SIZE EFFECT ON SHEAR CAPACITY OF REINFORCED CONCRETE BEAMS STRENGTHENED BY FRP U-SHAPE JACKET

L. NGUYEN-MINH[1*], T. TRAN-QUOC[2], D. HO-DUC[1], C. HO-HUU[1], M. T. TRINH-LAM[3], O. W. KEONG[4†], and M. ROVŇÁK[5]

[1] *Faculty of Civil Engineering, Ho Chi Minh City University of Technology, Vietnam*

[2] *Faculty of Civil Engineering, Ton Duc Thang University, Vietnam*

[3] *Tat & Associates D'Architectura Co., Ltd., Vietnam*

[4] *Fyfe Asia PTE Ltd., Singapore*

[5] *Department of Architecture, Faculty of Art, Technical University of Kosice, Slovakia*

ABSTRACT

This paper deals with effect of size on shear capacity of reinforced concrete (RC) beams strengthened with carbon and glass fiber-reinforced polymer (CFRP and GFRP) jacket. Eighteen RC beams strengthened with CFRP and GFRP U-shaped jacket of three proportional geometrical scales were tested. The results show that the high effectiveness of FRP jacket in shear strengthening and the increase of shear capacity are higher in the smaller specimens. The FRP jacket reduces tensile strain of stirrups, width of shear crack at failure and decreases brittleness of beam failure. The beam size affects considerably on strengthened beams, but not on rehabilitated beams (beams with pre-crack). The stirrups of the strengthened beams were not yielded at beam failure; however the stirrups of the rehabilitated beams were yielded in large scale. Therefore, the assumption of stirrup yielding in calculation of shear capacity of FRP strengthened beams in current recommendations or codes should be rechecked and considered separately for beams with or without pre-crack in order to avoid non-conservative results.

Keywords: size effect; beams; shear strengthening, fiber-reinforced polymer (FRP); shear capacity.

1. INTRODUCTION

The use of fiber reinforced polymer (FRP), as non-corrosive material having high strength to weight ratio, has become popular alternative to strengthen, repair or rehabilitate structures beside traditional materials and techniques such as externally bonded steel plates, steel or concrete jackets and external post-tensioning. External bonding of FRP sheets or jackets has been known to increase considerably shear capacity of reinforced concrete (RC) beams. Nevertheless, the shear

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* Corresponding author: Email: minhlong_nguyen@yahoo.com

† Presenter: Email: ong_wk@fyfeasia.com

strengthening of RC members with FRP is actually a complex problem that is far from being completely solved and is still under investigation (Pellegrino and Modena, 2006).

The shear capacity of a traditional RC beam is influenced by several parameters, in which, the depth of beam or size effect is an important parameter (Kani, 1967; Bazant and Kim, 1984; Shioya et al. 1989). It has been shown that the average shear stress to cause failure of the larger beam was smaller than the average shear stress to cause failure of the smaller beam (ACI-ASCE 445R, 1999). Regarding FRP strengthened beams, the importance of size effect becomes more clearly since it decides to relation between effective bond length and effective FRP strain then corresponding shear capacity (Khalifa and Nanni, 2000; Deniaud and Cheng 2001). Despite this fact, relatively few works dealing with size effect in RC beams strengthened by CFRP U-shaped or fully wrapped-shape strip have been published (Leung et al. 2007, Godat et al. 2010). The problem, how does beam size affects on strengthening effectiveness of external FRP jacket on corresponding shear capacity in beams of practice size, are still not clear. Moreover, the assumption of stirrup yielding is accepted widely for calculation of shear capacity of FRP strengthened beams in current recommendations and codes (ACI 440.1R, 2006). However, some recent researches show that not all stirrups in an FRP strengthened beams intersected by critical shear crack can reach yielding at shear failure (Deniaud and Cheng, 2001; Pellegrino and Modena, 2008). This problem should be clarified to avoid un-conservative predictions.

The paper deals with an experimental study on size effect in shear behavior and resistance of large-scale RC beams strengthened by GFRP and CFRP U-shaped jacket, in which, total eighteen beams of three different sizes are tested. The principal motivation of the experimental program is for a better understanding of size effect on the axial strains in the FRP jackets and the corresponding shear capacity of the beams in practical size.

2. EXPERIMENTAL INVESTIGATION

2.1. Materials and testing specimens

Test beams were made from concrete, which contained cement PC40 (490 kg/m^3), natural sand (0-4 mm, 720 kg/m^3), coarse aggregate (22 mm, 1225 kg/m^3), water (167 l/m^3) and plasticizer (5 l/m^3). The average compressive concrete strengths $f_{c,cube}$ and splitting tensile strengths $f_{sp,cube}$ (cube edge of 150 mm) are 25 MPa and 2.25 MPa, respectively. Steel re-bars of 10, 14, 20, 22 and 25 mm diameter were used as longitudinal tensile reinforcement. Stirrups were of 6 mm and 8 mm diameter. Rebars of 8, 10, 12, 14 and 16 mm diameter were used for top rebar. The mechanical properties of steel reinforcement were determined by the tensile tests from an average of six samples. The average yield stress and ultimate tensile strength of tensile longitudinal rebars were 420 MPa and 590 MPa. These values for stirrups were 350 MPa and 510 MPa, respectively. The modulus of elasticity of steel reinforcement was 200 GPa. The composite material is an unidirectional carbon and glass fiber fabric of 1.1 and 1.3 mm thick and was applied in a continuous U-shape with one, two and three layers.

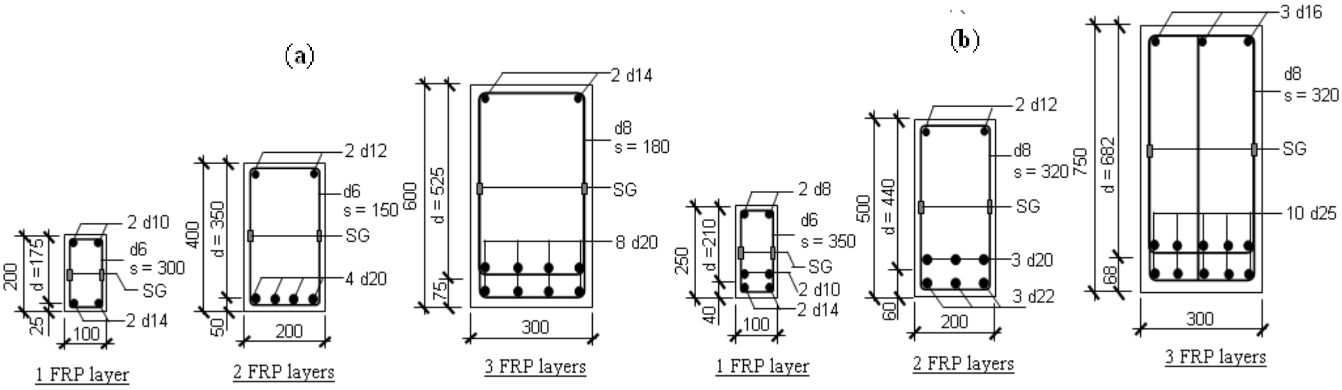

Fig. 1: Cross-sectional view of beams and reinforcement arrangement: (**a**) group G1; (**b**) group G2 (all dimensions in mm)

Table 1: Details of beams

Group	Signature of	Dimensions (mm)	L (mm)	d (mm)	a/d	ρ_s (%)	ρ_w (%)	t_f (mm)
G1	G1-RC1	100×200×1200	1000	175	1.7	1.7	0.19	-
	G1-GFRP1A							1.3
	G1-GFRP1B							1.3
	G1-RC2	200×400×2400	2000	350				-
	G1-GFRP2A							2.6
	G1-GFRP2B							2.6
	G1-RC3	300×600×3600	3000	525				-
	G1-GFRP3A							3.9
	G1-GFRP3B							3.9
G2	G2-RC1	100×250×1800	1500	210	2.0	2.4	0.16	-
	G2-GFRP-1A							1.3
	G2-CFRP-1							1.1
	G2-RC2	200×500×3600	3000	440				-
	G2-GFRP-2A							2.6
	G2-GFRP-2							2.2
	G2-RC3	300×750×5400	4500	682				-
	G2-GFRP-3A							3.9
	G2-GFRP-3							3.3

A total of eighteen beams were divided into 2 groups (**Table 1**). Group G1 includes nine beams those are strengthened by FRP jacket after concrete reached to 28 days (beams without pre-crack). Group G2 consists of other nine beams those first were loaded to obtain a required shear crack with limited width of 0.3 mm and then they were rehabilitated by FRP jacket. For group G1, all beams had shear span to depth ratio a/d = 1.7, where the shear span is defined as the distance from the loading point to the center of the nearest support. They had the same flexural (ρ_s = 1.7%) and shear reinforcement ratio (ρ_w = 0.19%). For group G2, there parameters are as follows: a/d = 2.0; ρ_s = 2.4%; and ρ_w = 0.19%. Reinforcement arrangement of the specimens is shown in **Fig. 1.**

2.2. Test procedure and instrumentation

The beams, simply supported, were tested under the two concentrated load (**Fig. 2**). Eight electrical gauges were bonded on steel stirrups to measure their strain, and another six gauges were used to measure strain of the FRP jackets. Strain of longitudinal tensile rebars was measured by one gauge bonded at mid-span and six another gauges bonded at shear-span (to assess dowel action of the rebars). Another four gauges were used to measure compressive concrete strain along the beam height. Five linear variable differential transformers (LVDTs) were used to determine deflections at the mid-span, at loading points, and at the supports of the beams. The beams were tested by a hydraulic testing machine (capacity 1000 kN) under load control in increments of 30~50 kN up to failure. The loading rate was approximately 10 kN / minute. At each load level, deflections, concrete strain, rebar strain, stirrups and FRP reinforcement strain, and crack development were recorded.

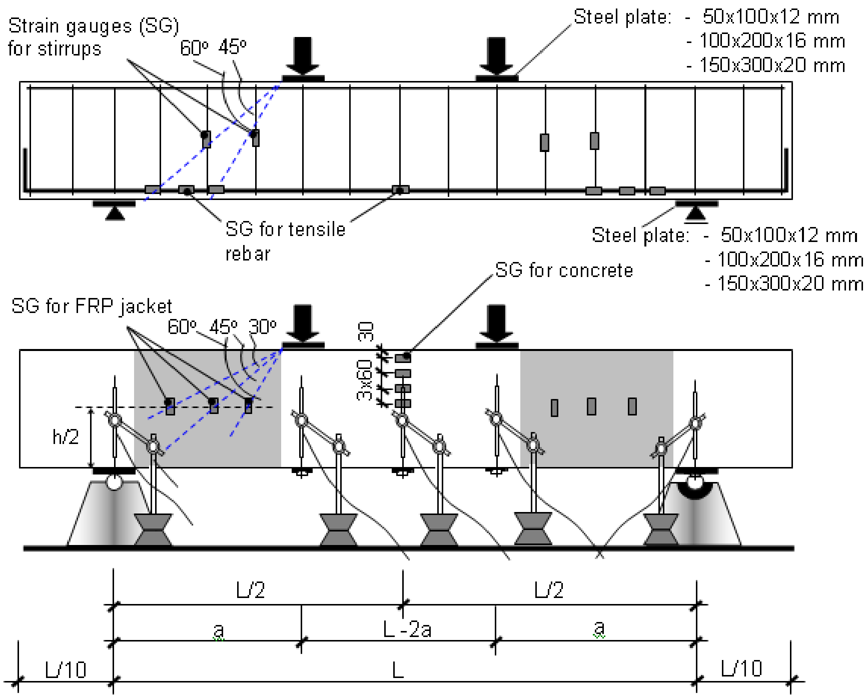

Fig. 2: Details of test specimens and test arrangement (all dimensions in mm)

3. TEST RESULTS AND DISCUSSION

3.1. Failure of specimens

All tested beams failed in shear combined with debonding of FRP jacket. The crack patterns for typical beams are illustrated in **Fig. 3.** The debonding of FRP jackets was occurred locally at the places where diagonal cracks initiated. The debonded jacket carried with them a thin layer of concrete, indicating a strong adhesive/concrete interface that leads to failure within the concrete. The inclination of shear cracks in all test beams were approximately from 38 to 45^o and the FRP jackets do not affect much on this inclination. The control beams failed more brittle and crack development was very fast in comparison with the strengthened beams. After carefully peeling off the FRP jackets, see that, the cracks in the strengthened beams (group G1) were distributed more uniformly with much smaller width than in the control beams. Opposite to the rehabilitated beams (group G2), the failure of these beams was governed strongly by development of the pre-crack. At failure, width of largest crack of the strengthened beams is smaller than of the control beams from 8 to 14 times (beams of group G1). For the beams of group G2, width of largest crack was equal to that of the control beams, approximately 6 mm.

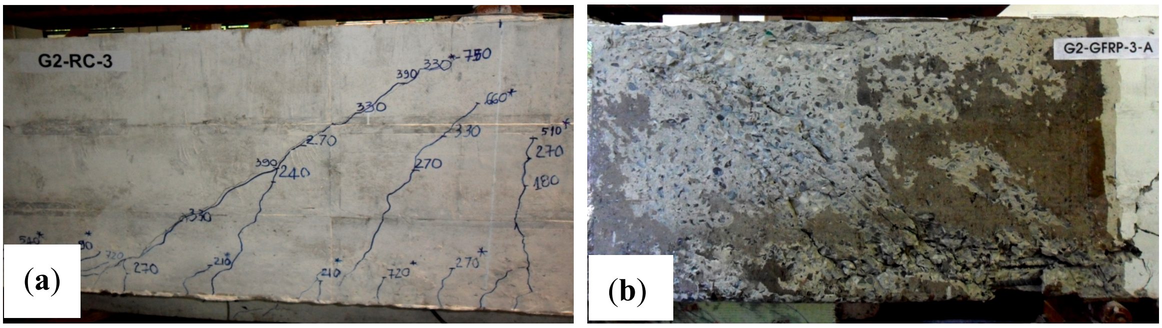

Fig. 3: Typical failure pattern of test beams: (**a**) control beams; (**b**) strengthened beams

3.2. Load-deflection responses

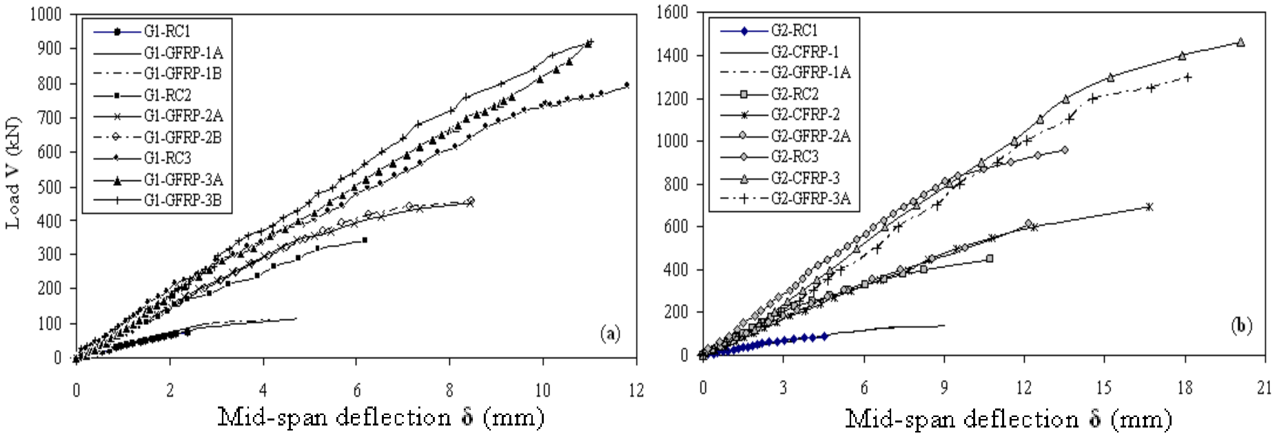

Fig. 4: Load – deflection diagrams of tested beams: (**a**) group G1; (**b**) group G2

The test results are summarized in **Table 3.** The load-displacement diagrams of tested beams are shown in **Fig. 4**. For the beams of group G1 (**Fig. 4a**), the control and strengthened beams behave similarly before initiation of shear crack. However, after shear cracking, their behavior is different. For the beams of smallest size, at the failure load, deflection of the control beams were 2.4 mm, while, average deflection of the strengthened beams were 2.2 mm that indicated slight reduction of 13%. Similarly, for the beams of middle and largest size (group 2 and 3), the deflection of the strengthened beams reduced up to 28% and 26%, respectively, in comparison with corresponding control beam. For the beams of group G2, the shear pre-crack causes the rehabilitated beams to become weaker than the control beams at first loading levels (to approximately 55% $V_{u,tot,exp}$) which results in their deflection to be slightly larger than that of the corresponding control beams (**Fig. 4b**). At the later loading levels (from 55% $V_{u,tot,exp}$ to failure), the deflection of the rehabilitated beams was smaller than of control beams. For the beams of smallest size, at the failure load of the control beam, the middle-span deflection of the rehabilitated beams is inconsiderably smaller than the one of the control beam about 4%, however, for the beams of middle and largest size, the deflection was reduced up to 21% and 15%, respectively.

3.3. Axial strain in FRP jackets and stirrups

Table 3: Test results

Group	Signature of beams	Beam size (mm)	$V_{u,tot,exp}$ (kN)	$V_{u,exp}$ (kN)	ε_{fu} (%)	ε_{wu} (%)	ε_{cu} (%)	ε_{su} (%)	$\varepsilon_{su,dow}$ (%)	δ_u (mm)
G1	G1-RC-1	100×200 ×1200	77	38.5	-	0.23	0.15	0.19	-	2.4
	G1-GFRP-1A		116	58.0	0.50	0.17	0.18	0.19	-	4.7
	G1-GFRP-1B		112	56.0	0.46	0.17	0.18	0.18	-	4.2
	G1-RC-2	200×400 ×2400	340	170.0	-	0.24	0.15	0.17	-	6.2
	G1-GFRP-2A		450	225.0	0.45	0.17	0.16	0.20	-	8.4
	G1-GFRP-2B		456	228.0	0.44	0.16	0.17	0.19	-	8.5
	G1-RC-3	300×600 ×3600	789	394.5	-	0.25	0.17	0.18	-	11.8
	G1-GFRP-3A		917	458.5	0.41	0.16	0.17	0.19	-	11.0
	G1-GFRP-3B		921	460.5	0.43	0.17	0.16	0.18	-	12.2
G2	G2-RC-1	100×250 ×1800	90	45	-	0.20	0.10	0.15	0.19	4.53
	G2-CFRP-1		140	70	0.39	0.22	0.20	0.18	0.18	9.04
	G2-GFRP-1A		125	62.5	0.51	0.19	0.18	0.16	0.17	7.14
	G2-RC-2	200×500 ×3600	450	225	-	0.25	0.09	0.16	0.19	10.7
	G2-CFRP-2		690	345	0.395	0.22	0.21	0.19	0.17	16.6
	G2-GFRP-2A		610	305	0.52	0.20	0.18	0.19	0.17	12.2
	G2-RC-3	300×750 ×5400	940	470	-	0.25	0.08	0.16	0.18	13.5
	G2-CFRP-3		1460	730	0.38	0.19	0.19	0.19	0.17	20.1
	G2-GFRP-3A		1300	650	0.51	0.23	0.18	0.19	0.19	18.1

Note: $V_{u,tot,exp}$ (kN) is failure load; $V_{u,exp}$ (kN) is shear force; ε_{fu}, and ε_{we} are tensile strain of FRP sheet and stirrups at beam failure; ε_{cu} is compressive strain of concrete at mid-span at beam failure; ε_{su} is max. tensile strain of longitudinal rebar at mid-span; $\varepsilon_{su,dow}$ is tensile strain of longitudinal rebar in shear span at beam failure; and δ_u (mm) is max. mid-span deflection of beam.

The load-strain diagrams of the FRP jackets and stirrups are presented in **Fig**. **5**. There is general trend that as load increases and approaches ultimate load, the strain rate of the jacket increases faster than the strain rate of the stirrups. For beams of group G1 (beams without pre-crack), the maximum stirrup strain of the control beams was 0.24 to 0.25%, indicating clearly that the stirrups were yielded (**Fig. 5a**). While, the maximum stirrup strain of the strengthened beams was only approximately 0.16 ~ 0.17% which are lower than their yielding value, showing evidently that the stirrups were not fully utilized in these cases and therefore the assumption of stirrup yielding in most current recommendations or codes in predicting shear resistance of FRP strengthened beams may be non-conservative. For beams of group G2, the existing shear crack results in high strain values of both jackets and stirrups at first loading steps (**Fig. 5b**). The maximum stirrup strain of the rehabilitated beams ranged from 0.19 to 0.25% that is approximately equal to that of the corresponding control beams, indicating that the stirrups were yielded in large scale. It is interesting that this remark is opposite to the remark in case of the strengthened beams of group G1. Therefore, the assumption of stirrup yielding in calculation of shear capacity of FRP strengthened beams should be rechecked and considered separately for beams with or without pre-crack.

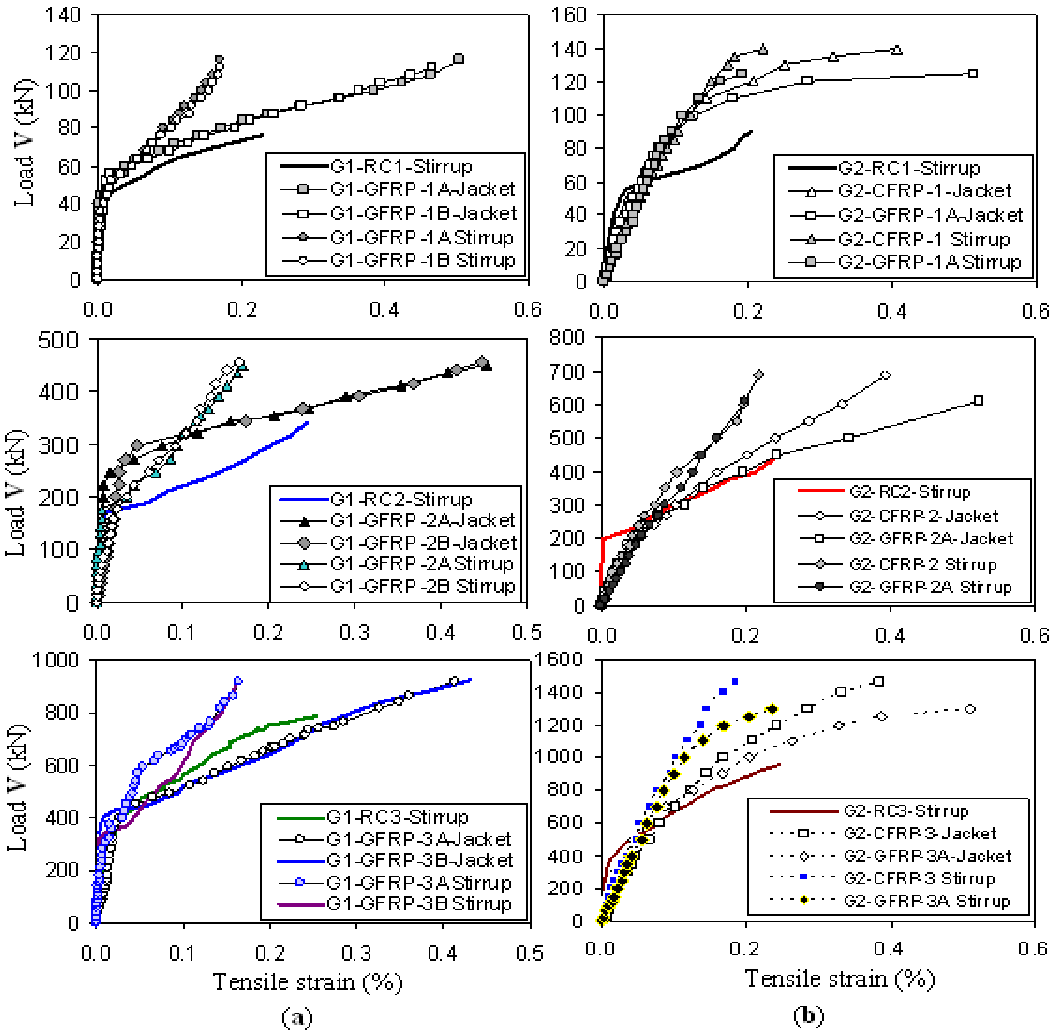

Fig. 5: Load –strain diagrams of FRP jacket and stirrups: (**a**) group G1; (**b**) group G2

3.4. Size effect in shear resistances

From the results summarized in **Table 3** follows that FRP jackets considerably enhanced the shear resistance of the beams. The size of tested beams affected significantly on shear resistances of the strengthened beams (group G1), however, it did not influence on shear resistances of the rehabilitated beams (group G2). For the beams of group G1, there is trend that as beam size increases, the increase of shear resistance decreases, in which, the average increase of the shear resistance of the strengthened beams in comparison with the control beam was approximately 48.1% for the beams of smallest size, 33.2% and 16.5% for the beams of middle size and largest size. For the beams of group G2, another trend is observed where the increase of the shear resistance of the beams remained mostly quasi-constantly with varying beam sizes that was approximately 53% for CFRP beams and 35% for GFRP beams.

4. CONCLUSIONS

Based on the results obtained from the study, the following conclusions can be drawn:

(1) The beam size affected on strengthened beams and rehabilitated beams (beams with pre-crack) in different ways. For strengthened beams, beam size affected significantly on tensile strain of FRP jackets. As beam size increases, the strain of the jackets decreases. For the rehabilitated beams, the

size of beams did not affect on tensile strain of FRP jackets. For all beams, as load increases and approaches ultimate load, the strain rate of the jacket increases faster than of the stirrups.

(2) The stirrups of the strengthened beams were not yielded at beam failure; however the stirrups of the rehabilitated beams were yielded in large scale. Therefore, the assumption of stirrup yielding in calculation of shear capacity of FRP strengthened beams should be rechecked and considered separately for beams with or without pre-crack in order to avoid non-conservative results.

(3) For the strengthened beams, the increase of shear resistance of the beams is inversely propotional with the size of test beams. The usage of FRP jackets increases shear resistance of the beams approximately from 25 to 58% and reduces beam deflection from 13 to 28%. For the rehabilitated beams, the increase of the shear capacity of the beams remained mostly quasi-constantly with varying beam sizes that was approximately 53% for CFRP beams and 35% for GFRP beams. FRP jackets reduce deflection of the beams from 4% to 21%.

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