flexural capacity and stiffness of RC beams but with the reduction of their deflection ductility.

7.3. Numerical and experimental strain results

The maximum tensile strain in the main steel bars obtained from the FE analysis was compared with the experimental values obtained using the strain gages. Fig. 14a–c illustrate this comparison for beams CB, B90CP and B90GP, respectively. The table also shows that a good agreement between measured and predicted steel strains was achieved. It is clear that the control (as-built) beam specimen, CB, had higher ductile behavior with a strain ductility (ratio of steel strain at ultimate load to steel yield strain – refer to Eq. (1)) of 12, which is typical for under-reinforced beams failing in flexure. Nevertheless, strain ductility of 3.4 and 5.5 was predicted for B90CP and B90GP, respectively.

In addition to longitudinal steel, strain gages were attached to the bottom FRP composite face sheets to measure their strain during the experiment. Fig. 15 and Table 5 present a comparison between experimental and numerical tensile strain results for the bottom FRP composite face sheet of strengthened beams. As shown, a satisfactory correlation between the experimental and predicted strains was achieved. From Table 5, one can clearly see that for beam B90CP; the composite face of the sandwich panel did not reach its tensile capacity due to cover delamination as both measured and predicted tensile FRP strains at peak load were only 76% and 70% of its rupture strain, respectively. The failure of strengthened beam specimen B90GP was due to tensile rupture of the composite sandwich panel. However, both measured and predicted FRP strains at peak load were very close to the values at rupture strain recorded from the tests (14,887 $\mu\epsilon$) as measured experimentally.

8. Effect of length and end conditions of composite sandwich panels: a parametric study

The FE analysis was utilized to conduct parametric study on three parameters that may affect the overall performance of the

H-Lam strengthened RC beams. In this analysis, sandwich panels with identical physical and mechanical properties as for those evaluated experimentally were utilized. The three main parameters considered in this part of the study are:

- Effect of plate length:** Four different plate lengths (90, 75, 60 and 45 cm) were investigated numerically. The four plate lengths correspond, respectively, to plate length ratio within shear span (ratio of plate length within shear span (L_s) to shear span (a)) of 0.88, 0.71, 0.53 and 0.35. Both H-Lam-C and H-Lam-G composite sandwich panels were evaluated.
- Effect of the degree of attachment of the sandwich panel to beam soffit:** Two cases were studied numerically. The first case is for combined bonded/bolted sandwich panels. This case simulates an RC beam strengthened with H-Lam panels that are both adhesively bonded in addition of the use of concrete screws. In such case, interfacial plate debonding is inhibited and, as a result, perfect bond was assumed between the plate and the concrete substrate. The second case simulates a condition where the sandwich panels are

(a) Beam B90CP

(b) Beam B90GP

Fig. 15. Experimental and numerical load versus FRP strain for strengthened beams.

attached to RC beams soffits via epoxy adhesives only (without concrete screws). In this scenario, the bond between the sandwich panels and concrete substrate was modeled through the tiebreak surface-to-surface contact definition of LS-DYNA. The contact algorithm accounts for both normal and shear forces in the interface. Under tensile and shear loads, tiebreak allows the separation of the tied surfaces following an interface strength-based failure criterion. The following failure criterion was used in this work:

$$\left(\frac{|\sigma_n|}{NFLS}\right)^2 + \left(\frac{|\sigma_s|}{SFLS}\right)^2 \geq 1 \quad (2)$$

where: σ_n is the normal stress, σ_s is the shear stress, NFLS is the normal failure stress and SFLS is the shear failure stress.

After failure, this contact type behaves as a surface-to-surface contact with no thickness offsets. In addition, after failure, no interface tension is possible. Failure parameters were estimated based on Neale et al. [28] as follows:

$$NFLS = 0.62\sqrt{f'_c} \quad (\text{MPa}) \quad (3)$$

$$SFLS = 1.5\beta_w NFLS \quad (4)$$

where:

$$\beta_w = \sqrt{\frac{2.25 - b_f/b_c}{1.25 + b_f/b_c}} \quad (5)$$

(a) Set :1 Beams strengthened with CFRP -sandwich panels

(b) Set :2 Beams strengthened with GFRP-sandwich panels

(c) FE model with FRP end anchorage

Fig. 16. Details of FRP end restraints of strengthened beams.

where: f'_c = concrete compressive strength, b_f = width of composite plate and b_c = width of RC beam. The contact modeling procedure followed in the FE analysis was validated by Elsanadedy et al. [11].

$$A_{\text{anchor}} = \frac{(A_f f_{fu})_{\text{longitudinal}}}{(E_f \kappa_v \epsilon_{fu})_{\text{anchor}}} \quad (6)$$

(c) *Effect of panel's end restraint*: In order to assess the impact of end restraints on the behavior of H-Lam strengthened beams, two cases were numerically studied. In the first case, no end anchorage was provided; while in the second case, two-sided (U-shaped) CFRP laminates were bonded at the termination regions of the H-Lam plates acting as end anchors at these zones with high interfacial shear stresses. The area of these two-sided CFRP laminates was set as 50% of that required by Eq. (6) of the ACI 440.2R-08 [2]. Details of CFRP end anchors are shown in Fig. 16. Based on the ACI 440 requirements and knowing that the tensile strength of the H-Lam-C panel is about 1.8 times of the tensile strength of the H-Lam-G panel, one-ply and two-ply CFRP laminates were adhesively bonded to each side of the RC beams at ends of the H-Lam-C and the H-Lam-G sandwich panels, respectively (refer to Fig. 16a). As shown in Fig. 16c, the two-sided CFRP laminates were modeled using 4-node shell elements with material properties presented in Table 3. The bond

between CFRP side laminates and concrete substrate was modeled through the tiebreak surface-to-surface contact described previously in Eqs. (2)–(5).

In order to evaluate the effect of all three parameters discussed in the preceding paragraph, thirty cases were numerically simulated and evaluated. This is in addition to the numerical models that were developed for the two strengthened RC beam groups B90CP and B90GP that were evaluated experimentally. Details of all RC beams simulated numerically are listed in Table 6. The additional thirty numerical models listed in Table 6 have the same dimensions and material properties as those used for beams B90CP and B90GP. As shown in Table 6, strengthened beams are divided into two sets, viz. Set (1) is for beams strengthened with CFRP sandwich panels (H-Lam-C) and Set (2) is for beams strengthened with GFRP sandwich panels (H-Lam-G). For beams of Set (1), H-Lam rupture was detected for beams upgraded with 90 cm sandwich panels which was also provided with CFRP flat laminate end anchors (specimens B90CPW and B90CPW-AB). However, for all other specimens of Set (1), failure was due to either concrete cover delamination or H-Lam interfacial end debonding. For specimens of Set (2), H-Lam panel rupture was observed for: beams with fully bonded 90-cm plate (B90GP & B90GPW); beam with fully bonded 75 cm panels that was provided with CFRP end anchors (B75GPW); and beams strengthened with 90 cm and 75 cm adhesively bonded sandwich panels that were provided with CFRP flat laminate

Table 6
Details of strengthened RC beams specimens used in FE parametric analysis.

Beam ID	Length of strengthening plate (mm)	Length of strengthening plate in shear span (L_s) (mm)	Plate length ratio within shear span (L_s/a)	Bond condition of strengthening plate	Provision of FRP end anchorage
<i>Set 1: Beams strengthened with CFRP sandwich panels</i>					
B90CP	900	375	0.88	Fully bonded	None
B90CPW	900	375	0.88	Fully bonded	Yes
B90CP-AB	900	375	0.88	Adhesively bonded	None
B90CPW-AB	900	375	0.88	Adhesively bonded	Yes
B75CP	750	300	0.71	Fully bonded	None
B75CPW	750	300	0.71	Fully bonded	Yes
B75CP-AB	750	300	0.71	Adhesively bonded	None
B75CPW-AB	750	300	0.71	Adhesively bonded	Yes
B60CP	600	225	0.53	Fully bonded	None
B60CPW	600	225	0.53	Fully bonded	Yes
B60CP-AB	600	225	0.53	Adhesively bonded	None
B60CPW-AB	600	225	0.53	Adhesively bonded	Yes
B45CP	450	150	0.35	Fully bonded	None
B45CPW	450	150	0.35	Fully bonded	Yes
B45CP-AB	450	150	0.35	Adhesively bonded	None
B45CPW-AB	450	150	0.35	Adhesively bonded	Yes
<i>Set 2: Beams strengthened with GFRP sandwich panels</i>					
B90GP	900	375	0.88	Fully bonded	None
B90GPW	900	375	0.88	Fully bonded	Yes
B90GP-AB	900	375	0.88	Adhesively bonded	None
B90GPW-AB	900	375	0.88	Adhesively bonded	Yes
B75GP	750	300	0.71	Fully bonded	None
B75GPW	750	300	0.71	Fully bonded	Yes
B75GP-AB	750	300	0.71	Adhesively bonded	None
B75GPW-AB	750	300	0.71	Adhesively bonded	Yes
B60GP	600	225	0.53	Fully bonded	None
B60GPW	600	225	0.53	Fully bonded	Yes
B60GP-AB	600	225	0.53	Adhesively bonded	None
B60GPW-AB	600	225	0.53	Adhesively bonded	Yes
B45GP	450	150	0.35	Fully bonded	None
B45GPW	450	150	0.35	Fully bonded	Yes
B45GP-AB	450	150	0.35	Adhesively bonded	None
B45GPW-AB	450	150	0.35	Adhesively bonded	Yes

end anchors (B90GPW-AB and B75GPW-AB). For all other Set (2) specimens, failure was controlled by either concrete cover delamination or plate interfacial end debonding. The relationship between H-Lam panel length ratio within shear span (L_s/a) and percent gain in flexural capacity as compared to the control (unstrengthened beam specimen capacity) is shown in Fig. 17a and b for Sets (1) and (2), respectively. In general, as the sandwich panel length ratio within the shear span zone increases, the percent gain in flexural capacity increases. For the same sandwich panel length, the addition of FRP flat laminate end anchors significantly enhances the gain in flexural capacity. For the same sandwich panel length, numerical results indicated that the percent gain in flexural capacity for fully bonded sandwich panels is approximately the same as that for adhesively bonded sandwich panels when FRP flat laminate end anchors are used. However, in case of absence of FRP flat laminate end anchorage, for plate length ratio within shear span of 0.88, the percent gain in flexural capacity for fully-bonded sandwich panels are considerably more than that of adhesively bonded sandwich panels. A summary of FE

analysis results for the strengthened RC beams is listed in Table 7.

9. Conclusions and recommendations

9.1. Conclusions

Based on both the experimental and numerical results and observations, the following major conclusions are drawn:

1. Experimental results indicated that the innovative H-Lam sandwich panel strengthening systems are shown to be effective in increasing both flexural strength and stiffness of strengthened RC beams. For example, an average gain in the flexural capacity of 55% and 80% was added for beams strengthened with GFRP and CFRP sandwich panels, respectively. However, and as for the general reported trend for conventional FRP strengthening, deflection ductility was reduced.
2. Experimental results indicated that H-Lam sandwich panel systems are capable of enhancing flexural stiffness of the strengthened RC beams at service load level. For example, an average increase in flexural stiffness of beams strengthened with GFRP and CFRP sandwich panels of 34% and 38% was attained, respectively. This gain in flexural stiffness is translated to an appreciable deflection reduction of RC beams at service load level.
3. Based on the experimental observations, the use of combined bolted/bonded joint between the H-Lam panels and the concrete prevented the typical interfacial panel debonding that is common for conventional FRP flat laminates strengthening technique. In this regard, H-Lam strengthening systems may be considered to have a superior performance and higher reliability as compared to conventional externally bonded FRP flat laminates.
4. Numerical results obtained from the FE parametric analysis performed in this study, it is concluded that in general, as the sandwich panel length ratio increases within shear span, the percent gain in flexural capacity increases. It is also evident that the provision of bonding two-sided FRP laminates, with an area of at least 50% of that required by Eq. (6) of the ACI 440.2R-08 [2] at the ends of the H-Lam sandwich panels, is efficient in preclusion or even delaying the unwanted failure modes such as concrete cover delamination and plate end interfacial debonding.

9.2. Recommendations

Based on the outcomes of this study, the following recommendations for further research to be conducted in this new area includes:

- (1) Investigation on the long-term (durability) of the H-Lam system is needed.
- (2) Additional studies on the effect of changing geometrical and mechanical properties of the H-Lam systems such as the use of different types of core materials such as aluminum, foam, etc. and panels with different thicknesses are needed.
- (3) Further research on dynamic and fatigue behavior of RC beams strengthened with this innovative system is also recommended.
- (4) Extending the applications of this system to cover reinforced concrete slabs, bridge decks, walls, columns and masonry structural elements are also recommended.
- (5) In order to validate the field performance of this system, as it was done for steel bridge and collision applications [21,22], a study on utilizing this system in a bridge or building that is monitored under service load is also recommended.

(a) Set (1): Beams strengthened with CFRP-sandwich panels

(b) Set (2): Beams strengthened with GFRP-sandwich panels

Fig. 17. Effect of panel length on flexural capacity of upgraded beams (based on FE analysis).

Table 7

FE parametric analysis results for strengthened RC beams.*

Beam ID	P_y (kN)	P_u (kN)	Δ_y (mm)	Δ_u (mm)	K_s (kN/m)	μ_Δ	ϵ_{su} ($\mu\epsilon$)	$\epsilon_{FRP,u}$ ($\mu\epsilon$)	Failure mode
<i>Set 1: RC beams strengthened with CFRP sandwich panels</i>									
B90CP	54.93	90.69	3.00	10.50	30549	3.50	8844	9138	DL
B90CPW	57.85	105.99	3.40	14.60	30114	4.29	13766	12880	FR
B90CP-AB	55.85	85.97	3.00	9.70	29363	3.23	8043	8055	End DB
B90CPW-AB	57.02	105.47	3.30	13.90	30081	4.21	13584	12876	FR
B75CP	55.67	79.24	3.30	8.60	30163	2.36	6553	6033	DL
B75CPW	55.17	91.61	3.00	11.00	30008	3.60	10508	8422	DL
B75CP-AB	55.39	78.68	3.40	8.30	29579	2.26	5998	5958	DL
B75CPW-AB	55.68	89.62	3.00	10.80	29591	3.37	9234	7898	End DB
B60CP	54.16	66.35	3.40	9.20	29520	1.56	3159	4352	DL
B60CPW	53.40	75.69	3.30	9.90	30380	2.21	6097	6289	DL
B60CP-AB	54.50	64.63	3.70	9.10	28924	1.51	2870	4212	DL
B60CPW-AB	53.42	75.49	3.50	8.70	30095	2.14	5985	6089	DL
B45CP	52.93	57.93	4.20	23.50	23583	1.26	2884	3591	DL
B45CPW	51.81	63.10	3.60	18.10	23742	1.83	3233	4141	DL
B45CP-AB	51.53	56.90	4.20	19.10	23222	1.26	2734	3426	DL
B45CPW-AB	52.16	63.25	3.60	19.60	23852	1.72	3277	4225	DL
<i>Set 2: Beams strengthened with GFRP-sandwich panels</i>									
B90GP	51.59	86.06	3.00	13.00	26552	4.33	14515	14829	FR
B90GPW	51.59	86.06	3.00	13.00	26552	4.33	14515	14829	FR
B90GP-AB	49.91	79.85	2.90	13.60	26797	3.62	11920	11941	End DB
B90GPW-AB	50.15	86.89	2.80	11.90	26791	4.25	15344	14830	FR
B75GP	49.83	77.07	3.00	11.20	26219	3.57	10571	10237	DL
B75GPW	51.43	86.37	3.20	13.90	26771	4.34	15885	14807	FR
B75GP-AB	50.27	75.98	3.00	11.10	26091	3.27	10010	9150	DL
B75GPW-AB	51.84	85.80	3.10	13.20	27309	4.26	14832	14838	FR
B60GP	49.37	66.75	3.30	9.30	26429	2.12	6289	6559	DL
B60GPW	49.13	72.87	3.10	9.70	26576	3.03	9054	8976	DL
B60GP-AB	49.17	65.51	3.30	9.50	25231	1.88	5866	6142	DL
B60GPW-AB	49.23	71.92	3.10	9.30	27180	2.84	8351	8610	DL
B45GP	48.65	57.56	3.70	21.40	21115	1.46	3223	4314	DL
B45GPW	47.65	62.73	3.30	20.60	22491	1.91	5464	6214	DL
B45GP-AB	48.53	57.81	3.80	20.60	21991	1.42	3769	4329	DL
B45GPW-AB	47.61	62.93	3.40	21.40	22904	1.85	5586	6258	DL

* P_y and Δ_y = load and mid-span deflection at yielding of main steel; P_u = ultimate load; Δ_u = mid-span deflection at ultimate; K_s = effective stiffness at service load level; μ_Δ = deflection ductility ratio = Δ_u/Δ_y ; ϵ_{su} = main steel strain at ultimate load; $\epsilon_{FRP,u}$ = FRP strain at ultimate load; CC = concrete crushing; DL = concrete cover delamination; FR = plate rupture; End DB = plate interfacial end debonding.

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